Silicon Valley Rapid Transit Project
Tunnel Segment
Geotechnical Data Report
Volume I of VI

(P0503-D300-RPT-GEO-002, Rev.0)

Received SVRT PDCC Sep 29 2005

Silicon Valley
Rapid Transit Project

Tunnel Segment
Geotechnical Data Report
Volume I of VI

P0503-D300-RPT-GEO-002
Rev. 0

HMM/Bechtel

Prepared by
HMM/Bechtel SVRT,
a Joint Venture

VTA Silicon Valley Rapid Transit Corridor 2000 Measure A Project
BART to Milpitas San Jose Santa Clara

B0508-C006

This page intentionally left blank.

GEOTECHNICAL DATA REPORT FOR

PRELIMINARY DESIGN SERVICES FOR THE TUNNEL SEGMENT

Contract No. S03099

Submitted by: HMM/Bechtel SVRT, a Joint Venture

Andrew Liu, P.E. 9/23/05
Resident Geotechnical Engineer

Reviewed by:

Ignacio Arango, Ph.D., P.E., R.G.E. 9/23/05
Design Section Manager

John Hawley, P.E. 9/23/05
Tunnel Segment Design Manager

This page intentionally left blank.

VOLUME 1

TABLE OF CONTENTS

LIST OF APPENDICES

VOLUME VI

LIST OF TABLES

LIST OF FIGURES

This page intentionally left blank.

0.0 Executive Summary

The Santa Clara Valley Transportation Authority (VTA) is currently in the 35% Preliminary Engineering (PE) phase of the Silicon Valley Rapid Transit (SVRT) Project in Santa Clara County, California. This project consists of a 16.1-mile long extension of the Bay Area Rapid Transit (BART) heavy rail rapid transit system from the Warm Springs Extension (currently under design) in Warm Springs, to the Santa Clara Station in Santa Clara. The alignment is divided into three segments: a Line Segment approximately 9.7 miles (51,211 ft) long from Warm Springs to northeast San Jose; a Tunnel Segment approximately 5.3 miles (27,900 ft) long, consisting of twin-bored tunnels and cut-and-cover structures through San Jose, and a 1.1 mile long Yards and Shops Segment, extending from the west end of the Tunnel Segment to Santa Clara Station.

The Tunnel Segment includes two portals, constructed using cut-and-cover, U-wall, and retaining wall methods; three cut-and-cover stations; a cut-and-cover track crossover structure; and underground twin circular tunnels. The twin circular tunnels will be constructed using a closed-face tunnel-boring machine (TBM) to interconnect the stations and portals. The total length of the bored tunnels is 22,700 ft. The three underground stations are Alum Rock, Downtown San Jose, and Diridon/Arena Stations. Two ventilation shafts, several traction power substations and gap breakers, and several sump structures are also proposed along the tunnel segment.

As part of the 10% Conceptual Engineering, field explorations and geotechnical laboratory testing for the SVRT Project were carried out by URS Corporation in 2002 and 2003. The effort primarily targeted soil conditions at the locations of the proposed underground stations along Santa Clara Street in downtown San Jose.

A review of the available geotechnical information was performed at the early stages of 35% Preliminary Engineering. Based on the review, a supplemental comprehensive field and laboratory investigation program was prepared and implemented from October 2004 through April 2005.

The scope of the Tunnel Segment geotechnical investigation included exploratory borings and sampling; in-situ testing (cone penetration tests, vane shear tests, pressuremeter tests, downhole geophysical logging, and slug tests); observation well and vibrating-wire piezometer installations; and laboratory testing (classification, strength, stress-strain, and compressibility of the various materials encountered).

This Geotechnical Data Report presents the results of the investigations obtained for the 35% PE efforts up to May 2005. The new data verifies the validity and expands the information gained from the 10% Conceptual Engineering effort. Results of additional explorations/testing that may be required to further define conditions at specific locations, such as mid-tunnel ventilation shafts, surface structures, and tunnel cross-passages, will be presented as addenda to this report.

This page intentionally left blank.

1.0 Introduction

1.1 Project Description

The Santa Clara Valley Transportation Authority (VTA) is currently in the 35% Preliminary Engineering (PE) phase of the Silicon Valley Rapid Transit (SVRT) Project in Santa Clara County, California. This project consists of a 16.1-mile long extension of the Bay Area Rapid Transit (BART) heavy rail rapid transit system from the Warm Springs Extension (currently under design) in Warm Springs, to the Santa Clara Station in Santa Clara. The proposed route begins just south of the planned future Warm Springs Station and extends both at-grade and on aerial structures along the Union Pacific Railroad (UPRR) right-of-way through Milpitas to San Jose. The alignment then descends underground through twin tunnels (denoted S1 for "outbound" track to San Jose and S2 for "inbound" track towards Oakland) immediately east of the US 101 and UPRR overcrossing. The underground alignment proceeds south beneath US 101 at the McKee Road/Julian Street overcrossing and connects into Alum Rock Station. The alignment then turns west near the 28th Street and Santa Clara Street intersection and proceeds west beneath Santa Clara Street to the Downtown San Jose Station. The alignment continues under Santa Clara Street, turning southwest to the Diridon/Arena Station, before bending north and traveling beneath Stockton Avenue. The proposed route daylights north of Interstate 880 and continues at-grade to the west end of the SVRT Project at Santa Clara Station. The alignment includes six proposed stations (three above-grade and three below-grade), and vehicle storage and maintenance facilities. An additional station (Calaveras Station) is planned as part of a separate, future project.

The alignment is divided into three segments: a Line Segment approximately 9.7 miles (51,211 ft) long from Warm Springs to northeast San Jose, consisting of at-grade, elevated and cut-and-cover track; a Tunnel Segment approximately 5.3 miles (27,900 ft) long, consisting of twin-bored tunnels and cut-and-cover structures through San Jose (Figure 1-1); and a Yards and Shops Segment, extending from the west end of the Tunnel Segment to Santa Clara Station, approximately 1.1 miles long. Segment lengths are based on the April 2005 alignment drawings.

The Tunnel Segment includes two portals constructed using cut-and-cover, U-wall, and retaining wall methods; three cut-and-cover stations; a cut-and-cover track crossover structure; and underground twin circular tunnels. The twin circular tunnels will be constructed using a closed-face tunnel-boring machine (TBM) to interconnect the stations and portals. The total length of the bored tunnels along the S1 track alignment is 22,700 ft (based on the April 2005 alignment drawings). The three underground stations are Alum Rock, Downtown San Jose, and Diridon/Arena Stations. Two ventilation shafts, several traction power substations and gap breakers, and several sump structures are also proposed along the tunnel segment.

The following table presents the length of structures based on the April 2005 PE Alignment (S1 Track):

Table 1-1. Summary Lengths of Tunnel Segment Structures.

Location	Length (ft)	Structure Type
East Portal to Alum Rock Station	580	Retained Cut
	160	Cut and Cover
	2951	Bored Tunnel
Alum Rock Station	874	Cut and Cover
Alum Rock Station to Crossover Structure	8245	Bored Tunnel
Crossover Structure and Downtown San Jose Station	1632	Cut and Cover
Downtown San Jose Station to Diridon/Arena Station	2602	Bored Tunnel
Diridon/Arena Station	874	Cut and Cover
Diridon/Arena Station to West Portal	8902	Bored Tunnel
	201	Cut and Cover
	746	Retained Cut
West Portal	120	Open Cut/At Grade

1.2 Purpose and Scope

As part of the 10% Conceptual Engineering, field explorations and geotechnical laboratory testing for the SVRT Project were carried out by URS Corporation in 2002 and 2003. The effort primarily targeted soil conditions at the location of the proposed underground stations along Santa Clara Street in downtown San Jose. Limited information was obtained at other locations along the alignment.

A review of the available geotechnical information was performed at the early stages of 35% Preliminary Engineering (HMM/Bechtel, 2004a). Based on the review, a supplemental comprehensive field and laboratory investigation program was prepared in June (HMM/Bechtel, 2004b). The program was subsequently implemented from October 2004 through April 2005. Modifications to the original program were implemented at various times to accommodate changes in station locations, tunnel alignments, and tunnel depths. Modifications were also made to further investigate soil conditions as additional information became available (HMM/Bechtel, 2005a). It should be recognized that the decision to consider only one station in the downtown area was made after a considerable portion of the investigation had already been carried out at the now deleted Civic Plaza Station location, east of the currently proposed Crossover structure location.

The scope of the Tunnel Segment geotechnical investigation included exploratory borings and sampling; in-situ testing (including cone penetration tests, vane shear tests, pressuremeter tests, downhole geophysical logging, and slug tests); observation well and vibrating-wire piezometer installations; and laboratory testing (including classification, strength, and compressibility of the various materials encountered).

This Geotechnical Data Report presents the results of the investigations obtained for the 35% PE efforts up to May 2005. Some additional explorations/testing may be required to further define conditions at specific locations such as mid-tunnel ventilation shafts, surface structures, and tunnel cross-passages. The results of future work will be presented as addenda to this report.

1.3 Available Data and Information

Geologic, hydrogeologic, hydrologic, seismic, and geotechnical data from various sources were reviewed in preparation of this report. Information was obtained from the following sources: URS Corporation, the City of San Jose (CSJ), Santa Clara Valley Water District (SCVWD), Santa Clara Department of Public Works (SCDPW), California Department of Transportation (Caltrans), California Department of Water Resources (DWR), United States Geologic Survey (USGS), United States Army Corp of Engineers (USACOE), Federal Emergency Management Agency (FEMA), Earthquake Engineering Research Institute (EERI), National Earthquake Hazards Reduction Program (NEHRP), private files of several local geotechnical consulting organizations, and numerous papers available in technical publications. The list of references in Chapter 9 documents the specific reports from the various sources that were reviewed in preparation of this report.

1.4 Report Organization

Chapters 2 through 5 of this report deal with the geologic, seismologic, hydrologic and hydro-geologic setting of the project site, respectively. These chapters are followed by a description of the field investigation (Chapter 6) and the laboratory investigation (Chapter 7). The first part of Chapter 8 presents a general description of the surface and subsurface conditions along the alignment, ground water conditions, and air and vapor monitoring. The information in the second part of Chapter 8 (local stratigraphy, field testing, and physical soil properties including water content and unit weights, plasticity, gradation, compressibility, stress-strain behavior, and drained and undrained shear strength) is presented in groups corresponding to each of the seven individual study sections in which the Tunnel Segment alignment has been divided. The last section of Chapter 8 presents the engineering properties of materials, based on field and laboratory testing.

Results of the field investigations are presented in eleven appendices as follows:

- Logs of Borings, SPT Energy Calibration, and Gas Monitoring Measurements (Appendix 1)
- Field Vane Shear Tests (Appendix 2)
- Pressuremeter Tests (Appendix 3)
- P/S Wave Suspension Logging (Appendix 4)
- Vibrating Wire Piezometers (Appendix 5)
- Observation Wells (Appendix 6)
- Slug Testing Program (Appendix 7)
- Cone Penetration Test (CPT) Results (Appendix 8)

- Seismic Cone Penetration Test (SCPT) Results (Appendix 9)
- Cone Penetration Test (CPT) Hydropunch Water Sampling, Hydropunch Laboratory Test Results (Appendix 10)
- Dissipation Test Results (Appendix 11)

Laboratory test results are presented in seven appendices, as follows:

- Classification Tests (Appendix 12)
- Constant Rate of Strain (CRS) Consolidation Test Results (Appendix 13)
- Static Direct Simple Shear Test Results (Appendix 14)
- Consolidated Drained Triaxial Compression Test Results (Appendix 15)
- K_0 Consolidated Undrained Triaxial Test Results (Appendix 16)
- K_0 Consolidated Undrained Triaxial Compression Test Results (Bishop Method) (Appendix 17)
- Corrosion Testing Results (Appendix 18)

Appendix 19 contains a collection of photographs taken along the alignment from the East Portal to the West Portal. Appendix 20 contains X-ray images of tube samples sent to the Fugro laboratory in Houston for specialized testing.

1.5 Limitations

The geotechnical data presented in this report are the results of the investigation performed by HMM/Bechtel for the 35% Preliminary Design effort of the SVRT project. Data obtained by others for the 10% Conceptual Design are not included. To obtain additional data at specific locations along the alignment, future investigations will be required both during the Preliminary and the Final Design phases of the project. The results of any additional investigations will be presented as addenda to this report.

Figure 1-1. Map of Tunnel Segment Alignment for SVRT Project.

This page intentionally left blank.

2.0 Geologic Setting

2.1 General

The Tunnel Segment of the SVRT Project is located in the Santa Clara Valley, which is bounded by San Francisco Bay to the north, the Diablo Range to the northeast and the Santa Cruz Mountains to the southwest. The valley is covered by alluvial fan, levee, and active stream channel deposits with marine estuary deposits along the Bay margins.

The Tunnel Segment is located on alluvial deposits that are underlain, at depths greater than 1000 ft, by Tertiary-age (65 to 1.8 million years old), Upper Cretaceous-age (78 to 65 million years old) marine sedimentary rocks, and Cretaceous-age (144 to 65 million years old) Franciscan Complex bedrock, Figure 2-1 (Knudsen et al., 2000, Wagner et al. 1990). The older rocks appear at the ground surface in the mountain ranges to the northeast and southwest of the tunnel alignment.

The alluvium is Holocene-age (less than 10,000 years old) and can be classified as:

- Alluvial fan deposits (Qhf). These deposits consist of sand, gravel, silt, and clay, deposited by mountain canyon streams onto alluvial valley floors or plains.
- Fine-grained alluvial fan deposits (Qhff). These deposits occur on the flatter distal portion of fans and consist primarily of silt- and clay- rich sediments with interbedded lobes of coarser sand and occasional gravel.
- Alluvial fan levee deposits (Qhl). These deposits have formed naturally where streams have overtopped their banks and deposited sand, silt, and clay adjacent to the channel.

2.2 Faulting

The SVRT project is located in a highly active seismic region, bounded by the San Andreas Fault to the west, and the Hayward and Calaveras Faults to the east. Each of these faults has produced damaging earthquakes in the past. The valley margins are marked by belts of active thrust faults: the Foothills Fault system to the southwest and the East Valley Thrusts (the southeast extension of the Hayward Fault) to the northeast.

Faults located in the SVRT project area are shown in Figures 2-2 and 3-1.

The three active fault sources with the greatest contribution to the ground motion shaking hazard of the SVRT Tunnel Segment are the Hayward, San Andreas and Calaveras Faults. A description of the project seismic design criteria and corresponding expected characteristics of the earthquake ground motions as well as a more detailed discussion of the three fault sources mentioned above are presented in Chapter 3.

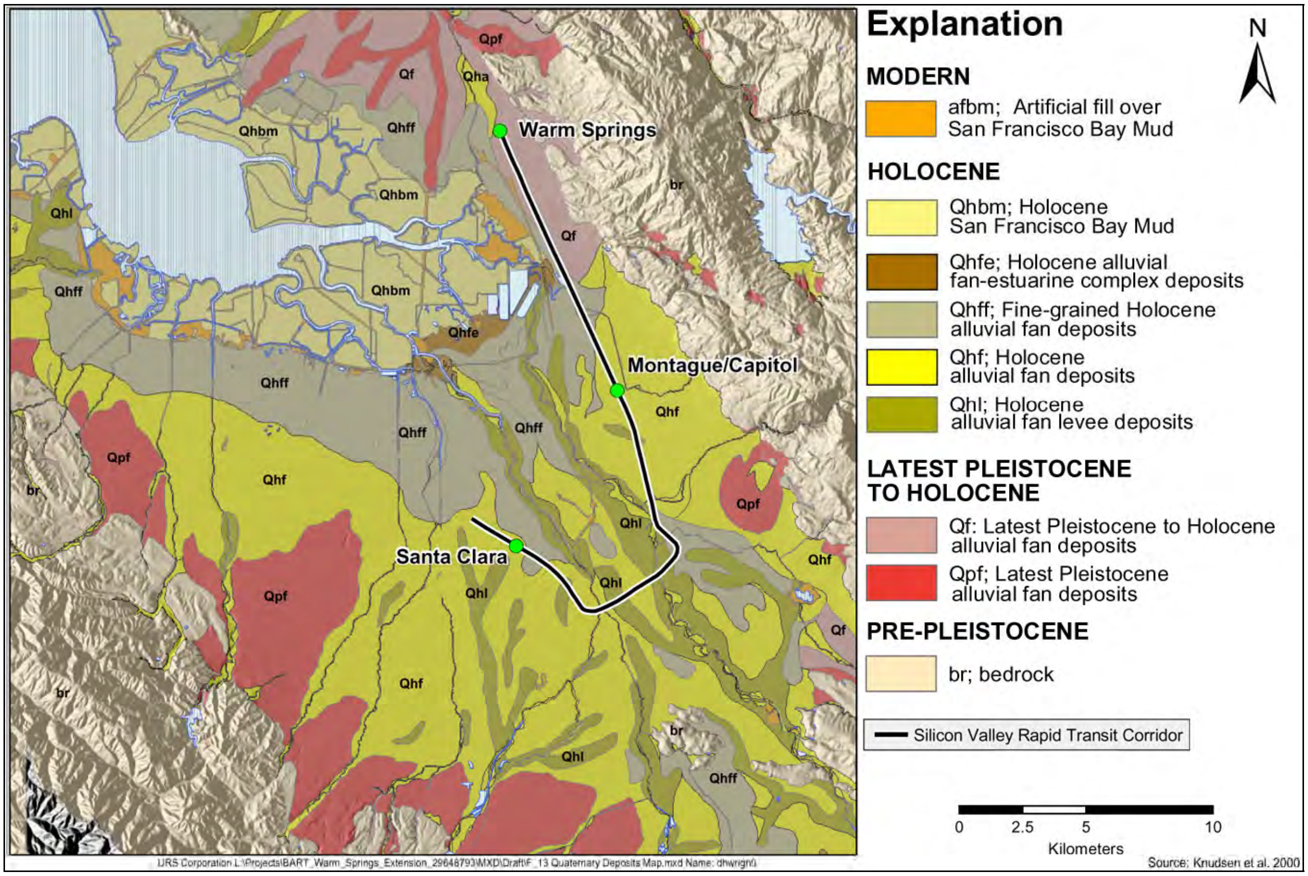


Figure 2-1. Geology of the Santa Clara Valley [from URS, 2003].

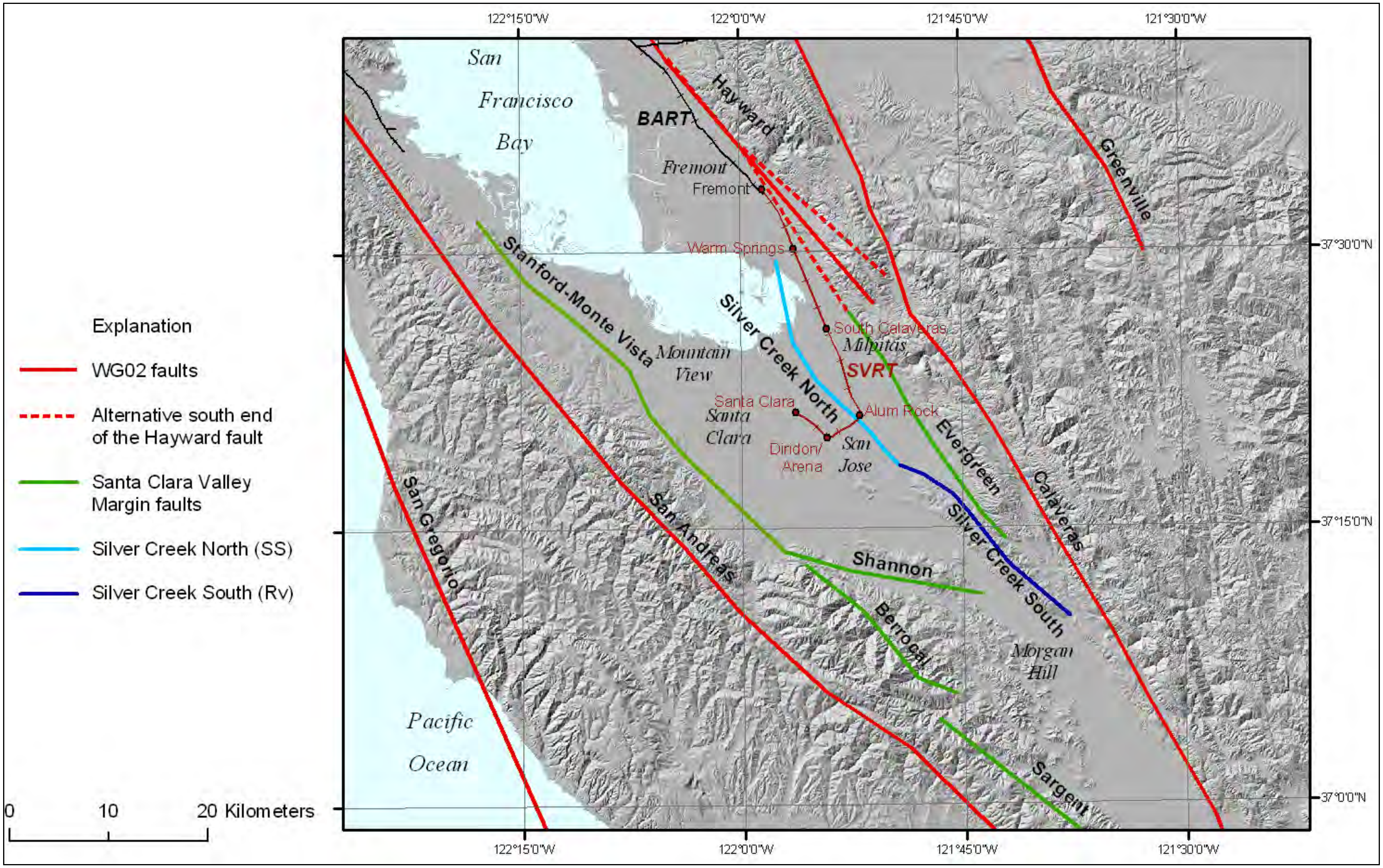


Figure 2-2. Detail Map of Fault Sources in the Santa Clara Valley.

This page intentionally left blank.

3.0 Seismic Setting

3.1 General Seismic Setting

The Tunnel Segment of the SVRT Project lies in a highly active seismic region. Based on the Unified Building Code (1997) seismic zonation map, the project site is situated in Seismic Zone 4 with three major active fault sources dominating the seismic ground motion shaking hazard: the San Andreas Fault, the Hayward-Rodgers Creek Fault, and the Calaveras Fault (Figure 3-1). Various additional fault sources exist in the general region and are described in more detail in "Report on Seismic Design Ground Motions (HMM/Bechtel, 2005b)" referred to as "seismic report" from hereon. The seismic report describes the three major active fault sources as follows:

3.1.1 San Andreas Fault

The northern San Andreas Fault, as considered by the Working Group on California Earthquake Probabilities (WGCEP) in 2003, referred to as WG02, extends south from offshore of Shelter Cove to San Juan Bautista with a length of approximately 470 km. The WG02 divides the northern San Andreas Fault into four major segments based on paleoseismic information and historical seismicity data. The four segments are, from north to south: the Offshore (SAO), North Coast (SAN), Peninsula (SAP), and Santa Cruz Mountains (SAS) (Figure 3-1). The 1906 M 7.9 earthquake ruptured all four segments and is the longest rupture considered by the WG02. The WG02 study uses nine rupture scenarios plus a floating earthquake. Because the paleoseismic data suggest that most previous ruptures appear to be similar to the 1906 rupture, most of the slip is accommodated by ruptures that extend along all four segments.

3.1.2 Hayward-Rodgers Creek Fault

The Hayward-Rodgers Creek Fault extends over a length of about 140 km from near Healdsburg, south to Fremont. The WG02 identifies three segments, the Rodgers Creek Fault (RC), and the Hayward Fault North (HN) and Hayward Fault South (HS) segments (Figure 3-1). Most or the entire southern segment apparently ruptured during the October 21, 1868, M 6.81 Hayward earthquake. The rupture extended from the Warm Springs area of Fremont northward to the Montclair area of Oakland, or slightly further north. The WG02 used six rupture scenarios plus a floating earthquake.

3.1.3 Calaveras Fault

The approximately 123-km long Calaveras Fault extends from south of Hollister to near Danville in Contra Costa County. The fault, which is classified as active, has been associated with historical earthquakes of M 5.6 (1861), M 5.6 (1866), M 6.2 (1897), M 5.8 (1899), M 6.6 (1911), M 5.8 (1979), M 6.2 (1984), and ML 5.1 (1988).

The WG02 segmentation model identifies three segments on the Calaveras Fault. These are a northern segment (CN) extending from Calaveras Reservoir north to Danville, a central segment (CC) in the Morgan Hill-Gilroy area, and a southern segment (CS) near Hollister (Figure 3-1). The northern segment displays clear paleoseismic evidence of past large earthquakes. There is a high degree of uncertainty as to whether the central and southern segments can produce large events or fail predominantly with moderate earthquakes and creep (Oppenheimer et al., 1990).

The WG02 characterization of the fault included six rupture scenarios plus a floating earthquake scenario.

3.2 Fault Rupture Displacement

The only known possible occurrence of a fault intersection with the tunnel alignment occurs at the northern reach of the Silver Creek Fault. A thorough study concluded that the potential for fault offset through the alignment along the Silver Creek North fault is negligible (Geomatrix Consultants, 2004). This result is consistent with the assessments of the California Geological Survey, California Department of Transportation, Santa Clara County, and the City of San Jose, none of who consider the Silver Creek North Fault, as defined in this report, to be a rupture hazard.

3.3 Soil Profile Type

Based on guidelines provided by the NEHRP Provisions (BSSC, 1991b), the SVRT Tunnel Segment alignment falls under Soil Profile Type 'D', described as a deep, stiff soil with an average shear-wave velocity within the upper 30 meters between 180 m/s (meters per second) and 360 m/s, typical undrained shear strengths between 1 ksf (kips per square foot) to 2 ksf, and Standard Penetration Test blow counts, N, between 15 and 50.

3.4 Seismic Design Criteria

The SVRT Project Seismic Design Criteria for ground motions state that the higher ground motions from a site-specific 10% in 50-year probabilistic analysis ground motion or the median deterministic ground motions from the San Andreas, Hayward and Calaveras Faults maximum magnitude events shall be used in the design of bridges and revenue structures.

Revenue structures are those whose structural integrity is necessary for continued operation of trains, and include aerial guideways, passenger stations, tunnels, portals, cut-and-cover subway structures, ventilation structures, and earth retaining structures.

The design of the temporary excavation support structures of the SVRT Tunnel Segment, erected for the construction of permanent structures, shall use a reduced design ground motion level based on the 10% in 10 year probabilistic ground motion with peak horizontal ground acceleration not less than 0.2g.

The maximum magnitude events shall be as follows:

San Andreas	- magnitude 8.0
Hayward	- magnitude 7.25
Calaveras	- magnitude 7.0

Two horizontal (fault normal [FN] and fault parallel [FP]) and vertical response spectra shall be developed.

Adopting NEHRP soil type 'D' based on regional subsurface conditions for SVRT project-wide use, baseline probabilistic and deterministic ground motion estimates shall be made for the free-field ground surface. Modifications of this baseline estimate may be made as appropriate either to account for detailed knowledge of site-specific foundation conditions indicating departure from NEHRP soil type 'D' conditions at the ground surface, or to account for differences between surface motions and the depth of subsurface facilities.

3.5 Seismic Design Ground Motions

The SVRT Project design ground motions for the temporary and permanent revenue structures are presented in the seismic report (HMM/Bechtel, 2005b). In this report, the ground motions are subdivided into three spectra according to geography. The "North" and "Central" spectra are applicable to the Line Segment of the SVRT Project. The "South" spectra presented in the report are applicable to NEHRP soil type 'D' ground motions for the Tunnel Segment of the SVRT Project. In addition to the spectra presented in the seismic report, NEHRP soil type 'C' spectra were also generated to account for differences between motions near the surface and at depth (HMM/Bechtel, 2005c).

This page intentionally left blank.

4.0 Hydrologic Setting

4.1 Climate

The climate in San Jose is characterized by warm, dry summers dominated by sea breezes, which are caused by the temperature difference between the ocean and the inland valleys. The winters are moderately wet, with large rainfalls usually associated with storms from the Pacific Northwest. Table 4-1 shows the mean monthly temperatures based on data obtained from the California Department of Water Resources (DWR) for 130 years of records from 1873 to 2004 (California DWR, 2005). The temperature data were recorded at Station E60 7821 000 located near the existing Civic Center at Hedding and 1st Streets in San Jose.

Table 4-1. Mean Monthly Temperatures in San Jose.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Avg	49.0	52.1	54.8	57.7	61.4	65.8	68.1	67.8	66.7	61.9	54.8	49.5
Max	57.7	59.0	61.5	63.9	70.6	76.1	73.6	74.4	72.9	67.2	60.8	57.9
Min	40.4	45.3	48.5	50.9	50.1	59.7	63.8	62.4	60.9	55.8	48.5	43.9

DWR Data for the period 1873-2004 in degrees Fahrenheit

Table 4-2 shows the maximum and minimum daily temperatures for a given month as obtained from the DWR data for a period from about 1878 through 2002. This gives a range of extreme temperatures for downtown San Jose on a monthly basis.

Table 4-2. Maximum and Minimum Daily Temperatures for a Given Month in San Jose.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Max	79	82	88	95	104	107	108	105	106	101	85	79
Min	18	24	26	29	32	35	41	42	37	31	21	19

DWR data for the period 1878-2002 in degrees Fahrenheit

4.2 Precipitation

Precipitation over the drainage basins for the streams crossing the proposed SVRT alignment for the Tunnel Segment is highly variable. The average annual precipitation in downtown San Jose is approximately 14 inches, but may be as high as 44 inches in the Santa Clara Mountains, which contribute to runoff in the Guadalupe River. The rainfall distribution for the Guadalupe River watershed is highly variable over the basins contributing to flooding for the SVRT Project.

There are nearly 100 years of records from precipitation gages in Los Gatos, (7.5 miles southwest of downtown San Jose), San Jose, and Santa Clara University. Data obtained at the San Jose Civic Center (Hedding and 1st Streets in San Jose) from 1874 through mid 2004 show the average annual precipitation in downtown San Jose is about 14 inches (California DWR, 2005). The mean monthly precipitation is distributed as shown in Table 4-3. Approximately 81% of the total annual rainfall occurs between November and March, with 94% occurring between October and April.

Table 4-3. Monthly Average Precipitation in Downtown San Jose.

	Annual	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
Percent of Average	100%	4.9%	10.7%	17.0%	20.0%	17.5%	16.2%	7.7%	3.2%	0.7%	0.1%	0.3%	1.6%
Average in inches	14.32	0.70	1.53	2.44	2.86	2.51	2.32	1.10	0.46	0.10	0.02	0.05	0.23

Percent of average annual precipitation in inches, period of record 1874-2004

4.3 Intensity-Duration Frequency (IDF) Curves for SVRT Facilities Design and Local Flooding for Construction in the Tunnel Segment

4.3.1 Sources of Data

Data were obtained from the California DWR for Station E60 7821 000 located near the existing Civic Center in San Jose. Since this station has a precipitation record from 1874 to 2004, it is clearly representative of rainfall in downtown San Jose. These data are shown in Table 4-4 and plotted in Figure 4-1.

Table 4-4. Intensity-duration Frequency Data for Downtown San Jose Based on Readings from Station E60 7821 00.

Return Period in Years	Intensities in inches/hr							
	5 min	10 min	15 min	30 min	1 Hr	2 Hr	3 Hr	6 Hr
2	1.379	1.002	0.840	0.564	0.393	0.275	0.215	0.152
5	1.938	1.409	1.181	0.793	0.552	0.387	0.303	0.213
10	2.312	1.681	1.409	0.947	0.659	0.462	0.361	0.254
25	2.781	2.022	1.695	1.138	0.793	0.555	0.434	0.306
50	3.124	2.272	1.904	1.279	0.891	0.624	0.488	0.343
100	3.462	2.517	2.110	1.417	0.987	0.691	0.541	0.381

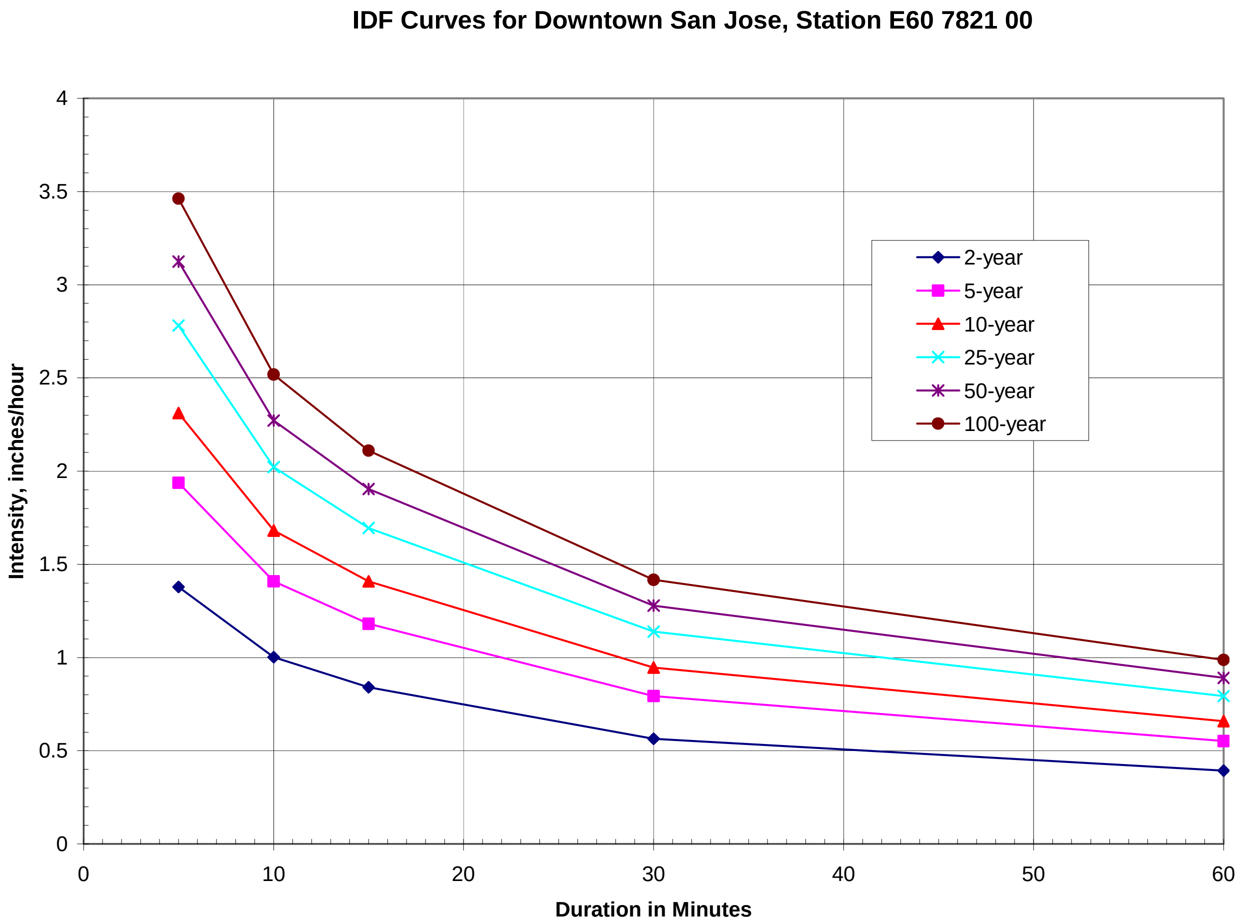


Figure 4-1. Intensity-Duration Frequency Data for Downtown San Jose using DWR Period of Record 1874-2004.

This page intentionally left blank.

5.0 Hydrogeologic Setting

5.1 Hydrostratigraphy

An understanding of the hydrostratigraphy in the Santa Clara Valley is important in the evaluation of water data. The conceptual model of hydrostratigraphy along the alignment, as defined in HMM/Bechtel (2005d), consists of the following layers, defined from ground surface downward:

- Confining Layer. Composed of clays and silts, with some ancient channels of sand. The pervious channels are most common near Guadalupe River and Las Gatos Creek. The layer thickness varies from 50 to 80 ft at the station locations.
- Upper Aquifer. Composed of silty sand, sand, gravelly sand, and sandy gravel. It includes intersecting and coalescing channels of varying thickness and differing permeability. The top of this unit varies from 50 to 80 ft below ground surface.
- Major Aquitard. Primarily clays and silts, but can include deposits of sand and silty sand. The top of this unit may be approximately 80 to 150 ft below ground surface (about 110 to 150 ft at the station locations).
- Lower Aquifer. Zone of major groundwater withdrawals, composed of sand and gravel zones with intervening clay and silt layers. The top of this unit may be about 200 to 250 ft below ground surface. The thickness may be approximately 800 ft or more.

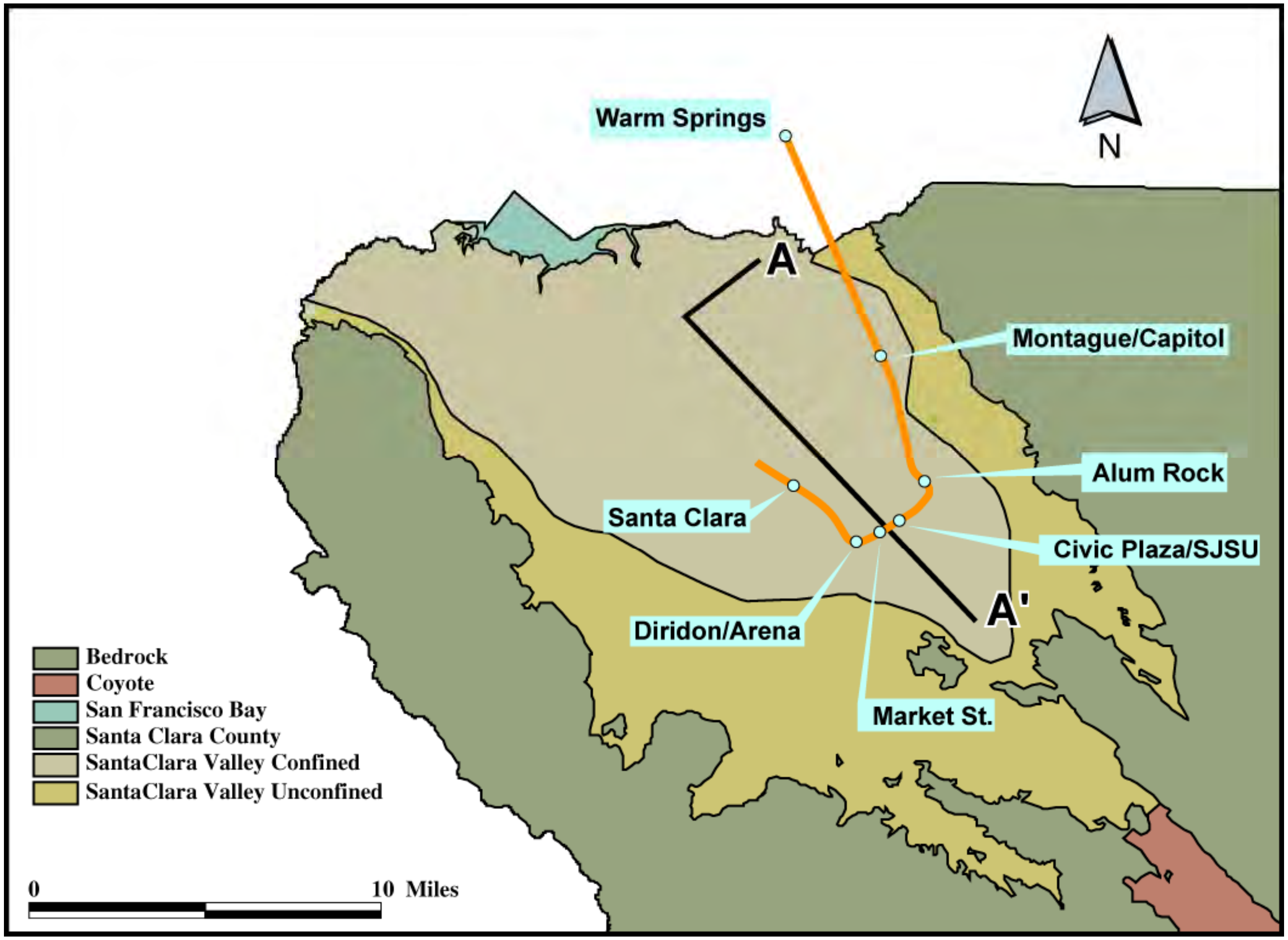
Profiles of the stratigraphy along the alignment up to a 200-foot depth, based on boring logs and CPTs from this field investigation, are presented in Figures 8-1 through 8-50. Deeper stratigraphy is discussed in Section 5.2.

5.2 Major Aquitard

Figure 5-1 shows the regional extent of the Major Aquitard (referred to as "Santa Clara Valley Confined" on the figure), and the location of a cross-section through downtown San Jose. Figure 5-2 shows a generalized cross-section and identifies the significant hydrogeologic units as a "Clay cap" (shallow confining layer), "Upper Aquifer Zone," "Major Aquitard", and "Lower Aquifer Zone."

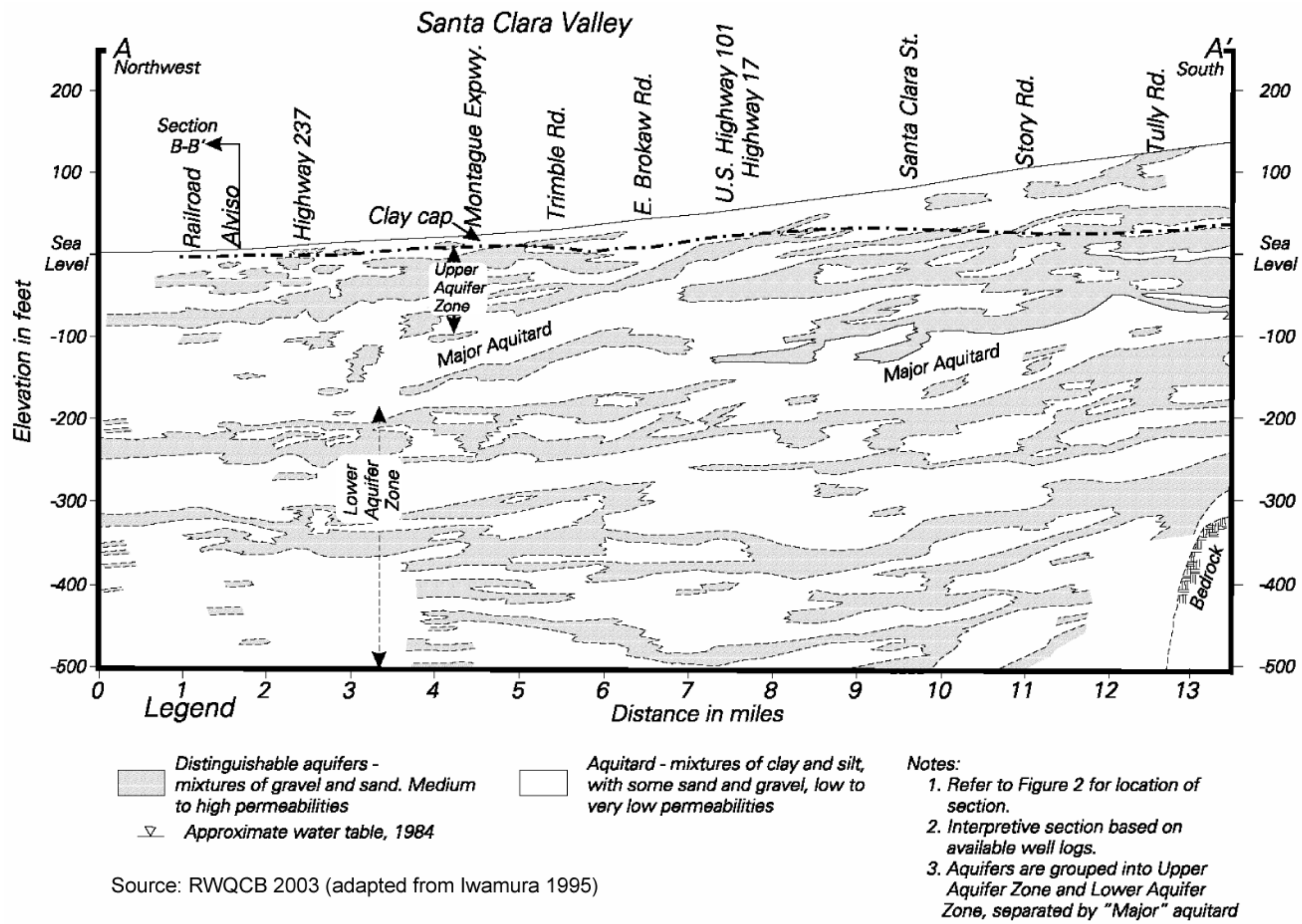
The aquifer zones consist of mixtures of gravel and sand with layers of clay and silt. The confining layer and aquitard consist of clay and silt mixtures, with some sand and gravel. It should be emphasized that the Major Aquitard is not a single layer of fine-grained soil, and that the term "Major Aquitard" may be misrepresentative (USGS, 2004a). The zone is probably better characterized, as stated above, as a mixture of clay and silt deposits with some sand and gravel channels.

Figure 5-1. Extent of Major Aquitard in Santa Clara Valley Basin.



Source: SCVWD (2002).
Note: "Santa Clara Valley Confined" unit is the extent of Major Aquitard.

Figure 5-2. Generalized Regional Cross Section.

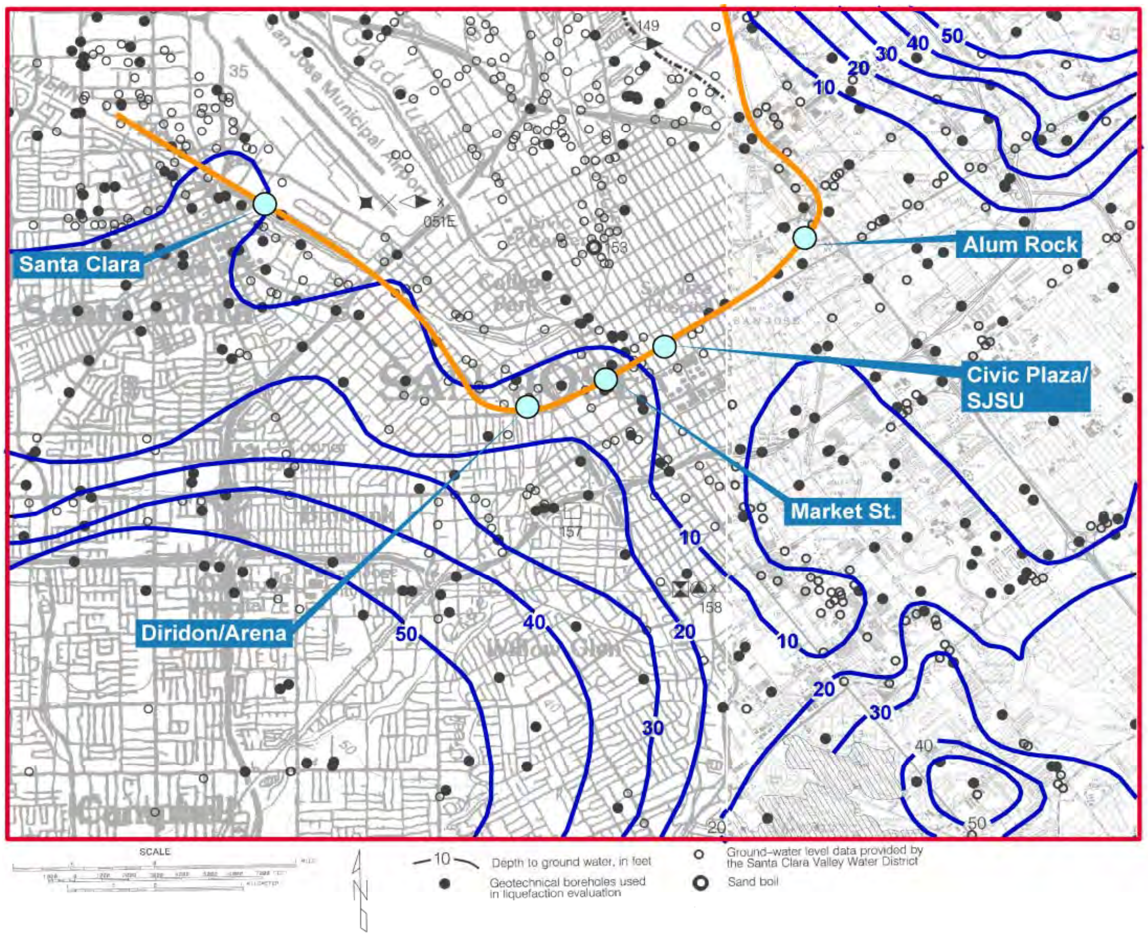


Note: SVRT alignment intersects this cross section at Santa Clara St.

5.3 Regional Water Levels

Historic and recent water level contour maps were reviewed to determine water level trends. Water levels along Santa Clara Street were approximately at elevation (El.) 90 ft above mean sea level (MSL) in 1914-1916. By 1932, water levels in the Lower Aquifer had dropped to El. 0 to 10 ft along Santa Clara Street. In Spring 2001, water levels in the Lower Aquifer were about El. 30 to 35 ft along Santa Clara Street.

“Historically high” water table depth contours are shown in Figure 5-3. Along Santa Clara Street, water table depths were about 5 to 15 ft bgs, typically less than 10 ft bgs, but somewhat greater at the location of the proposed downtown station. Based primarily on the SCVWD data for Well 105, and extrapolation of data for Wells 133 and 140, the maximum historical water levels were estimated in the vicinity of each station as shown in Figure 5-4 and Table 5-1.



Sources: (1) CDMG 2000, Open-File Report 2000-010 (Plate 1.2).
(2) CDMG 2002, Seismic Hazard Zone Report 058 (Plate 1.2).

Notes: (1) Contours are based on "first-encountered" water in geotechnical and geoenvironmental boreholes (borehole water levels may not be as reliable as wells or piezometers, particularly in fine-grained soil and indicate a reading on one date).
(2) Figure shows alignment and stations for SVRT Conceptual Design.

"Historically High" Water Table Depth 1967-1997

Job No.: 24965
May 2005

Hydrogeology Report
SVRT -BART to San Jose Project
San Jose, California

Figure 5-3. Water Level Contours – “Historically High” Water Table Depth (1967-1997).

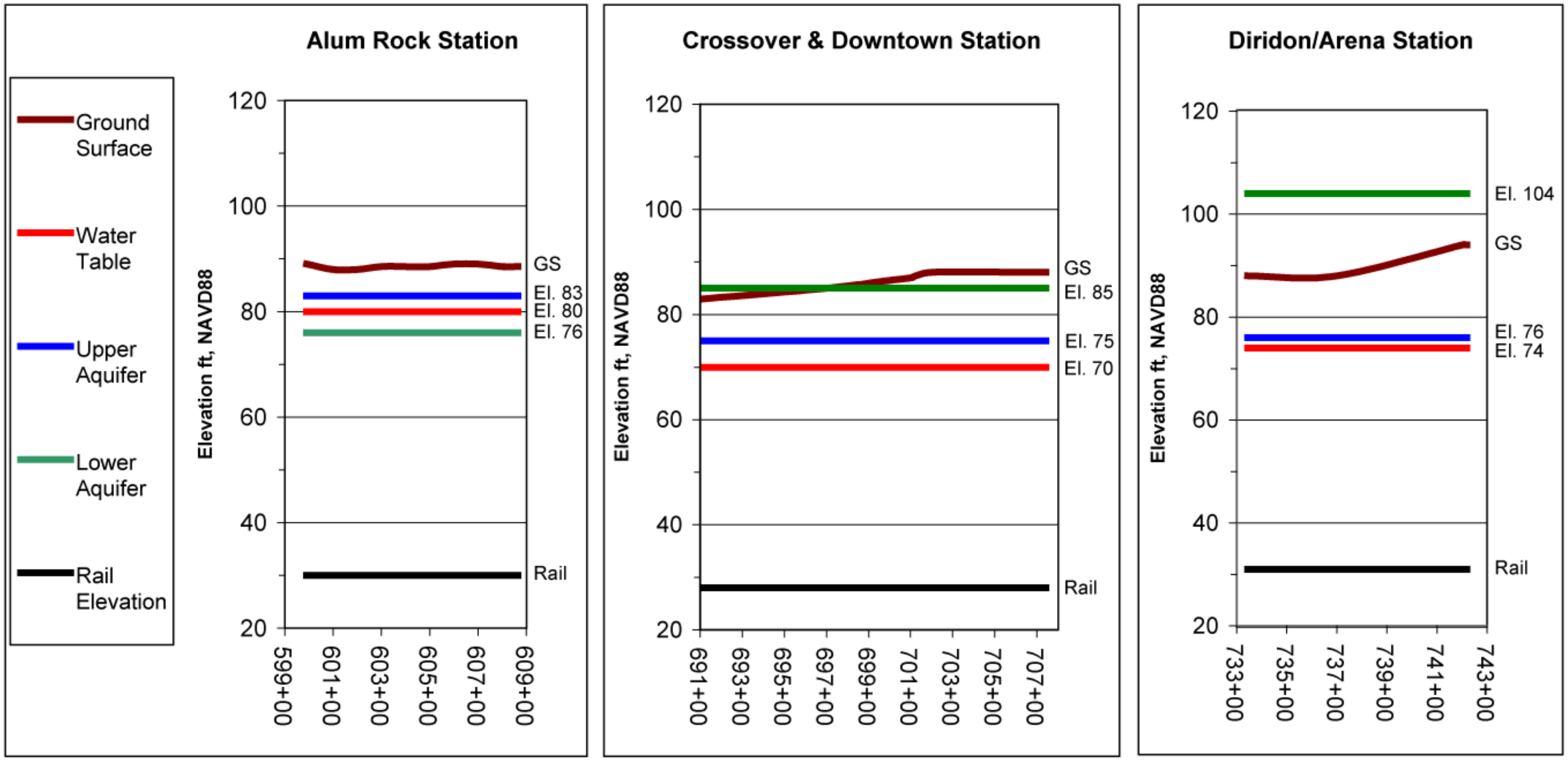


Figure 5-4. Maximum Historical Water Levels at Station Locations.

Table 5-1. Water Level Estimates for Station Locations.

Historical Maximum Water Levels			
	Water Level Elevation (ft, NAVD88)		
Hydrostratigraphic Zone	Alum Rock Station	Downtown Station and Crossover	Diridon/Arena Station
Ground Surface (NAVD88)	El. 87-88	El. 81-87	El. 87-92
Water Table	El. 80 (7 ft bgs)	El. 70 (11 ft bgs)	El. 74 (13 ft bgs)
Upper Aquifer	El. 83 (5 ft bgs)	El. 75 (6 ft bgs)	El. 76 (11 ft bgs)
Major Aquitard (interpolated)	El. 80 (8 ft bgs)	El. 80 (1 ft bgs)	El. 90 (3 ft ags)
Lower Aquifer	El. 76 (11 ft bgs)*	El. 85 (4 ft ags)*	El. 104 (17 ft ags)

5.4 Land Subsidence

Land subsidence in the San Jose area is illustrated by monthly water elevation data measurements from San Jose's "Index Well" that penetrates into the Lower Aquifer. The "Index Well" is a well that was designated by the Santa Clara Valley Water District as representative of general ground-water elevation trends within the ground-water basin. The well location is shown in Figure 5-6, adjacent to the tunnel alignment, between Downtown San Jose and Diridon/Arena Stations.

The hydrograph for this well appears in numerous publications on the groundwater of Santa Clara Valley (Iwamura, 1995; USGS, 1999; SCVWD, 2002). It dramatically shows the long-term subsidence and water level decline resulting from overdraft of the Lower Aquifer, during a period of below-normal precipitation, and subsequent periods of artificial recharge, water importation, and above-normal precipitation.

As shown in Figure 5-5, the "maximum annual depth" (lowest of each year's monthly readings) to groundwater decreased to an historic low of 235 ft at Santa Clara Street and Delmas Avenue in 1964. Subsidence in downtown San Jose at 1st Street and Saint James Street resulted in a ground surface change from El. 98 ft to El. 84 ft (14 ft drop) from 1910 to 1993. The water level responded to above-average annual precipitation with an increase of 30 ft in the maximum annual depth corresponding to two wet years in 1982 and 1983. A long-term rise of 195 ft from 1963 to 1993 also corresponds to above-average precipitation since 1970 with simultaneous importation of water for artificial recharge and decline in groundwater usage.

The first deliveries of imported water were received in 1965 through the State Water Project. Groundwater levels have generally increased annually since then, with the exceptions of two significant drought periods in the late 1970s and late 1980s (SCVWD, 2002).

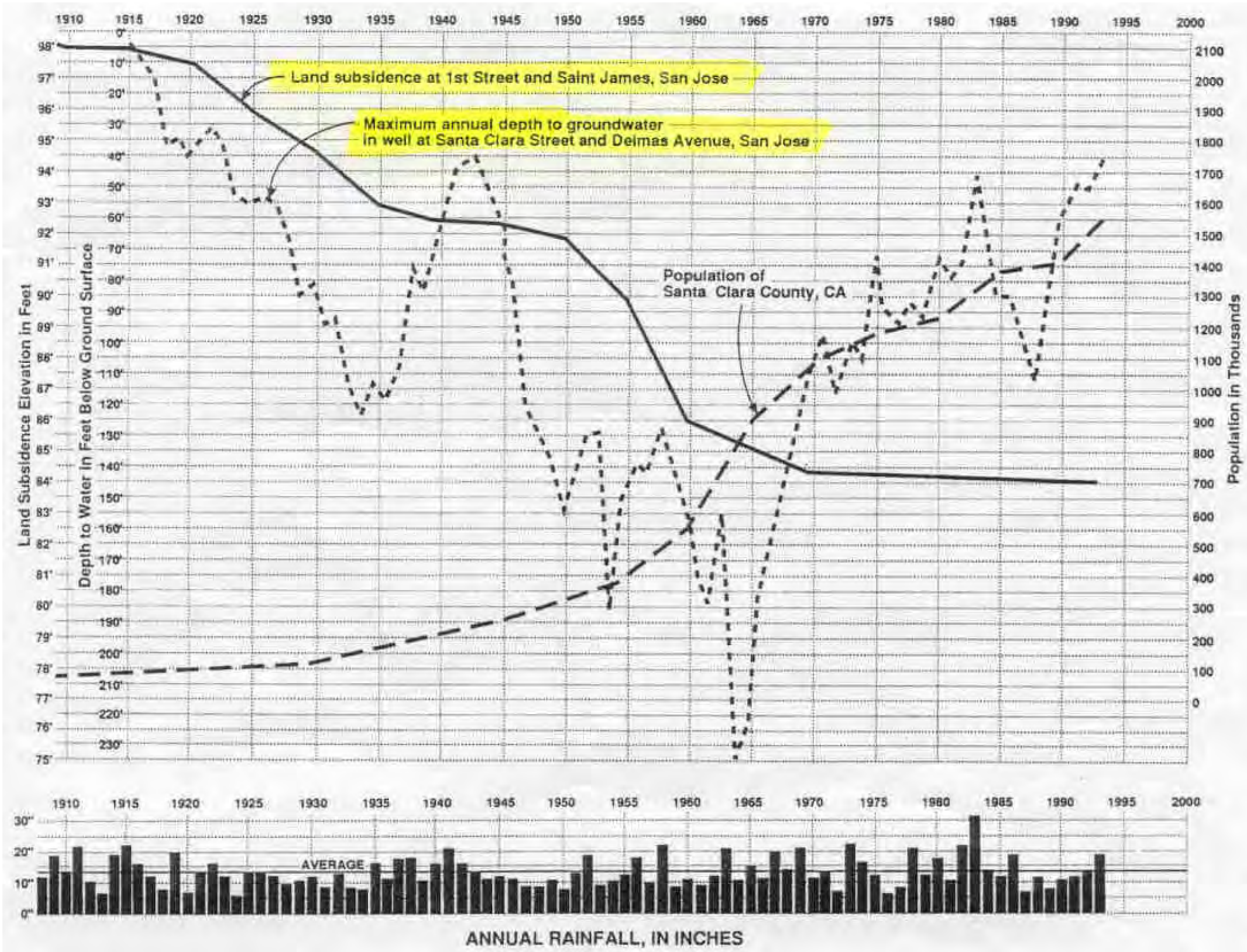


Figure 5-5. Well Hydrograph – San Jose "Index Well" (Lower Aquifer) 1915-1993 "Maximum Annual Depth" (with Subsidence and Precipitation 1908-1993).

5.5 Artesian Conditions

Artesian flow, characterized by continuously flowing water above the ground surface without the presence of pumping, occurred in the past in downtown San Jose (Clark, 1924). As stated by Iwamura (1995, p. 179), "In most recent years, pressures in the Lower Aquifer Zone have recovered to the extent that wells in the basin interior again became flowing artesian on an intermittent basis. However, water levels within the forebay and in certain large pumping areas within the basin have not recovered to their pre-overdraft levels." The tunnel alignment appears to be within the latter area; i.e., water levels have not recovered to their pre-overdraft levels.

Recovery to artesian conditions was reported to have occurred at the San Jose airport, causing groundwater buildup beneath the runways through leaking abandoned deep wells (HMM/Bechtel, 2005e). Recent data from the SCVWD compiled in Figure 5-6 indicate that several wells near the alignment, including the "Index Well", have returned to the artesian condition. Other deep wells along or near the alignment show no indication of a return to artesian conditions.

5.6 Presence of Gas and Temporary Discharges of Groundwater

During the 35% Preliminary Engineering Field Investigation, one of the wells shown in Figure 5-7 (ST-3) was observed to be releasing gas during auger drilling on February 10, 2005, based on a hissing sound heard within the hollow stem upon penetrating the Upper Aquifer.

Temporary discharges of ground water occurred at five CPT locations and one borehole during the geotechnical investigation for preliminary engineering. The locations of these occurrences are shown in Figure 5-7. As described below, the discharges ranged from 3 to 65 minutes in duration:

- CPT-25 (Nov. 3, 2004, between 19th and 20th Streets) – 20 minutes, estimated 20 gallons (1 gal/min average), with sand, silt and gravel discharges.
- CPT-30 (Dec. 29, 2004, on S. 15th Street) – 65 minutes, estimated 75 gal/min, with sand, silt, and gravel discharges up to 2.5 inches in diameter; one hour after flow had stopped, flow returned for 5 minutes.
- CPT-94 (Jan. 18, 2005, between 6th and 7th Streets) – 15 minutes, with silt discharge.
- CPT-124 (Jan. 26, 2005, at N. 28th Street and Five Wounds Lane) – 8 minutes, estimated 20 to 80 gallons (2.5 to 10 gal/min average), with sand, silt, and gravel discharges.
- BH-19 (Jan. 31, 2005, between 10th and 11th Streets) – 10 to 15 minutes, rising level in mud tub, encountered at depth of 65 to 70 ft.
- CPT-95 (Apr. 20, 2005, at 5th Street) – 3 minutes, with sand, silt, and gravel discharges.

The discharges occurred when all of the CPTs and BH-19 were in the process of penetrating into the confined Upper Aquifer. Flowing artesian conditions appear to have been absent at the time of these discharges. For the period between November 2004 and March 2005, static water levels in the Upper Aquifer, as measured at SVRT wells and piezometers, and by CPT dissipation tests, were all below ground surface (neglecting two readings that appear to be incorrect field measurements).

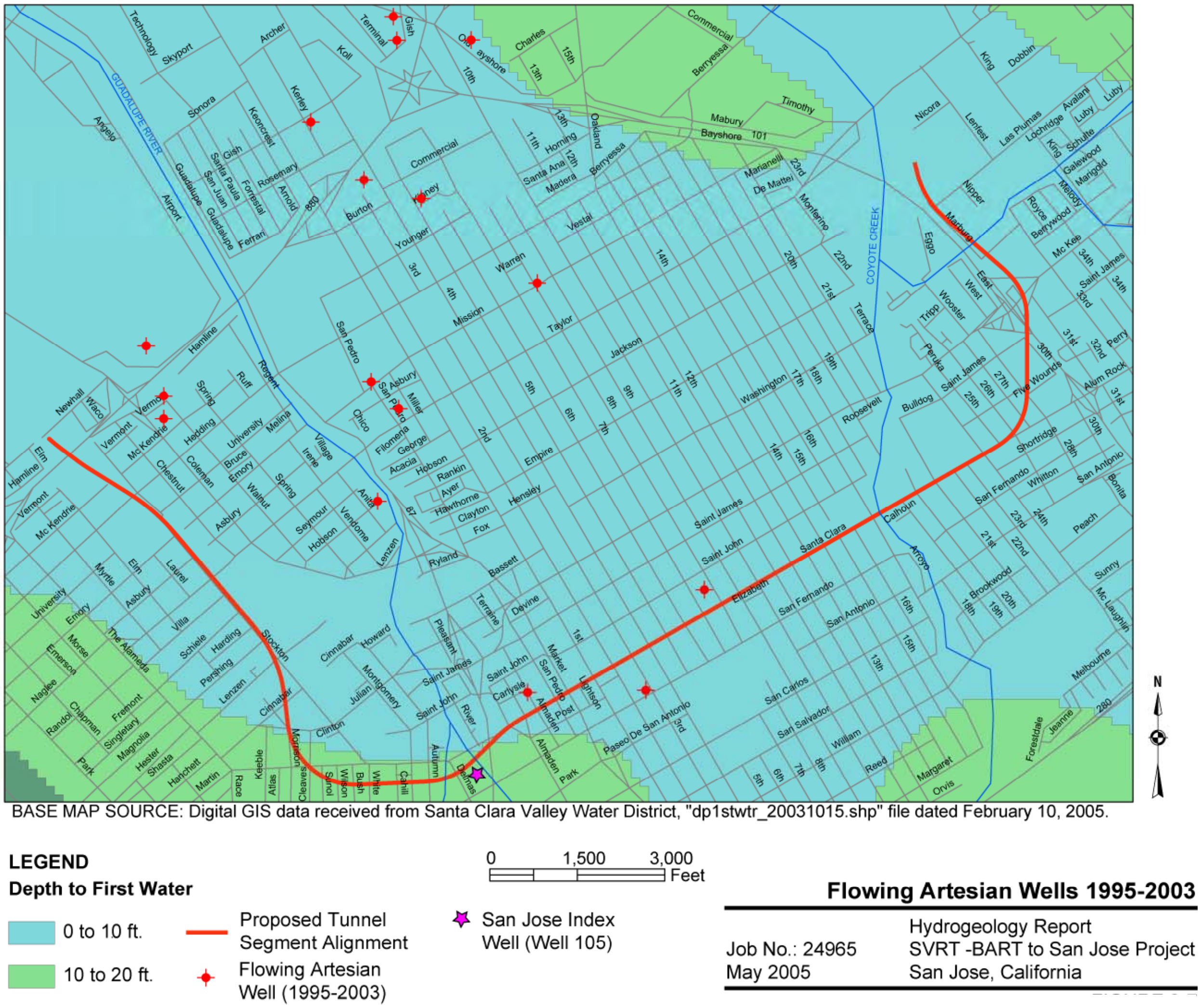


Figure 5-6. Downtown San Jose with Documented Locations of Flowing Artesian Wells [after SCVWD, 2005].

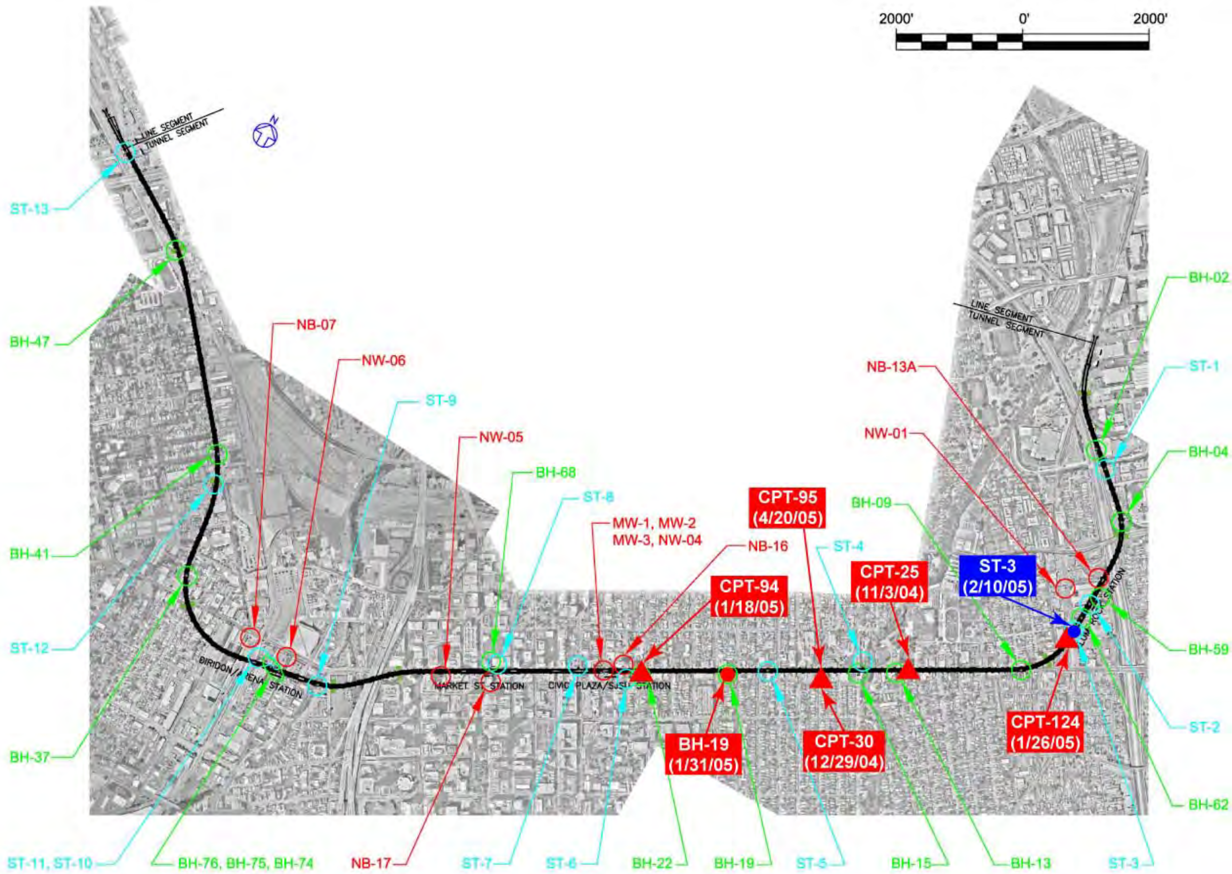


Figure 5-7. Locations of Ground Water Discharge.

6.0 Field Investigations

6.1 Introduction

The October 2003 10% conceptual engineering draft report, “Geotechnical Exploration Findings and Recommendations Report” (URS, 2003) incorporated information obtained from a 2001 field exploration for the Major Investment Study (MIS) (URS, 2001) phase of the project with URS Corporation’s 2002/2003 investigation. The investigations included a total of 21 borings, five CPTs, four vibrating wire piezometers and seven observation wells. In-situ field vane shear tests were performed at five borings. Data gathered during the 10% conceptual engineering study primarily focused on the proposed underground station locations.

Available information from the 10% Conceptual Design investigation was reviewed along with 43 relevant geotechnical reports. The review of available data was presented in a summary report (HMM/Bechtel, 2004a). Based on an evaluation of the available data, a field investigation program was developed to address data and information needed for the 35% Preliminary Engineering design.

The 35% Preliminary Engineering Field Investigation provides additional geotechnical data about the stratigraphy, groundwater, and physical and engineering characteristics of the soil along the alignment. Details of the field investigation are described in the following sections. Table 6-1 below lists the appendices where results of the field investigations are documented.

Table 6-1. Field Testing Program.

	Parikh Consultants	Fugro
Boring Logs	**Appendix 1**	
Standard Penetration Tests (SPT)	**Appendix 1**	
Field Vane Shear Tests	**Appendix 2**	
Pressuremeter Tests	**Appendix 3**	
Downhole Geophysical Logging	**Appendix 4**	
Vibrating Wire Piezometers	**Appendix 5**	
Observation Wells	**Appendix 6**	
Conventional CPTs		**Appendix 8**
Seismic CPTs		**Appendix 9**
Hydropunch Tests		**Appendix 10**
Dissipation Tests		**Appendix 11**

6.1.1 Organization of Team

Several geotechnical engineering, drilling and specialty testing firms were subcontracted as part of the field investigation team. Subcontractors included Fugro, Parikh Consultants, Pitcher Drilling, Hughes Insitu Engineering, GEOVision Geophysical Services, URS Corporation, Geomatrix Consultants and ABE Engineering.

Fugro's field investigation scope focused on the CPT explorations, which included seismic cone testing and pore pressure dissipation testing. Fugro also performed one hydropunch and subsequent in-situ water sampling. Pitcher Drilling provided the drill rigs and drill crews necessary to complete all geotechnical borings, soil sampling, and observation well installation.

Parikh Consultants provided technical oversight for Pitcher Drilling for all geotechnical borings. Field engineers from Parikh Consultants performed all field logging of borings. Parikh Consultants also employed Hughes Insitu for pressuremeter testing, and GEOVision Geophysical Services for downhole suspension logging. Staff from HMM/Bechtel and URS Corporation monitored all subcontractor field activities. Geomatrix Consultants performed well and piezometer monitoring on a monthly basis for a 6-month period and obtained gas monitoring measurements (for tunnel gas classification) within several observation wells. ABE Engineering calibrated Pitcher Drilling's automatic hammer on the Fraste Multidrill XL top-drive drill rig.

Kleinfelder, although not subcontracted by HMM/Bechtel, provided laboratory testing results for the water samples obtained during the CPT hydropunch. Kleinfelder, under subcontract to EarthTech for the permanent underground Stations preliminary design work, reviewed the scope of the field investigation and observed a small portion of the field exploration activities at underground Station locations.

6.1.2 Field Manual

A field manual (HMM/Bechtel, 2004c) was developed to serve as a reference tool for personnel overseeing on-site drilling operations for the Tunnel Segment of the SVRT Project. The purpose of the manual was to provide a guide for the on-site drilling operations, and included the following information:

- Project description
- Pre-drilling requirements for all exploration locations
- Field guidelines for drilling and sampling activities, personal behavior, and safety
- ASTM Standards
- Tunnel Segment specific guides for classification, logging, field testing and in-situ testing
- Location maps, forms, permits, and standards applicable to the geotechnical investigation.

The field manual was also a useful tool for the quality control program. For the field investigation, the manual provided guidance for the following activities:

- Verification that all safety and quality requirements per the Work Plan and Specifications were met
- Verification that all City of San Jose and Santa Clara Valley Transportation Authority (VTA) requirements were followed
- Compliance with drilling methods and procedures
- Compliance with logging and classification of soil types

6.1.3 Project Restrictions

During the field investigation, various restrictions were imposed by local agencies, private property owners, neighborhood organizations, and commercial and residential tenants, limiting the available locations and work schedule.

Encroachment permits were required by several public and private agencies to perform borings and CPTs along different portions of the alignment. These agencies included the City of San Jose (CSJ), California Department of Transportation (Caltrans), San Jose Water Company, Union Pacific Railroad (UPRR) and Santa Clara Valley Water District (SCVWD). CSJ and Caltrans also required traffic control permits, and SCVWD required exploration and well permits. These permitting requirements restricted the available locations, dates, and work hours of field operations.

Design revisions that developed during the field program were incorporated into the investigation as needed and when possible. Some of the major design revisions included the following changes:

- Tunnel alignment revised near Highway 101/Alum Rock Station area (borings and CPTs moved from station 587+00 to 618+00)
- Tunnel alignment at Coyote Creek deepened to avoid piles at Coyote Creek Bridge (borings and CPTs deepened and sampling intervals adjusted);
- Tunnel alignment at Guadalupe River deepened to avoid obstructions with the river retaining wall near Route 87 (borings and CPTs deepened, and sampling intervals adjusted)
- Tunnel alignment between Diridon/Arena Station and West Portal deepened (several borings deepened and one 200-foot deep boring added);
- Proposed Civic Plaza/SJSU and Market Street Stations combined and renamed as Downtown San Jose Station.

This report reflects the April 2005 tunnel alignment.

6.2 Boring Program

The boring program commenced on October 9, 2004. A total of 76 borings were completed as part of the field investigation program (Figure 6-1). Seventy-five borings were drilled between October 9, 2004 and March 5, 2005. An additional boring was drilled on April 18, 2005, where permitting requirements had delayed access to the site.

6.2.1 Overview

Of the 76 borings, 24 were drilled at the three proposed underground stations, 50 along the tunnel alignment, and two (2) at the portals. The boring frequencies, depths, sampling methods, sampling intervals and in-situ testing intervals were chosen based on design needs.

Borings drilled at the proposed station locations were spaced approximately 150 ft apart. Borings completed at the proposed Alum Rock Station and Diridon/Arena Station were drilled to depths of at least 150 ft, with an average depth of 159.5 ft. Borings drilled for the proposed Crossover and Downtown San Jose Station were drilled to depths of at least 130 ft, with an average depth of 151.0 ft. An average depth of at least 150 ft was chosen so that subsurface information could be obtained for design of seepage control measures. One boring for Alum Rock Station and one boring for Crossover and Downtown San Jose Station were drilled to depths of 200.5 ft and 216.0 feet, respectively, for downhole suspension logging by GEOVision Geophysical Services. One boring was drilled to a maximum depth of 200.5 feet at Diridon/Arena Station.

Continuous sampling was performed throughout the upper 70 ft at selected station borings: BH-58 at Alum Rock Station; BH-24 and BH-70 at the Crossover and Downtown San Jose Station; and BH-74 at Diridon/Arena Station. In-situ pressuremeter testing for the stations was performed in one borehole at Alum Rock Station, four borings at the Crossover and Downtown San Jose Station, and two borings at Diridon/Arena Station.

One boring was drilled for each of the two portal locations. Boring BH-56 was drilled to a depth of 42.5 ft at the proposed East Portal location. Boring BH-78 was drilled to a depth of 80.8 ft at the proposed West Portal location.

Borings drilled along the proposed tunnel alignment (tunnel borings) were spaced at about 300-foot intervals and were generally planned to depths of at least 20 ft below the tunnel invert, based on the baseline tunnel alignment at the time of drilling. For tunnel borings, field vane shear tests/attempts were performed near the tunnel crown, center and invert. Pressuremeter tests were also performed near the tunnel crown, center and invert. Continuous sampling in the “tunnel zone” (from 20 ft above the proposed tunnel crown to 20 ft below invert) was done at nine borings along the tunnel alignment.

At a potential future ventilation structure location, BH-79 was drilled 216.0 ft deep to investigate the soil conditions and for downhole suspension logging.

6.2.2 Drill Rig and Hammer Types

The drill rigs used for the project consisted of two types of truck-mounted equipment, a Fraste Multidrill XL drill rig and a Failing 1500 drill rig, and a track-mounted Fraste Multidrill XL drill rig. The Failing 1500 is one of several typical rig types commonly used for rotary wash drilling. Fraste Multidrill XL drill rigs are top-drive drill rigs that allow for “self-boring” pressuremeter testing (discussed in Section 6.4). The track-mounted rig was specifically needed for BH-04, due to sloping terrain at Route 101/McKee Road interchange

(Caltrans right-of-way). Failing 1500 and Fraste Multidrill XL drill rigs utilized Rope and Cathead systems, and Automatic Trip Hammer systems, respectively, to advance split-spoon and Modified California samplers.

The drill rigs were equipped with a standard 140-lb hammer to drive thick-walled samplers. ABE Engineering calibrated the efficiency of the automatic hammer (Fraste Rig) at BH-25 and BH-65 (Section 6.2.11). AW-size drill rods were used during drilling and sampling operations.

6.2.3 Sampling Methods and Equipment

6.2.3.1 Sampler Types

Three types of soil samplers were used: driven thick-walled samplers (split-spoon and Modified California), pushed thin-walled samplers (Shelby Tube) and rotated thin-walled samplers (Pitcher Barrel). Bag samples were retrieved at a few selected locations and from split-spoon samplers. Modified California (MC) samples were placed in plastic tubes.

A split-spoon sampler and a Modified California sampler were used to obtain standard penetration resistance data of granular materials such as sandy or gravelly soils. The 140-pound drive hammer used for sample collection and casing installation and removal was in conformance with ASTM D1586, Standard Method for Penetration Test and Split-Barrel Sampling of Soils. The split-spoon sampler used was in conformance with ASTM D1587, and had an outside diameter of 2 inches and an inside diameter of 1-3/8 inches. The Modified California sampler used was in general conformance with ASTM D3550, Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils. The MC sampler had an outside diameter of 3 inches and an inside diameter of 2.5 inches. The MC sampler was also used to obtain disturbed samples of sand and gravel soils. The MC sampler was able to retrieve larger gravel particles (up to 2.5 inches) that could not be obtained using the split-spoon sampler.

Soft to stiff clayey soils were generally sampled using a thin-walled Shelby Tube sampler in conformance with ASTM D1587. The Shelby Tube sampler consists of a 3-inch diameter, 36-inch long mild steel thin-walled tube that is hydraulically pushed by the drill rig. The sampler was used to obtain relatively undisturbed samples of clays and silts (fine-grained soils). For each push, the standard length of advancement was 30 inches.

Very dense cemented soils and stiff to very stiff clays were generally sampled using a Pitcher Barrel sampler in conformance with ASTM D1587. Pitcher Barrel samplers consist of double-tube core-barrels; the inner barrel, which consists of a Shelby tube, is affixed to a spring-loaded sampler head that extends or retracts relative to the cutting bit on the outer barrel with changes in soil stiffness.

The magnitude and change in hydraulic pressure during Shelby Tube and Pitcher Barrel sampler advancement were recorded on the boring logs. A change in hydraulic pressure qualitatively indicates a change of material type or consistency at each depth or location, but may not be comparable between two separate borings due to characteristics of hydraulic systems.

Pitcher Barrel sampling could not be performed in some gravelly formations. Thick drilling fluid is needed to lift the gravelly material from the bottom of the boring during the rotary wash process. The thick drilling fluid reduces the circulation within the Pitcher Barrel sampler and around the drill bit. If the drilling fluid becomes too thick and the circulation ports of the sampler plug, the cutting bit heats up, causing the cutting bit to wear out quickly or fracture.

6.2.3.2 Sampling Interval

Continuous sampling was performed in the upper 70 ft of specified station borings. Shelby Tube and Pitcher Barrel samples were typically taken in fine-grained soils at 5-foot intervals between 10 ft and 70 ft split-spoon and Modified California samples were taken in coarse-grained soils at 5-foot intervals between 10 ft and 70 ft. Below 70 ft, Pitcher Barrels in clay or split-spoon/Modified California samples in sand were typically taken at 5 or 10-foot intervals.

Sampling in tunnel borings was generally performed at 5-foot intervals and were limited to the proposed tunnel zone (20 ft above the proposed tunnel crown and 20 ft below invert). Continuous sampling was performed at nine of the tunnel boring locations within the tunnel zone.

Samples were also obtained at depths where material changes were detected for both station borings and tunnel borings. The cuttings in the drilling fluid were examined to identify changes in the soil conditions between sample locations. Material changes were also identified by the driller's observations of drill rig response (i.e. chattering of drill rig, loss of fluid, etc.).

Occasionally soil samples could not be recovered due to wet soil conditions or obstructions, such as gravel or slough in the samplers. When this occurred, the field engineer for Parikh Consultants typically directed the driller to drill out the zone where sampling had been attempted and to repeat the sampling attempt below the missing sample. Occasionally it was necessary to change sampling methods, or to use steel or plastic retainer baskets ("catchers") to retrieve samples.

6.2.4 Handheld Field Tests

In addition to visual observations of soil strength, handheld field tests using pocket penetrometer and pocket torvane were performed on the bottom of relatively undisturbed Shelby Tube and Pitcher Barrel samples. The estimated unconfined compressive strengths from pocket penetrometer tests are presented in the material description column on each boring log. Units for unconfined compressive strength are shown in tons per square foot (tsf). Though the pocket penetrometer was used to estimate the unconfined compressive strength for cohesive soils, readings from the pocket penetrometer were also converted to undrained shear strength in units of ksf. The pocket torvane was used to directly estimate the undrained shear strength for cohesive soils. Both handheld field tests were used as a guide to strength and consistency variations.

The undrained shear strength test results from handheld field tests are shown in kips per square foot (ksf) at the corresponding test depths on the boring logs presented in Appendix 1 and adjacent to the stick logs presented in Figures 8-26 through 8-50, Geotechnical Plan and Profile with Strength Parameters.

6.2.5 Groundwater Level Measurements

When feasible, groundwater levels were measured on the second or third day of drilling for each borehole. The groundwater levels are based on the assumption that the drilling fluid/mud reached equilibrium with natural groundwater level overnight. These measured groundwater levels should not be used for design. For design purposes, vibrating wire piezometers and observation wells were installed to provide groundwater level and pore-water pressure information.

6.2.6 Sample Handling

In order to obtain high-quality undisturbed samples for laboratory testing, every effort was made to minimize disturbance during handling and transportation of Shelby Tube and Pitcher Barrel samplers. Slough was typically removed from the tubes and empty spaces at the top and bottom of the sample tubes were filled with Styrofoam packaging peanuts prior to initial sealing in the field. Shelby Tubes and Pitcher Barrel samples were kept upright in wooden boxes.

Sample preservation and transportation followed ASTM D4220, Standard Practice for Preserving and Transporting Soil Samples. In general, all samples were protected from extreme temperatures and kept out of direct sunlight. Samples were carefully transported from the field to the laboratory and stored in locations where they were not exposed to extreme temperature changes and would not be disturbed.

Waxing of Shelby Tube sample tubes took place at Parikh Consultant's laboratory, within three (3) days of drilling. Waxing was performed in accordance with ASTM D4220.

6.2.7 Borehole Completion and Abandonment

Borings were generally terminated at the planned depth. A few borings, however, were terminated prematurely due to difficult drilling conditions, time constraints, permitting requirements, or access issues.

Prior to completion of each boring, the Santa Clara Valley Water District (SCVWD) was contacted for observation of grouting procedures. After the boring was completed, the borehole was grouted from the bottom up using a tremie pipe per SCVWD requirements. All Investigation Derived Waste (IDW) and any loose soil or cuttings from the drilling operation were removed from City of San Jose streets or private property and placed in 55-gallon drums. All drums containing IDW were characterized, labeled, and disposed of by Integrated Waste Management (IWM) in accordance with applicable regulatory requirements.

Pavement removed to drill borings was patched using a non-shrink, quick-dry grout. If the borehole was located on City of San Jose streets, traffic was restricted from crossing the grouted patch until the field engineer determined the grout had set.

6.2.8 Boring Log Organization and Presentation

Soil descriptions were made in general accordance with ASTM D2487 Standard Classification of Soil for Engineering Purposes (Unified Soil Classification System) and ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). The boring logs are presented in Appendix 1. Towill, Inc. surveyed the ground surface elevation of all borehole locations based on NAVD88 (North American Vertical Datum, 1988).

Boring logs were prepared for all 76 borings. The Boring Log Key (Figure A1-1) summarizes coarse-grained and fine-grained soils and corresponding group names. General notes, abbreviations, sampler types, soil structure definitions, consistency and relative density terminology and moisture content descriptions that are incorporated into each of the boring logs are also included on the Boring Log Key. Each boring log presents boring specific details including: Field Engineer (Logged By), Quality Control Manager (Checked By), Drilling Start and Completion Dates, Drilling Contractor and Operator Name, Project Location, Drilling Method, Hammer Type and Drill Rig Type. Drilling Start and End Times for each day of drilling are shown within the material description column.

The field engineer for Parikh Consultants recorded the soil conditions encountered as the borings were drilled. At depths where sampling was not performed, field engineers based soil information on soil cuttings recovered during the rotary wash drilling process and driller's comments regarding drilling response (i.e., "chattering" noise from drill rods during drilling in sands and gravels, changes in drilling pressures at soil layer intervals, etc.). Field engineers recorded handheld field test results from pocket penetrometer and pocket torvane tests on the field boring logs, as well as results of air monitoring tests of the breathing zone using a Photo-Ionization Detector (PID) and Lower Explosive Limit (LEL/O2) meter. The depth to water and final boring depth were also recorded. Depth to water values presented on the boring logs should not be used for design purposes.

The field engineer from Parikh Consultants also recorded observations of caving conditions and locations where a loss of drilling fluid occurred. Upon completion of each boring, information recorded on the field log was entered into a gINT database and printed out using a gINT boring log template.

Soil samples were visually classified in the laboratory (Section 7.1.1) prior to soil strength and property testing (Sections 7.2.2 through 7.2.6). The soil information presented on the gINT boring logs was revised based on the results of the laboratory visual classification and testing to address soil consistency, particle size and moisture content results. The resulting draft boring logs were reviewed for Quality Assurance by HMM/Bechtel and subsequently finalized.

6.2.9 Standard Penetration Test (SPT)

The Standard Penetration Test (SPT) is a measure of the resistance of the soil during sampling using the split-spoon sampler. This resistance is an indicator of the consistency in fine-grained soils and density and strength in coarse-grained soils. The standard penetration resistance of the soil is defined as the number of blows (N) required to drive the sampler one foot into the soil with a 140-pound hammer dropped 30 inches. The hammer is lifted using either a rope that is wrapped twice around a cathead, or a mechanical device to elevate the hammer (automatic hammer).

The number of blows required to advance the split-spoon samplers was counted and recorded for each 6-inch interval of driving by the field engineer. The SPT, in accordance with ASTM D1586, was halted if the total number of blows exceeded 100, the number of blows exceeded 50 in any 6-inch increment, or if the sampler was not advanced as a result of 10 consecutive blows. The distance driven for each of these refusal conditions was recorded. When the final penetration increment was less than 6 inches, the penetration was recorded to the nearest inch.

Assuming the sampler did not meet the refusal criteria, the SPT blow count shown on the boring logs is the sum of the blows for the two final 6-inch intervals. The first 6-inch interval is not presented on the boring logs unless the sampling interval was 6 inches or less. The Boring Log Key presents a summary of blow count information.

Undisturbed coarse-grained soil samples are not possible to obtain using typical driven thick-walled samplers or pushed thin-walled samplers. It is possible, however, to estimate the in-situ density using the SPT. For the SVRT Tunnel Segment, the SPT was generally performed only at locations and depths where granular material was expected.

A Modified California sampler was also used to sample coarse-grained soils at selected depths of chosen borings. The uncorrected blow count using a driven Modified California sampler was recorded and is shown on the boring logs in Appendix 1. In order to obtain a comparable correlation of strength and density of soils to the SPT blow count (N-value), the Modified California blow count can be corrected by multiplying by a correction factor. This correction factor is typically a function of sampler size and type of soil being sampled. Uncorrected Modified California blow counts are presented on the boring logs and are enclosed in parentheses to differentiate the values from SPT blow counts.

6.2.10 SPT Energy Calibration

The majority of the borings were drilled using Pitcher Drilling Company Failing 1500 drill rigs. However, in some locations, a Fraste Multidrill XL drill rig was needed to allow self-boring pressuremeter testing (discussed in Section 6.4) to be performed. The Fraste Multidrill XL drill rig uses a different hammer system than the Failing 1500 drill rigs. Instead of a Rope and Cathead System, an Automatic Trip Hammer System was used.

To estimate the energy transfer ratio of the hammer on the Fraste Multidrill XL drill rig, ABE Engineering calibrated the efficiency of the automatic hammer during drilling of two borings (BH-25 and BH-65). The results of the calibration showed that the mean energy transfer ratio, based on 265 blows of the automatic hammer, was approximately 75% of the theoretical energy (140-lb hammer at 30-inch drop). The results of the energy calibration are presented in Appendix 1 after Logs of Borings.

6.2.11 Air and Vapor Monitoring

Air monitoring of the work zone was conducted as part of the Work Plan to protect workers should exposure to contamination occur. The breathing zone around the drilling operations was monitored frequently using a Photo-Ionization Detector (PID) meter and a Low Explosive Limit/Oxygen (LEL/O_2) meter. The PID instrument used was an Environmental Instruments Co. Model "Determinator" Organic Vapor Meter (OVM) with a minimum detectable level of 0.1 parts per million (ppm). Monitoring of specific levels of methane, hydrogen sulfide, ethane, butane and propane was not carried out. The LEL/O_2 meter was a GASTECH Model GT-201 with a minimum detectable level of 0.1 ppm. The instruments were rented from Environmental Instruments, located in Concord, CA.

The initial work plan required air monitoring of the breathing zone surrounding the drill rig operation, primarily for worker safety. Beginning in November/December 2004, readings were also taken of the soil samples as the sampler was extracted from the borehole. Readings were also taken during drilling of the slug test wells (Section 6.8). Generally, a minimum of three PID and LEL/O_2 readings were each taken during drilling and sampling of all station and tunnel borings. Along the tunnel alignment, three readings were typically taken within the 60-foot tunnel zone.

Readings (OVM, LEL/O_2 and oxygen) are shown at the corresponding borehole depths on the Logs of Borings (Appendix 1). Readings are also shown at the corresponding well depths on the Logs of Wells (Appendix 7).

According to CalOSHA, "When the preliminary investigation of a tunnel project is conducted, the owner or agency proposing the construction of the tunnel shall submit the geological information to the Division for review and classification relative to flammable gas or vapors." To assist in the gas classification requirements, the above monitoring of the breathing zone around drill rigs was supplemented with headspace monitoring of the 10% Conceptual Engineering observation wells by Geomatrix Consultants. Although the existing wells were not constructed with the intent of measuring underground gases, the measurements can be considered qualitative and possibly indicative of gases in the soil.

The headspace in each well was measured by Geometrix Consultants for percentage lower explosive limit (%LEL), methane, combustible gases without methane, and hydrogen sulfide using an INNOVA 4-gas meter (December, 2004) and a QRAE PLUS 4-gas meter (February, 2005). Gas measurements were obtained at depths just above the water level in the well and two feet below the ground surface to measure gases heavier and lighter than air, respectively. The results of the gas monitoring measurements are presented in Appendix 1 following the SPT Energy Calibration and discussed in Section 8.2.4.

6.3 Field Vane Shear Testing

In-situ measurements of the undrained shear strength of fine-grained soils encountered during drilling were obtained from field vane shear tests at selected boreholes locations. The test is not suitable for sandy soils, since these soils allow for drainage during vane shear testing.

6.3.1 Field Procedures

Parikh Consultants performed field vane shear tests using a GEONOR H-10 Vane Borer. Testing was done in general accordance with ASTM D2573 and the GEONOR user's manual.

The GEONOR H-10 Vane Borer System consists of a vane and protection tube connected to strings of inner and outer rods, respectively. The vane size used for the project was 2.16” x 4.33” (55 mm x 110 mm). The general procedure to conduct the field vane shear test is described as follows:

After drilling and sampling to the desired test depth, the GEONOR H-10 Vane Borer System was assembled by connecting the protection tube (i.e. housing) and vane to the outer and inner rods, respectively. The vane and housing was then lowered to the bottom of the borehole and the drill rig’s hydraulic system was used to push the housing a minimum of six inches into native soil. The vane was subsequently extended an additional 19 inches into the native soil to the depth where the test was conducted. Before beginning the test, the last sections of inner and outer rods were connected and the readout unit was attached to the outer rod.

A manual cranking device was used to generate the shearing action. The crank was turned at 1 revolution per second, creating a rotational shearing rate of 0.2 degrees per second. Gauge readings were recorded at 30-second intervals.

The capacity of the GEONOR H-10 Vane Borer used on the SVRT Tunnel Segment was approximately 2.1 kips per square foot (ksf). Field vane shear tests could not be performed in clayey soils where the strength exceeded the equipment capacity. Within the station borings, vane shear tests were typically performed in the upper 50 ft below the existing ground surface. The strength of the clayey soils below 50 ft typically exceeded the vane shear capacity. For boring locations along the proposed tunnel alignment, vane shear tests were planned at the crown, center and invert of the tunnel profile at the time of the field investigation.

Remolded tests were performed after reaching the peak material strength. The soil was remolded by rotating the inner rods and vane a total of 25 revolutions. Remolded tests were not performed at locations where the shearing resistance exceeded equipment capacity. A more detailed description of vane shear testing is provided in Appendix 2.

6.3.2 Frequency of Testing

Field vane shear tests were performed in 8 station borings and 17 tunnel borings (Table A2-1). At the proposed station locations, vane shear tests were generally attempted continuously (2 to 2.5-foot intervals) in the upper 50 ft. Two borings (BH-61 and BH-63) at Alum Rock Station, four borings (BH-23, BH-66, BH-72 and BH-77) at the Crossover/ Downtown San Jose Station and two borings (BH-34 and BH-73) at Diridon/Arena Station had vane shear testing. Continuous testing was discontinued when the housing (outer rods) could not be pushed or the vane could not be extended from the housing.

Field vane shear testing was performed at the following tunnel boring locations:

- East Portal to Alum Rock Station – 3 borings (BH-01, BH-04 and BH-57);

- Alum Rock Station to Crossover – 8 borings (BH-07, BH-10, BH-12, BH-16, BH-19, BH-21, BH-50 and BH-54);
- Downtown San Jose Station to Diridon/Arena Station – 1 boring (BH-29);
- Diridon/Arena Station to West Portal – 5 borings (BH-37, BH-39, BH-41, BH-44 and BH-47).

6.3.3 Results

Tables A2-2 through A2-6 present a summary of the peak undrained and remolded field shear strength results for all vane shear tests. The peak undrained strengths are also presented on the boring logs in Appendix 1. The field vane shear test data are presented in Chapter 8, and in Appendix 2, as Figures A2-1 through A2-77. Figure A2-78 presents the calibration information on the GEONOR H-10 Vane Borer used for the SVRT project.

6.4 Pressuremeter Testing

Hughes Insitu Engineering, Inc. performed pressuremeter testing in selected boreholes on both granular and cohesive soils. The pressuremeter test is an in-situ stress-strain test performed against the wall of a borehole using a cylindrical probe that expands radially. The testing, similar to field vane shear testing, is a method used to estimate the in-situ undrained shear strength, lateral stress, and modulus (stress-strain properties) of the soil.

6.4.1 Field Procedures

Both pre-bored pressuremeter tests and self-boring pressuremeter tests were conducted. Failing 1500 drill rigs were used for pre-bored pressuremeter testing. A Fraste Multidrill XL drill rig (a top-drive rig) was used for self-boring pressuremeter testing. In the stiffer, fine-grained soils and in the coarse-grained soils, only pre-bored tests were conducted. Steel casing was used to prevent caving from sandy/gravelly formations above or at the depth of pressuremeter tests, which could cause damage or loss of the pressuremeter equipment. Nitrogen provided the pressure source for the tests. Three electronic sensors within the center of the instrument registered displacement data during testing.

Pre-bored and self-boring pressuremeters were coupled to the end of the AW-size drill rods before being placed into the borehole. To conduct pre-bored pressuremeter tests, a 2.5-foot long pilot hole was formed using a 3-inch outside diameter Shelby Tube. At several locations, two consecutive Shelby Tubes were pushed to form a 5-foot test zone. The first pressuremeter test was conducted in the bottom 2.5 feet of the test zone. Upon completion of the first test, the pressuremeter device was raised 18 inches and the second test was conducted. If the soil was granular in nature or Shelby Tube advancement was difficult, the pilot hole was drilled using a 2-15/16 inches tri-cone bit under controlled/limited circulation.

The self-boring pressuremeter differs from the pre-bored pressuremeter in that it contains a hollow core and is drilled into the ground rather than placed into a hole. A specially fabricated drill bit and drill rod fit into the hollow space within the pressuremeter, allowing drill cuttings to be forced up through the center of the apparatus. The advantage of the self-boring pressuremeter is that it can minimize disturbance of the surrounding soil caused by the drilling process and it avoids contraction of the borehole by soil swelling before the pressuremeter is inserted and inflated. The self-boring pressuremeter is intended for relatively uniform sands, silts and clays. Gravelly formations are not suitable for a self-boring pressuremeter, as gravel particles can wedge between the drill bit and cutting shoe. A more detailed description of pressuremeter testing operations is provided in the Hughes Insitu Engineering, Inc. report entitled "Summary of the Pressuremeter Testing for the Silicon Valley Rapid Transit Project" (Appendix 3).

6.4.2 Frequency of Testing

In-situ pressuremeter testing was performed in one borehole at Alum Rock Station (BH-60), four borings at the Crossover/Downtown San Jose Station (BH-24, BH-64, BH-65 and BH-71) and two borings at Diridon/Arena Station (BH-33 and BH-76) (Table A3-1).

At the planned station locations, pressuremeter testing was generally attempted at representative layers throughout the borehole depth. Estimates of undrained shear strength, lateral stresses, in-situ modulus, and friction angle were obtained to a maximum depth of 150 ft for design of the seepage cut-off walls planned at each of the stations.

Pressuremeter testing was performed at the following tunnel boring locations:

- East Portal to Alum Rock Station – 2 borings (BH-02 and BH-06);
- Alum Rock Station to Crossover – 5 borings (BH-08, BH-13, BH-18, BH-53, and BH-55);
- Downtown San Jose Station to Diridon/Arena Station – 1 boring (BH-31);
- Diridon/Arena Station to West Portal – 4 borings (BH-38, BH-42, BH-45, and BH-48).

Pressuremeter tests for tunnel borings were planned at the crown, center and invert of the tunnel profile available at the time of drilling.

6.4.3 Results

Pressuremeter test results, including undrained shear strength, friction angle, lateral stress, and shear modulus are presented in Chapter 8 and in Appendix 3. The pressuremeter test depths are also presented on the boring logs in Appendix 1.

The pressuremeter instrument had difficulties obtaining data below a depth of 100 ft at boring BH-60 due to interference with the electronic signals. Difficulties obtaining an electronic signal also arose when the SCVWD inspector attempted a Global Positioning System (GPS) survey at this borehole location.

6.5 Downhole Geophysical Logging

GEOVision Geophysical Services obtained suspension soil velocity measurements at three selected boreholes using a downhole suspension logging system. The purpose of the downhole geophysical logging was to acquire compression and shear wave velocities (P/S) as a function of depth, which can be used to characterize low-strain soil modulus for ground response analyses.

6.5.1 Field Procedures

The borings drilled for downhole suspension logging were approximately 216 to 220 ft deep. The additional depth beyond the intended test depth of 200 ft provided a “rat hole” to accommodate the P/S wave suspension logging probe (OYO Model 170). The drilling fluid used in the rotary wash drilling process was agitated immediately before deploying the probe.

The entire probe was suspended via an armored 7-conductor cable centered within the borehole by nylon “whiskers”. The source motion is, therefore, not coupled directly to the boring walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling the borehole and surrounding the source. This pressure wave propagates as P and S-waves in the surrounding soil as it impinges upon the borehole wall.

Prior to entering the borehole, the mechanical and electronic depth counters were set to zero. The probe was lowered to the bottom of the borehole, and then returned to grade, stopping at 0.5 m/1.6 ft intervals to collect data. The probe receives control signals from, and sends the amplified receiver signals to, instrumentation on the surface via the cable. No significant signal contamination from cultural (man-made) vibration was observed. A more detailed description of the downhole geophysical logging operations is provided in Appendix 4.

6.5.2 Frequency of Testing

Downhole geophysical logging was performed at two station boring locations, BH-59 (February 7, 2005) at Alum Rock Station and BH-68 (January 21, 2005) at the Crossover/Downtown San Jose Station, and at one tunnel boring location, BH-79 (March 2, 2005). The tunnel boring location also coincided with a planned deep ventilation structure.

6.5.3 Results

The results of the downhole geophysical logging are presented in Appendix 4.

6.6 Vibrating Wire Piezometers

Vibrating wire piezometers are instruments for measuring pore water pressure. They are used to monitor pore-water pressures for engineering works such as retaining walls, excavations, and tunnels. A vibrating wire piezometer consists of a tensioned steel wire that is clamped to both ends of a hollow cylindrical body, which is enclosed in a protective steel housing. The piezometer is designed so that a change in pressure causes a change in tension in the wire. An electro-magnetic coil is used to excite the wire at its natural frequency. The frequency signal generated by the coil is transmitted to a readout box. Using applicable calibration factors, an estimate of water pressure can be obtained based on the frequency signal obtained during a monitoring event.

For 35% Preliminary Engineering, vibrating wire piezometers (VWPs) were typically installed at the following three locations:

- Along the proposed tunnel alignment
- At proposed station locations

- Near stream crossings

The purpose of installing VWPs was to obtain initial values of piezometric levels and subsequent monthly readings throughout future design and construction of the SVRT Project. The readings obtained are of assistance in estimating design water pressure distributions within the Confining Layer, Upper Aquifer, and Major Aquitard.

6.6.1 Field Procedures

As part of the 35% Preliminary Engineering Design, vibrating wire piezometers were installed at borehole locations. The two types of instruments used were a Geokon Model 4500S Standard Piezometers (for piezometers at moderate pressure ranges) and a Geokon Model 4500AL Piezometers (for groundwater table level at low pressure ranges). Parikh Consultants, with the assistance of Pitcher Drilling Company, installed all 35% PE vibrating wire piezometers.

Prior to installation, VWPs were placed into a sand-filled filter sock and immersed in a bucket of water for a minimum of 24 hours. At each location, the prepared piezometer “socks” were attached to a 1-inch diameter PVC (polyvinyl chloride) pipe at predetermined depths. Upon completion of drilling and sampling, the pipe was lowered into the fluid-filled open borehole. The VWPs were then tremie-grouted in-place. Figure A5-1 presents a cross section showing typical installed VWPs on the SVRT Tunnel Segment project.

Figure A5-2 presents a cross section of the VWPs installed at BH-68. At BH-68, two piezometers were installed within the borehole at depths of 80 ft and 160 ft. An additional VWP was installed in an adjacent 30-foot borehole. BH-68 was drilled and logged to a depth of 216 ft before P/S wave suspension logging was performed. The borehole was then grouted to a depth of 160 ft depth and left overnight to set. The following day, the first VWP was installed at a depth of 160 ft with sand pack and bentonite pellets/seal for isolation. The rest of the borehole was tremie-grouted through the 1-inch diameter PVC, with a VWP attached at a depth of 80 ft.

6.6.2 Locations

As part of the 10% Conceptual Engineering (CE) program, URS Corporation, with the assistance of Robert Chew Geotechnical, Inc. and Pitcher Drilling Co., installed VWPs at four borehole locations (NB-07, NB-13A, NB-16 and NB-17) near the four proposed stations. At the time of conceptual engineering, four stations were planned. Two VWPs were installed within each of the four borehole locations. The Slope Indicator VWPs were installed between November 13, 2002 and November 25, 2002.

Piezometers P2-1 and P2-2 were installed within NB-07 near Diridon/Arena Station, south of the Alameda and east of White Street. NB-13A (P4-1 and P4-2) was drilled near Alum Rock Station along the north side of E. St. James Street approximately 50 ft west of 30th Street. NB-16 (P3-1 and P3-2) was drilled on 6th Street approximately 35 ft south of Santa Clara Street. NB-17 (P1-1 and P1-2) was drilled near the Downtown San Jose Station along the east side of Lightston Alley, approximately 90 ft south of Santa Clara Street. The locations of all 10% CE phase vibrating wire piezometers are presented on Figure 6-2. Of the four sets of VWPs installed for 10% CE phase, one (1) location (NB-07) has been abandoned; piezometers installed in NB-07 are no longer accessible due to recent asphalt concrete pavement that was constructed in the area.

Thirty-one vibrating wire piezometers were installed at 17 borehole locations during the 35% PE investigation. Two VWPs were typically installed in selected tunnel borings. Along the tunnel alignment one VWP was typically placed at approximately 25 to 30 ft below ground surface (bgs) and one deeper VWP was typically installed at tunnel depth. The two exceptions were boreholes BH-79 and BH-80 where three VWPs and one VWP, respectively, were installed. VWPs were typically installed within granular layers so that relatively quick responses in piezometric levels could be observed.

At proposed station locations, VWPs were placed in three different hydrogeologic layers: the Confining Layer (to obtain water table information), the Upper Aquifer and the Major Aquitard. A summary of VWPs installed at the proposed stations are as follows:

- Alum Rock Station - BH-58 (P-58) at a depth of 30.5 ft (Confining Layer); BH-63 (P-63) at a depth of 81.0 ft (Upper Aquifer).
- Downtown San Jose Station - BH-68 (P-68-1, P-68-2 and P-68-3) at depths of 30.0 ft (Confining Layer), 80.0 ft (Upper Aquifer) and 160.0 ft (Major Aquitard).
- Diridon/Arena Station - BH-74 (P-74) at a depth of 30.0 ft (Confining Layer); BH-76 (P-76) at a depth of 105.0 ft (Major Aquitard).

6.6.3 Results

The 10% CE VWPs were monitored by URS Corporation on a biweekly basis from November 2002 through March 2003. Monitoring was not conducted between March 2003 and October 2004. In October 2004, Geomatrix Consultants resumed monitoring the 10% CE vibrating wire piezometers on a monthly basis through March 2005. Parikh Consultants took over VWP monitoring duties in May 2005 and have been monitoring VWPs on a monthly basis since then.

Geomatrix Consultants monitored the 35% PE vibrating wire piezometers on a monthly basis from October 2004 through March 2005. Parikh Consultants began monitoring the VWPs in May 2005 and are currently monitoring VWPs on a monthly basis.

The locations of 35% PE vibrating wire piezometers are also presented on Figure 6-2. Table A5-1 (Appendix 5) includes a summary of all VWP locations installed during the 35% PE investigation. Table A5-2 summarizes VWP installation information. The monitoring results of all VWPs (10% CE and 35% PE) are presented in Table A5-3. Factory calibration sheets for each VWP are presented in Appendix 5 as Figures A5-3 through A5-33. VWPs are defined with a dual labeling system based on borehole location. For example, the location of VWPs at BH-19 is labeled as “BH-19 (P-19)”. An ascending numerical suffix refers to the specific VWP within the location. For example, the deeper VWP (out of two VWPs) installed in BH-19 is “P-19-2”.

6.7 Observation Wells

Observation wells were typically installed at the following three locations:

- Along the proposed tunnel alignment
- At proposed station locations
- Near stream crossings

The purpose of installing observation wells was to obtain initial water level readings and subsequent monthly readings throughout future design and construction. Monitoring was performed to assist in the estimation of design water pressure distributions within the Confining Layer, Upper Aquifer and Major Aquitard.

6.7.1 Field Procedures

As part of the 35% Preliminary Engineering Design, deep standpipe piezometers were installed at two borehole locations for well monitoring. The screened interval of the two piezometers was positioned within the major aquitard (~200 ft deep). The depths, screened intervals, sand pack intervals, and bentonite pellet layer intervals of the two standpipe piezometers are shown on Figure A6-1. The standpipe piezometers were constructed in accordance with the Santa Clara Valley Water District (SCVWD) standards and guidelines.

Slug testing wells were also used for groundwater level monitoring. A more detailed discussion of the construction and testing of slug testing wells is presented in Section 6.8. The depths of the standpipe piezometers, screen depths, sand pack intervals, and bentonite pellet layer intervals are presented on the well logs in Appendix 7, Slug Testing Program. Slug testing wells were constructed in accordance with the Santa Clara Valley Water District (SCVWD) standards and guidelines, and were developed using the techniques of surging, bailing and pumping.

6.7.2 Locations

Prior to and as part of the 10% Conceptual Engineering (CE) program, observation wells were installed at seven borehole locations near the four proposed stations. At the time of the CE program, a four-station concept was being investigated. The seven observation wells (NW-01, NW-04, NW-05, NW-06, MW-1, MW-2, and MW-3) were installed between September 05, 2001 and March 2, 2003. Logs of the 10% CE observation wells are presented in the 10% Conceptual Engineering draft report entitled, "Geotechnical Exploration Findings and Recommendations Report", prepared by URS Corporation (2003). The locations of all 10% CE vibrating wire piezometers are presented on Figure 6-2. Observation wells NW-01, NW-04, NW-05, and NW-06 were installed at the Alum Rock Station, Civic Plaza/SJSU Station (previously proposed), Market Street Station (previously proposed) and Diridon/Arena Station, respectively. Observation wells MW-1 through MW-3 were installed near the corner of 5th Street and Santa Clara Street at the previously proposed Civic Plaza/SJSU Station.

For the 35% Preliminary Engineering Design, two deep standpipe piezometers, OW-1 (in

BH-59) and OW-5 (in BH-75), were installed. Ten slug testing wells (ST-1, ST-2, ST-3, ST-5, ST-7, ST-8, ST-10, ST-11, ST-12, and ST-13) were also used for groundwater level monitoring.

6.7.3 Results

URS Corporation monitored groundwater levels at wells NW-01, NW-04, NW-05, and NW-06 from October 2001 through April 2003. Observation wells MW-1 through MW-3 and a nearby historic observation well, Well #18, were also monitored by URS Corporation on three occasions between March and April 2003. Monitoring was not conducted at any wells between April 2003 and October 2004. Geomatrix Consultants monitored the 10% CE observation wells on a monthly basis from October 2004 through March 2005. Parikh Consultants began monitoring the observation wells in May 2005 and are currently monitoring wells on a monthly basis. Of the seven observation wells installed for 10% CE, five are operational, two (NW-04 and MW-3) have been abandoned (per SCVWD requirements), and one (MW-1) is currently being monitored but in need of redevelopment to remove debris within the well.

Geomatrix Consultants monitored the 35% PE observation wells on a monthly basis from October 2004 through March 2005. Parikh Consultants began monitoring the observation wells in May 2005 and are currently monitoring the wells on a monthly basis. The locations of all operational 10% CE and 35% PE wells are presented on Figure 6-2. Table A6-1 (Appendix 6) includes a summary of all observation well locations installed during the 35% PE investigation. Water level readings of all observation wells (10% CE and 35% PE) are summarized in Table A6-2.

6.8 Slug Testing

Slug testing was used to estimate hydraulic conductivity of granular layers along the tunnel alignment, at stations, and near creeks intersecting the alignment. A total of 13 slug test wells were originally planned. Two locations were deleted due to private property restrictions and one location was omitted when the Civic Plaza/SJSU Station and Market Street Stations were combined into a single Downtown San Jose Station. The ten slug test wells were installed between February 10, 2005 and April 19, 2005.

6.8.1 Field Procedures

The slug test procedure used for the project followed the guidelines outlined in ASTM D4044, (Field Procedure) for "Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers". A slug test involves the instantaneous lowering or raising of the water level in a well and measuring the response of the water level as it returns to its static level. The test is performed by dropping a slug, commonly a sealed PVC pipe of known volume into a well to displace an equivalent volume of water. Once the water level in the well has returned to its static level, the slug is then removed. During both the "slug-in" and "slug-out" parts of the test, the water level is monitored with either a water level meter or pressure transducer until the water level has recovered to at least 80% of its initial displacement. Following the collection of data in the field, analytical techniques can be used to interpret the data and determine aquifer properties.

The tests were conducted using a pressure transducer to automatically measure the water level, and a 5-foot long, 2-inch diameter, Schedule 40 polyvinyl-chloride, sealed bailer. The slug test was performed by manually inserting the bailer into the well and measuring the water level until it returned to a minimum of 80% of its initial displacement. The tests were also conducted by removing the bailer from the well and measuring the recovery of the water level. The equipment and procedures used for performing the slug tests is described in more detail in Appendix 7.

Slug tests were performed in each of the designated wells between March 2 and April 20, 2005. Between four and twelve slug tests were performed in each well, based on the response time.

Six of the borings, for the purposes of well installation and slug testing, were drilled using a direct rotary wash drill rig and four were drilled using a hollow-stem auger drill rig. The borings were drilled to depths ranging from 36 ft (ST-13) to 90 ft (ST-1).

All borings for slug test wells were initially drilled with a 6-inch bit to allow for soil sampling, and subsequently reamed out with the larger diameter 10-inch bit. An attempt was made to collect one sample from each five-foot screened interval. Samples were sent to the laboratory for sieve analysis, and where applicable, hydrometer analysis. The sample depths are presented in Table A7-1.

The wells were constructed using one of three combinations of filter packs/screen slot sizes shown in the well logs in Appendix 7. For construction of all wells, an SCVWD inspector observed the placement of the grout seal.

Slug test wells were developed a minimum of 48 hours after well installation. The purpose of development was to remove fines from the filter pack. All wells were developed using a combination of bailing, surging and pumping (with a submersible pump).

Pitcher Drilling installed all the slug test wells and provided well development services. HMM/Bechtel, with the assistance of URS Corporation, oversaw installation.

6.8.2 Locations

Ten slug testing wells (ST-1, ST-2, ST-3, ST-5, ST-7, ST-8, ST-10, ST-11, ST-12, and ST-13) were installed at different locations along the alignment. The locations of the 10 slug testing wells are presented in Figure 6-2.

6.8.3 Results

The summary results of the slug tests are presented in Table A7-2. It should be recognized that the slug tests have significant limitations for assessing the hydraulic permeability of soil formations. Because only a small volume of the aquifer is tested, the estimated permeability values are representative only of the condition in the immediate vicinity of the well, a volume of soil that may have been disturbed during drilling and purging operations.

6.9 Cone Penetration Testing Program

The CPT program commenced on October 6, 2004. The majority of CPTs were conducted between October 6, 2004 and February 23, 2005. Two additional CPTs (CPT-93 and CPT-157) were conducted on April 19, 2005 at the VTA Newhall Yard, where UPRR permitting restrictions postponed work. One additional CPT (CPT-95) was performed on April 20, 2005 at the location where upward water flow was observed at a previous CPT location (CPT-30). In addition to continuous CPT soundings, dissipation tests were conducted and downhole seismic shear wave velocity measurements were obtained. The locations of all of the CPTs are presented in Figure 6-1 and Figure A8-1.

6.9.1 Conventional CPTs

A total of 146 CPTs were conducted. The following sections describe the equipment, procedures, locations and results of the CPT program.

6.9.1.1 Equipment

Equipment utilized in conducting CPTs included a self-contained 25-ton CPT rig with hydraulic pushing system, a piezocone, cone rods and casing, a data acquisition system and a support truck and trailer.

The CPTs were performed using an International 25-ton capacity truck mounted rig with a self-contained power supply unit. The rig was equipped with hydraulic jacking systems to lift and level the pushing platform. The “dead weight” of the rig provided the reaction weight necessary for advancing the CPT tools. The conventional instrumented piezocone assembly used for the SVRT project included a cone tip with a 60-degree apex and a cone base area of 15 cm^2, a sleeve segment with a surface area of 200 cm^2, and a pore pressure transducer near the base (shoulder) of the cone tip (designated the u2 location).

Fugro’s CPT cone rods are manufactured from high tensile strength steel and have a cross sectional area adequate to sustain up to 700 tsf tip pressure without buckling. A steel casing was generally placed in the upper clayey strata and was typically extended to depths of 20 to 75 ft, when used. The casing provided lateral support to prevent bending or buckling of the slender 10-foot sections of steel rod as they were hydraulically pushed into the ground.

The data acquisition system converted an analog signal from the cone penetrometer to a digital signal, which was monitored, recorded and presented in near-real time on the laptop computer. A support pickup truck/trailer contained a grout pump and mixer to properly abandon CPT holes after completion, a pressure wash system for cleaning the work area and maintaining clean equipment throughout field program, a steam cleaning system for environmental protocol if needed, and tools and supplies for daily operations.

6.9.1.2 Procedures

Prior to testing, the truck was lifted up and leveled on four pads to provide a stable reaction for the cone thrust. During the test, the instrumented cone was hydraulically pushed into the ground at a rate of about 2 centimeters per second (cm/s), and readings of cone tip resistance, sleeve friction, and pore pressure were digitally recorded every second. As the test progressed, the CPT operator monitored the cone resistance and its deviation from verticality.

Information collected during a push was stored digitally. The data files included project description and location, operator, data format information, and other pertinent information about the sounding.

After completing a CPT, the hole was backfilled with cement-bentonite grout by the tremie method using a grout pump and mixer. The surface of the CPT holes was finished with rapid setting quickcrete. Grout mix and grouting procedures were completed in accordance with Santa Clara Valley Water District regulations. The work area was cleaned per City of San Jose requirements.

Fugro conducted the CPTs in general accordance with ASTM D5778. The continuous CPT soundings were typically advanced to refusal (500 to 700 tsf tip pressure), which ranged from approximately 42 to 158 ft in depth. Each CPT lasted between 3 and 8 hours.

More detailed descriptions of the procedures and equipment specifications of the CPT operations can be found in Appendix 8.

6.9.1.3 Locations

CPTs performed along the proposed tunnel alignment ("tunnel CPTs") were spaced at 200 to 300-foot intervals. All CPTs at tunnel locations were completed to depths of at least 20 ft below tunnel invert depth or to refusal. CPTs performed at the proposed stations ("station CPTs") were spaced approximately 100 ft apart. All station CPTs were pushed until refusal.

6.9.1.4 Results

The CPT logs present the measured cone (tip) resistance in tons per square foot (tsf), the measured sleeve friction in tsf, the friction ratio in percentage (including color coding denominating the Soil Behavior Type according to Robertson and Campanella (1990); see CPT correlation chart in Figure 8-0, Key to Plan and Profiles), the measured pore pressure in tsf at the u2 location, and the estimated soil undrained shear strength (s_u) in ksf.

Some of the data presented on the CPT logs is interpreted and based on assumptions that need to be verified with site-specific data. The interpreted data include the soil behavior type and the estimated soil undrained shear strength. The soil behavior type and estimated undrained shear strength are influenced by the soil unit weight (and resulting in-situ total stress condition), and the N_k-value. The range of selected N_k values was based on calibrations performed by Fugro comparing the CPT tip resistance with the strength determined from field vane shear testing in adjacent borings. A more detailed discussion regarding the undrained shear strength calibration is presented in Appendix 8.

The CPT logs show the range of undrained shear strengths calculated from CPT cone tip resistances (corrected for unequal end area effects) based on cone bearing capacity factors (N_k) of 12 and 15. The key to CPT logs is presented on Figure A8-4. CPT sounding logs for the 146 CPTs are presented on Figures A8-5 through A8-150.

6.9.2 Seismic CPTs

A total of 10 SCPTs were conducted. The following sections describe the equipment, procedures, locations and results of the SCPT program.

6.9.2.1 Equipment

Downhole seismic shear wave velocity measurements were conducted using Fugro's seismic CPT system. The seismic CPT system includes the basic thrust system, a seismic cone assembly, a seismic wave source, and a digital recording seismograph.

6.9.2.2 Procedures

The seismic cone assembly is similar to the conventional cone assembly, with the addition of a three-component array of geophones. The geophones are orthogonally mounted inside the assembly about 15 cm above the cone tip. The seismic CPT system consists of a heavy metal beam that is positioned parallel to the cone truck and held firmly against the ground by the weight of the beam and additional weights placed on it. The beam is positioned at least 10 ft from the cone rods. Seismic waves are generated by striking each end of the beam with a 12-pound sledgehammer. The hammer blow from opposite ends of the beam generates shear waves with opposite polarity. Conventional CPT testing was temporarily halted at 5-foot intervals to perform the seismic testing and collect seismic data.

The hammer blows trigger the seismograph to record the time histories of the generated seismic waves as they travel through the soil. If the shear wave signal is clearly defined, the waveform is selected for stacking and the arrival time of the wave is recorded. Additional blows were similarly examined and stacked. A more detailed discussion regarding the signal selection and stacking is presented in Appendix 8.

Waveforms are digitally recorded and saved in the seismograph's hard drive for further processing. After a complete set of seismic data is recorded, the cone is advanced to the next depth, and the procedure is repeated until the hole reaches the required depth or refusal.

The shear wave arrival time at each depth is determined from the recorded "stacked" signals. The average arrival time is determined and based on the horizontal offset of the seismic source from the CPT rods, a strike angle is estimated. The average vertical arrival time is determined by taking the sine of the strike angle. The incremental seismic velocity is the difference in vertical average arrival time between two depth increments, divided by the length of the increment (typically 5 ft). This seismic velocity is presented on the seismic CPT logs (Appendix 9).

Seismic CPT testing was performed in accordance with ASTM D577 and "Seismic Cone Penetration Test," by Robertson, Campanella, and Gillespie (1986).

6.9.2.3 Locations

Seismic shear wave velocity tests were conducted at ten locations. Tests were performed at each of the proposed station locations and at the crossover area. Inconsistent/noisy seismic data were obtained at four of the ten CPT locations, CPT-106, CPT-128, CPT-145, and CPT-145A. The seismic data at these four locations were not of sufficient quality for interpretation of shear wave velocities. The possible reasons for the inconsistent/noisy data are discussed in Appendix 9.

Seismic cone testing was successfully performed at the following six locations:

- Two Seismic CPTs at Alum Rock Station
- Two Seismic CPTs at Downtown San Jose Station
- Two Seismic CPTs at Diridon/Arena Station

The locations of all 10 seismic CPTs are shown in Appendix 9 on Figure A9-1.

6.9.2.4 Results

CPT sounding logs for the six seismic CPTs are presented on the Logs of seismic CPTs (Figures A9-3 through A9-8). The seismic CPT logs provide graphical plots of the same data presented on conventional CPT logs, along with measured shear wave velocity in ft per second (fps).

6.9.3 Hydropunch Testing

During the October 2004 to March 2005 exploration, four CPTs (CPT-25, CPT-30, CPT-94, and CPT-124) and one boring (BH-19) revealed the existence of temporary upward groundwater flow conditions. These locations are primarily within the eastern stretch of the tunnel alignment along Santa Clara Street (between 6th and 28th). A discussion of the phenomenon was presented in Section 5.5.

At these CPT locations, water and sediments were ejected from the CPT holes upon withdrawal of the rods. At CPT-30, the location of the largest temporary discharge, the field engineer onsite estimated that water flowed out of the CPT hole for more than 1 hour and at a peak rate of about 75 gallons per minute (gpm). During rotary wash drilling at boring location BH-19, a rise in the fluid of the drilling tub was observed for 10 minutes after the auger breached a well-graded gravel layer at about 65 ft bgs. A detailed summary of the locations where temporary upward groundwater conditions existed is discussed in Technical Memorandum: Observed Upward Groundwater Flow (HMM/Bechtel, 2005e)

To investigate this phenomenon, hydropunch H-1 and CPT-95 were conducted on April 20, 2005, 35 ft and 50 ft south of CPT-30, respectively (Figure A10-1). A pore pressure dissipation test was performed at CPT-95 to record in-situ pore pressures. CPT-95, as anticipated, also exhibited upward groundwater flow. The flow at CPT-95, however, lasted only 3 minutes. A surface groundwater sample was taken from the pavement surface as it flowed out of the CPT hole, but was not tested.

6.9.3.1 Equipment

The hydropunch sampling system utilized the same 25-ton, truck-mounted CPT system used to conduct the CPT soundings for the project. The hydropunch sampler, shown in Figure A10-2, makes use of a retrievable stainless steel screen, 18 inches in length, with 0.005-inch laser cut slots. The CPT rods were steam-cleaned prior to sampling to minimize the potential for introducing outside contaminants into the water sample.

6.9.3.2 Procedures

The hydropunch allowed in-situ water samples to be obtained at the depth of the anticipated pressurized layer while minimizing contamination from other water-bearing layers. Groundwater samples were captured for laboratory testing of water quality and dissolved gases.

At H-1, the hydropunch sampler was advanced to a depth of 64 ft using the truck-mounted CPT hydraulic system. The CPT rods were retracted approximately one foot to expose the screen and allow the groundwater to flow into the sampler. A hand pump system consisting of plastic tubing and a Waterra check valve was used to retrieve the water samples. Upon completion, the hydropunch sampler and CPT rods were retrieved, leaving the disposable tip in the ground.

Twelve water sample bottles were filled, and the samples were then given to a representative of Kleinfelder for laboratory testing. It should be noted that water samples were exposed to air at the ground surface for a short time (5 to 10 minutes) during the sampling process.

The samples were analyzed for dissolved gases, general inorganic properties, and properties indicative of treated potable water (trihalomethanes) and of waste water (oil and grease, coliform, nitrate).

6.9.3.3 Results

The laboratory results of the hydropunch testing are presented in Table A10-1.

The purpose of the hydropunch sampling was to determine the source of the upward water flow. Values for common anions, total dissolved solids, and hardness are within typical values reported for the Upper Aquifer (Table A10-2). Test results for dissolved gases, show that the nitrogen concentration is near the equilibrium concentration at atmospheric pressure. Furthermore, the absence of trihalomethanes suggests no influence from leaking potable water lines, and the lack of oil and grease, coliform bacteria, and high nitrate levels, suggest no influence from leaking sewer lines or storm drains. Therefore, the results of the laboratory tests on the hydropunch sample indicate that the upward water flow originated in the Upper Aquifer.

6.9.4 Dissipation Testing

Dissipation tests were conducted at 27 CPT locations. The following sections describe the equipment, procedures, locations and results of the Dissipation Testing program.

6.9.4.1 Equipment

Fugro conducted dissipation tests using a conventional Fugro truck-mounted 25-ton cone apparatus in general accordance with ASTM D5778.

6.9.4.2 Procedures

HMM/Bechtel typically selected the target test depths based on an evaluation of the stratigraphy from adjacent explorations and the anticipated depth of the structure. Detailed procedures and equipment specifications for the dissipation testing are provided in Appendix 11.

The cone is advanced in the hole to the estimated test depth and then halted. During a pause in penetration at a specific depth, any excess pore pressures generated around the cone will start to dissipate. In clays, pore pressure data are then recorded until at least approximately 50 to 75 percent of the induced excess pore pressure is dissipated. In sands, tests are generally conducted to 100 percent of the excess pore pressure dissipation. The resulting dissipation test duration is typically on the order of 15 minutes to 3½ hours depending on soil permeability, with pore pressures dissipating quicker in sandy soils than in clays. Pore pressure data recorded during the test are digitally recorded for subsequent analyses. After each dissipation test is completed, the cone is advanced to the next depth. A detailed discussion of the interpreted coefficients of consolidation and permeability are discussed in Appendix 11.

6.9.4.3 Locations

One to four dissipation tests were conducted per CPT, at the following locations:

- Two CPTs at Alum Rock Station (CPT-112 and CPT-120)
- Six CPTs at Crossover and Downtown San Jose Station (CPT-44, CPT-134A, CPT-133, CPT-137, CPT-140, and CPT-143)
- Two CPTs at Diridon/Arena Station (CPT-146 and CPT-153)
- One CPT location at the West Portal (CPT-93)
- One CPT location at Lower Silver Creek (CPT-04)
- One CPT location at Coyote Creek (CPT-27)
- Two CPT location between Guadalupe River and Los Gatos Creek (CPT-55 and CPT-55A)
- Twelve CPT locations along the tunnel alignment (CPT-09, CPT-17, CPT-18, CPT-95, CPT-35, CPT-96, CPT-98, CPT-102, CPT-103, CPT-68, CPT-79, and CPT-84)

6.9.4.4 Results

A summary of the dissipation test locations, depths, soil types, start and final measured pore pressures, and estimated water table depth is presented in Table A11-1. Based on the results of the pore pressure dissipation tests, estimates of the coefficient of horizontal consolidation (c_h) and the coefficient of horizontal permeability (k_h) were made by Fugro. These estimates were based on several assumptions including: water table depth, effective vertical stress (using correlated unit weights) and rigidity index. The results of the dissipation test interpretations for c_h and k_h are presented in Table A11-2. Table A11-2 also presents Fugro's estimated effective stress, estimated water table depth, estimated static and initial pore pressure, and percentage of pore pressure dissipated for each test.

6.9.5 CPT Completion and Abandonment

CPT locations were generally terminated at refusal. At a few CPT locations, however, the operation had to be terminated prematurely due to time constraints, permitting requirements, or access issues.

Prior to completion of the CPT, the Santa Clara Valley Water District (SCVWD) was contacted for observation of grouting procedures. After CPT testing was completed, the CPT hole was grouted from the bottom up using a tremie pipe per SCVWD requirements. All Investigation Derived Waste (IDW) and any loose soil or cuttings from the CPT operation were removed from City of San Jose streets or private property and were placed in 55-gallon drums. All drums containing IDW were characterized, labeled, and disposed of in accordance with applicable regulatory requirements. Integrated Waste Management (IWM) processed all drums containing IDW.

Pavement removed to perform CPTs was patched using a non-shrink, quick-dry grout. If the CPT was located on City of San Jose streets, traffic was restricted from crossing the grouted patch until the Field Engineer determined the grout had set.

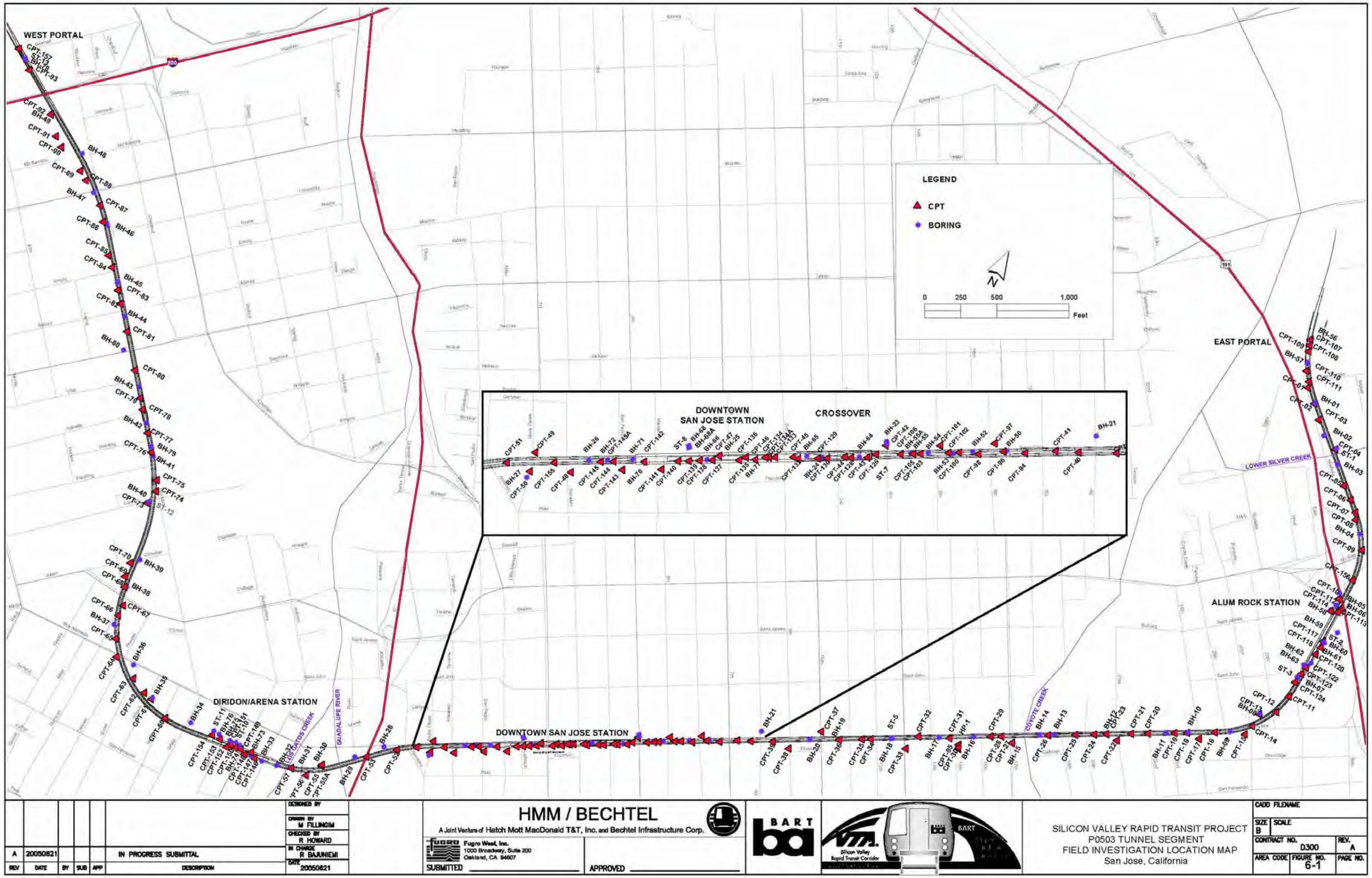


Figure 6-1. Summary of 35% Preliminary Engineering Geotechnical Boring and Cone Penetration Test Locations.

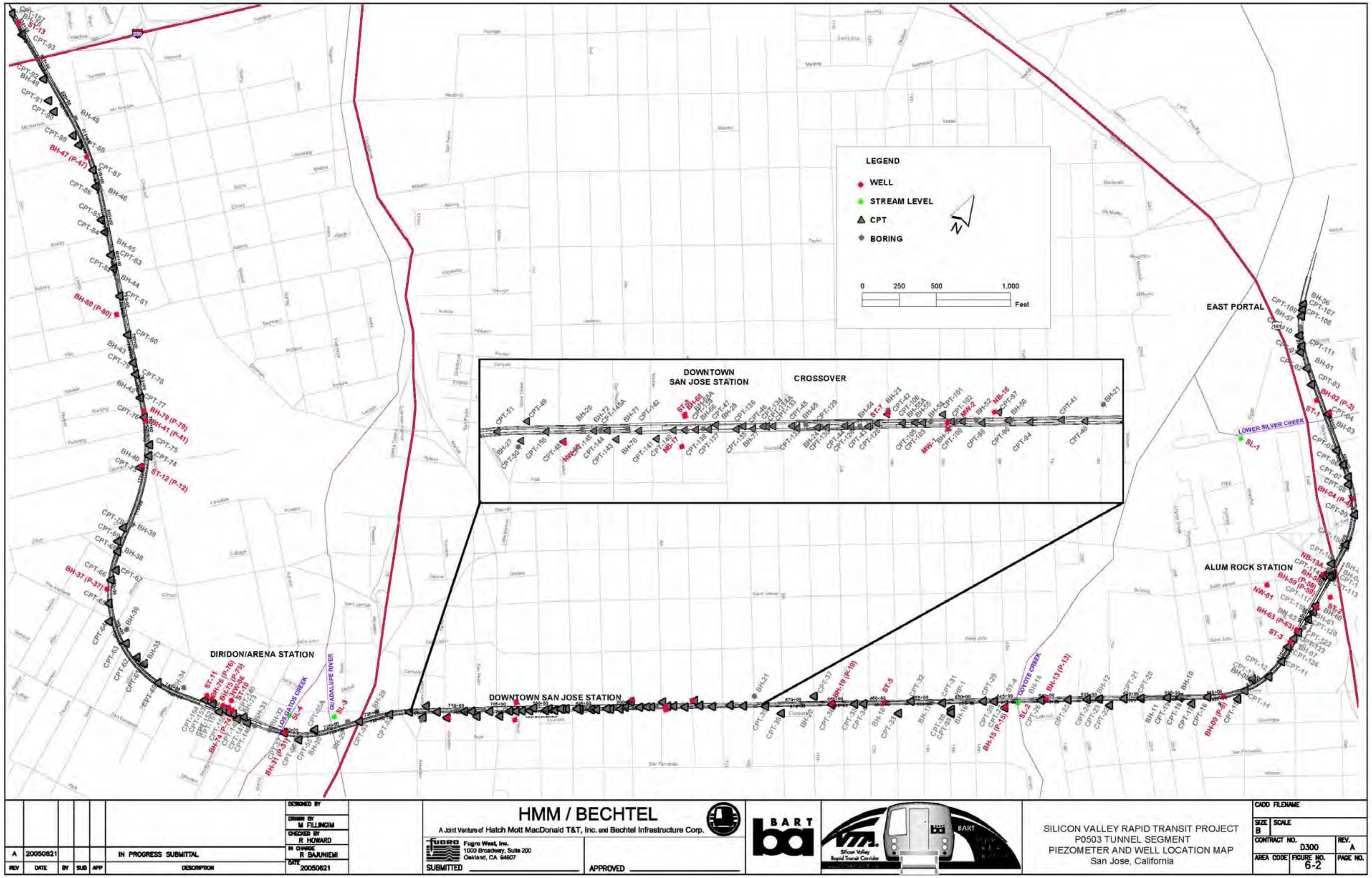


Figure 6-2. Summary of 35% Preliminary Engineering Piezometer and Well Locations.

7.0 Laboratory Investigations

7.1 Introduction: 10% and 35% Designs, Organization

During 10% Conceptual Engineering, laboratory testing was carried out by EarthTech through the Massachusetts Institute of Technology (MIT) Laboratories, Signet Testing Laboratories in Hayward, California, and at the URS Corporation Laboratory in San Jose, California. The testing program consisted of 1) index tests that included Atterberg Limits, gradation analyses, moisture content and density determinations; 2) unconfined compression tests to assist in determining shear strength properties; 3) constant rate of strain (CRS) consolidation tests to determine compressibility characteristics of clays; 4) unconsolidated undrained triaxial compression tests to evaluate undrained shear strengths; and 5) K_0-consolidated undrained direct simple shear tests to evaluate shear strength. The laboratory results from 10% Conceptual Engineering are contained in the Geotechnical Exploration Findings and Recommendations Report – Volume 2: Tunnel and Underground Stations Segment (URS, 2003).

To provide additional laboratory test data, HMM/Bechtel carried out a laboratory test program with samples collected during 35% PE field investigation. The testing was performed by Parikh Consultants Laboratory in Milpitas, California and Fugro's laboratory in Houston, Texas and consisted of index tests, unconfined compression tests, laboratory minivane tests, compressibility tests, and advanced testing to determine stress-history, stress-strain properties, and strength characteristics of the soils. Table 7-1 below lists the appendices where the results of the laboratory investigations are documented.

Table 7-1. Laboratory Testing Program.

	Parikh Consultants	Fugro
Visual Classification	Appendix 12	
Unconfined Compression Tests	Appendix 1	
Moisture Content	Appendix 12	Appendix 13
Unit Weight	Appendix 12	Appendix 13
Specific Gravity		Appendix 13
Sieve Analyses	Appendix 12	
Sieve Analysis and Hydrometer	Appendix 12	
Materials Finer than No. 200 Sieve	Appendix 12	
Atterberg Limits	Appendix 12	Appendix 13
Laboratory Vane Shear	Appendix 1	
X-Ray Radiography		Appendix 20
1-D Constant Rate of Strain (CRS) Consolidation		Appendix 13
Consolidated Drained (CD) Triaxial Compression		Appendix 15
Static Direct Simple Shear		Appendix 14
Ko-Consolidated Undrained Triaxial Compression and Extension		Appendix 16
Ko-Consolidated Undrained Triaxial Compression (Bishop Test)		Appendix 17

Classification and index testing consisted of laboratory visual classification of soil and a suite of tests, including moisture content, unit weight, specific gravity, sieve analyses, sieve and hydrometer analyses, materials finer than No. 200 sieve, and Atterberg Limits.

7.1.1 Laboratory Visual Classification

Laboratory classification of soils was carried out in general accordance with ASTM D2487, Test Method for Classification of Soils for Engineering Purposes, and ASTM D2488, Practice for Description and Identification of Soils (Visual-Manual Procedures). Visual classification of soils collected in undisturbed Shelby Tubes was performed on the soil at the bottom of the Shelby Tube after removing excess disturbed material from the bottom of the tube. It is noted that this methodology does not capture change of soil conditions that may potentially occur within the Shelby Tube unless the soil is extruded for additional testing and/or classification.

Laboratory soil classifications were performed for all samples where recovery was sufficient. Field classifications were adjusted based on laboratory visual classifications and supplemented with laboratory testing. Final classifications appear in the boring logs (Appendix 1), in the Plan and Profiles (Figures 8-1 through 8-50), and in the laboratory classification test summary (Appendix 12) of this report.

Shelby Tube and Pitcher Barrel samples from four (4) continuously sampled borings for proposed stations and eight (8) continuously sampled borings along the proposed tunnel alignment were extruded and visually classified in detail (HMM/Bechtel, 2005f) at Parikh's lab. Pocket penetrometer, torvane, and water content determinations were generally made at approximately 6-inch intervals in the extruded tube samples to better assess stratigraphy and soil property variations at small depth intervals.

7.1.2 Moisture Content

Moisture content testing was assigned to a selected portion of samples from each boring so that a value representing each soil type could be determined. The testing was performed in accordance with ASTM D2216, Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures.

Moisture content tests were conducted within three (3) days of the samples arriving at the laboratory. Shelby Tube samples that were not tested within the three (3) days were sealed with heated microcrystalline wax. Moisture content data appears at the corresponding sample depth in the boring logs (Appendix 1) as well as Parikh Consultant's laboratory report (Appendix 12). Moisture content data are also presented in Chapter 8.

7.1.3 Unit Weight

Unit weight testing was assigned to a selected portion of the samples from each boring so that a value representing each soil type could be determined. The total unit weight was obtained by dividing the weight of the sample by its volume. The weight and volume of the sample were determined by measurement. The dry unit weight of the sample was obtained by heating the sample and measuring the change in weight. This change in weight is used to determine the moisture content. The tests were performed in accordance with U.S. Army Corps of Engineers "Engineer Manual", EM 1110-2-1906 (1970). Dry unit weight data appears at the corresponding sample depth in the boring logs (Appendix 1) as well as Parikh Consultant's laboratory report (Appendix 12). The data are also presented in Chapter 8.

7.1.4 Specific Gravity

Specific gravity testing was assigned to clay samples sent to Fugro's Houston Laboratory for Constant Rate of Strain Consolidation testing. The tests were performed in accordance with ASTM D854, Test Method for specific gravity of Soils. The test results for Specific Gravity can be found in Appendix 13.

7.1.5 Sieve Analysis

At a minimum, three sieve analyses were assigned to borings located along the tunnel alignment. At borings located in the area of the cut-and-cover structures, sieve analyses were assigned within the depths of the excavation and cutoff walls. In addition, extra tests were assigned at borings near water crossings and at locations where granular materials are more abundant.

All sieve analysis tests of soils were carried out in accordance with ASTM D422, Standard Method for Particle-Size Analysis of Soils. As applicable, test results included percentage by weight finer than each of the ASTM Sieves 3 in., 2 in., 1-1/2 in., 1 in., 3/4 in., 1/2 in., 3/8 in., No. 4, No. 10, No. 20, No. 40, No. 60, No. 100, and No. 200 for each sample tested. Test results for sieve analyses in the form of gradation curves (particle size versus percent passing by dry unit weight) can be found in Appendix 12. Also, the fines content determined by the percentage (by weight) of material passing the No. 200 sieve is indicated in the boring logs (Appendix 1). Fines content data are also presented in Chapter 8.

7.1.6 Sieve and Hydrometer Analysis

Combined sieve and hydrometer analyses were performed on a limited number of fine-grained and coarse-grained samples. These tests were performed in accordance with ASTM D422.

The results are presented in a summary table and as gradation curves in Appendix 12. The fines content determined by the percentage of material (by weight) passing the No. 200 sieve is also indicated in the boring logs (Appendix 1). The sieve and hydrometer data are also presented in Chapter 8.

7.1.7 Materials Finer than No. 200 Sieve

The determination of the total amount of material in soils finer than the No. 200 Sieve was performed in accordance with ASTM D1140, Standard Test Method for Amount of Material in Soils Finer Than the No. 200 Sieve. The test results are presented in the boring logs, Appendix 1 as well as on the gradation curves in Appendix 12.

7.1.8 Atterberg Limits

The Liquid Limit, Plastic Limit, and Plasticity Index were determined in accordance with ASTM D4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.

Both Parikh Consultants and Fugro performed Atterberg Limits testing. The test results for each soil sample include values of Liquid Limit (determined from the flow curve), Plastic Limit, and Plasticity Index and are shown on the boring logs (Appendix 1 for Parikh Consultants' data), and figures and tables in Appendices 12 and 13 (for both Parikh Consultants' and Fugro's data). Parikh Consultant's Atterberg Limits data are presented in Chapter 8.

7.2 Specialty Testing

Specialty testing was performed on relatively undisturbed Shelby Tube samples of material classified as clay. The program was designed to evaluate strength, compressibility, stress-strain properties, and stress conditions of the clayey soils along the alignment. The testing included Constant Rate of Strain (CRS) Consolidation, Isotropically-Consolidated Drained Triaxial Compression, Static Direct Simple Shear, K_0-Consolidated Undrained Triaxial Compression and Extension, and K_0-Consolidated Triaxial (Bishop's Procedure).

7.2.1 Shipping and X-ray

All specialty testing was performed on relatively undisturbed Shelby Tube samples collected by Parikh Consultants during the 35% PE field investigation. The sealed Shelby Tubes were shipped to Fugro's laboratory in Houston, Texas in wooden containers that maintained the tubes in a vertical position. The specially-fabricated, padded containers were designed to minimize disturbance of the samples.

All Shelby Tubes received by Fugro's laboratory in Houston were x-rayed to determine the availability and quality of the material inside the Shelby Tubes. Interpretation of soils using x-ray radiographs were performed in accordance with ASTM D4452, Methods for X-Ray Radiography of Soil Samples, with slight modifications that are described in detail in Appendix 20. Images of the x-ray sample radiography are presented in Appendix 20.

7.2.2 Constant Rate of Strain Consolidation Tests

Constant Rate of Strain (CRS) consolidation tests were conducted on clayey specimens to determine the rate and magnitude of consolidation and their stress history. The tests provide data on the load versus strain behavior and the coefficient of consolidation of the soils. The compressibility-related properties that are obtained from CRS consolidation tests are the compression (CR), recompression (RR), and swelling ratios (SR). The rate of consolidation is characterized by the coefficient of consolidation (c_v). The consolidation test data were utilized to estimate pre-consolidation stress using various methods reported in Chapter 8. The pre-consolidation stress determined from these tests provides an indication of the maximum past consolidation pressure that the soil was subjected to in the ground.

All CRS consolidation tests were performed in general accordance with ASTM D 4186. The test specimens were taken from x-rayed thin-wall Shelby Tubes and tested for general index properties such as unit weight, moisture content, and Atterberg Limits. Detailed procedures and results of the CRS consolidation tests are documented in Appendix 13. Chapter 8 presents a summary of the properties discussed above.

7.2.3 Consolidated Drained Triaxial Tests

Isotropically consolidated-drained (IC-D) triaxial compression tests were performed to evaluate the drained strength characteristics, friction angle, and stress-strain relationship of clayey materials encountered in the borings. Generally, two to three specimens from each selected

Shelby Tube sample were tested at different effective consolidation stresses to define a strength envelope.

All IC-D triaxial specimens were performed in general accordance with U.S. Army Corps of Engineer's test standard EM 1110, as well as the proposed ASTM test method currently under development. The test specimens were taken from relatively undisturbed x-rayed Shelby Tubes and tested for general index properties such as unit weight, moisture content, and Atterberg Limits. Detailed procedures and results of the IC-D consolidation tests are described in Appendix 15. Chapter 8 presents a summary of the IC-D test results.

7.2.4 Static Direct Simple Shear Tests

Static Direct Simple Shear (DSS) tests were conducted to measure constant volume (undrained) shear strength and stress-strain characteristics of the clays at depths corresponding to the bottom of cut-and cover station boxes. The specimens were first stressed to a normally-consolidated state and then either relaxed or maintained at an assigned level of over-consolidation prior to applying the constant rate of simple shear deformation.

All DSS tests were performed in general accordance with ASTM D6528. The test specimens were taken from relatively undisturbed x-rayed Shelby Tubes and tested for general index properties such as unit weight, moisture content, and Atterberg Limits. Detailed procedures and test results are described in Appendix 14. The normalized undrained shear strength values, as a function of the over-consolidation ratio, are summarized in Chapter 8.

7.2.5 K_0-Consolidated Undrained Triaxial Compression and Extension Tests

K_0-Consolidated Undrained Triaxial Compression and Extension (CK_0UC and CK_0UE) tests were conducted to estimate the static strength properties and stress-strain characteristics of clays under a range of confining stresses and over-consolidation ratios. The test specimens were first stressed under drained conditions to a normally-consolidated state and then either relaxed to or maintained at the assigned level of over-consolidation. During testing, the specimen diameter is kept constant by adjusting the horizontal confining stress. The load is then applied under undrained conditions in both compression and extension in order to define the anisotropic characteristics of the soil shear strength.

All CK_0UC and CK_0UE tests were performed in general accordance with ASTM D4767-95 with some exceptions as described in Appendix 16. The test specimens were taken from relatively undisturbed x-rayed Shelby Tubes and tested for general index properties such as unit weight, moisture content, and Atterberg Limits. Detailed procedures and results of the CK_0UC and CK_0UE tests are described in Appendix 16. The normalized undrained shear strength values, as a function of the over-consolidation ratio, are summarized in Chapter 8.

7.2.6 K_0-Consolidated Undrained Triaxial Compression (Bishop Method) Tests

K_0-Consolidated Undrained Triaxial Compression tests, henceforth referred to as K_0-Bishop Tests, were conducted on both the upper and lower clays to determine the at-rest lateral earth pressure coefficient (K_0) as a function of the over-consolidation ratio. The tests were performed by initially consolidating the sample to a normally consolidated state, and then unloading and reloading at several values of over-consolidation ratio (OCR) while maintaining zero lateral strain by adjusting the horizontal confining stress. The ratio of horizontal to vertical stress at each OCR was taken as the value of K_0 corresponding to that specific OCR. After completion of K_0 determinations, the sample was reconsolidated to normally consolidated condition and sheared to failure under undrained conditions to determine the shear strength.

K_0-Bishop tests were performed in general accordance with ASTM D4767 with certain variations as described in Appendix 17. The test specimens were taken from relatively undisturbed x-rayed Shelby Tubes and tested for general index properties such as unit weight, moisture content, and Atterberg Limits. Chapter 8 presents a summary of all K_0 (Bishop Method) tests.

7.3 Corrosion Testing

A corrosion testing program is currently underway. Both soil and groundwater samples will be collected and tested for chemical analyses to identify the presence or absence of potentially corrosive substances that could affect underground structures. The results and recommendations of the study will be presented in an addendum to this report.

8.0 Surface and Subsurface Soil Conditions along the Alignment

8.1 Surface Conditions

The tunnel alignment from the East Portal to Alum Rock Station crosses several industrial facilities and extends under HWY 101 and Silver Creek. Alum Rock Station will be constructed off the main streets in a light industrial area where no significant structures are present. From the station to the intersection near 28th Street and Santa Clara Street, the tunnel alignment is planned beneath a residential area consisting primarily of one-story structures. The alignment along Santa Clara Street extends under Coyote Creek between 17th and 18th Streets before reaching downtown San Jose. The Crossover Structure and Downtown San Jose Station will be constructed in a busy section of the Santa Clara Street right-of-way, adjacent to many local business and several high-rise office buildings (up to 18 stories high). The alignment west of Downtown San Jose Station crosses beneath Route 87, the Guadalupe River, and Los Gatos Creek.

The Diridon/Arena Station, planned west of the two stream crossings, will be constructed in a relatively open area. From this station to the West Portal, the alignment underlies a stretch of residential structures along Stockton Avenue, crosses the UPPR tracks at the intersection with University Avenue, and runs parallel to the UPRR tracks from about West Hedding Street to the West Portal.

Along the proposed alignment, the ground surface elevations range from 70 to 95 ft (NAVD88). The ground surface at the western end of the alignment is generally lower than the eastern end.

Appendix 19 presents photographs taken along the proposed alignment from the East to the West Portals.

8.2 Generalized Subsurface Conditions

8.2.1 Geologic Deposits

In general, the two major geologic deposits along the alignment include:

- *Holocene alluvial deposits*. These deposits consist of clay, silty clay, silt, sand, and gravel. Clayey materials are low to high plasticity, medium stiff to very stiff in consistency, and correspond to the Confining Layer, Upper Aquifer, Major Aquitard and Lower Aquifer).

- *Late Pleistocene alluvium*. The composition of these deposits is similar to Holocene alluvium.

The review of (1) available logs from approximately 100 geotechnical borings drilled by others near the proposed alignment, (2) information received from several government agencies, and (3) observations recorded during this investigation, indicate the absence of boulders (particle size greater than 12 inches or 300 mm). Cobbles with a particle size between 3 inches (75 mm) and 12 inches (300 mm) may be present along the alignment and were encountered in the downtown San Jose area and near the West Portal.

8.2.2 Applicable Geotechnical Subsurface Information

Figures 8-1 through 8-25 and Figures 8-26 through 8-50 depict the plan and profile view of the alignment layout with the locations of the boreholes and CPTs from this 35% PE investigation. Profiles with the subsurface stratigraphy from the boring logs or CPT traces are shown below the plan view. Figures 8-1 through 8-25 also present the results of classification tests (moisture content, Atterberg Limits, unit weights, and percent finer than 0.0078 mm size) adjacent to the stick logs. These results are also reported in Appendix 12. Figures 8-26 through 8-50 present strength properties including in-situ undrained shear strength derived from the field tests (vane shear, mini-vane shear torvane and pocket penetrometer tests on soil samples, and penetration resistance from CPTs) reported in Appendices 1 and 2, and secant shear modulus, lateral earth pressure coefficient and effective friction angle derived from the pressuremeter tests (Appendix 3). Boring logs and CPT traces are presented in Appendices 1 and 8, respectively.

The soil type symbols presented on the Logs of Borings differ from the symbols on the profiles in Figures 8-1 through 8-50. Due to software limitations, exact soil type symbols cannot be imported into the Plan and Profile stick logs.

8.2.3 Groundwater Table Information

The groundwater table is typically encountered in either (1) the unconfined surficial layer of sand and silt (Upper Aquifer) or (2) the Upper Confining Layer. Monitoring wells and piezometers have been installed to monitor both of these zones. The depth of the water table varies seasonally and increases slightly in elevation through the spring season between March and May. The low points in the water table occur near Coyote Creek, Guadalupe River, and the West Portal (HMM/Bechtel, 2005d).

Hydrographs for monitoring wells at SCVWD "clean-up" sites show water levels have increased through the 1990's, corresponding to a period of above-average rainfall between 1989 and 1998. Peak seasonal levels over the past two years appear comparable to those in the late 1990s, and appear higher in May 2005 than in previous years. The magnitude of the seasonal fluctuation in the water table varies between 3 and 5 ft. Additional information on ground water table is contained in Chapter 5.

8.2.4 Air and Vapor Monitoring

Throughout drilling of the borings and wells, oxygen readings were typically in the range of 19 to 20 percent. Results are presented at the corresponding depth on the boring logs and slug test well logs in Appendices 1 and 7, respectively.

OVM and LEL readings were typically below the detection levels. One exception was boring BH-30 (employee parking lot of San Jose Water Co.), where a gasoline odor was noticed during drilling from 15 to 20 ft depth. The maximum OVM reading observed was 100 ppm, but levels quickly stabilized to 40 to 60 ppm, and decreased to 0.2 ppm when drilling reached a depth of 23 ft. The soil cuttings and drilling fluid from BH-30 were drummed, labeled and isolated for testing by IWM. Based on the test results, the material was classified as a Non-Hazardous Solid and disposed of at an approved landfill.

Another exception was at slug test well ST-3. A hissing sound was heard after the hollow stem auger had penetrated the Upper Aquifer at ST-3. The Lower Explosive Limit/Oxygen (LEL/O_2) meter registered the oxygen level at 0% when the nozzle (sniffer) of the instrument was placed into the top of the hole. Vapors were also observed releasing from the top of the hole.

To assist in gas classification requirements for future construction of the tunnel, vapor monitoring of the air space, within six existing observation wells that had been installed during the 10% CE program, was performed on December 30, 2004 and February 3, 2005 by Geomatrix Consultants. Because the instrument did not detect the presence of target gases, with the exception of two un-sustained carbon monoxide readings, vapor samples were not collected (refer to Appendix 1).

8.3 Detailed Stratigraphy

For presentation purposes, the Tunnel Segment alignment was split into seven study sections that are defined by project features: three sections corresponding to the stations (Alum Rock, Downtown San Jose, and Diridon/Arena); and four sections corresponding to bored tunnel stretches (East Portal to Alum Rock Station, Alum Rock Station to Crossover, Downtown San Jose Station to Diridon/Arena Station, and Diridon/Arena Station to West Portal).

A detailed description of the stratigraphy and geotechnical properties of the soils along the alignment is presented below for each geotechnical study section, based on the 35% Preliminary Engineering field investigation and laboratory testing program. Descriptions of relative densities (i.e. loose, medium dense, etc.) of granular soils based the correlation of N values from SPTs to relative density (see chart in Figure A1-1) are also presented for each study section below. Global engineering properties (drained and undrained shear strength, compressibility and stress-strain behavior) derived from field and laboratory testing and applicable to the soil conditions along the entire alignments are described separately at the end of Chapter 8.

8.3.1 Geotechnical Study Section 1: East Portal to Alum Rock Station

Study Section 1 extends from about Station 562+00 to about Station 599+00 (Figures 8-1 through 8-3, Figures 8-26 through 8-28). The subsurface profile consists primarily of clayey soils (Upper Clay) to a depth of 60 to 70 ft, with lenses or layers of sand and silt generally less than 10 ft thick. The Upper Aquifer generally extends to depths between 60 and 90 ft below the surface and underlies the Upper Clay. The Lower Clay was encountered below the Upper Aquifer at the location of several boreholes.

Test results, reported in Table 8-1 and presented in Figures 8-51 and 8-52, indicate that most clays are low to medium plasticity, with Liquid Limits from 26 to 61 percent and Plasticity Indices from 11 to 32 percent. The in-situ moisture contents are closer to the Plastic Limits, indicating that the clays are pre-consolidated. Average total and dry unit weights are 124.8 pcf and 100.0 pcf, respectively. The variation of the unit weight with depth is shown in Table 8-2 and Figure 8-53.

The uncorrected penetration blow counts of the granular soils, obtained with the standard split-spoon and with the Modified California sampler, are shown in Table 8-3 and presented in Figure 8-54. These results indicate that the granular soils are medium-dense to very dense and that density generally increases with depth. Typical particle size distribution curves for Study Section 1 are shown in Figure 8-55.

Field undrained shear strengths derived from the CPTs were calibrated against the results of the field vane shear tests that did not meet refusal. The CPTs indicate strengths generally varying from about 1 to 2 ksf in the Upper Clay, typically increasing to 3 ksf or more in the Lower Clay. The high undrained shear strength peaks, shown on the CPT plots, indicate granular materials at the corresponding depths and should not be interpreted as the undrained strength of cohesive materials. Failure modes corresponding to handheld field tests and pressuremeter tests (shown in Figures 8-26 through 8-50) differ, and cannot be directly compared to vane shear/CPT results.

Secant shear moduli and friction angle data presented in Figures 8-26 through 8-50 were taken from pressuremeter data (Table A3-3 and Table 8-23). Table A3-3 also includes qualitative information on the quality of the individual pressuremeter tests.

8.3.2 Geotechnical Study Section 2: Alum Rock Station

Study Section 2 extends from about Station 599+00 to about Station 608+00 (Figures 8-3 through 8-5, Figures 8-28 through 8-30). The subsurface profile consists primarily of clayey soils (Upper Clay) to a depth of about 70 ft, with lenses or layers of sand and silt varying in thickness between less than 1 foot and up to 5 ft thick. The clay is underlain by a granular horizon varying in thickness from a few feet up to 30 ft thick (Upper Aquifer) consisting primarily of sands and silty sands with silt and clay seams. The granular stratum is in turn underlain by clayey soils (Lower Clay) that extend beyond the maximum explored depths.

Test results, reported in Table 8-4 and presented in Figures 8-56 and 8-57, indicate that the fine-grained soils are low to high plasticity with Liquid Limits from 21 to 63 percent and Plasticity Indices from 2 to 33 percent. The in-situ moisture contents are generally closer to the Plastic Limits, indicating that the clays are moderately pre-consolidated. Unit weights, both total and dry, generally increase with depth, as shown in Table 8-5 and Figure 8-58. Average total and dry unit weights of the Upper and Lower Clays are about 125.7 pcf and 101.0 pcf, respectively.

The uncorrected penetration blow counts of the granular soils obtained with the standard split-spoon and with the Modified California sampler are summarized in Table 8-6 and presented in Figure 8-59. These results indicate that the granular soils are medium dense in the upper 20 ft and dense to very dense below a depth of 65 ft. Typical particle size distribution curves for Study Section 2 are shown in Figure 8-60. Test results obtained from samples taken where either loss of drilling fluid or cave-ins in the field was noted, are identified in the figure.

Field undrained shear strengths derived from the CPTs were calibrated against the results of the field vane shear tests that did not meet refusal. The CPTs indicate strengths generally varying from about 1 to 2 ksf in the Upper Clay, typically increasing to 3 ksf or more in the Lower Clay. The high undrained shear strength peaks, shown on the CPT plots, indicate granular materials at the corresponding depths and should not be interpreted as the undrained strength of the clayey materials. Failure modes corresponding to handheld field tests and pressuremeter tests (shown in Figures 8-26 through 8-50) differ, and cannot be directly compared to vane shear/CPT results.

The secant shear moduli and friction angles presented in Figures 8-26 through 8-50 were taken from Table A3-3 and from Table 8-23. Table A3-3 also includes qualitative information on the quality of the individual pressuremeter tests.

8.3.3 Geotechnical Study Section 3: Alum Rock Station to Crossover

Study Section 3 extends from about Station 608+00 to about Station 691+00 (Figures 8-5 through 8-12, Figures 8-30 through 8-37). The subsurface profile consists primarily of clayey soils (Upper Clay) to depths of 50 to 60 ft, with lenses or layers of sand and silt less than about 5 ft thick. The clay is underlain by a granular horizon varying in thickness from a few feet up to 55 ft thick, consisting of primarily sands and silty sands (Upper Aquifer). The granular stratum is in turn underlain by clayey soils (Lower Clay) that extend beyond the maximum explored depths.

Test results, reported in Table 8-7 and presented in Figures 8-61 and 8-62, indicate that the clays are of low to high plasticity with Liquid Limits from 24 to 72 percent and Plasticity Indices from 4 to 44 percent. The in-situ moisture contents are generally closer to the Plastic Limits, indicating that the clays are moderately pre-consolidated. Unit weights, both total and dry, generally increase with depth, as shown in Table 8-8, and Figure 8-63. Average total and dry unit weights of the Upper and Lower Clays are about 124.1 pcf and 98.7 pcf, respectively.

The uncorrected penetration blow counts of the granular soils obtained with the standard split-spoon and with the Modified California sampler are summarized in Table 8-9 and presented in Figure 8-64. These results indicate that the granular soils in the upper 85 ft are loose to very dense, becoming dense to very dense below a depth of 85 ft. Typical particle size distribution curves for Study Section 3 are shown in Figure 8-65. Test results obtained from samples taken where either loss of drilling fluid or cave-ins in the field was noted, are identified in the figure.

Field undrained shear strengths derived from the CPTs were calibrated against the results of the field vane shear tests that did not meet refusal. The CPTs indicate strengths generally varying from about 1 to 2 ksf in the Upper Clay, typically increasing to 3 ksf or more in the Lower Clay. The high undrained shear strength peaks, shown on the CPT plots, indicate granular materials at the corresponding depths and should not be interpreted as the undrained strength of the clayey materials. Failure modes corresponding to handheld field tests and pressuremeter tests (shown in Figures 8-26 through 8-50) differ, and cannot be directly compared to vane shear/CPT results.

The secant shear moduli and friction angles presented in Figures 8-26 through 8-50 were taken from Table A3-3 and from Table 8-23. Table A3-3 also includes qualitative information on the quality of the individual pressuremeter tests.

8.3.4 Geotechnical Study Section 4: Crossover/Downtown San Jose Station

Study Section 4 extends from about Station 691+00 to about Station 707+00 (Figures 8-11 through 8-14, Figures 8-36 through 8-39). The subsurface profile consists primarily of clayey soils (Upper Clay) to depths of 60 to 70 ft, with an abundance of sand and silt layers ranging from a few feet to as thick as 20 ft. Below the Upper Clay is the Upper Aquifer consisting of granular materials with lenses or layers of clayey and silty soils. The Upper Aquifer extends 90 to 100 ft below the surface, at which depth the clayey soils become more abundant (Lower Clay).

Test results, reported in Table 8-10 and presented in Figures 8-66 and 8-67, indicate that the clays are of low to high plasticity with Liquid Limits from 22 to 70 percent and Plasticity Indices from 1 to 42 percent. The in-situ moisture contents are generally closer to the Plastic Limits, indicating that the clays are moderately pre-consolidated. Unit weights, both total and dry, generally increase with depth, as shown in Table 8-11 and Figure 8-68. Average total and dry unit weights of the Upper and Lower Clays are about 124.4 pcf and 100.1 pcf, respectively.

The uncorrected penetration blow counts of the granular soils obtained with the standard split-spoon and with the Modified California sampler are summarized in Table 8-12 and presented in Figure 8-69. The results indicate that the granular soils in the upper 70 ft are loose to very dense, becoming dense to very dense below 70 ft, with the exception of one SPT at a depth of ~115 ft. Typical particle size distribution curves for Study Section 4 are shown in Figure 8-70. Test results obtained from samples taken where either loss of drilling fluid or cave-ins in the field was noted, are identified in the figure.

Field undrained shear strengths derived from the CPTs were calibrated against the results of the field vane shear tests that did not meet refusal. The CPTs indicate strengths generally varying from about 1 to 2 ksf in the Upper Clay, typically increasing to 3 ksf in the Lower Clay. The high undrained shear strength peaks, shown on the CPT plots, indicate granular materials at the test depths and should not be interpreted as the undrained strength of the clayey materials. Failure modes corresponding to handheld field tests and pressuremeter tests (shown in Figures 8-26 through 8-50) differ, and cannot be directly compared to vane shear/CPT results.

The secant shear moduli and friction angles presented in Figures 8-26 through 8-50 were taken from Table A3-3 and Table 8-23. Table A3-3 also includes qualitative information on the quality of the individual pressuremeter tests.

8.3.5 Geotechnical Study Section 5: Downtown San Jose Station to Diridon/Arena Station

Study Section 5 extends from about Station 707+00 to about Station 733+00 (Figures 8-14 through 8-16, Figures 8-39 through 8-41). In general, an abundance of granular lenses and layers are present within the Upper Clay. Below the Upper Clay is the Upper Aquifer, consisting of granular materials with lenses or layers of clayey and silty soils. The aquifer extends 90 to 100 ft below the surface, at which depth the clayey soils become more abundant (Lower Clay).

The test results, reported in Table 8-13 and presented in Figures 8-71 and 8-72, indicate that the clays are of low to medium plasticity with Liquid Limits from 24 to 49 percent and Plasticity Indices from 5 to 24 percent. The in-situ moisture contents are generally closer to the Plastic Limits, indicating that the clays are moderately pre-consolidated. Unit weights, both total and dry, generally increase with depth, as shown in Table 8-14 and Figure 8-73. Average total and dry unit weights of the Upper and Lower Clays are about 128.0 pcf and 107.0 pcf, respectively.

The uncorrected penetration blow counts of the granular soils obtained with the standard split-spoon and with the Modified California sampler are summarized in Table 8-15 and presented in Figure 8-74. The results indicate that the granular soils below 40 ft depth are dense to very dense. Typical particle size distribution curves for Study Section 5 are shown in Figure 8-75.

Test results obtained from samples taken where either loss of drilling fluid or cave-ins in the field was noted, are identified in the figure.

Field undrained shear strengths derived from the CPTs were calibrated against the results of the field vane shear tests that did not meet refusal. The CPTs indicate strengths generally varying from about 1 to 2 ksf in the Upper Clay, typically increasing to 3 ksf in the Lower Clay. The high undrained shear strength peaks, shown on the CPT plots, indicate granular materials at the test depths and should not be interpreted as the undrained strength of the clayey materials. Failure modes corresponding to handheld field tests and pressuremeter tests (shown in Figures 8-26 through 8-50) differ, and cannot be directly compared to vane shear/CPT results.

The secant shear moduli and friction angles presented in Figures 8-26 through 8-50 were taken from Table A3-3, and Table 8-23. Table A3-3 also includes qualitative information on the quality of the individual pressuremeter tests.

8.3.6 Geotechnical Study Section 6: Diridon/Arena Station

Study Section 6 extends from about Station 733+00 to about Station 742+00 (Figures 8-16 through 8-19, Figures 8-41 through 8-44). The Upper Clay generally exists to a depth of about 60 to 70 ft below the ground surface, but contains interspersed lenses of granular materials. The Upper Aquifer underlies the Upper Clay, but is poorly defined along this stretch of the proposed tunnel alignment. The Lower Clay, containing lenses and layers of granular materials, underlies the Upper Aquifer. The presence of granular materials within the Lower Clay is more abundant in the subsurface profile for Study Section 6 than in Study Sections 1 through 5.

Test results, reported in Table 8-16 and presented in Figures 8-76 and 8-77, indicate that the fine-grained soils vary from non-plastic to high plasticity, with Liquid Limits from 11 to 57 percent and Plasticity Indices from 0 to 30 percent. The in-situ moisture contents are generally closer to the Plastic Limits, indicating that the clays are moderately pre-consolidated. Unit weights, both total and dry, generally increase with depth, as shown in Table 8-17, and Figure 8-78. Average total and dry unit weights of the Upper and Lower Clays are about 125.5 pcf and 103.0 pcf, respectively.

The uncorrected penetration blow counts of the granular soils obtained with the standard split-spoon and with the Modified California sampler are summarized in Table 8-18 and shown in Figure 8-79. These results indicate that the granular soils in the upper 70 ft are medium dense to very dense, becoming very dense below a depth of 70 ft. Typical particle size distribution curves for Study Section 6 are shown in Figure 8-80. Test results obtained from samples taken where either loss of drilling fluid or cave-ins in the field was noted, are identified in the figure.

Field undrained shear strengths derived from the CPTs were calibrated against the results of the field vane shear tests that did not meet refusal. The CPTs indicate strengths generally varying from about 1 to 2 ksf in the Upper Clay, typically increasing to 3 ksf in the Lower Clay. The high undrained shear strength peaks, shown on the CPT plots, indicate granular materials at the test depths and should not be interpreted as the undrained strength of the clayey materials. Failure modes corresponding to other field strength tests shown in Figures 8-26 through 8-50 are different. Therefore, their results cannot be directly compared with the results from the vane shear/CPT probes.

The secant shear moduli and friction angles presented in Figures 8-26 through 8-50 were taken from Table A3-3 and Table 8-23. Table A3-3 also includes qualitative information on the quality of the individual pressuremeter tests.

8.3.7 Geotechnical Study Section 7: Diridon/Arena Station to West Portal

Study Section 7 extends from about Station 742+00 to about Station 835+00 (Figures 8-19 through 8-25, Figures 8-44 through 8-50). The thickness of the Upper Clay varies between about 30 ft to about 70 ft along the alignment. The thickness and depth of the Upper Aquifer is poorly defined and interbedded with clayey layers. The aquifer is underlain by the Lower Clay. The presence of granular materials within the Lower Clay in Study Section 7 is more abundant in the subsurface profile for Study Section 7 than in Study Sections 1 through 5.

Test results, reported in Table 8-19 and presented in Figures 8-81 and 8-82, indicate that the plasticity of the cohesive soils varies from low to high plasticity, with Liquid Limits from 27 to 65 percent and Plasticity Indices from 6 to 38 percent. The in-situ moisture contents are generally closer to the Plastic Limits, indicating that the clays are moderately pre-consolidated. Unit weights, both total and dry, generally increase with depth, as shown in Table 8-20 and Figure 8-83. Average total and dry unit weights of the Upper and Lower Clays are about 124.5 pcf and 100.7 pcf, respectively.

The uncorrected penetration blow counts of the granular soils obtained with the standard split-spoon and with the Modified California sampler are summarized in Table 8-21 and shown in Figure 8-84. The results indicate that the density of the granular soils in the upper 20 ft below the ground surface are loose to very dense. Granular soils deeper than 20 ft range from dense to very dense, with the exception of one SPT at a depth of 55 ft. Typical particle size distribution curves for Study Section 7 are shown in Figure 8-85. Test results obtained from samples taken where either loss of drilling fluid or cave-ins in the field was noted, are identified in the figure.

Field undrained shear strengths derived from the CPTs were calibrated against the results of the field vane shear tests that did not meet refusal. The CPTs indicate strengths generally varying from about 1 to 2 ksf in the Upper Clay, typically increasing to 3 ksf or more in the Lower Clay. The high undrained shear strength peaks, shown on the CPT plots, indicate granular materials at the test depths and should not be interpreted as the undrained strength of the clayey materials. Failure modes corresponding to handheld field tests and pressuremeter tests (shown in Figures 8-26 through 8-50) differ, and cannot be directly compared to vane shear/CPT results.

The secant shear moduli and friction angles presented in Figures 8-26 through 8-50 were taken from Table A3-3 and Table 8-23. Table A3-3 also includes qualitative information on the quality of the individual pressuremeter tests.

8.4 Geotechnical Soil Properties

The information presented in the following sections are the result of the 35% Preliminary Engineering investigations described in the previous chapters. Table 8-22 identifies the three main groups in which geotechnical soil properties can be associated: shear strength, compressibility and hydraulic permeability, and stress-strain behavior. The table also lists the various geotechnical properties within each of the above-mentioned categories, and the appendices where the data are presented.

8.4.1 Undrained Shear Strength

The results of field and laboratory tests provide information regarding the undrained shear strength of the clays along the alignment. Field tests that provide undrained shear strength information include vane shear tests, pressuremeter tests, CPTs, and Standard Penetration Tests (SPTs). Laboratory tests providing undrained shear strength information include K_0-consolidated triaxial compression/extension and K_0-consolidated simple shear tests.

8.4.1.1 Field Vane Shear Tests

Figures 8-86 through 8-92 present, for each study section, the undrained shear strength variation with depth obtained using a field vane shear device. The shear resistance of the clays exceeded the capacity of the instrument in many cases (see discussion in Appendix 2). For this reason, the maximum undrained strength and "remolded" strength of the clay could not be obtained at several locations.

To identify the cases where both maximum and remolded strengths could be obtained, both the undisturbed and the remolded strengths are connected in the figure by a straight line. Sensitivity of the clay to disturbance, as defined by the ratio of maximum undrained strength to "remolded" strength, ranges from 1 to 6 for all of the data. Specific values of undrained strengths, "remolded" strengths, and sensitivities can be found in Tables A2-2 through A2-6 in Appendix 2.

8.4.1.2 Pressuremeter Tests

Table 8-23 summarizes the following data obtained by Hughes Insitu Engineering from the pressuremeter tests: undrained shear strength (clays), friction angle (granular soils), initial tangent shear modulus and secant shear modulus, and at-rest earth pressure coefficient. The initial tangent modulus corresponds to the average slope of the initial part of the pressuremeter curves. This modulus, expressed as a Young Modulus, corresponds to the "pressuremeter modulus" defined in Section 9.5 of ASTM D4719. The relationship between shear modulus (G), Young's modulus (E), and Poisson's ration (μ) is $E=2G(1+\mu)$.

As shown in Appendix 3, the undrained shear strength of the clays was determined by comparing the results of the field pressuremeter tests with an ideal "model" pressuremeter curve, based on an assumed set of properties. The material properties required by this model are the shear strength, the lateral stress, and the shear modulus (secant shear modulus from zero strain to the initiation of failure). Adjustments were made to these properties until a mathematical curve was obtained that matched the field data.

The data presented in Table 8-23 are a selection of the values presented in Tables A3-1 and A3-3 (Hughes, 2005) of Appendix 3, and in Figures 8-26 through 8-50. Values presented in Table 8-23 include only those values where the 'model' secant modulus is equal to or within 20% of the value of the unload-reload pressuremeter curves obtained in the field. These moduli are likely to be more representative of the behavior of the in-situ material.

The data in Table 8-23 and in Figure 8-93 show the increase of the undrained shear strength with depth to values in excess of 4 ksf at depths below 100 ft.

8.4.1.3 CPT Undrained Shear Strength Calibration and Results

Appendix 8 documents the calibration of the cone tip resistance to the field undrained shear strength of the clays. Results indicated that an N_k factor of 12 to 15 would provide a good correlation between the undrained strength and the cone tip resistance. The cone-derived undrained shear strengths are presented in the subsurface profiles, Figures 8-26 to 8-50. The strengths shown in these figures correspond to N_k factors between 12 and 15.

8.4.1.4 Triaxial Tests

Compression and extension K_0-consolidated undrained triaxial tests were performed in the laboratory following the SHANSEP methodology developed at MIT (Ladd and Foott, 1974), to define the relationship between over-consolidation ratio (OCR) and normalized undrained shear strength. Results are summarized in Table 8-24 and Figure 8-94 (also presented in Appendix 16, Tables A16-2 through A16-22). The trends and increasing strengths with higher OCRs exhibited by both the triaxial compression and extension curves are in agreement with the trends reported in technical literature for other clays.

8.4.1.5 Laboratory Static Direct Simple Shear Tests

Results from the K_0-consolidated simple shear tests are summarized in Table 8-25 (also presented in Appendix 14, Tables A14-2 and A14-2b), and shown in Figure 8-95. As anticipated, the undrained shear strength under simple shear conditions increases with increasing OCR. This trend is in agreement with data from other clays and with results of the triaxial tests discussed in Section 8.4.1.4. This data, combined with the results of the triaxial K_0 tests, define the anisotropic behavior of the clays.

8.4.2 Effective Shear Strength Properties

8.4.2.1 Pressuremeter Tests

Effective friction angles of granular material obtained from pressuremeter tests are summarized in Table 8-23. These data are plotted against depth in Figure 8-96. Effective friction angle values range from 33 to 35 degrees.

8.4.2.2 Triaxial Tests

Effective friction angles of clays can be obtained from the K_0-consolidated undrained triaxial with pore water pressure measurements (Appendix 16) and from consolidated drained triaxial tests. The results are summarized in Table 8-26 (also presented in Appendix 15, Tables A15-2a through A15-2d). The "p'-q (mean effective normal stress, mean shear stress) at failure" plots presented in Figure 8-97, combine the results of both tests separately for the Upper and Lower Clays.

8.4.2.3 Standard Penetration Test Blow Counts

Uncorrected blow count distributions with depth are individually shown for each study section in Figures 8-54, 8-59, 8-64, 8-69, 8-74, 8-79, and 8-84. By applying appropriate correction factors to the blow counts, such as an energy calibration, the relative density and effective friction angle of the material can be estimated.

8.4.3 Compressibility, Load History and Hydraulic Conductivity

8.4.3.1 Consolidation Tests

Table 8-27 and Figures 8-98 through 8-100 summarize the results of the strain-controlled laboratory consolidation tests. The data in the table are graphically presented as follows: Figure 8-98 presents the general decrease in initial void ratio with depth within the profile; Figure 8-99 presents the relationship between initial void ratio and compression, recompression, and swell ratios; and Figure 8-100 presents the maximum past pressures estimated using the Casagrande, Becker, and Becker Minimum methods.

The void ratio generally decreases with depth, with higher values in the Upper Clay. Correspondingly, compression ratios increase with increasing void ratio. The maximum past pressure values increase with depth and are greater than the current overburden pressures, regardless of the method used (Casagrande, Becker, or Becker Minimum).

8.4.3.2 At-Rest Earth Pressure Coefficient

At-rest earth pressure coefficients from the pressuremeter tests in Table 8-23 are plotted versus depth in Figure 8-101 for fine-grained soils, and in Figure 8-102 for coarse-grained soils. With the exception of four points, all test data from fine-grained soils fall within a range of 0.4 to 0.7. Estimates of the coefficient of at-rest earth pressure were also obtained in the laboratory from Bishop-type K_0-consolidated triaxial tests. Results are reported in Table 8-28 and plotted on Figure 8-103. For OCRs between 2 and 3, the range of at-rest earth pressure coefficients is between 0.4 and 0.9.

8.4.3.3 Coefficient of Hydraulic Conductivity

Horizontal permeability coefficients were obtained from the results of field dissipation tests performed at various locations and depths in selected CPTs. Dissipation test data (Appendix 11) are plotted in Figure 8-104. Results from slug tests (Appendix 7) may also be used to estimate hydraulic conductivity.

8.4.4 Stress-Strain Properties

8.4.4.1 Initial Tangent Shear Modulus

Initial Tangent Shear Modulus (also defined in Appendix 3) is "the average slope of the initial part of the pressuremeter curve". Values in Table 8-23 and in Figures 8-105 and 8-106 summarize the initial tangent shear modulus for fine-grained and coarse-grained soils, respectively. The values of the Young's Modulus can be estimated using the equation presented in Section 8.4.1.2.

8.4.4.2 Secant Modulus

Secant Modulus values are presented in Table 8-23 and are plotted against depth in Figures 8-107 and 8-108 for fine-grained and coarse-grained soils, respectively. As discussed in Section 8.4.1.2, the secant moduli values include only those values where the 'model' secant modulus is equal to or within 20% of the value of the unload-reload pressuremeter curves obtained in the field. These moduli values are likely to be more representative of the behavior of the in-situ material.

8.4.4.3 Small-Strain P- and S- Wave Velocities, Poisson's Ratio (μ)

Shear wave velocity values obtained by field suspension logging methods within boreholes (Appendix 4) and from seismic CPTs (Appendix 9) are presented in Figure 8-109. Down-hole suspension logging was performed at three boring locations along the alignment. Both shear and compression wave velocities were recorded for these tests. Six seismic CPTs yielded good quality seismic shear wave velocity data.

In Figure 8-110, the mean and ± 1 standard deviation of the data obtained from field suspension logging and SCPTs have been superimposed adjacent to shear wave velocity readings obtained by USGS (2004b) at three locations near the proposed alignment. USGS shear wave velocity test data from (1) Coyote Creek Outdoor Classroom (CCOC) located south of Downtown San Jose, (2) Santana Park, San Jose (STPK) located southwest from Diridon/Arena Station, and (3) Guadalupe River, San Jose (GUAD) located north of the West Portal are also shown in this figure.

Figures 8-111 through 8-113 present the three shear and compression wave velocity data sets obtained from field suspension logging within boreholes. These figures also show the value of the calculated Poisson ratio.

Borehole	Depth feet	Moisture Content %	Liquid Limit %	PI %
BH-01	17.5	30.1	42	20
BH-01	27.5	28.8	40	17
BH-01	46.75	16		
BH-01	51.3	9.8		
BH-01	61	11.1		
BH-02	34.8	21.3	27	11
BH-02	47.5	25	26	11
BH-02	52	9.8		
BH-03	32.5	37		
BH-03	37	18		
BH-03	45	26		
BH-03	55	15		
BH-03	65	22		
BH-03	74.8	15		
BH-03	85	24		
BH-03	89.8	27		
BH-04	32.1	40		
BH-04	47.5	22	38	22
BH-04	50.5	41		
BH-04	61	25		
BH-04	76.83	20	28	10
BH-04	81.1	9		
BH-04	91.4	27		
BH-05	37	38		
BH-05	47.5	23		
BH-05	57.5	35		
BH-05	62.5	24		
BH-05	67.5	16		
BH-05	77.5	32		
BH-05	87	25		
BH-06	32.4	34		
BH-06	39.5	41	61	32
BH-06	52.1	19		
BH-06	54.2	26	34	6
BH-06	64	28		
BH-06	77	30		
BH-56	17.25	28.1	45	21
BH-56	32.5	33.5	45	19
BH-57	11.3	26.7		
BH-57	22.2	28.4		
BH-57	32.5	36.3		
BH-57	42.4	23.6		

Table 8-1. Moisture Content and Atterberg Limits, Study Section 1: East Portal to Alum Rock Station.

Borehole	Depth feet	Total Unit Weight pcf	Dry Unit Weight pcf
BH-01	17.5	120.9	92.9
BH-01	27.5	121.2	94.1
BH-01	47	131.1	113.0
BH-02	34.8	132.7	109.4
BH-02	47.5	125.1	100.1
BH-03	32.5	116.2	84.8
BH-03	37	136.1	115.3
BH-03	45	124.7	99.0
BH-03	55	134.8	117.2
BH-03	65	128.2	105.1
BH-03	74.8	127.0	110.4
BH-03	85	125.1	100.9
BH-04	32.1	112.6	80.4
BH-04	47.5	125.8	103.1
BH-04	50.5	111.0	78.7
BH-04	61	127.0	101.6
BH-04	76.83	129.6	108.0
BH-04	81.1	147.4	135.2
BH-04	91.4	125.5	98.8
BH-05	47.5	126.7	103.0
BH-05	62.5	124.0	100.0
BH-05	67.5	136.0	117.2
BH-06	52.1	130.9	110.0
BH-06	54.2	124.1	98.5
BH-56	17.25	121.7	95.0
BH-56	32.5	115.9	86.8
BH-57	11.3	120.9	95.4
BH-57	32.5	117.4	86.1
BH-57	42.4	125.5	101.5

Table 8-2. Total and Dry Unit Weights, Study Section 1: East Portal to Alum Rock Station.

Borehole	Sample Number	Depth feet	Sampler	Soil Type	Blow Count[1,2]	Plotted Blow Count
BH-01	11	50	MOD CAL	Sand	85	85
BH-01	13	60	MOD CAL	Sand	96/11"	100
BH-02	8	50.5	MOD CAL	Gravel	81	81
BH-02	9	54	MOD CAL	Gravel	21	21
BH-03	16	75	SPT	Sand	32	32
BH-04	11	80	MOD CAL	Gravel	78	78

[1]Reported values are blows/ft unless indicated as blows/x"

[2]"Ref/3" (typical) indicates 50 blows drove sampler 3" during initial 6" seating interval (Ref=Refusal)

Table 8-3. Uncorrected SPT and Modified California Blow Counts, Study Section 1: East Portal to Alum Rock Station.

Borehole	Depth feet	Moisture Content %	Liquid Limit %	PI %
BH-58	6.5	22.4		
BH-58	7.5	22.2	29	11
BH-58	9.5	26.6	24	6
BH-58	10	26.6		
BH-58	10.5	27.5		
BH-58	12	28.2	44	18
BH-58	16	22.5		
BH-58	17	21.5	21	6
BH-58	19	18.9		
BH-58	20	30.9	35	11
BH-58	21.5	29.5	31	7
BH-58	22.5	24.8		
BH-58	23.3	26.2		
BH-58	24.3	31.1	39	15
BH-58	26	31.7		
BH-58	26.5		35	9
BH-58	27	26.5	30	8
BH-58	28.5	30.3		
BH-58	29.5	34.4	50	24
BH-58	31.1	25		
BH-58	32		39	12
BH-58	32.1	26.8		
BH-58	33.5	30		
BH-58	34.5	30	38	9
BH-58	36	33.1		
BH-58	37	31.7	37	10
BH-58	38.5	33.9		
BH-58	39.5	35.5	63	31
BH-58	41	40.4		
BH-58	42	31.3	48	21
BH-58	43.5	33		
BH-58	44.5	31.3	51	26
BH-58	46	28.8		
BH-58	47	29.7	47	23
BH-58	48.5	22	45	28
BH-58	49.5	18.2	32	18
BH-58	52	25.5	31	11
BH-58	53.87	24.9		
BH-58	54.5		35	10
BH-58	54.87	26.7		
BH-58	56	23		
BH-58	57	32.4	46	20
BH-58	59.5	29.6	42	
BH-58	61	25.7		15
BH-58	62	28.8	33	
BH-58	63.5	19.9		12
BH-58	64.5	25.9	25	
BH-58	66	23		2
BH-58	67	26.4	35	
BH-58	68.5	28.2		16
BH-58	69.5	21.8	27	
BH-58	91.2	22.9		8
BH-58	110.75	10.4		
BH-58	116.5	24	37	
BH-58	130.5	5.9		14
BH-58	141	10.2		
BH-58	151.5	20		

Borehole	Depth feet	Moisture Content %	Liquid Limit %	PI %
BH-59	30.5	28	44	24
BH-59	60.5	22	30	11
BH-59	68.75	9		
BH-59	79.65	20		
BH-59	90	25		
BH-59	99.5	21		
BH-59	108.75	11		
BH-59	120	24		
BH-59	129.5	14		
BH-59	160.5	22	32	11
BH-59	180.5	24		
BH-59	200.5	21		
BH-60	10	31.3	28	8
BH-60	32.5	24.1	33	10
BH-60	42.5	34	63	33
BH-60	62.5	24	35	15
BH-60	75	23.5	30	11
BH-60	81	23.3		
BH-60	92	26.3	36	11
BH-60	101.3	22.5		
BH-60	109.5	9.1		
BH-60	125.5	20.5		
BH-61	17.5	22.8		
BH-61	37.5	28.8		
BH-61	57.5	24.6		
BH-61	72.5	22.7	34	13
BH-61	91.75	27.4	43	18
BH-61	101.5	21.7		
BH-61	110.5	9.3		
BH-61	115.5	9.2		
BH-61	121.5	29.8		
BH-61	132.5	28.9		
BH-61	142	22.5		
BH-61	151.5	21.1		
BH-62	7.5	18.7		
BH-62	22.5	24.7		
BH-62	37.5	31.8		
BH-62	52.5	21.7		
BH-62	62.5	23.6		
BH-62	71	5		
BH-62	82.2	28.7	40	16
BH-62	92.5	26.8		
BH-62	111.5	9.6		
BH-62	119.8	9.7		
BH-62	130.5	13.5		
BH-62	149.5	22.5		
BH-63	21.5	26.4		
BH-63	42.5	30.2		
BH-63	66.5	12.3		
BH-63	75.5	8.9		
BH-63	91.5	18.7	29	8
BH-63	93	24.6		
BH-63	100.5	25		
BH-63	110.5	23.2		
BH-63	119.5	11.3		
BH-63	124.5	12.3		
BH-63	136.5	23.4		

Table 8-4. Moisture Content and Atterberg Limits, Study Section 2: Alum Rock Station.

Borehole	Depth feet	Total Unit Weight pcf	Dry Unit Weight pcf
BH-58	7.5	121.6	99.5
BH-58	12	123.2	96.1
BH-58	17	132.9	109.4
BH-58	20	130.6	99.8
BH-58	22.5	122.6	98.2
BH-58	24.3	123.9	94.5
BH-58	27	126.6	100.1
BH-58	29.5	121.2	90.2
BH-58	32.1	124.4	98.1
BH-58	34.5	122.5	94.2
BH-58	37	122.9	93.3
BH-58	39.5	116.9	86.3
BH-58	42	121.7	92.7
BH-58	44.5	119.1	90.7
BH-58	47	120.5	92.9
BH-58	49.5	129.3	109.4
BH-58	52	128.3	102.2
BH-58	54.87	122.3	96.5
BH-58	57	121.5	91.8
BH-58	59.5	121.0	93.4
BH-58	62	123.9	96.2
BH-58	64.5	125.5	99.7
BH-58	67	124.3	98.3
BH-58	69.5	127.2	104.4
BH-58	91.2	126.1	102.6
BH-58	111	142.7	129.3
BH-58	116.5	130.1	104.9
BH-58	141	143.7	130.4
BH-58	151.5	132.2	110.2
BH-59	79.65	126.0	105.0
BH-59	90	123.5	98.8
BH-59	99.5	122.5	101.2
BH-59	120	124.6	100.5
BH-59	160.5	128.2	105.1
BH-59	180.5	128.3	103.5
BH-59	200.5	132.4	109.4
BH-60	10	122.4	93.2
BH-60	32.5	123.9	99.8
BH-60	42.5	116.6	87.0
BH-60	62.5	124.0	100.0
BH-60	81	124.5	101.0
BH-60	101.3	128.0	104.5
BH-60	109.5	137.1	125.7
BH-60	125.5	128.0	106.2
BH-61	17.5	126.0	102.6
BH-61	37.5	123.1	95.6
BH-61	57.5	121.6	97.6
BH-61	72.5	128.2	104.5
BH-61	91.75	124.6	97.8
BH-61	101.5	125.6	103.2
BH-61	121.5	123.4	95.1
BH-61	132.5	122.2	94.8
BH-61	142	126.8	103.5
BH-61	151.5	131.2	108.3
BH-62	7.5	123.3	103.9
BH-62	22.5	123.2	98.8
BH-62	37.5	119.1	90.4
BH-62	52.5	125.1	102.8
BH-62	62.5	123.7	100.1
BH-62	92.5	122.4	96.5
BH-62	130.5	134.8	118.8
BH-62	149.5	126.3	103.1
BH-63	21.5	126.0	99.7
BH-63	42.5	120.7	92.7
BH-63	91.5	133.2	112.2

Table 8-5. Total and Dry Unit Weights, Study Section 2: Alum Rock Station.

Borehole	Sample Number	Depth feet	Sampler	Soil Type	Blow Count[1,2]	Plotted Blow Count
BH-58	29	100	MOD CAL	Sand	50/5"	100
BH-58	30	110	MOD CAL	Sand	50/4"	100
BH-58	34	130	MOD CAL	Gravel	55/6"	100
BH-58	35	135	MOD CAL	Sand	70/6"	100
BH-58	36	140	MOD CAL	Gravel	50/6"	100
BH-59	2	18.5	MOD CAL	Gravel	24	24
BH-59	7	68	MOD CAL	Gravel	50/3"	100
BH-59	11	108	MOD CAL	Gravel	60/6"	100
BH-59	14	138	MOD CAL	Gravel	Ref/5"	100
BH-59	15	148	MOD CAL	Gravel	50/3"	100
BH-60	15	108.5	MOD CAL	Gravel	52/6"	100
BH-60	16	118	MOD CAL	Gravel	53	53
BH-61	12	110	MOD CAL	Sand	65/6"	100
BH-61	13	115	MOD CAL	Sand	65/6"	100
BH-62	15	70	MOD CAL	Sand	50/5"	100
BH-62	19	110	MOD CAL	Gravel	Ref/5"	100
BH-62	20	114	MOD CAL	Gravel	50/5"	100
BH-62	21	119	MOD CAL	Gravel	50/4"	100
BH-62	22	124	MOD CAL	Gravel	50/5"	100
BH-63	8	65	SPT	Sand	54	54
BH-63	9	70	SPT	Sand	50/6"	100
BH-63	10	74	SPT	Sand	86	86
BH-63	11	79	SPT	Sand	89	89
BH-63	16	109.1	SPT	Sand	68	68
BH-63	17	114	SPT	Sand	70/8"	100
BH-63	18	119	SPT	Gravel	60/6"	100
BH-63	19	124	SPT	Sand	50/6"	100

[1]Reported values are blows/ft unless indicated as blows/x"

[2]"Ref/3" (typical) indicates 50 blows drove sampler 3" during initial 6" seating interval (Ref=Refusal)

Table 8-6. Uncorrected SPT and Modified California Blow Counts, Study Section 2: Alum Rock Station.

Borehole	Depth feet	Moisture Content %	Liquid Limit %	PI %
BH-07	32.5	32		
BH-07	42.5	34		
BH-07	57.25	25		
BH-07	65.5	15		
BH-07	70.5	14		
BH-07	80.75	6		
BH-08	37.5	31		
BH-08	47.5	30	39	16
BH-08	60	29	39	17
BH-08	76.2	9		
BH-08	81.2	7		
BH-08	85.9	10		
BH-09	52	29.3		
BH-09	62	28.2		
BH-09	66	23.6		
BH-09	71	13.3		
BH-09	75.9	11.7		
BH-09	81	12		
BH-09	86	13.1		
BH-09	91	8.7		
BH-09	96	11.7		
BH-10	59.5	21	36	16
BH-10	66.5	14	25	4
BH-10	69.5	8		
BH-10	79.5	8		
BH-10	90	12		
BH-11	60	23.7		
BH-11	67.5	24.5		
BH-11	75	21.7		
BH-11	78.3	8.7		
BH-11	88.2	17.5		
BH-11	93.5	8.7		
BH-11	99	20.5		
BH-12	32	29.4		
BH-12	37	13.2		
BH-12	42	30.7	35	11
BH-12	47	37.4		
BH-12	52	26.6		
BH-12	57	32.4		
BH-12	66	29	34	11
BH-12	71	25		
BH-12	80	13		
BH-12	86	22.6	33	13
BH-12	95	9		
BH-12	105	11		
BH-12	115	34		
BH-13	76	8		
BH-13	91.3	9		
BH-13	101.5	29		
BH-13	111.5	13		
BH-13	117.5	16		
BH-13	125.5	11		
BH-14	71.3	11		
BH-14	81	11		
BH-14	90.8	10		
BH-14	95.9	9		
BH-14	100.87	8		
BH-14	105.5	12		
BH-14	110.7	13		
BH-14	115.4	11		
BH-14	125.25	7		

Borehole	Depth feet	Moisture Content %	Liquid Limit %	PI %
BH-15	70.8	41.5		
BH-15	73.5	10.6		
BH-15	81	8.8		
BH-15	101	11.8		
BH-15	116	32		
BH-15	123	12.2		
BH-16	57.5	31.6		
BH-16	65	41.6	51	25
BH-16	76	6.9		
BH-16	90.5	11.7		
BH-16	101.5	26.6	33	9
BH-16	112.3	26.1		
BH-16	116.5	25.9		
BH-17	52	32		
BH-17	57.5	20		
BH-17	66	9		
BH-17	82.5	20		
BH-17	92	24		
BH-18	42.5	36		
BH-18	52.5	31	54	27
BH-18	67	7		
BH-18	73.5	21	31	13
BH-18	82.5	22		
BH-18	95.5	23		
BH-19	32	36		
BH-19	42.3	43.9		
BH-19	47	34		
BH-19	61.5	12.1		
BH-19	66	7.3		
BH-19	71.5	23.1		
BH-19	86	6.8		
BH-20	37	45.9	66	36
BH-20	42.5	38.5		
BH-20	49.5	31.4		
BH-20	55.5	8.3		
BH-20	68.5	15.4		
BH-20	73.5	28	31	5
BH-20	81	11		
BH-21	60.5	10		
BH-21	65.5	9.3		
BH-50	3.5	22.9		
BH-50	6.5	22.4	37	15
BH-50	12.5	30.9	38	16
BH-50	16.5	23.2	43	22
BH-50	25	31.8		
BH-50	27.5	31.8		
BH-50	32.5	31.1	35	15
BH-50	37.5	41.2	56	29
BH-50	45	24.8		
BH-50	47.5	26.5		
BH-50	52.5	31.5	36	13
BH-50	57.5	28.7		
BH-50	61.5	23.6		
BH-50	70.5	22.8		
BH-50	80.5	8.5		
BH-50	90	7.7		
BH-50	101.5	23.3	33	11
BH-50	111.5	22.1	27	8
BH-50	120.5	25.1	36	17
BH-50	130.75	24.7		

Borehole	Depth feet	Moisture Content %	Liquid Limit %	PI %
BH-50	140.5	26	35	14
BH-50	150.5	23.4	31	7
BH-52	3.5	22.2		
BH-52	7	21.1		
BH-52	12	30		
BH-52	14.5	20.7		
BH-52	16.2	19		
BH-52	19.5	18	26	9
BH-52	21.5	31.5		
BH-52	24	26.1		
BH-52	26.5	28.3		
BH-52	29	24		
BH-52	31.5	30.3	38	18
BH-52	34	36.8		
BH-52	36.5	42.7	51	26
BH-52	39	39.7	72	44
BH-52	41.5	25.5	46	27
BH-52	44	28.6		
BH-52	46.5	28.4	36	15
BH-52	49	33.2		
BH-52	51.5	26.4		
BH-52	54	21		
BH-52	56.5	27.8	33	11
BH-52	59	25.7		
BH-52	61	27.7		
BH-52	63.7	8.3		
BH-52	65.5	9.8		
BH-52	68.5	19.2		
BH-52	70.5	25.1		
BH-52	75	12.2		
BH-52	81	28.2		
BH-52	85	9.6		
BH-52	90	12.3		
BH-52	94.5	17.9		
BH-52	101	18.5		
BH-52	106	23		
BH-52	111	21.5		
BH-52	116	21.3	33	12
BH-52	121	18.4		
BH-52	130	21.8		
BH-52	141	23.1		
BH-52	150	24.9		
BH-53	4.5	21.1	41	18
BH-53	7	26.8		
BH-53	12	26.7	32	11
BH-53	17	21	36	17
BH-53	22.5	31.8		
BH-53	24.5	33.2	32	6
BH-53	27	24.7	31	9
BH-53	31	32.2		
BH-53	37	58.7	73	38
BH-53	42	23.6	44	24
BH-53	44.5	23.5		
BH-53	47	28.7	33	10
BH-53	52	25		
BH-53	54	23.7	30	6
BH-53	57.5	29.3	34	4
BH-53	62	24.4		
BH-53	66	28		
BH-53	71	19		

Borehole	Depth feet	Moisture Content %	Liquid Limit %	PI %
BH-53	76	27		
BH-53	81	8.1		
BH-53	91	10.9		
BH-53	100	29.4	46	24
BH-53	110	22.1		
BH-53	116	20.3	27	12
BH-53	120	19		
BH-53	125	31.5	37	12
BH-53	131	32.9		
BH-53	136	24.3	41	17
BH-53	141	23.6	23	2
BH-53	146	23.4		
BH-53	148.5	32.1	44	19
BH-54	3.5	24.8		
BH-54	6	29.4		
BH-54	11.5	31.4	29	9
BH-54	17	21.6	30	12
BH-54	22.5	27.8	28	7
BH-54	27.5	22.2		
BH-54	32	27.5	32	12
BH-54	37.5	45.6		
BH-54	42.5	22.3	37	21
BH-54	47.5	28.3		
BH-54	52.5	23.1	39	19
BH-54	57.5	24.5		
BH-54	62.5	19.6	27	11
BH-54	71.5	21		
BH-54	81	8.3		
BH-54	91	25.6	34	15
BH-54	101.2	24.3		
BH-54	111	9.2		
BH-54	120.87	19.1		
BH-55	3.1	19.4		
BH-55	7.5	14.3		
BH-55	12.5	15.6	35	12
BH-55	16.7	21.6	32	14
BH-55	21.5	23.8		
BH-55	26.6	27.2	31	11
BH-55	32.3	34.8		
BH-55	37.5	46.5	69	35
BH-55	42.5	22.2		
BH-55	45	23.8	37	20
BH-55	47.5	29.5	35	12
BH-55	52.5	29.7		
BH-55	57.5	25.9		
BH-55	62.5	19.2	24	7
BH-55	67.5	22.6		
BH-55	71.5	23.2	23	2
BH-55	82.5	9.5		
BH-55	86.8	9.7		
BH-55	90.8	12.8		
BH-55	94.5	9.4		
BH-55	100.5	24.3		
BH-55	111	23		
BH-55	115.9	22.6		
BH-55	120.5	24		
BH-55	125.4	25.8		
BH-55	131.5	21.1		
BH-55	136.5	22.8	39	18
BH-55	141.5	22.6		
BH-55	147.5	19.4		

Table 8-7. Moisture Content and Atterberg Limits, Study Section 3: Alum Rock Station to Crossover/Downtown San Jose Station.

This page intentionally left blank.

Borehole	Depth feet	Total Unit Weight pcf	Dry Unit Weight pcf
BH-07	32.5	119.7	90.7
BH-07	42.5	116.6	87.0
BH-07	57.25	121.1	96.9
BH-07	65.5	136.9	119.0
BH-07	70.5	137.9	121.0
BH-08	37.5	121.0	92.4
BH-08	47.5	118.3	91.0
BH-08	60	120.7	93.6
BH-09	52	120.2	93.0
BH-09	62	121.7	94.9
BH-09	66	124.2	100.5
BH-10	59.5	127.1	105.0
BH-10	66.5	128.8	113.0
BH-11	60	130.4	105.4
BH-11	67.5	128.0	102.8
BH-11	75	131.4	108.0
BH-11	88.2	131.8	112.2
BH-11	99	132.2	109.7
BH-12	32	123.3	95.3
BH-12	37	115.7	102.2
BH-12	42	123.1	94.2
BH-12	47	118.6	86.3
BH-12	52	124.1	98.0
BH-12	57	117.7	88.9
BH-12	66	122.2	94.7
BH-12	69	119.6	95.7
BH-12	86	126.0	102.8
BH-12	114	116.6	87.0
BH-13	111.5	132.8	117.5
BH-13	117.5	137.5	118.5
BH-13	125.5	140.8	126.8
BH-14	100.87	139.9	129.5
BH-16	57.5	118.8	90.3
BH-16	65	112.6	79.5
BH-16	101.5	124.1	98.0
BH-16	112.3	125.8	99.8
BH-17	52	121.0	91.7
BH-17	57.5	129.6	108.0
BH-17	82.5	129.6	108.0
BH-17	92	124.1	100.1
BH-18	52.5	115.9	88.5
BH-18	73.5	127.1	105.0
BH-19	32	117.4	86.3
BH-19	47	116.7	87.1
BH-19	71.5	125.7	102.1
BH-20	37	109.0	74.7
BH-20	42.5	113.6	82.0
BH-20	49.5	116.8	88.9
BH-20	73.5	122.5	95.7
BH-50	6.5	127.4	104.1
BH-50	12.5	120.3	91.9
BH-50	16.5	129.2	104.9
BH-50	25	117.3	89.0
BH-50	27.5	119.9	91.0
BH-50	32.5	118.1	90.1
BH-50	37.5	112.5	79.7
BH-50	45	127.3	102.0
BH-50	47.5	125.0	98.8
BH-50	52.5	118.6	90.2
BH-50	101.5	129.2	104.8
BH-50	111.5	124.4	101.9

Borehole	Depth feet	Total Unit Weight pcf	Dry Unit Weight pcf
BH-52	12	121.8	93.7
BH-52	14.5	131.1	108.6
BH-52	16.2	130.9	110.0
BH-52	19.5	132.0	111.9
BH-52	21.5	135.3	102.9
BH-52	24	124.0	98.3
BH-52	26.5	126.0	98.2
BH-52	29	123.5	99.6
BH-52	31.5	119.5	91.7
BH-52	34	118.1	86.3
BH-52	36.5	112.3	78.7
BH-52	39	112.5	80.5
BH-52	41.5	126.8	101.0
BH-52	44	122.7	95.4
BH-52	46.5	119.8	93.3
BH-52	49	119.6	89.8
BH-52	51.5	126.5	100.1
BH-52	54	130.4	107.8
BH-52	56.5	117.1	91.6
BH-52	59	128.7	102.4
BH-52	106	133.9	108.9
BH-52	111	131.6	108.3
BH-52	116	126.0	103.9
BH-52	121	133.1	112.4
BH-52	141	127.8	103.8
BH-53	4.5	125.0	103.2
BH-53	12	125.6	99.1
BH-53	17	129.1	106.7
BH-53	24.5	117.3	88.1
BH-53	27	124.5	99.8
BH-53	37	104.7	66.0
BH-53	42	124.3	100.6
BH-53	47	120.2	93.4
BH-53	57.5	121.2	93.7
BH-53	62	122.8	98.7
BH-53	116	133.2	110.7
BH-53	141	131.3	106.2
BH-53	148.5	119.2	90.2
BH-54	11.5	124.8	95.0
BH-54	17	131.8	108.4
BH-54	22.5	122.9	96.2
BH-54	27.5	127.0	103.9
BH-54	32	121.1	95.0
BH-54	37.5	109.6	75.3
BH-54	42.5	126.1	103.1
BH-54	47.5	122.3	95.3
BH-54	52.5	123.7	100.5
BH-54	62.5	128.9	107.8
BH-54	71.5	121.0	100.0
BH-55	12.5	124.3	107.5
BH-55	16.7	131.3	108.0
BH-55	26.6	122.2	96.1
BH-55	37.5	112.8	77.0
BH-55	45	125.5	101.4
BH-55	47.5	121.0	93.4
BH-55	62.5	132.9	111.5
BH-55	71.5	127.4	103.4
BH-55	111	124.8	101.5
BH-55	115.9	124.9	101.9
BH-55	131.5	130.2	107.5
BH-55	136.5	127.6	103.9
BH-55	141.5	125.5	102.4

Table 8-8. Total and Dry Unit Weights, Study Section 3: Alum Rock Station to Crossover/Downtown San Jose Station.

This page intentionally left blank.

Borehole	Sample Number	Bottom Depth, ft	Sampler	Soil Type	Blow Count[1,2]	Plotted Blow Count
BH-07	9	65.5	MOD CAL	Sand	Ref/6"	100
BH-07	10	70.5	MOD CAL	Sand	Ref/6"	100
BH-07	11	76	MOD CAL	Sand	50/6"	100
BH-07	12	81	MOD CAL	Gravel	50/5"	100
BH-08	8	70	MOD CAL	Gravel	80	80
BH-08	9	76.5	MOD CAL	Gravel	87	87
BH-08	10	81.5	MOD CAL	Gravel	83	83
BH-08	11	86.5	MOD CAL	Sand	92/11.5"	100
BH-08	12	90.5	MOD CAL	Sand	70/6"	100
BH-09	7	71.5	SPT	Sand	42	42
BH-09	8	76.5	SPT	Sand	46	46
BH-09	9	81.5	SPT	Sand	44	44
BH-09	10	86.5	SPT	Sand	54	54
BH-09	11	91.5	SPT	Sand	77	77
BH-09	12	96.5	SPT	Sand	51	51
BH-10	6	70.5	MOD CAL	Gravel	88	88
BH-10	7	75	MOD CAL	Gravel	50/6"	100
BH-10	8	80	MOD CAL	Gravel	50/3"	100
BH-10	9	85	MOD CAL	Sand	50/4.5"	100
BH-10	10	90	MOD CAL	Sand	50/4"	100
BH-10	13	105.5	MOD CAL	Sand	95/11"	100
BH-11	12	78.5	MOD CAL	Sand	50/6"	100
BH-11	13	81.5	MOD CAL	Gravel	96/11"	100
BH-11	16	88.5	MOD CAL	Sand	50/5"	100
BH-11	17	91.5	MOD CAL	Sand	75	75
BH-11	18	94	MOD CAL	Sand	94/11"	100
BH-12	10	75.5	SPT	Sand	53	53
BH-12	11	80.5	SPT	Sand	53	53
BH-12	13	90.5	SPT	Sand	60	60
BH-12	14	95	SPT	Gravel	50/5.5"	100
BH-12	15	100.5	SPT	Gravel	69	69
BH-12	16	105.5	SPT	Gravel	61	61
BH-12	17	110.5	SPT	Gravel	61	61
BH-13	1	71.5	MOD CAL	Sand	90	90
BH-13	2	76.5	MOD CAL	Gravel	81	81
BH-13	3	81.5	MOD CAL	Sand	36	36
BH-13	4	86.5	MOD CAL	Sand	98/11.5"	100
BH-13	5	91.5	MOD CAL	Gravel	64	64
BH-13	6	96.5	MOD CAL	Sand	90	90
BH-13	9	106	MOD CAL	Sand	50/6"	100
BH-13	10	111.5	MOD CAL	Sand	46	46
BH-13	13	125.5	MOD CAL	Sand	95/6"	100
BH-13	14	131.5	MOD CAL	Sand	90/11.5"	100
BH-14	1	71.5	SPT	Sand	45	45
BH-14	2	76.5	SPT	Sand	61	61
BH-14	3	81.5	SPT	Gravel	39	39
BH-14	4	86.5	SPT	Gravel	54	54
BH-14	5	91.5	MOD CAL	Gravel	57	57
BH-14	6	96.5	MOD CAL	Gravel	99/11"	100
BH-14	7	101.5	MOD CAL	Sand	96/10"	100
BH-14	8	106	MOD CAL	Sand	50/6"	100
BH-14	9	111	MOD CAL	Gravel	50/5.5"	100
BH-14	10	116	MOD CAL	Gravel	50/4"	100
BH-14	11	121.5	MOD CAL	Gravel	75	75
BH-14	12	125.5	MOD CAL	Gravel	Ref/3"	100
BH-14	13	127	SPT	Gravel	91	91
BH-15	2	74	SPT	Sand	71	71
BH-15	3	76.5	SPT	Sand	48	48
BH-15	4	79	SPT	Sand	50	50
BH-15	5	81.5	SPT	Sand	66	66
BH-15	6	84	SPT	Sand	65	65
BH-15	7	86.5	SPT	Sand	64	64
BH-15	8	89	SPT	Sand	40	40
BH-15	9	91	SPT	Sand	50/5"	100
BH-15	10	94	SPT	Gravel	55	55
BH-15	11	96	SPT	Gravel	50/6"	100
BH-15	12	98.5	SPT	Gravel	50/6"	100
BH-15	13	101.5	SPT	Gravel	47	47
BH-15	14	104	SPT	Gravel	80	80
BH-15	17	111.5	SPT	Sand	60	60
BH-15	18	114	SPT	Sand	75	75
BH-15	19	116.5	SPT	Sand	36	36
BH-15	20	118.5	SPT	Sand	50/5"	100
BH-15	21	121	SPT	Sand	50/6"	100
BH-15	22	123.5	SPT	Sand	50/6"	100
BH-15	23	126	SPT	Sand	50/6"	100
BH-15	24	128.5	SPT	Sand	50/6"	100
BH-16	4	71	MOD CAL	Gravel	50/4"	100
BH-16	5	76	MOD CAL	Gravel	50/5"	100
BH-16	6	80.5	MOD CAL	Gravel	Ref/6"	100
BH-16	8	90.8	MOD CAL	Gravel	Ref/6"	100
BH-16	9	95.5	MOD CAL	Gravel	Ref/5"	100
BH-17	5	66.5	SPT	Sand	84	84
BH-17	6	71	SPT	Sand	50/6"	100
BH-18	5	57.5	MOD CAL	Sand	61	61
BH-18	6	67.5	MOD CAL	Gravel	82	82
BH-19	7	61.5	MOD CAL	Sand	88/11"	100
BH-19	8	66.5	MOD CAL	Gravel	79	79
BH-19	10	81.5	MOD CAL	Sand	79	79
BH-19	11	86	MOD CAL	Gravel	50/4"	100
BH-20	11	56	SPT	Gravel	69	69
BH-20	12	59	SPT	Gravel	84	84
BH-20	13	61.5	SPT	Gravel	50	50
BH-20	14	64	SPT	Gravel	49	49
BH-20	15	66.5	SPT	Gravel	13	13
BH-20	16	69	SPT	Gravel	6	6
BH-20	19	78.5	SPT	Sand	50/6"	100
BH-20	20	81.5	SPT	Sand	58	58
BH-20	21	86	SPT	Sand	50/6"	100
BH-20	22	91.5	SPT	Sand	60	60
BH-21	8	56.5	SPT	Sand	49	49
BH-21	9	60.5	SPT	Sand	46	46
BH-21	10	65.5	SPT	Sand	83/11.5"	100
BH-21	12	75	SPT	Sand	50/6"	100
BH-21	13	80	SPT	Gravel	50/6"	100
BH-50	13	61.5	SPT	Sand	77	77
BH-50	15	80.5	SPT	Gravel	59	59
BH-50	16	90.5	SPT	Gravel	68	68
BH-50	20	130.5	SPT	Sand	65	65
BH-52	25	66	SPT	Sand	46	46
BH-52	27	71	SPT	Sand	11	11
BH-52	29	81.5	SPT	Sand	20	20
BH-52	30	85.5	SPT	Sand	50	50
BH-52	31	90.5	SPT	Sand	56	56
BH-52	38	130.5	SPT	Sand	67	67
BH-52	40	150.5	SPT	Sand	45	45
BH-53	17	66.5	SPT	Sand	30	30
BH-53	19	76.5	SPT	Sand	63	63
BH-53	20	81.5	SPT	Sand	68	68
BH-53	21	91.5	SPT	Sand	74	74
BH-53	25	120.5	SPT	Sand	52	52
BH-54	15	81.5	SPT	Sand	52	52
BH-54	17	101.5	SPT	Sand	43	43
BH-54	18	111.5	SPT	Sand	66	66
BH-54	19	121.5	SPT	Sand	45	45
BH-55	17	82.5	SPT	Gravel	52	52
BH-55	18	87.5	SPT	Sand	74	74
BH-55	19	91.5	SPT	Sand	43	43
BH-55	20	95.5	SPT	Gravel	68	68

[1]Reported values are blows/ft unless indicated as blows/x"

[2]"Ref/3" (typical) indicates 50 blows drove sampler 3" during initial 6" seating interval (Ref=Refusal)

Table 8-9. Uncorrected SPT and Modified California Blow Counts, Study Section 3: Alum Rock Station to Crossover/Downtown San Jose Station.

Borehole	Depth feet	Moisture Content %	Liquid Limit %	PI
BH-23	3.5	21.8		
BH-23	11.5	33.9		
BH-23	22	27.6		
BH-23	25	27.2		
BH-23	28	32.1	35	14
BH-23	33	30.6		
BH-23	47	25.7	30	8
BH-23	51	19.8		
BH-23	59.5	25.2		
BH-23	67	21.8		
BH-23	76	13.6		
BH-23	86	7.6		
BH-23	96	21.6		
BH-23	107	20.1		
BH-23	117	24.5		
BH-23	119	15.3		
BH-23	121	23.6		
BH-23	125	9.4		
BH-23	130	22		
BH-24	5	14.7		
BH-24	12	27.8		
BH-24	14.5	24.5	43	24
BH-24	17	29.9	35	19
BH-24	19.5	24.3		
BH-24	22	31.8		
BH-24	24.5	23.1		
BH-24	27	28.1	31	9
BH-24	29.5	29.3		
BH-24	32	26.3	25	1
BH-24	34.5	44		
BH-24	37	28.1		
BH-24	39.5	19.5		
BH-24	40.5	19.2		
BH-24	43.8	12.3		
BH-24	46	8.9		
BH-24	50	25.3		
BH-24	53	24.7		
BH-24	54.5	25.6	29	7
BH-24	57	21.5		
BH-24	59.5	20.9		
BH-24	62	23.4		
BH-24	64.5	23		
BH-24	67	27		
BH-24	70	16.5		
BH-24	81	21.2		
BH-24	90.5	14.5		
BH-24	101	17.5		
BH-24	111	20.8	27	8
BH-24	121	12.7		
BH-24	129.5	7.7		
BH-24	141	18.5	26	5
BH-24	151	24.7		
BH-25	6.5	7.3		
BH-25	18	35.2	38	15
BH-25	32.5	26.5		
BH-25	36.5	39.4		
BH-25	40.6	30.1		
BH-25	41.1	28.5	47	21
BH-25	52.25	9.3		
BH-25	52.75	12.6		
BH-25	56.5	27.5	33	12
BH-25	61.5		24	4
BH-25	61.7	20.9		
BH-25	70.5	21	22	1
BH-25	71	23.1	33	15
BH-25	81.5	12.5		
BH-25	104	27.1	34	16
BH-25	113.5	16.8		
BH-25	121.5	8.3		
BH-25	129.5	26.6		
BH-25	142	9.6		
BH-25	147.5	26.9		
BH-26	27	31.1		
BH-26	32.5	25.2		
BH-26	56.5	12.9		
BH-26	67	22.2		
BH-26	77	25.3		
BH-26	91	26.4		
BH-26	111	20.6		
BH-26	125.5	9.6		
BH-26	147	22.7		
BH-64	4	12.9	26	12
BH-64	12.5	31.5	31	2
BH-64	22.5	27		
BH-64	25	26.5	33	15
BH-64	32.5	37.6	40	15
BH-64	42.5	22.6		
BH-64	45.7	10		
BH-64	52.5	20.7	23	1
BH-64	54.7	21.4		
BH-64	61	25.7		
BH-64	72.5	24.2	35	14
BH-64	74	9.2		
BH-64	80	3.8		
BH-64	84.4	9.6		
BH-64	91	10.1		
BH-64	102.5	26.5		
BH-64	107	24.8	37	16
BH-64	117.5	27.3	38	17
BH-64	122	24.1		
BH-64	126	8.9		
BH-64	131	13.1		
BH-64	137	23	35	15
BH-65	9	29.1	31	7
BH-65	22	25.1		
BH-65	32	27.1	30	10
BH-65	46	9.3		
BH-65	58	22.6		
BH-65	67	21.4		
BH-65	76	11.1		
BH-65	86	11.1		
BH-65	96.3	20.1		
BH-65	109	19.2		
BH-65	114	16		
BH-65	121	22.6		
BH-65	126	14		
BH-65	135	23		
BH-65	147	21.7		
BH-66	4	18.9		
BH-66	6.2	29.2		
BH-66	11.5	10.2		
BH-66	19	36.6		
BH-66	24	26.9		
BH-66	27.5	23.8		
BH-66	32.5	24.7		
BH-66	35.5	17.4		
BH-66	41.5	37.2		
BH-66	47.5	25.5		
BH-66	52.5	25.6		
BH-66	62.5	25.1	35	19
BH-66	72.5	20.2	22	6
BH-66	81	9.5		
BH-66	91	12.9		
BH-66	100	27.4		
BH-66	112.5	24.2	37	18
BH-66	116.2	19.6		
BH-66	120.5	11.5		
BH-66	128.5	19	25	7
BH-68	21	40		
BH-68	29.5	18		
BH-68	71	22		
BH-68	79.5	9		
BH-68	99.5	24.9		
BH-68	119.5	10		
BH-68	151	26		
BH-68	159.5	22		
BH-68	169	8		
BH-68	180	26		
BH-70	8.5	15.8		
BH-70	19	36.3		
BH-70	21.5	33.1	34	6
BH-70	22	17.8		
BH-70	23.5	27.1		
BH-70	24.5	21.2		
BH-70	25	20.5	32	13
BH-70	26.5	23	28	7
BH-70	27.5	19.2		
BH-70	33.5	14.9		
BH-70	38.5	19.1		
BH-70	44	26.7	45	20
BH-70	44.5	24.6		
BH-70	45	23	30	11
BH-70	47	43.8	32	14
BH-70	47.5	26		
BH-70	49.25	25.4		
BH-70	51	22.8		
BH-70	59	27.9		
BH-70	60	20	33	12
BH-70	62	25.4		
BH-70	68.5	18.9		
BH-70	69	19.3	22	5
BH-70	80	11		
BH-70	111	24.5		
BH-70	115.5	26.7		
BH-70	140	40.1		
BH-71	11	9		
BH-71	21	39.3	70	42
BH-71	42	36.2	51	25
BH-71	51.75	8.7		
BH-71	62	25.9	36	16
BH-71	72	18.7		
BH-71	79.8	9.5		
BH-71	99.5	18.4	33	17
BH-71	115.5	20.1		
BH-71	125.5	22.1	28	8
BH-71	135	35.2	25	1
BH-71	147.9	14.3		
BH-72	72.5	25.2		
BH-72	81	10.4		
BH-72	100.3	24.3		
BH-72	111	10.5		
BH-72	121.2	20.2		
BH-72	146	19.5		
BH-72	156.5	24.2		
BH-77	2.5	6.8		
BH-77	7.5	18.4		
BH-77	11	40.8	47	23
BH-77	16.5	26.1	23	NP
BH-77	21	30.7		
BH-77	28	21.1	28	10
BH-77	33	31.2	41	19
BH-77	35.3	23.2		
BH-77	36.5	26.8		
BH-77	40.5	17.8	28	13
BH-77	51	10.9		
BH-77	61.5	24.5		
BH-77	71.7	19.7	30	10
BH-77	81	7.6		
BH-77	91	12.3		
BH-77	102	27	30	9
BH-77	110.5	20.8		
BH-77	116	9.6		
BH-77	121	21.4		
BH-77	126	24.2		
BH-77	132.5	15.5	29	6

Table 8-10. Moisture Content and Atterberg Limits, Study Section 4: Crossover/Downtown San Jose Station.

Borehole	Depth feet	Total Unit Weight pcf	Dry Unit Weight pcf
BH-23	28	94.1	71.2
BH-23	47	128.0	101.8
BH-23	67	126.7	104.0
BH-23	107	128.0	106.6
BH-23	117	124.6	100.1
BH-24	12	123.8	96.9
BH-24	14.5	122.0	98.0
BH-24	17	125.1	96.3
BH-24	19.5	124.5	100.2
BH-24	22	121.1	91.9
BH-24	24.5	129.1	104.9
BH-24	27	121.2	94.6
BH-24	29.5	122.3	94.6
BH-24	32	122.6	97.1
BH-24	34.5	113.3	78.7
BH-24	37	127.6	99.6
BH-24	39.5	131.5	110.0
BH-24	53	126.7	101.6
BH-24	54.5	147.1	117.1
BH-24	57	129.9	106.9
BH-24	59.5	127.3	105.3
BH-24	62	125.3	101.5
BH-24	64.5	126.6	102.9
BH-24	67	125.6	98.9
BH-24	69.5	134.3	115.3
BH-24	110.5	126.8	105.0
BH-24	140.5	128.8	108.7
BH-24	150.5	123.2	98.8
BH-25	18	116.5	86.2
BH-25	71	124.7	101.3
BH-25	113.5	127.4	109.1
BH-25	129.5	120.3	95.0
BH-25	147.5	123.9	97.6
BH-26	27	120.7	92.1
BH-26	67	128.4	105.1
BH-26	77	124.2	99.1
BH-26	147	126.6	103.2
BH-64	12.5	121.5	92.4
BH-64	25	123.5	97.6
BH-64	32.5	120.8	87.8
BH-64	52.5	130.5	108.1
BH-64	72.5	127.2	102.4
BH-64	74	107.1	98.1
BH-64	107.5	125.3	100.4
BH-64	117.5	118.6	93.2
BH-64	138	125.5	102.0
BH-65	9	120.5	93.3
BH-65	22	150.9	120.6
BH-65	32	124.2	97.7
BH-65	58	126.3	103.0
BH-65	67	126.9	104.5
BH-65	109	127.7	107.1
BH-65	121	127.5	104.0
BH-65	135	127.6	103.7
BH-65	147	121.5	99.8
BH-66	24	119.4	94.1
BH-66	27.5	125.3	101.2
BH-66	32.5	124.3	99.7
BH-66	62.5	124.0	99.1
BH-66	72.5	128.6	107.0
BH-66	112.5	125.4	101.0
BH-66	128.5	130.9	110.0
BH-68	21	113.1	80.8
BH-68	29.5	137.1	116.2
BH-68	71	126.4	103.6
BH-68	99.5	126.6	101.4
BH-68	151	122.9	97.5
BH-68	159.5	137.3	112.5
BH-68	180	124.6	98.9
BH-70	8.5	117.9	101.8
BH-70	21.5	115.8	87.0
BH-70	22	107.9	91.6
BH-70	24.5	123.7	102.1
BH-70	25	120.6	100.1
BH-70	26.5	122.5	99.6
BH-70	44.5	124.8	100.2
BH-70	45	125.6	102.1
BH-70	47	136.2	94.7
BH-70	47.5	125.4	99.5
BH-70	49.25	118.0	94.1
BH-70	59	131.1	102.5
BH-70	60	143.7	122.0
BH-70	62	124.4	99.2
BH-70	68.5	128.8	108.3
BH-70	69	132.9	111.4
BH-70	115.5	124.0	97.9
BH-70	140	113.9	81.3
BH-71	21	114.6	82.3
BH-71	42	117.7	86.4
BH-71	62	123.4	98.0
BH-71	72	129.0	108.7
BH-71	125.5	125.2	102.5
BH-72	72	122.7	98.0
BH-72	121.5	123.9	103.1
BH-72	146	131.2	109.8
BH-72	156	124.4	100.2
BH-77	7.5	119.5	100.9
BH-77	16.5	117.7	93.3
BH-77	21	119.9	91.7
BH-77	28	128.0	105.7
BH-77	33	119.8	91.3
BH-77	40.5	130.8	111.0
BH-77	71.7	131.6	109.9
BH-77	102	122.9	96.8
BH-77	126	125.2	100.8
BH-77	132.5	129.7	112.3

Table 8-11. Total and Dry Unit Weights, Study Section 4: Crossover/Downtown San Jose Station.

Borehole	Sample Number	Depth feet	Sampler	Soil Type	Blow Count[1,2]	Plotted Blow Count
BH-23	14	75	SPT	Sand	72	72
BH-23	15	85	SPT	Gravel	55	55
BH-23	20	120	SPT	Sand	47	47
BH-23	21	124	SPT	Gravel	90	90
BH-24	16	45	SPT	Gravel	45	45
BH-24	17	49	SPT	Gravel	10	10
BH-24	27	89	SPT	Sand	60	60
BH-24	31	129	SPT	Sand	Ref/5"	100
BH-25	1	5	SPT	Sand	6	6
BH-25	2	10	SPT	Gravel	17	17
BH-25	12	80	SPT	Sand	34	34
BH-25	13	90	SPT	Sand	49	49
BH-25	19	114.5	SPT	Sand	13	13
BH-25	20	120	SPT	Gravel	58	58
BH-25	25	140.5	SPT	Sand	72	72
BH-26	4	35	SPT	Sand	26	26
BH-26	8	55	SPT	Sand	68	68
BH-26	14	90	SPT	Sand	39	39
BH-26	18	116	SPT	Sand	81/11"	100
BH-26	20	125	SPT	Gravel	50/5"	100
BH-64	7	45	SPT	Sand	47	47
BH-64	10	60	SPT	Sand	34	34
BH-64	14	83	SPT	Sand	32	32
BH-64	15	90	SPT	Sand	70	70
BH-64	21	125	SPT	Gravel	60	60
BH-64	22	130	SPT	Gravel	100	100
BH-65	5	45	SPT	Gravel	56	56
BH-65	8	75	SPT	Sand	36	36
BH-65	9	85	SPT	Sand	40	40
BH-65	10	95	SPT	Sand	39	39
BH-65	12	113	SPT	Sand	40	40
BH-65	14	125	SPT	Sand	47	47
BH-65	15	130	SPT	Sand	43	43
BH-65	19	147.5	SPT	Sand	44	44
BH-66	8	35	SPT	Sand	16	16
BH-66	14	80	SPT	Gravel	68	68
BH-66	15	90	SPT	Sand	45	45
BH-66	19	120	SPT	Sand	Ref/6"	100
BH-66	20	125	SPT	Sand	36	36

Borehole	Sample Number	Depth feet	Sampler	Soil Type	Blow Count[1,2]	Plotted Blow Count
BH-68	5	28.5	MOD CAL	Sand	28	28
BH-68	11	78.5	MOD CAL	Sand	Ref/6"	100
BH-68	15	118.5	MOD CAL	Gravel	50/3"	100
BH-68	19	158.5	MOD CAL	Sand	50/4"	100
BH-68	20	168.5	MOD CAL	Gravel	50/4"	100
BH-70	2	10	SPT	Sand	20	20
BH-70	9	27.5	SPT	Sand	23	23
BH-70	10	30	SPT	Sand	49	49
BH-70	11	32.5	SPT	Sand	34	34
BH-70	12	35	SPT	Sand	30	30
BH-70	13	37.5	SPT	Sand	16	16
BH-70	18	50	SPT	Sand	42	42
BH-70	19	52.5	SPT	Sand	50/5.5"	100
BH-70	26	80	SPT	Gravel	Ref/1.5"	100
BH-70	27	90	SPT	Gravel	Ref/6"	100
BH-70	30	110	SPT	Sand	72	72
BH-70	32	120	SPT	Sand	Ref/4"	100
BH-70	37	145	SPT	Sand	Ref/6"	100
BH-70	38	145.5	SPT	Sand	50/5.5"	100
BH-71	2	5	SPT	Sand	16	16
BH-71	3	10	SPT	Sand	24	24
BH-71	11	50.5	SPT	Sand	72	72
BH-71	15	79	SPT	Sand	76	76
BH-71	16	89	SPT	Sand	40	40
BH-71	18	110	SPT	Sand	95	95
BH-71	19	114	SPT	Sand	44	44
BH-71	20	119	SPT	Sand	97	97
BH-71	25	147	SPT	Sand	64/6"	100
BH-72	2	80	SPT	Gravel	62	62
BH-72	5	110	SPT	Sand	90/10"	100
BH-72	6	114	SPT	Sand	85	85
BH-72	8	124	SPT	Sand	82/11"	100
BH-72	9	129	SPT	Sand	56	56
BH-72	10	134	SPT	Sand	50	50
BH-77	1	2	SPT	Gravel	Ref/5.5"	100
BH-77	11	50	SPT	Sand	34	34
BH-77	14	80	SPT	Gravel	50	50
BH-77	15	90	SPT	Sand	68	68
BH-77	18	115	SPT	Sand	77	77

[1]Reported values are blows/ft unless indicated as blows/x"

[2]"Ref/3" (typical) indicates 50 blows drove sampler 3" during initial 6" seating interval (Ref=Refusal)

Table 8-12. Uncorrected SPT and Modified California Blow Counts, Study Section 4: Crossover/Downtown San Jose Station.

Borehole	Depth feet	Moisture Content %	Liquid Limit %	PI %
BH-27	22.5	20.7		
BH-27	32.5	33.2	49	24
BH-27	41	9		
BH-27	51	11.8		
BH-27	68	7.1		
BH-27	75.5	17.8		
BH-27	131.5	24.3	33	13
BH-28	26	31.9		
BH-28	40.7	10.1		
BH-28	76.5	20.5		
BH-29	61.5	9		
BH-29	65.5	28		
BH-29	70.8	24	32	15
BH-29	76.5	21		
BH-29	79.5	24	24	5
BH-29	86.5	24		
BH-29	95.87	17		
BH-29	102	19		
BH-30	56.5	20.1		
BH-30	59.8	5.6		
BH-30	70	7.7		
BH-30	75.5	21.5	37	22
BH-30	84.5	8		
BH-30	90	6.7		
BH-30	94.5	16.8		
BH-30	105	10.2		
BH-31	40.3	21		
BH-31	51	20.9		
BH-31	55.8	22.4		
BH-31	58.4	11		
BH-31	70.2	22.8		
BH-31	76.5	21.5		
BH-31	80	21.6		
BH-31	86	8.6		
BH-31	89	13.6		
BH-31	94.5	5.4		
BH-32	42.5	25.4		
BH-32	52.3	23.2	27	9
BH-32	61	8.4		
BH-32	66.9	23.4	36	18
BH-32	71.5	17.4		
BH-32	77	22.9		
BH-32	80.5	12.2		
BH-32	92.5	21.1		

Table 8-13. Moisture Content and Atterberg Limits, Study Section 5: Downtown San Jose Station to Diridon/Arena Station.

Borehole	Depth feet	Total Unit Weight pcf	Dry Unit Weight pcf
BH-27	22	131.1	108.6
BH-27	32	121.1	90.9
BH-27	131.5	122.3	98.4
BH-29	76.5	130.4	107.8
BH-29	86.5	129.1	104.1
BH-29	95.87	139.3	119.1
BH-30	56.5	123.7	103.0
BH-30	75.5	126.2	103.9
BH-30	94.5	130.6	111.8
BH-30	105	136.9	124.2
BH-31	55.8	109.3	89.3
BH-31	58.4	135.4	122.0
BH-31	70.2	125.6	102.3
BH-31	80	131.8	108.4
BH-31	89	133.4	117.4
BH-32	42.5	117.0	93.3
BH-32	52.3	125.0	101.5
BH-32	61	148.1	136.6
BH-32	71.5	131.1	111.7
BH-32	77	125.4	102.0
BH-32	80.5	138.3	123.3
BH-32	92.5	125.9	104.0

Table 8-14. Total and Dry Unit Weights, Study Section 5: Downtown San Jose Station to Diridon/Arena Station.

Borehole	Sample Number	Depth feet	Sampler	Soil Type	Blow Count[1,2]	Plotted Blow Count
BH-27	6	40	SPT	Gravel	67	67
BH-27	7	45	SPT	Gravel	69	69
BH-27	8	50	SPT	Sand	48	48
BH-27	12	66.5	SPT	Sand	81	81
BH-27	13	74	SPT	Sand	67	67
BH-27	15	84	SPT	Sand	66	66
BH-27	18	99	SPT	Gravel	80/10"	100
BH-27	19	109	SPT	Sand	Ref/5"	100
BH-28	5	40	SPT	Gravel	34	34
BH-28	6	45	SPT	Sand	46	46
BH-28	16	100	SPT	Sand	50	50
BH-29	3	60	SPT	Sand	65	65
BH-29	6	75	MOD CAL	Sand	74	74
BH-29	8	85	MOD CAL	Sand	34	34
BH-30	3	59	MOD CAL	Gravel	Ref/3"	100
BH-30	N/A	64	MOD CAL	Gravel	Ref/6"	100
BH-30	4	69	MOD CAL	Gravel	50/6"	100
BH-30	7	84	MOD CAL	Gravel	Ref/6"	100
BH-30	8	89	MOD CAL	Gravel	50/4"	100
BH-30	11	104	MOD CAL	Gravel	50/6"	100
BH-31	10	88.5	MOD CAL	Sand	50/5"	100
BH-31	11	93.5	MOD CAL	Sand	71	71
BH-32	7	60	MOD CAL	Gravel	50/5"	100
BH-32	11	80	MOD CAL	Gravel	Ref/6"	100

[1]Reported values are blows/ft unless indicated as blows/x"

[2]"Ref/3" (typical) indicates 50 blows drove sampler 3" during initial 6" seating interval (Ref=Refusal)

Table 8-15. Uncorrected SPT and Modified California Blow Counts, Study Section 5: Downtown San Jose Station to Diridon/Arena Station.

Borehole	Depth feet	Moisture Content %	Liquid Limit %	PI
BH-33	10	22.3		
BH-33	20	19.9		
BH-33	41.5	8.8		
BH-33	54.5	24.7	31	7
BH-33	71	21.3		
BH-33	91	8.8		
BH-33	110	24.5	33	14
BH-33	116	15.6		
BH-33	131.5	20.7		
BH-33	148.2	21.9		
BH-34	12.5	30		
BH-34	32.5	21		
BH-34	52.5	22		
BH-34	80.75	16.1		
BH-34	101	20.1		
BH-34	127	28.4		
BH-34	142.5	24.1		
BH-34	150.8	22.5		
BH-73	7.5	26		
BH-73	27.5	32.4		
BH-73	32.5	10.7		
BH-73	41.5	10.1		
BH-73	56.5	10.7		
BH-73	82	33.8	29	4
BH-73	92.5	22.6		
BH-73	112.5	21.1	22	1
BH-73	116.5	18.7		
BH-73	126.3	14.8		
BH-73	150.5	11.5		
BH-74	6.3	28		
BH-74	9.5	26	40	21
BH-74	10	24		
BH-74	12	19	28	8
BH-74	15	25.1	34	10
BH-74	17	21	24	3
BH-74	19.87	24	30	12
BH-74	22.5	24	28	4
BH-74	24	26		
BH-74	25	27	43	20
BH-74	26.5	33.7	57	25
BH-74	27.5	33	57	30
BH-74	29.2	25	36	17
BH-74	31	24.5		
BH-74	31.5		37	17
BH-74	32	12		
BH-74	34.5	21	27	5
BH-74	37.2	28	36	14
BH-74	39.3	20.7	27	4
BH-74	41.1	21.5		
BH-74	42.1	26	11	0
BH-74	43.3	24.2		
BH-74	44.3	21.6		
BH-74	44.8	21	27	12
BH-74	46.5	19.8		
BH-74	47.5	25	30	10
BH-74	49	7	24	5
BH-74	50	20	20	7
BH-74	51.5	25	30	5
BH-74	52.5	25	34	15
BH-74	53	27.9		
BH-74	54	17.1		
BH-74	55	27	29	6
BH-74	56.3	19.1		
BH-74	57.3	20	34	16
BH-74	59	24.6		
BH-74	60	25	35	15
BH-74	61.5	23.4		
BH-74	62.5	19	30	6
BH-74	63.5	31.3		
BH-74	64		32	4
BH-74	64.5	20		
BH-74	66.88	17		
BH-74	71.5	9		
BH-74	77.5	23	30	13
BH-74	91.75	23		
BH-74	95.75	11		
BH-74	107.5	24		
BH-74	110.5	9		
BH-74	120.5	11		
BH-74	131.5	21		
BH-74	146	19		
BH-74	150.5	9		
BH-75	21	23		
BH-75	40.5	21		
BH-75	51	23		
BH-75	70.5	18		
BH-75	89.5	23		
BH-75	99	9		
BH-75	121	20		
BH-75	128.5	12		
BH-75	141	20		
BH-75	160	21		
BH-75	179	13		
BH-75	200.5	15		
BH-76	20	25.7		
BH-76	39.5	8.3		
BH-76	55.5	22.8		
BH-76	69.4	9.5		
BH-76	87	20.8	28	8
BH-76	111	7.9		
BH-76	117.5	17.1		
BH-76	132.5	20.2		
BH-76	141	22.6		

Table 8-16. Moisture Content and Atterberg Limits, Study Section 6: Diridon/Arena Station.

Borehole	Depth feet	Total Unit Weight pcf	Dry Unit Weight pcf
BH-33	10	119.4	97.6
BH-33	54.5	121.7	97.6
BH-33	71	128.7	106.1
BH-33	91	140.1	128.8
BH-33	110	124.4	99.9
BH-33	131.5	130.6	108.2
BH-34	12.5	119.3	91.8
BH-34	32.5	121.8	100.7
BH-34	52.5	129.2	105.9
BH-34	80.75	135.5	116.7
BH-34	101	126.7	105.5
BH-34	127	121.1	94.3
BH-34	142.5	125.7	101.3
BH-73	7.5	113.0	89.7
BH-73	27.5	117.4	88.7
BH-73	82	120.0	89.7
BH-73	112.5	125.3	103.5
BH-74	9.5	121.0	96.0
BH-74	12	124.4	104.5
BH-74	15	125.1	100.0
BH-74	17	127.3	105.2
BH-74	19.87	125.5	101.2
BH-74	22.5	124.7	100.6
BH-74	24	123.5	98.0
BH-74	25	126.2	99.4
BH-74	26.5	116.9	87.4
BH-74	27.5	115.8	87.1
BH-74	29.2	124.1	99.3
BH-74	31	124.9	100.3
BH-74	32	125.8	112.3
BH-74	34.5	123.5	102.1
BH-74	37.2	119.0	93.0
BH-74	39.3	129.8	107.5
BH-74	42.1	121.2	96.2
BH-74	44.8	128.1	105.9
BH-74	47.5	121.4	97.1
BH-74	50	130.7	108.9
BH-74	52.5	125.6	100.5
BH-74	55	122.7	96.6
BH-74	57.3	129.5	107.9
BH-74	60	123.3	98.6
BH-74	62.5	128.3	107.8
BH-74	64.5	124.2	103.5
BH-74	77.5	127.8	103.9
BH-74	91.75	122.5	99.6
BH-74	107.5	120.7	97.3
BH-74	131.5	117.5	97.1
BH-74	146	134.5	113.0
BH-75	21	128.8	104.7
BH-75	40.5	124.4	102.8
BH-75	89.5	127.4	103.6
BH-75	99	134.3	123.2
BH-75	121	128.5	107.1
BH-75	141	125.3	104.4
BH-75	160	128.4	106.1
BH-76	20	125.2	99.6
BH-76	39.5	131.9	121.8
BH-76	55.5	126.6	103.1
BH-76	69.4	138.4	126.4
BH-76	87	129.0	106.8
BH-76	111	130.3	120.8

Table 8-17. Total and Dry Unit Weights, Study Section 6: Diridon/Arena Station.

Borehole	Sample Number	Depth feet	Sampler	Soil Type	Blow Count[1,2]	Plotted Blow Count
BH-33	4	40	MOD CAL	Sand	103/11"	100
BH-33	10	90	MOD CAL	Sand	100/10.5"	100
BH-33	13	115	MOD CAL	Sand	85	85
BH-33	21	150	MOD CAL	Sand	50/4"	100
BH-34	8	70	MOD CAL	Sand	32	32
BH-34	9	80	MOD CAL	Sand	50/3"	100
BH-34	10	90	MOD CAL	Gravel	50/6"	100
BH-34	11	100	MOD CAL	Sand	50/5"	100
BH-73	6	31	SPT	Gravel	37	37
BH-73	7	35	SPT	Gravel	61	61
BH-73	8	40	SPT	Sand	84	84
BH-73	N/A	45	SPT	Sand	15	15
BH-73	9	50	SPT	Gravel	50/6"	100
BH-73	10	55	SPT	Gravel	77	77
BH-73	17	115	SPT	Sand	72	72
BH-73	24	150	SPT	Sand	65/6"	100
BH-74	27	71	MOD CAL	Gravel	50/6"	100
BH-74	32	95	MOD CAL	Gravel	50/3"	100
BH-74	N/A	100	MOD CAL	Gravel	50/6"	100
BH-74	34	110	MOD CAL	Gravel	90/6"	100
BH-74	35	115	MOD CAL	Sand	55/6"	100
BH-74	36	120	MOD CAL	Sand	50/3"	100
BH-74	38	130	MOD CAL	Sand	87	87
BH-74	40	140	MOD CAL	Gravel	50/4"	100
BH-74	42	150	MOD CAL	Gravel	70/6"	100
BH-75	10	98.5	MOD CAL	Gravel	92/11"	100
BH-75	13	128.5	MOD CAL	Sand	100/8"	100
BH-75	18	178.5	MOD CAL	Sand	100/8"	100
BH-76	4	38.5	MOD CAL	Sand	53	53
BH-76	8	68.5	MOD CAL	Gravel	72	72
BH-76	13	110	MOD CAL	Gravel	56/6"	100
BH-76	18	131.5	MOD CAL	Sand	59/6"	100

[1]Reported values are blows/ft unless indicated as blows/x"

[2]"Ref/3" (typical) indicates 50 blows drove sampler 3" during initial 6" seating interval (Ref=Refusal)

Table 8-18. Uncorrected SPT and Modified California Blow Counts, Study Section 6: Diridon/Arena Station.

Borehole	Depth feet	Moisture Content %	Liquid Limit %	PI
BH-35	24.5	28.4		
BH-35	37	22.2		
BH-35	42	24.7		
BH-35	49	25		
BH-35	59.5	24.9		
BH-35	67	20.4		
BH-35	73	9.7		
BH-36	32	28.2	50	31
BH-36	42	25.7	32	13
BH-36	51	9.1		
BH-36	62	16.5		
BH-36	67	24	37	18
BH-36	80.5	9.5		
BH-37	31.5	23.4		
BH-37	36	8.4		
BH-37	42	24.7		
BH-37	52	24.5		
BH-37	56	23.6		
BH-37	66	21.9		
BH-37	77	23.4		
BH-38	31.5	28	45	27
BH-38	40	14.7		
BH-38	55.5	21.8		
BH-38	71.5	20.6	33	16
BH-38	78.8	24.4		
BH-38	95	20.1	27	6
BH-39	39.4	21.1		
BH-39	54.5	16		
BH-39	64.25	7.3		
BH-39	81	24.8		
BH-39	91	25		
BH-40	14	39.4		
BH-40	24	37.6	54	32
BH-40	32	28.5		
BH-40	42	25.5	27	6
BH-40	52	26.9		
BH-40	59.5	23.9		
BH-40	66	15.5		
BH-41	17	40.2		
BH-41	32	22.7		
BH-41	41	21.2		
BH-41	46	15.3		
BH-41	59.5	9		
BH-42	6	15.9		
BH-42	27.5	28.5		
BH-42	37.5	28.3		
BH-42	42.3	17.8		
BH-42	45.9	11.4		
BH-42	51	26.5	38	20
BH-42	62.5	20	31	15
BH-43	16.9	36.4		
BH-43	22.5	23.7		
BH-43	27	37.6		
BH-43	29.5	22.3		
BH-43	33.5	7.7		
BH-43	40.5	10.1		
BH-43	48	11.8		
BH-43	59.5	27.8		
BH-44	11	23.1		
BH-44	17	32		
BH-44	32	26.1		
BH-44	39.5	23.8		
BH-44	44.5	7.8		
BH-44	51	8.6		
BH-45	31.5	26		
BH-45	41.5	26		
BH-45	54.5	15		
BH-45	76.2	21		
BH-45	80.3	17		
BH-46	14	29.3		
BH-46	22	29.9	42	20
BH-46	27	40.6		
BH-46	31	13		
BH-46	41	10.9		
BH-46	46	11.6		
BH-46	52	29.6	36	11
BH-46	59.5	24.6		
BH-47	3.1	15		
BH-47	6	8		
BH-47	20	30		
BH-47	35.9	13		
BH-47	45.75	11		
BH-47	57.5	29		
BH-48	40	19.2	29	11
BH-48	44	24.3		
BH-48	54	22.5		
BH-48	66.5	9.7		
BH-48	70.8	11.9		
BH-48	77.5	23.9	38	20
BH-49	26.5	24.3		
BH-49	30.5	8		
BH-49	47	17.3		
BH-49	52	21.7		
BH-49	56.5	25.2		
BH-78	6.5	43.1		
BH-78	17.5	34.9		
BH-78	31.2	9		
BH-78	41.2	19.7		
BH-78	51.2	20.7		
BH-78	61	11.6		
BH-78	72.5	24.6		
BH-78	75.75	9.6		
BH-79	22.5	37.7	65	38
BH-79	41.5	18.9		
BH-79	62.5	25.3	33	16
BH-79	82.5	23.2	35	18
BH-79	112	20.5		
BH-79	132.5	24.8	32	9
BH-79	150.5	22.4		
BH-79	160.5	9.3		
BH-79	180.5	11.6		
BH-79	200.5	10.9		
BH-80	40.5	19.7		
BH-80	45.5	8.9		
BH-80	50.5	24.2	29	8
BH-80	55	27.6		
BH-80	69	22.7	29	9
BH-80	74.5	25.8		
BH-80	86.5	9.4		

Table 8-19. Moisture Content and Atterberg Limits, Study Section 7: Diridon/Arena Station to West Portal.

Borehole	Depth feet	Total Unit Weight pcf	Dry Unit Weight pcf
BH-35	24.5	119.5	93.1
BH-35	37	125.3	102.5
BH-35	42	126.4	101.4
BH-35	49	125.3	100.2
BH-35	59.5	125.5	100.5
BH-35	67	129.7	107.7
BH-36	32	119.4	93.1
BH-36	42	124.2	98.8
BH-36	62	130.5	112.0
BH-36	67	123.4	99.5
BH-37	31.5	127.2	103.1
BH-37	42	149.3	119.7
BH-37	52	124.4	99.9
BH-37	77	126.0	102.1
BH-38	31.5	120.6	94.2
BH-38	71.5	127.6	105.8
BH-38	78.8	122.8	98.7
BH-38	95	131.1	109.2
BH-39	81	124.1	99.4
BH-39	91	127.0	101.6
BH-40	14	113.1	81.1
BH-40	24	113.9	82.8
BH-40	32	120.4	93.7
BH-40	42	122.7	97.8
BH-40	52	119.9	94.5
BH-40	59.5	122.8	99.1
BH-41	17	114.1	81.4
BH-41	32	126.5	103.1
BH-41	41	127.0	104.8
BH-42	6	132.9	114.7
BH-42	37.5	120.3	93.8
BH-42	42.3	128.9	109.4
BH-42	51	117.4	92.8
BH-42	62.5	128.5	107.1
BH-43	16.5	113.9	83.5
BH-43	22	123.2	99.6
BH-43	27	114.9	83.5
BH-43	29.5	123.4	100.9
BH-43	59.5	120.9	94.6

Borehole	Depth feet	Total Unit Weight pcf	Dry Unit Weight pcf
BH-44	17	119.5	90.5
BH-44	32	119.4	94.7
BH-44	39.5	126.2	101.9
BH-45	41.5	122.3	97.1
BH-45	54.5	130.9	113.8
BH-45	76.2	120.8	99.8
BH-45	80.3	133.8	114.4
BH-46	14	118.3	91.5
BH-46	22	118.3	91.1
BH-46	27	113.7	80.9
BH-46	52	119.0	91.8
BH-46	59.5	123.7	99.3
BH-47	3.1	132.7	115.4
BH-47	6	124.2	115.0
BH-47	20	121.2	93.2
BH-47	35.9	135.0	119.5
BH-47	45.75	142.5	128.4
BH-47	57.5	137.3	106.4
BH-48	40	129.9	109.0
BH-48	77.5	124.5	100.5
BH-49	26.5	123.2	99.1
BH-49	47	132.9	113.3
BH-49	52	115.6	95.0
BH-49	56.5	126.0	100.6
BH-79	22.5	114.4	83.1
BH-79	41.5	126.7	106.6
BH-79	62.5	124.9	99.7
BH-79	82.5	126.6	102.8
BH-79	112	127.9	106.1
BH-79	132.5	124.9	100.1
BH-80	40.5	125.8	105.1
BH-80	50.5	126.1	101.5
BH-80	55	122.8	96.2
BH-80	69	127.1	103.6
BH-80	86.5	144.6	132.2

Table 8-20. Total and Dry Unit Weights, Study Section 7: Diridon/Arena Station to West Portal.

Borehole	Sample Number	Bottom Depth, ft	Sampler	Soil Type	Blow Count[1,2]	Plotted Blow Count
BH-35	19	73	SPT	Gravel	Ref/6"	100
BH-35	20	75.5	SPT	Gravel	Ref/6"	100
BH-35	21	78	SPT	Gravel	Ref/5.5"	100
BH-36	7	51.5	SPT	Sand	80.0	80
BH-36	13	81	SPT	Gravel	50/6"	100
BH-37	4	36.5	SPT	Sand	76.0	76
BH-37	9	61	SPT	Sand	50/6"	100
BH-37	10	66.5	SPT	Sand	46.0	46
BH-37	11	71.5	SPT	Sand	46.0	46
BH-38	5	40.5	MOD CAL	Gravel	58.0	58
BH-38	6	47	MOD CAL	Gravel	33.0	33
BH-38	9	52.5	MOD CAL	Sand	55.0	55
BH-38	10	55.5	MOD CAL	Sand	82.0	82
BH-38	12	66.5	MOD CAL	Sand	90/9"	100
BH-38	16	82.5	MOD CAL	Sand	80/11"	100
BH-38	17	86	MOD CAL	Sand	50/5.5"	100
BH-38	18	90.5	MOD CAL	Sand	Ref/6"	100
BH-39	2	40	MOD CAL	Sand	67.0	67
BH-39	4	49.5	MOD CAL	Gravel	50/4"	100
BH-39	5	54.5	MOD CAL	Gravel	50/6"	100
BH-39	7	64.8	MOD CAL	Gravel	50/3"	100
BH-40	20	66	SPT	Sand	50/4"	100
BH-40	21	68.5	SPT	Sand	50/4"	100
BH-41	9	46.5	SPT	Sand	68.0	68
BH-41	11	56	SPT	Sand	50/6"	100
BH-41	12	60	SPT	Sand	60.0	60
BH-42	2	6.5	MOD CAL	Sand	14.0	14
BH-42	9	46	MOD CAL	Gravel	50/4"	100
BH-43	12	34	SPT	Gravel	79.0	79
BH-43	13	36.5	SPT	Gravel	51.0	51
BH-43	14	39	SPT	Gravel	57.0	57
BH-43	15	41.5	SPT	Gravel	44.0	44
BH-43	16	44	SPT	Gravel	45.0	45
BH-43	17	46.5	SPT	Gravel	73.0	73
BH-43	18	49	SPT	Sand	61.0	61
BH-44	3	11.5	SPT	Sand	8.0	8
BH-44	10	45	SPT	Gravel	63.0	63
BH-44	11	46.5	SPT	Gravel	77.0	77
BH-44	12	51.5	SPT	Gravel	60.0	60
BH-44	13	59.5	SPT	Sand	67.0	67
BH-44	14	61.5	SPT	Sand	70.0	70
BH-45	6	52	MOD CAL	Sand	50/4.5"	100
BH-45	7	54.5	MOD CAL	Sand	50/6"	100
BH-45	9	64.5	MOD CAL	Gravel	34.0	34

Borehole	Sample Number	Bottom Depth, ft	Sampler	Soil Type	Blow Count[1,2]	Plotted Blow Count
BH-45	11	76.5	MOD CAL	Sand	61.0	61
BH-45	12	80.5	MOD CAL	Sand	70.0	70
BH-46	3	11.5	SPT	Sand	19.0	19
BH-46	11	31.5	SPT	Sand	60.0	60
BH-46	12	36.5	SPT	Sand	52.0	52
BH-46	13	39	SPT	Sand	57.0	57
BH-46	14	41.5	SPT	Sand	90.0	90
BH-46	15	44	SPT	Gravel	12.0	12
BH-46	16	46.5	SPT	Gravel	78.0	78
BH-47	2	6.5	MOD CAL	Sand	35.0	35
BH-47	7	31.5	MOD CAL	Gravel	94.0	94
BH-47	8	36	MOD CAL	Sand	50/5"	100
BH-47	9	41.5	MOD CAL	Sand	84.0	84
BH-47	10	46.5	MOD CAL	Sand	98/11"	100
BH-48	1	16.5	MOD CAL	Sand	20.0	20
BH-48	9	66.5	MOD CAL	Sand	94/11.5"	100
BH-48	10	71.5	MOD CAL	Sand	35.0	35
BH-48	13	86.5	MOD CAL	Sand	82.0	82
BH-49	4	30.5	MOD CAL	Gravel	Ref/6"	100
BH-78	1	2.5	MOD CAL	Sand	20.0	20
BH-78	6	26.5	MOD CAL	Gravel	43.0	43
BH-78	7	31.5	MOD CAL	Gravel	71.0	71
BH-78	8	37	MOD CAL	Gravel	50/6"	100
BH-78	9	41.5	MOD CAL	Sand	42.0	42
BH-78	12	56	MOD CAL	Gravel	50/5"	100
BH-78	13	61.5	MOD CAL	Sand	65.0	65
BH-78	16	76	MOD CAL	Gravel	50/6"	100
BH-78	17	81	MOD CAL	Sand	50/3"	100
BH-79	10	101	MOD CAL	Sand	50/6"	100
BH-79	16	151.5	MOD CAL	Sand	50/3"	100
BH-79	17	160.5	MOD CAL	Gravel	65/6"	100
BH-79	19	180.5	MOD CAL	Sand	65/6"	100
BH-79	21	200.5	MOD CAL	Sand	Ref/6"	100
BH-80	1	6.5	MOD CAL	Sand	52.0	52
BH-80	2	10.5	MOD CAL	Gravel	93/11"	100
BH-80	9	45.5	MOD CAL	Gravel	67.0	67
BH-80	12	60	MOD CAL	Sand	50/5"	100
BH-80	13	65.5	MOD CAL	Sand	87/11"	100
BH-80	16	81.5	MOD CAL	Sand	71.0	71
BH-80	17	86.5	MOD CAL	Sand	91/11"	100
BH-80	18	91	MOD CAL	Sand	50/4"	100
BH-80	19	96.5	MOD CAL	Sand	33.0	33

[1]Reported values are blows/ft unless indicated as blows/x"

[2]"Ref/3" (typical) indicates 50 blows drove sampler 3" during initial 6" seating interval (Ref=Refusal)

Table 8-21. Uncorrected SPT and Modified California Blow Counts, Study Section 7: Diridon/Arena Station to West Portal.

1 - SHEAR STRENGTH

Material	Soil Parameter	Source (Appendix) Field Tests		Laboratory	
Clays	Undrained Shear Strength	Vane Shear	(A-2)	K_0 - Triaxial	(A-16)
	Undrained Shear Strength	CPT	(A-8)	Simple Shear	(A-14)
	Undrained Shear Strength	PMT	(A-3)		
Clays	Effective Friction Angle	PMT	(A-3)	CD - Triaxial	(A-15)
Granular	Effective Friction Angle	PMT	(A-3)		
		SPT	(A-1)		
2 - COMPRESSIBILITY AND HYDRAULIC CONDUCTIVITY					
Clays	1-D Compression and Permeability	Dissipation Tests	(A-11)	1-D Consolidation Tests	(A-13)
		Slug Tests	(A-7)		
Clays	At-rest K_0 Coefficient	PMT	(A-3)	Bishop-type Triaxial Tests	(A-17)
Granular	At-rest K_0 Coefficient	PMT	(A-3)		
3 - STRESS-STRAIN BEHAVIOR					
Clays	Initial Tangent Modulus	PMT	(A-3)	Triaxial Tests (clays only)	(A-15), (A-16)
&	Secant Modulus	PMT	(A-3)	Triaxial Tests (clays only)	(A-15), (A-16)
Granular	Low-Strain Modulus	Down-hole	(A-4)		
		CPT	(A-8)		
	Poisson's Ratio	Down-hole	(A-4)		

Table 8-22. Engineering Properties of Soils Along Tunnel Alignment: Reference List.

Test Number	Borehole	Depth feet	Material Type[1]	Undrained Shear Strength (Su) ksf	Effective Friction Angle Degree	Initial Tangent Modulus (GI) ksf	Secant Modulus (Gsec) ksf	Ko
1	BH-55	25	coh	0.72	0	18.72	144	0.5
3	BH-55	45	coh	2.016	0	46.08	216	0.6
19	BH-64	25	coh	1.152	0	43.2	216	0.5
20	BH-64	23.5	fri	2.016	34	57.6	288	0.6
21	BH-64	54.5	coh	3.744	34	108	360	0.6
22	BH-64	53	fri	2.304	34	72	576	0.5
23	BH-64	74	fri	3.456	34	79.2	576	0.5
24	BH-71	25	fri	2.016	0	28.8	144	1.3
25	BH-71	23.5	fri	1.296	34	20.16	72	0.5
27	BH-71	43.5	coh	1.728	0	20.16	86.4	0.5
28	BH-71	65	coh	2.304	0	25.92	100.8	0.6
30	BH-25	21	coh	1.44	0	86.4	316.8	0.6
31	BH-25	23	coh	2.16	0	144	360	0.6
33	BH-25	50	coh	2.448	0	201.6	316.8	0.5
34	BH-25	74	coh	3.168	0	144	288	0.6
36	BH-25	107	fri	6.336	34	216	864	0.6
37	BH-25	105.5	fri	7.92	34	432	2160	0.7
48	BH-25	114.5	coh	3.6	0	115.2	388.8	0.5
49	BH-25	113	fri	5.04	0	115.2	518.4	0.6
50	BH-25	129	coh	5.04	34	95.04	302.4	0.6
52	BH-25	150	coh	7.92	34	136.8	388.8	0.6
53	BH-25	148.5	coh	6.48	34	141.12	432	0.5
38	BH-65	13	coh	0.432	0	24.48	57.6	0.4
40	BH-65	38	coh	1.008	0	54.72	72	0.5
41	BH-65	40	coh	1.296	0	100.8	172.8	0.8
42	BH-65	54	fri	2.592	0	216	1296	0.6
46	BH-65	111.5	fri	3.888	34	115.2	288	0.5
104	BH-18	86	fri	0	35	141.12	720	0.5
111	BH-42	33	coh	1.728	0	64.8	288	0.5
119	BH-38	80	fri	4.32	35	57.6	288	0
121	BH-02	39	fri	2.304	35	28.8	216	0.5
122	BH-02	50	fri	5.76	35	201.6	1152	0.6
124	BH-02	60	fri	0	35	216	864	0.5
125	BH-02	58.5	fri	4.32	33	100.8	1008	0.7
129	BH-45	58.5	gra	11.52	35	1008	5760	1
130	BH-45	70	fri	3.6	35	187.2	576	0.5
132	BH-06	44	coh	2.16	0	43.2	115.2	0.7
133	BH-06	46	coh	2.304	0	50.4	115.2	0.7
135	BH-06	65	fri	6.48	0	216	1152	0.5
136	BH-06	63.5	coh	2.304	0	72	432	0.3
137	BH-33	13	coh	3.744	0	158.4	576	0.6
139	BH-33	23	coh	2.448	0	129.6	576	0.5
140	BH-33	25	coh	1.44	0	129.6	360	0.5
141	BH-33	45	cem	14.4	35	576	8640	1.4
144	BH-33	74.5	fri	2.592	0	72	576	0.4
145	BH-33	90	fri	4.032	35	129.6	576	0.5
147	BH-33	115	fri	4.32	33	216	720	0.5
149	BH-76	13	coh	1.44	0	72	216	0.5
150	BH-76	15	coh	1.728	0	57.6	216	0.6
152	BH-76	25	coh	1.728	0	36	216	0
155	BH-76	75	fri	0	33	129.6	864	0.5
156	BH-76	73.5	gra	10.08	35	374.4	4320	0.9
157	BH-76	95	fri	3.744	33	144	864	0.4
158	BH-76	93.5	fri	3.744	33	158.4	864	0.4
159	BH-48	30.5	coh	1.728	0	57.6	288	0.4
161	BH-48	50	fri	3.6	33*	115.2	1296	0.5
162	BH-48	48.5	fri	3.6	33*	129.6	792	0.5
163	BH-48	60	fri	2.304	33*	72	864	0.7
165	BH-60	13	fri	1.728	0	72	216	0.6
166	BH-60	15	coh	2.304	0	129.6	216	0.7
170	BH-60	35	coh	2.16	0	57.6	432	0
171	BH-60	33.5	coh	1.728	0	115.2	288	0
172	BH-60	45	coh	2.16	0	259.2	288	0.9
175	BH-60	73.5	coh	2.448	0	43.2	216	0.7
182	BH-08	54.5	coh	2.88	0	201.6	576	0.5
183	BH-08	63	coh	2.88	0	216	576	0.6
184	BH-08	64.5	fri	2.88	0	345.6	432	0.6

[1] Material Types: coh = cohesive, fri = frictional, gra = gravelly, cem = cemented

Table 8-23. Strength and Stress-Strain Properties Derived from Pressuremeter Tests.

Borehole	Sample Number	Sample Bottom Depth, ft	Over-Consolidation Ratio OCR	Normalized Undrained Shear Strength at Maximum Obliquity
K_0 Triaxial Compression Test Results				
BH-9	3c	52.20	6.14	1.33
BH-18	9c	80.95	2.03	0.55
BH-23	9b	41.20	1.00	0.29
BH-23	17c	106.50	3.84	0.78
BH-24	3b	14.85	1.00	0.28
BH-24	7b	24.45	1.00	0.28
BH-25	4b	32.05	3.14	0.76
BH-33	6b	54.00	4.29	1.12
BH-37	5a	42.30	1.00	0.29
BH-42	7d	36.95	5.06	0.99
BH-60	11a	74.35	1.00	0.30
BH-60	18a	132.40	1.00	0.27
BH-64	5d	31.50	1.00	0.28
BH-64[1]	19b	116.90	1.00	0.27
BH-68	3a	13.10	1.00	0.30
BH-68	18b	149.40	1.00	0.27
BH-70	35b	136.80	4.33	1.03
BH-75	15b	150.85	2.96	0.71
BH-75	16a	160.00	1.00	0.30
BH-77	16b	101.45	1.00	0.27
K_0 Triaxial Extension Test Results				
BH-23	9c	41.75	1.00	0.14
BH-23	18c	117.00	4.94	0.21
BH-24	7c	24.85	1.00	0.21
BH-25	4a	32.50	3.13	0.50
BH-33	6a	54.40	4.37	0.71
BH-42	7c	37.35	5.14	0.69
BH-60	11b	73.95	1.00	0.16
BH-60	18b	131.85	1.00	0.20
BH-64	5c	31.90	1.00	0.15
BH-64	19c	117.25	1.00	0.12
BH-68	3b	12.65	1.00	0.23
BH-68	4a	20.70	6.95	0.85
BH-68	18c	149.80	1.00	0.21
BH-70	35a	137.30	4.04	0.52
BH-75	15c	150.45	2.66	0.33
BH-77	16c	101.85	1.00	0.06

[1]Replacement from original assignment due to sample deficiency

Table 8-24. Laboratory-Derived Relationships between Over-Consolidation Ratio (OCR) and Normalized Undrained Strength Triaxial Shear Tests (q/σ_{vc}').

Borehole	Sample Number	Sample Bottom Depth, ft	Over-Consolidation Ratio OCR	Normalized Shear Strength
BH-25	16b	110.65	1.00	0.26
BH-25	21b	127.50	4.99	0.82
BH-55	26	131.35	3.50	0.62
BH-61	15f	127.10	4.00	0.68
BH-61	17c	136.00	2.99	0.56
BH-75	15d	151.00	2.17	0.41

Table 8-25. Laboratory-Derived Relationship between Over-Consolidation Ratio (OCR) and Normalized Undrained Strength Simple Shear Tests (q/σ_{vc}')

Borehole	Sample Number	Sample Bottom Depth, ft	Upper/Lower Clay	Axial Strain ε_a, %	Shear Stress[1] q, ksf	Effective Confining Pressure[1] p', ksf
BH-09	4b	56.80	Upper Clay	14.65	16.98	44.75
BH-09	4a	57.20	Upper Clay	10.38	12.47	31.23
BH-21	6a	45.50	Upper Clay	5.99	2.00	3.70
BH-21	6c	45.90	Upper Clay	18.66	15.07	39.85
BH-21	6b	46.30	Upper Clay	18.72	10.42	26.89
BH-24	6d	21.55	Upper Clay	11.60	8.17	14.16
BH-24	6c	21.95	Upper Clay	9.25	2.60	4.36
BH-24	6b	22.35	Upper Clay	8.98	1.61	2.53
BH-33	7c	61.65	Upper Clay	19.51	25.11	45.90
BH-33	7b	62.05	Upper Clay	18.99	13.23	25.16
BH-33	7a	62.50	Upper Clay	16.02	5.00	9.20
BH-38	14c	75.45	Upper Clay	3.68	3.83	6.55
BH-38	14b	75.85	Upper Clay	10.51	6.47	11.62
BH-60	3c	16.65	Upper Clay	7.04	11.98	18.78
BH-60	3b	17.05	Upper Clay	5.14	3.09	4.79
BH-60	3a	20.00	Upper Clay	4.82	1.44	2.34
BH-66	6d	26.20	Upper Clay	19.14	16.95	35.05
BH-66	6b	27.00	Upper Clay	20.08	11.52	23.17
BH-66	6a	27.40	Upper Clay	19.72	7.46	14.43
BH-25	22b	129.10	Lower Clay	10.48	4.62	8.83
BH-25	22a	129.50	Lower Clay	16.07	41.87	91.81
BH-61	15c	126.10	Lower Clay	10.26	5.07	9.08
BH-61	15d	126.50	Lower Clay	17.95	8.68	16.85
BH-61	15e	126.90	Lower Clay	19.67	14.17	30.19
BH-64	18d	106.10	Lower Clay	11.76	27.22	45.84
BH-64	18c	106.50	Lower Clay	11.65	11.31	18.17
BH-64	18b	106.90	Lower Clay	5.86	6.04	9.68
BH-75	14c	140.20	Lower Clay	14.79	39.87	77.88
BH-75	14b	140.60	Lower Clay	14.91	30.88	53.96
BH-75	14a	141.00	Lower Clay	20.23	12.65	21.56

[1]At maximum obliquity

Table 8-26. Effective Strength Parameters from Drained Triaxial Tests.

Borehole	Sample Number	Sample Bottom Depth, ft	Compression Ratio CR	Recompression Ratio RR	Swell Ratio R	Initial Void Ratio e_0	Maximum Past Pressure, ksf		
							Casagrande	Becker	$Becker_{min}$
BH-09	3d	52.35	0.159	0.031	0.033	0.831	12.75	12.37	11.53
BH-18	9a	80.15	0.104	0.011	0.010	0.640	18.12	18.62	16.42
BH-23	9a	41.35	0.100	0.012	0.007	0.635	10.04	10.06	6.18
BH-23	17	106.80	0.108	0.014	0.012	0.515	25.02	25.35	19.04
BH-23	18a	117.30	0.140	0.015	0.013	0.625	20.59	20.79	16.54
BH-24	3a	15.00	0.122	0.024	0.021	0.702	9.20	9.36	7.12
BH-24	6a	22.50	0.132	0.019	0.014	0.664	13.24	13.72	11.76
BH-24	7a	25.00	0.104	0.014	0.011	0.603	8.87	8.54	5.99
BH-24	8a	27.50	0.128	0.015	0.016	0.738	10.91	10.87	9.33
BH-24	29	110.50	0.110	0.016	0.011	0.610	19.58	19.19	12.16
BH-25	16a	112.50	0.092	0.011	0.008	0.540	20.59	23.74	20.94
BH-33	1a	9.75	0.113	0.014	0.012	0.705	6.21	6.27	5.71
BH-33	9a	82.25	0.155	0.015	0.015	0.713	18.63	18.61	15.97
BH-42	7a	37.50	0.104	0.010	0.014	0.680	12.92	12.52	12.14
BH-45	4a	41.40	0.106	0.010	0.009	0.608	17.53	14.94	13.48
BH-50	10a	47.50	0.157	0.028	0.023	0.725	10.80	10.66	8.22
BH-50	17	101.10	0.134	0.015	0.015	0.708	19.16	18.40	15.83
BH-52	8a	24.20	0.158	0.010	0.007	0.717	26.05	26.38	19.17
BH-52	11	31.90	0.147	0.028	0.022	0.765	8.88	8.85	7.61
BH-52	12a	34.50	0.114	0.012	0.012	0.854	8.52	8.53	6.16
BH-52	34a	106.50	0.093	0.018	0.013	0.622	30.60	30.44	18.72
BH-52	35a	111.50	0.130	0.020	0.017	0.565	21.38	21.97	18.30
BH-52	36a	116.50	0.137	0.016	0.013	0.604	29.19	27.74	19.27
BH-55	7a	32.35	0.123	0.019	0.012	0.828	7.54	7.36	5.11
BH-55	22a	110.80	0.113	0.011	0.007	0.591	20.15	19.47	15.25
BH-55	26a	131.50	0.127	0.017	0.013	0.604	25.46	25.38	17.85
BH-59	5a	50.70	0.110	0.024	0.022	0.623	10.89	10.91	9.54
BH-59	17	170.00	0.188	0.035	0.034	0.816	33.48	33.01	27.37
BH-61	5	47.20	0.149	0.024	0.027	0.933	10.57	10.42	8.00
BH-61	15	125.55	0.147	0.020	0.022	0.706	25.62	23.44	18.29
BH-61	17	135.45	0.168	0.028	0.026	0.769	23.42	22.12	17.45
BH-64	5	32.50	0.105	0.012	0.005	0.820	20.17	18.23	13.37
BH-64	18	107.05	0.117	0.019	0.012	0.574	24.23	23.18	17.29
BH-64	19	117.40	0.153	0.018	0.010	0.824	31.26	31.70	24.08
BH-65	13	121.60	0.126	0.026	0.017	0.580	18.14	18.32	12.12
BH-71	6	25.00	0.153	0.044	0.039	0.904	9.00	9.60	8.17
BH-77	16	102.00	0.124	0.018	0.011	0.519	16.14	15.11	12.39

Table 8-27. Laboratory Strain-Controlled Consolidation Tests: Data Summary.

Borehole	Sample Bottom Depth, ft	Effective Vertical Stress ksf	Ko	Over-Consolidation Ratio OCR
BH-09 Sa 3b	52.2	31.84	0.5764	1.0000
		21.14	0.6352	1.5062
		15.81	0.6634	2.0139
		7.90	0.8129	4.0318
		5.25	0.9943	6.0611
		7.88	0.8565	4.0384
		15.89	0.6702	2.0034
		21.19	0.6510	1.5026
		52.64	0.5884	1.0000
BH-18 Sa 9b	80.55	50.77	0.5040	1.0000
		33.67	0.5550	1.5079
		25.32	0.5980	2.0051
		12.62	0.7450	4.0230
		8.36	0.8690	6.0730
		12.69	0.7080	4.0008
		25.41	0.5710	1.9980
		33.80	0.5420	1.5021
		69.00	0.5080	1.0000
BH-25 Sa 3a	18	25.90	0.5100	1.0000
		17.20	0.6000	1.5000
		12.90	0.6500	2.0000
		6.50	0.8200	4.0000
		4.30	0.9750	6.1000
		6.50	0.7400	4.0000
		13.00	0.5800	2.0000
		17.30	0.5500	1.5000
		40.40	0.5300	1.0000
BH-25 Sa 16c	111.2	36.00	0.5410	1.0000
		23.80	0.5660	1.5126
		17.86	0.5620	2.0157
		8.80	0.6450	4.0909
		5.83	0.6850	6.1750
		8.93	0.6270	4.0000
		17.93	0.5440	2.0078
		23.94	0.5460	1.5038
		45.90	0.5500	1.0000
BH-33 Sa 1b	9.6	14.46	0.5388	1.0000
		9.59	0.6672	1.5081
		7.21	0.7495	2.0053
		3.56	1.0850	4.0637
		2.36	1.3270	6.1211
		3.60	1.0730	4.0206
		7.19	0.7430	2.0122
		9.63	0.6640	1.5010
		36.00	0.5460	1.0000
BH-33 Sa 9b	82.1	47.62	0.4870	1.0000
		31.56	0.5310	1.5089
		23.80	0.5540	2.0008
		11.86	0.6900	4.0152
		7.89	0.7900	6.0355
		11.95	0.6520	3.9849
		23.85	0.5300	1.9966
		32.10	0.5130	1.4835
		63.00	0.4900	1.0000

Borehole	Sample Bottom Depth, ft	Effective Vertical Stress ksf	Ko	Over-Consolidation Ratio OCR
BH-42 Sa 7b	36.55	50.41	0.4350	1.0000
		33.30	0.4756	1.5139
		25.13	0.4940	2.0061
		12.52	0.5898	4.0267
		8.37	0.6510	6.0203
		12.64	0.5250	3.9898
		25.31	0.4580	1.9919
		33.70	0.4450	1.4960
		74.87	0.4420	1.0000
BH-45 Sa 4b	41.25	71.93	0.4780	1.0000
		47.84	0.4890	1.5036
		35.87	0.5010	2.0053
		17.99	0.5940	3.9990
		11.78	0.6820	6.1061
		18.18	0.5560	3.9565
		36.16	0.4920	1.9894
		48.12	0.4790	1.4948
		86.62	0.4910	1.0000
BH-59 Sa 5b	50.55	28.77	0.5244	1.0000
		19.09	0.6014	1.5075
		14.32	0.6399	2.0091
		7.20	0.8158	3.9964
		4.74	0.9185	6.0713
		7.20	0.7900	3.9964
		14.41	0.5968	1.9966
		19.19	0.5578	1.4993
		54.55	0.5431	1.0000
BH-61 Sa 17b	135.85	51.00	0.5690	1.0000
		33.89	0.5970	1.5049
		25.36	0.6090	2.0110
		12.62	0.7080	4.0412
		8.31	0.7810	6.1372
		12.65	0.6950	4.0316
		25.75	0.5910	1.9806
		34.05	0.5750	1.4978
		71.93	0.5530	1.0000
BH-64 Sa 5b	32.35	28.80	0.4800	1.0000
		19.10	0.5600	1.5000
		14.30	0.6100	2.0000
		7.10	0.7400	4.0000
		4.70	0.8600	6.1000
		7.10	0.6600	4.0000
		14.30	0.5200	2.0000
		19.20	0.4900	1.5000
		50.40	0.4900	1.0000
BH-64 Sa 19b	117.4	58.40	0.4439	1.0000
		38.70	0.4800	1.5092
		28.91	0.5530	2.0198
		14.32	0.7160	4.0771
		9.50	0.8310	6.1445
		14.42	0.6310	4.0500
		29.16	0.4990	2.0025
		38.91	0.4600	1.5007
		80.34	0.4430	1.0000

Table 8-28. Laboratory-Derived Relationship between Over-Consolidation Ratio (OCR) and At-Rest Earth Pressure Coefficient (K_0).

This page intentionally left blank.

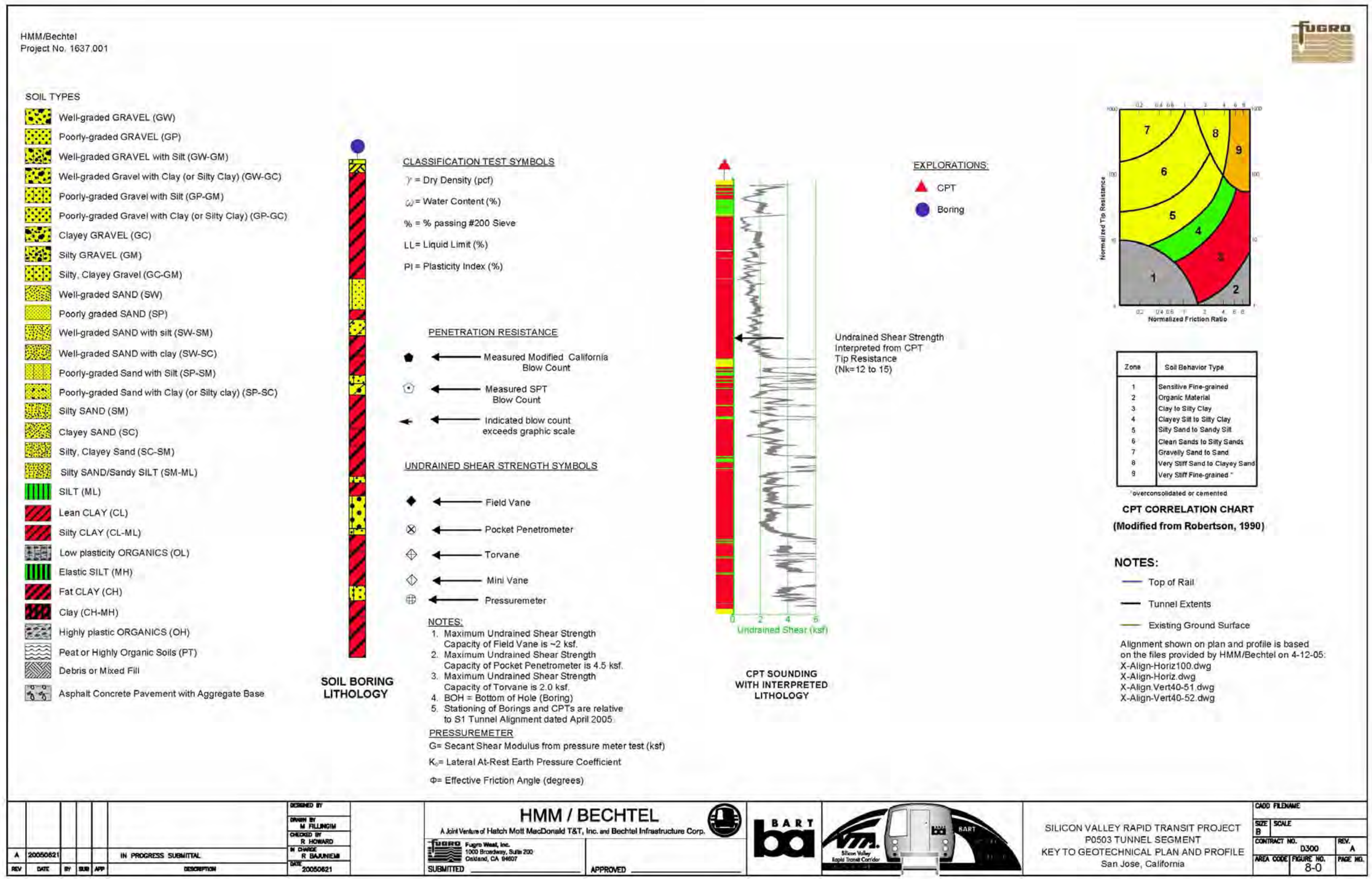


Figure 8-0. Key to Plan and Profiles shown in Figures 8-1 to 8-50.

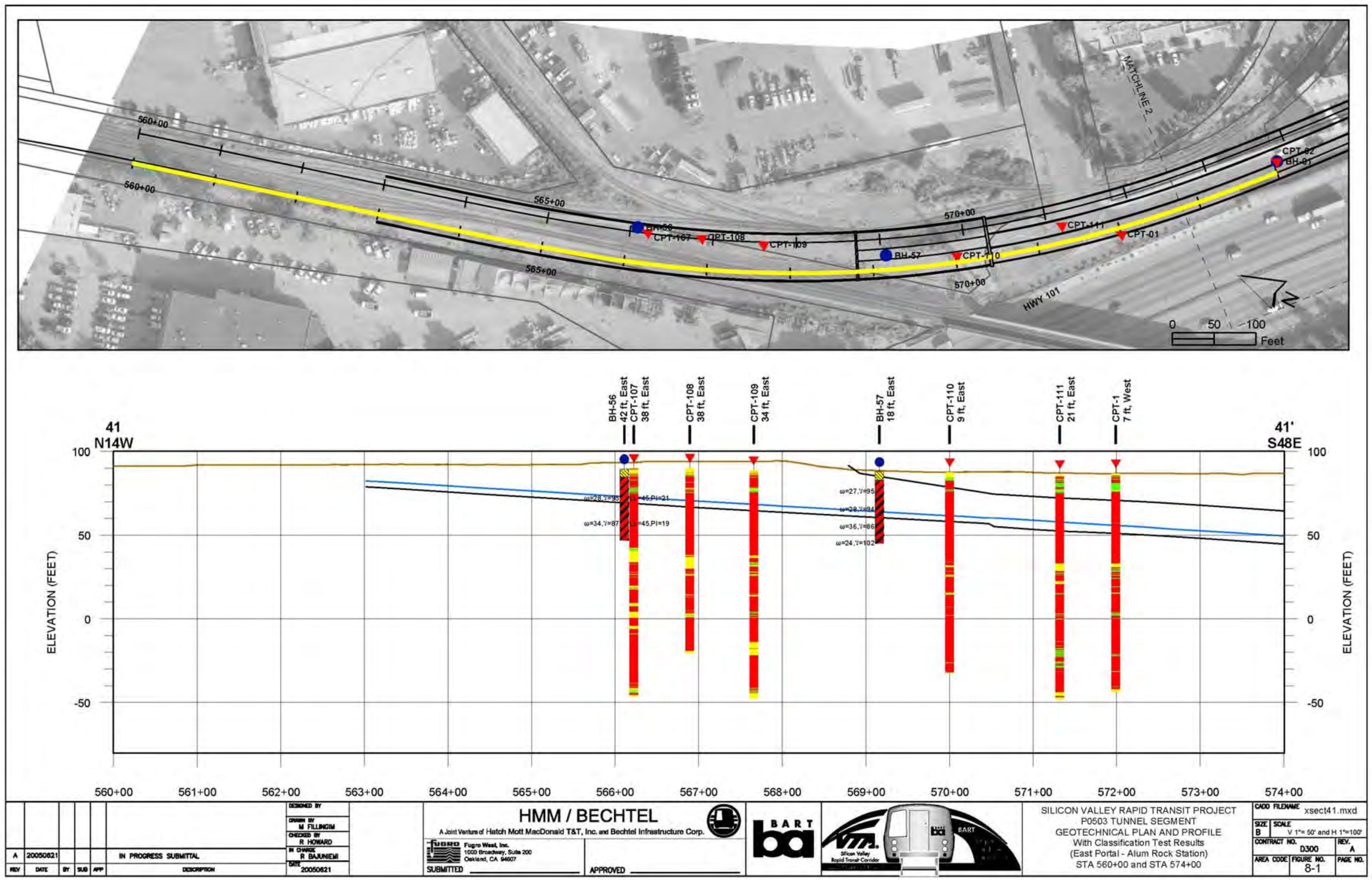


Figure 8-1. Geotechnical Plan and Profile with Classification Test Results: Station 560+00 to 574+00.

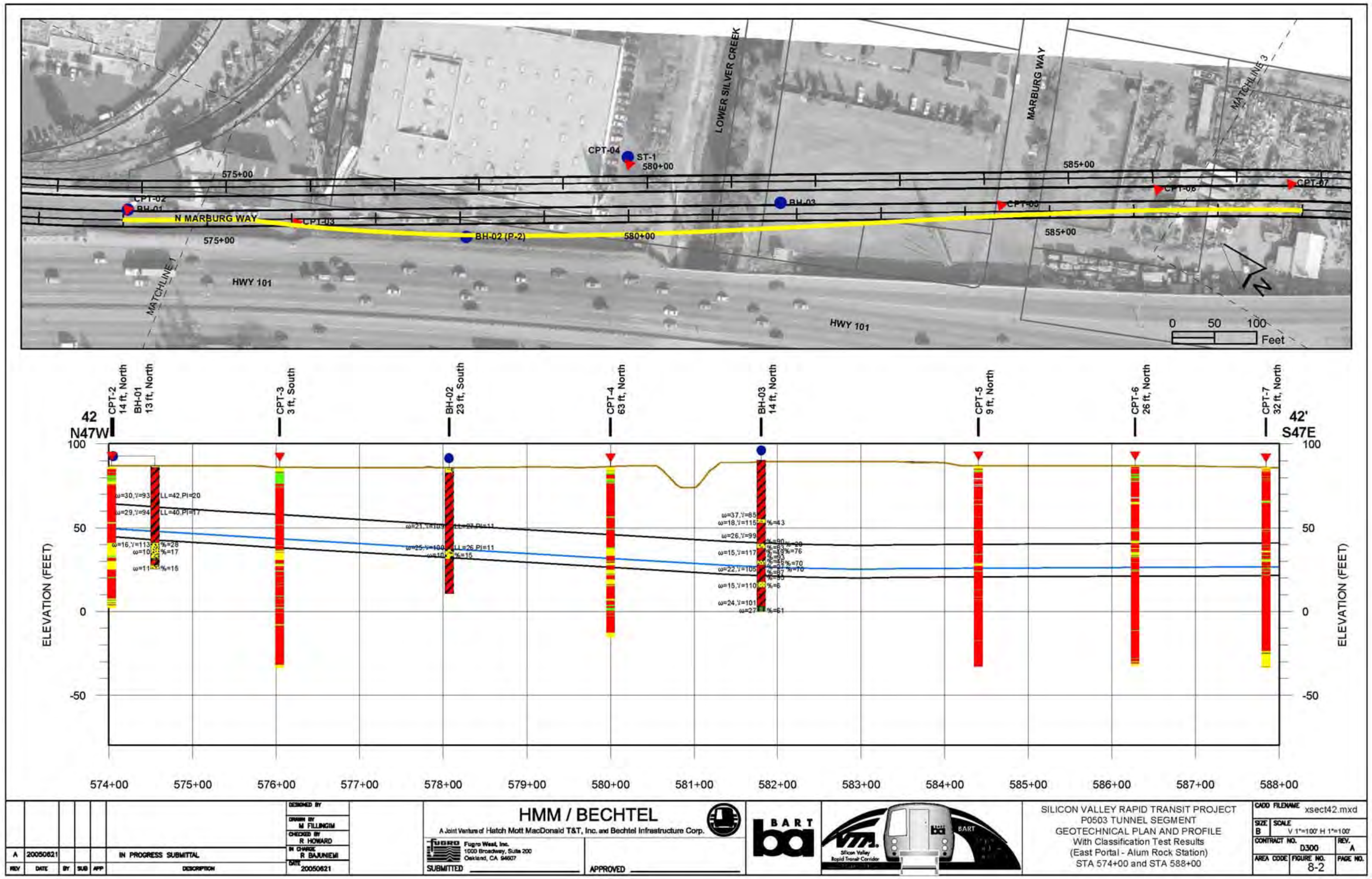


Figure 8-2. Geotechnical Plan and Profile with Classification Test Results: Station 574+00 to 588+00.

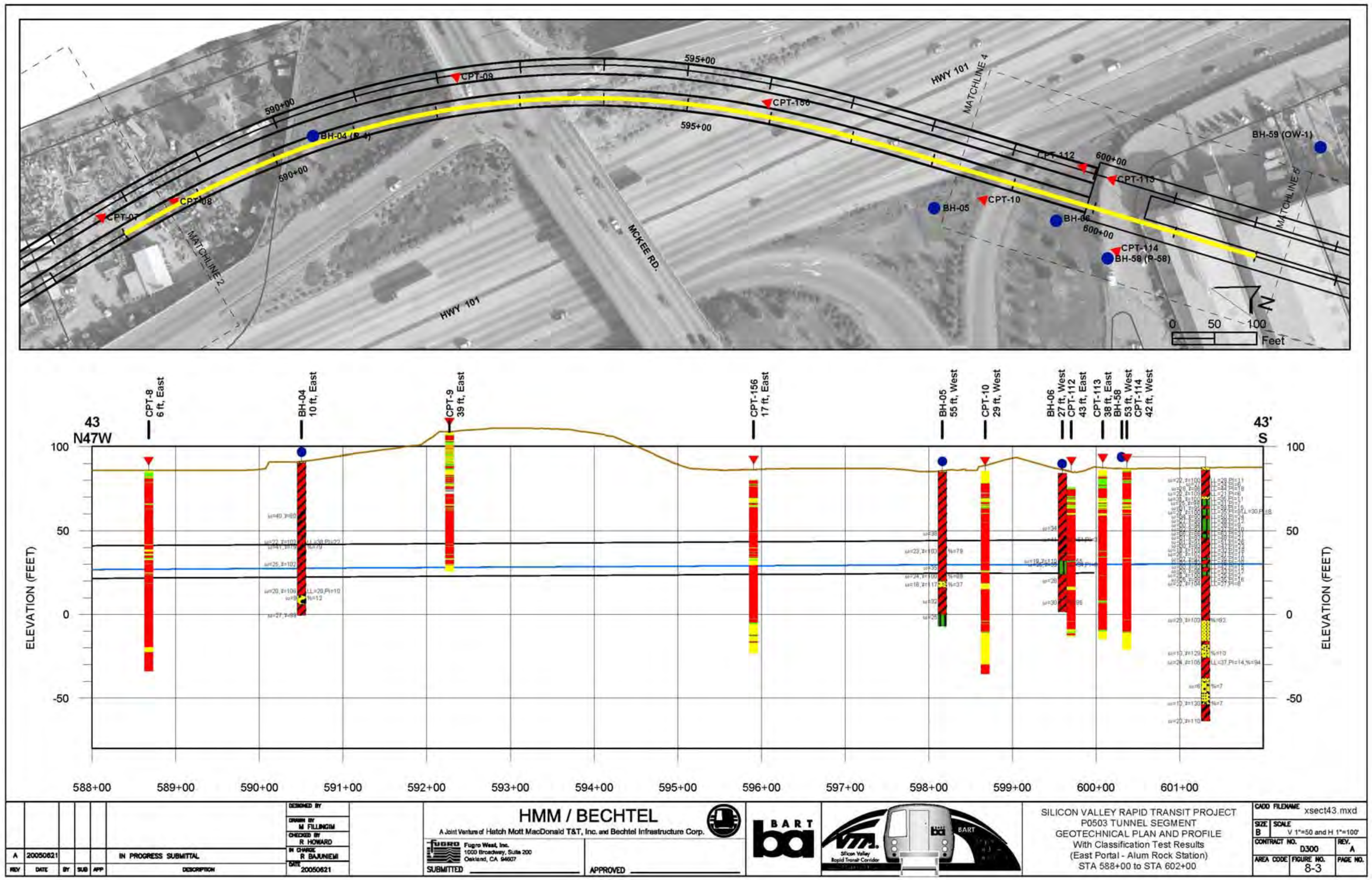


Figure 8-3. Geotechnical Plan and Profile with Classification Test Results: Station 588+00 to 602+00.

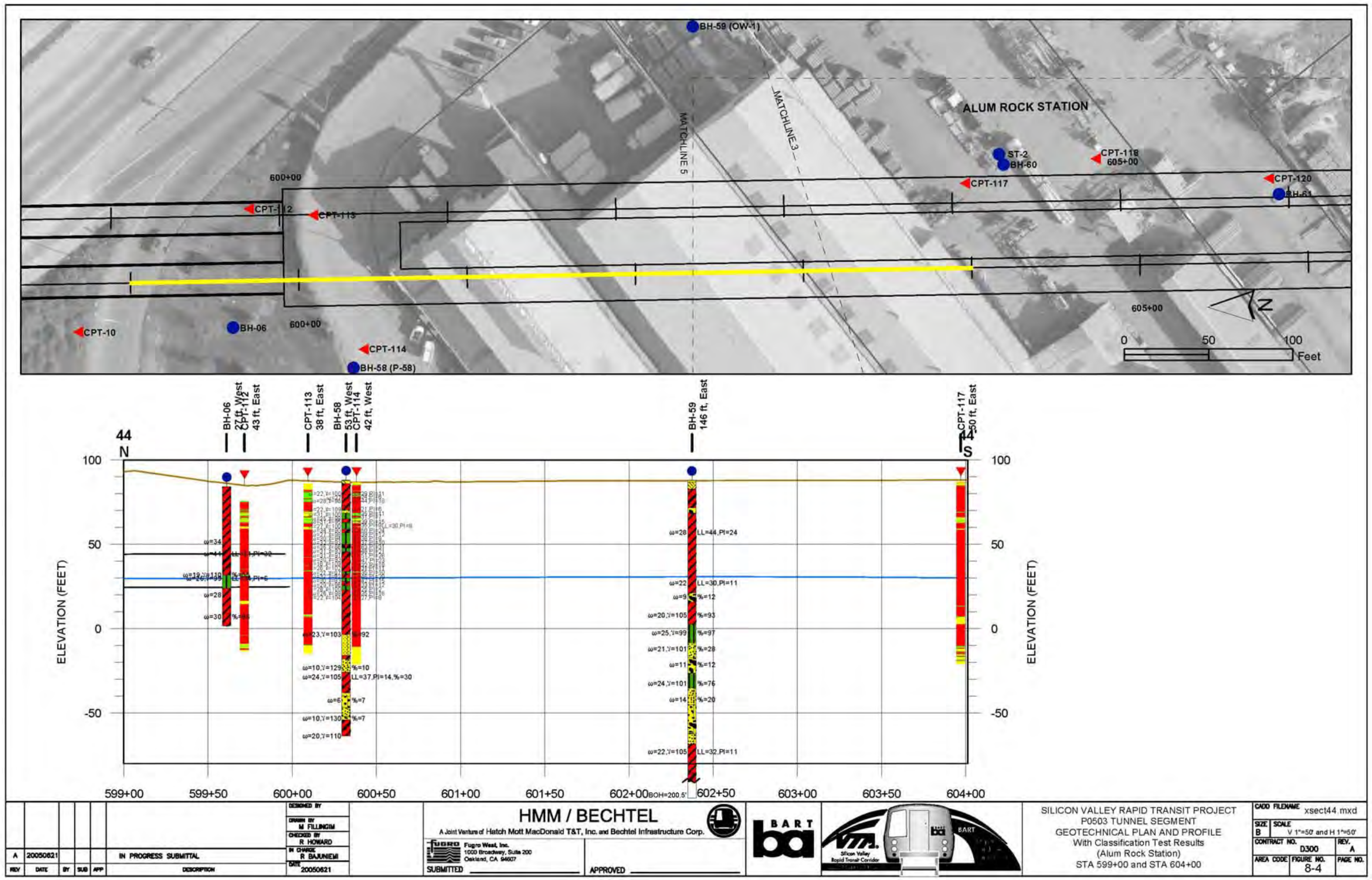


Figure 8-4. Geotechnical Plan and Profile with Classification Test Results: Station 599+00 to 604+00.

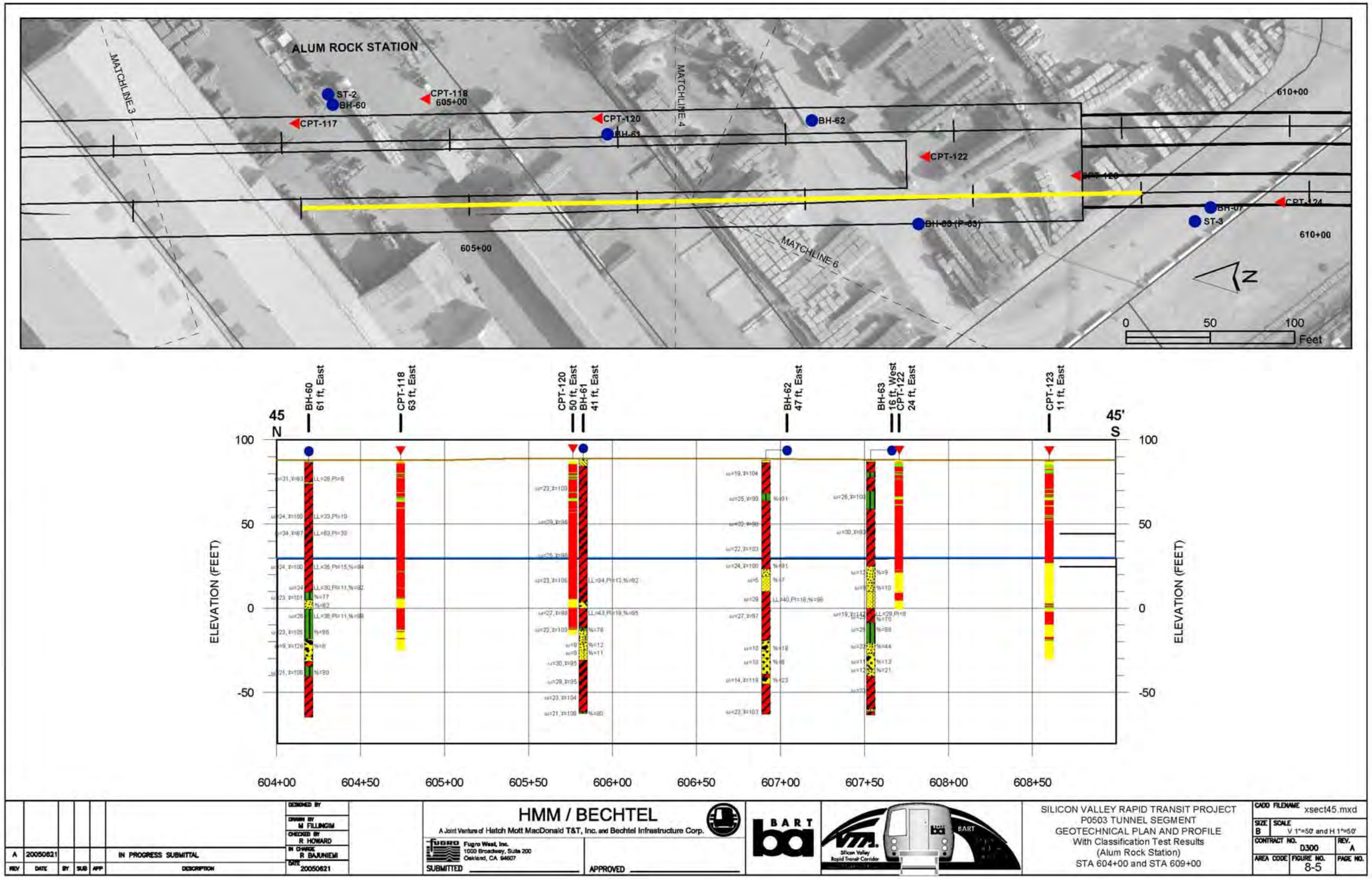


Figure 8-5. Geotechnical Plan and Profile with Classification Test Results: Station 604+00 to 609+00.

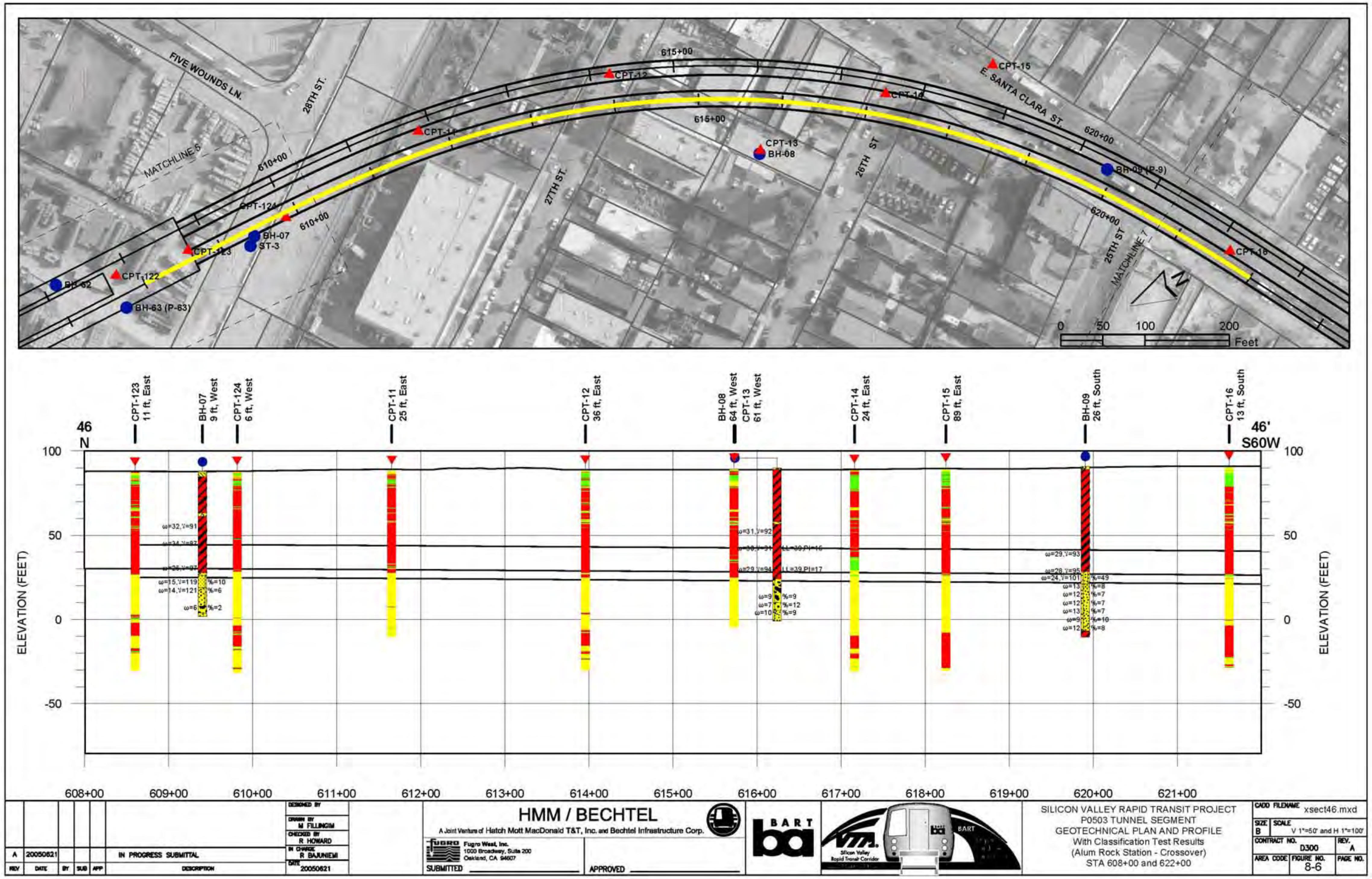


Figure 8-6. Geotechnical Plan and Profile with Classification Test Results: Station 608+00 to 622+00.

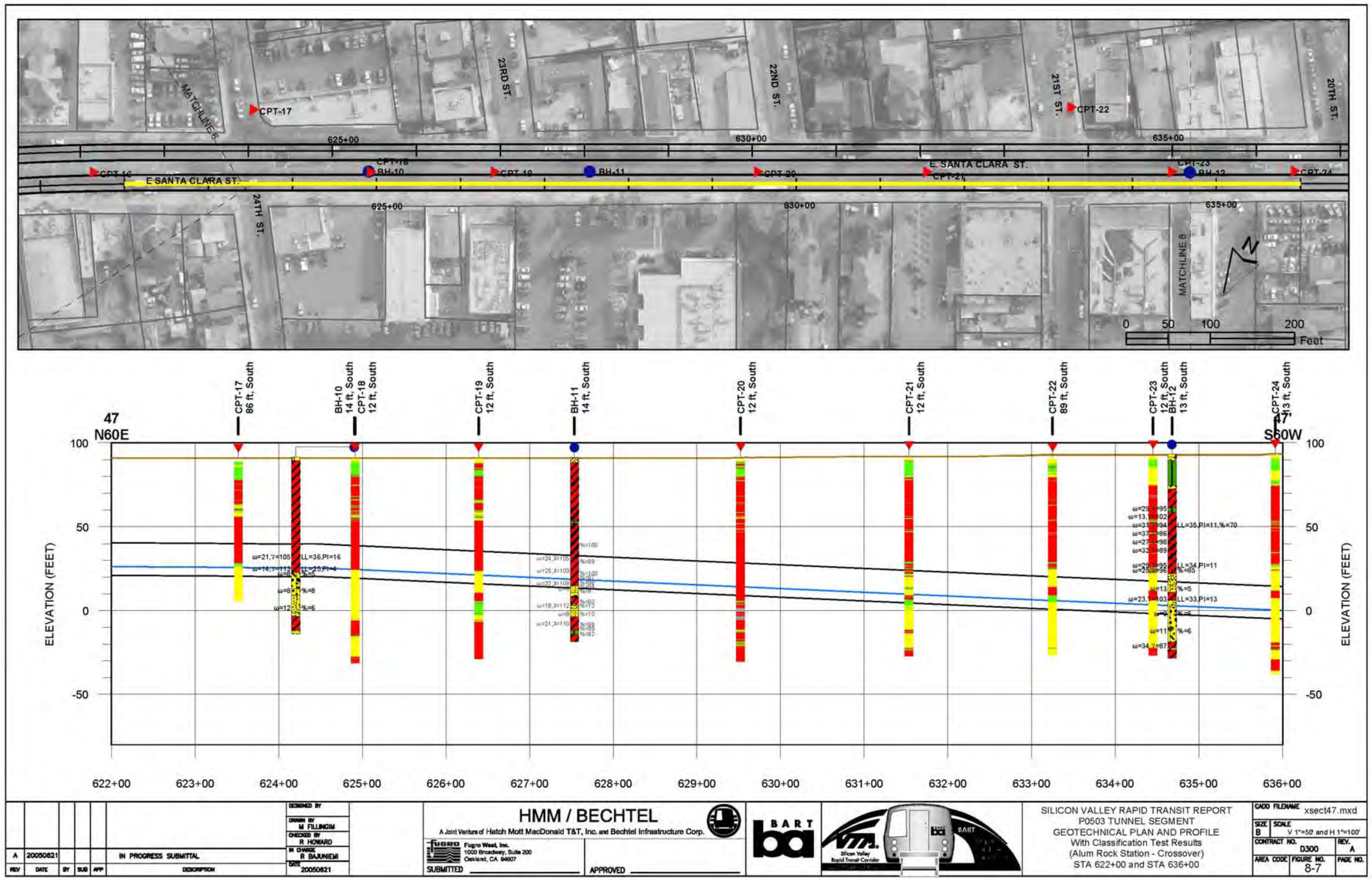


Figure 8-7. Geotechnical Plan and Profile with Classification Test Results: Station 622+00 to 636+00.

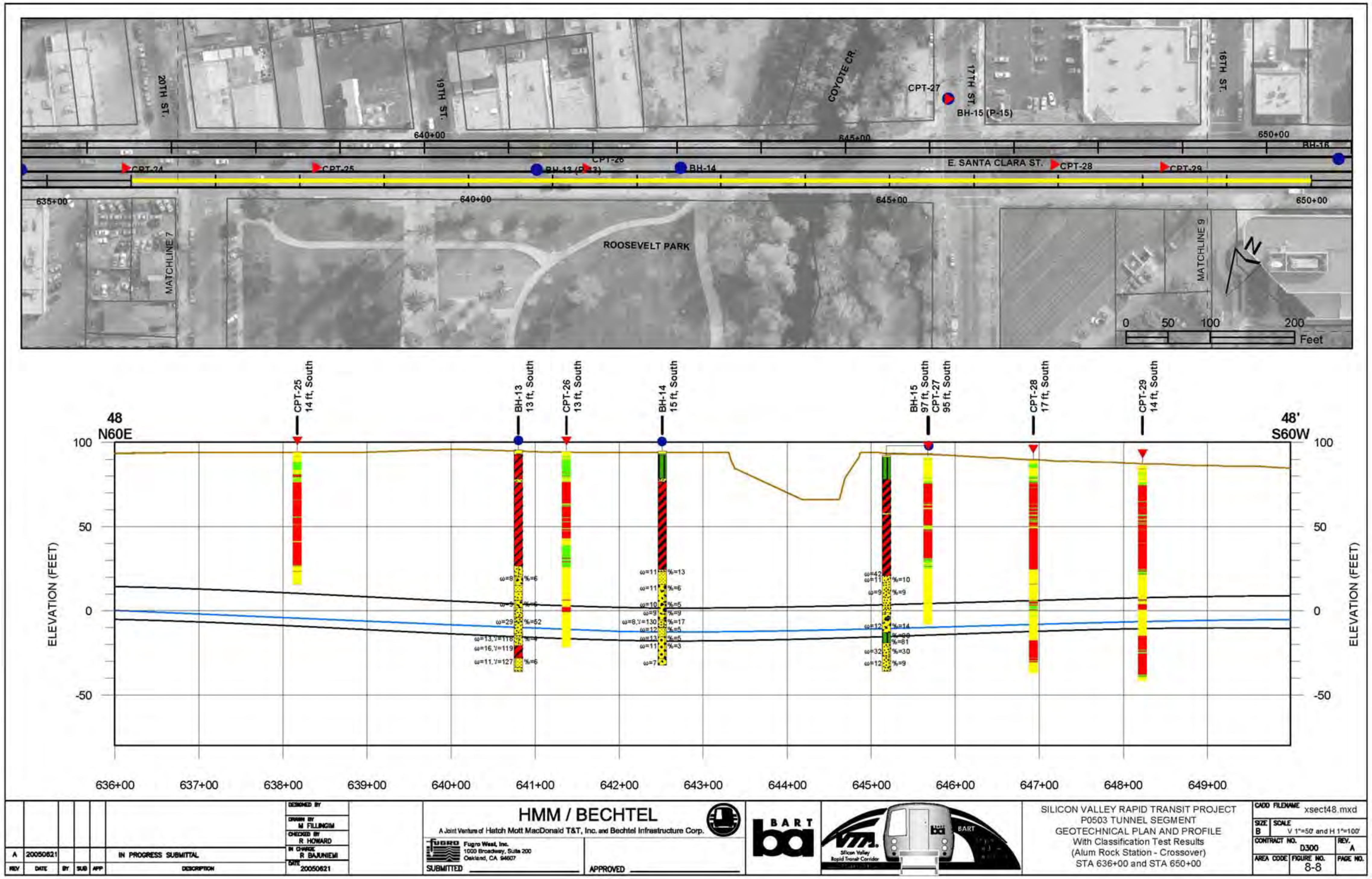


Figure 8-8. Geotechnical Plan and Profile with Classification Test Results: Station 636+00 to 650+00.

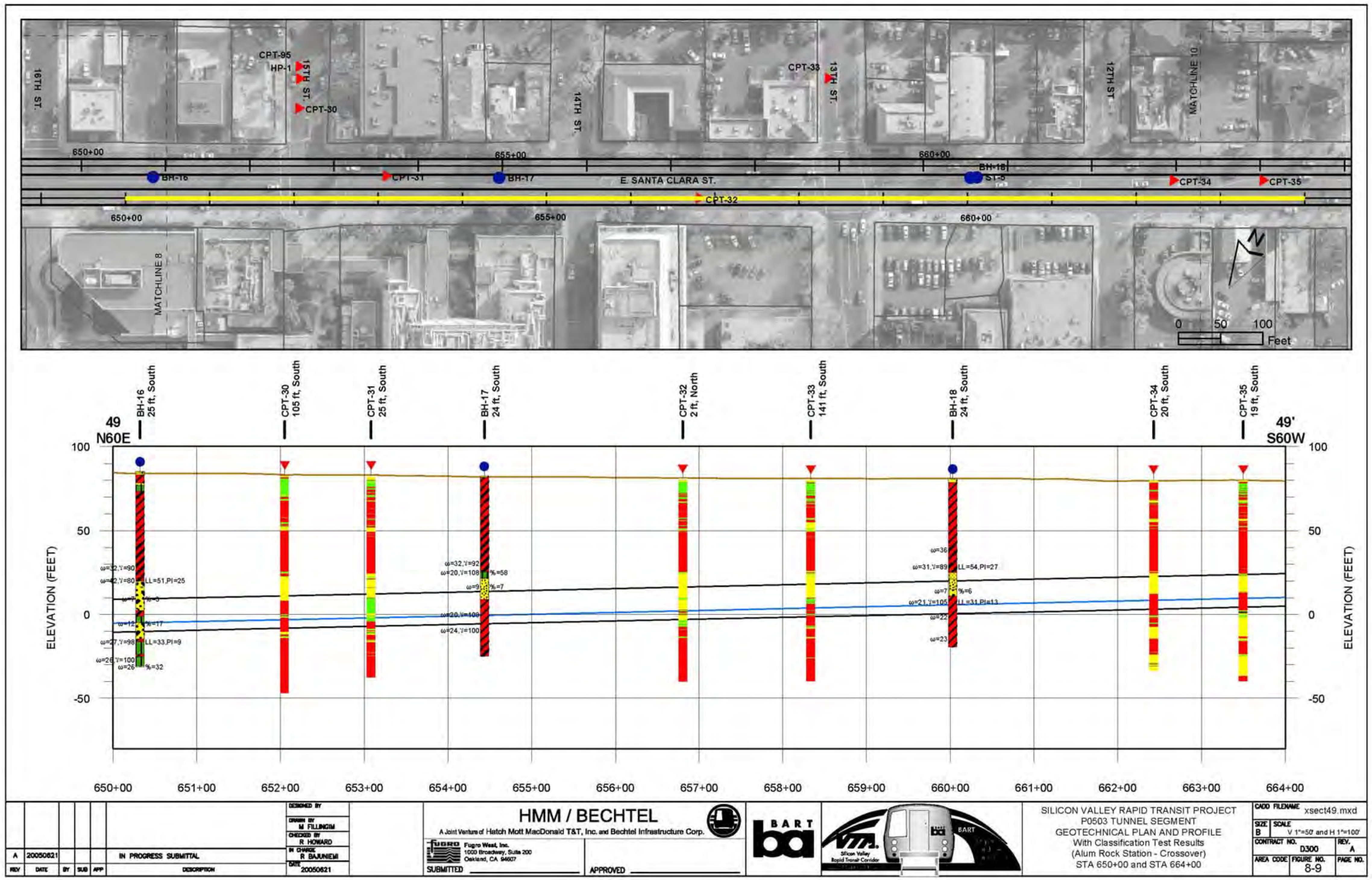


Figure 8-9. Geotechnical Plan and Profile with Classification Test Results: Station 650+00 to 664+00.

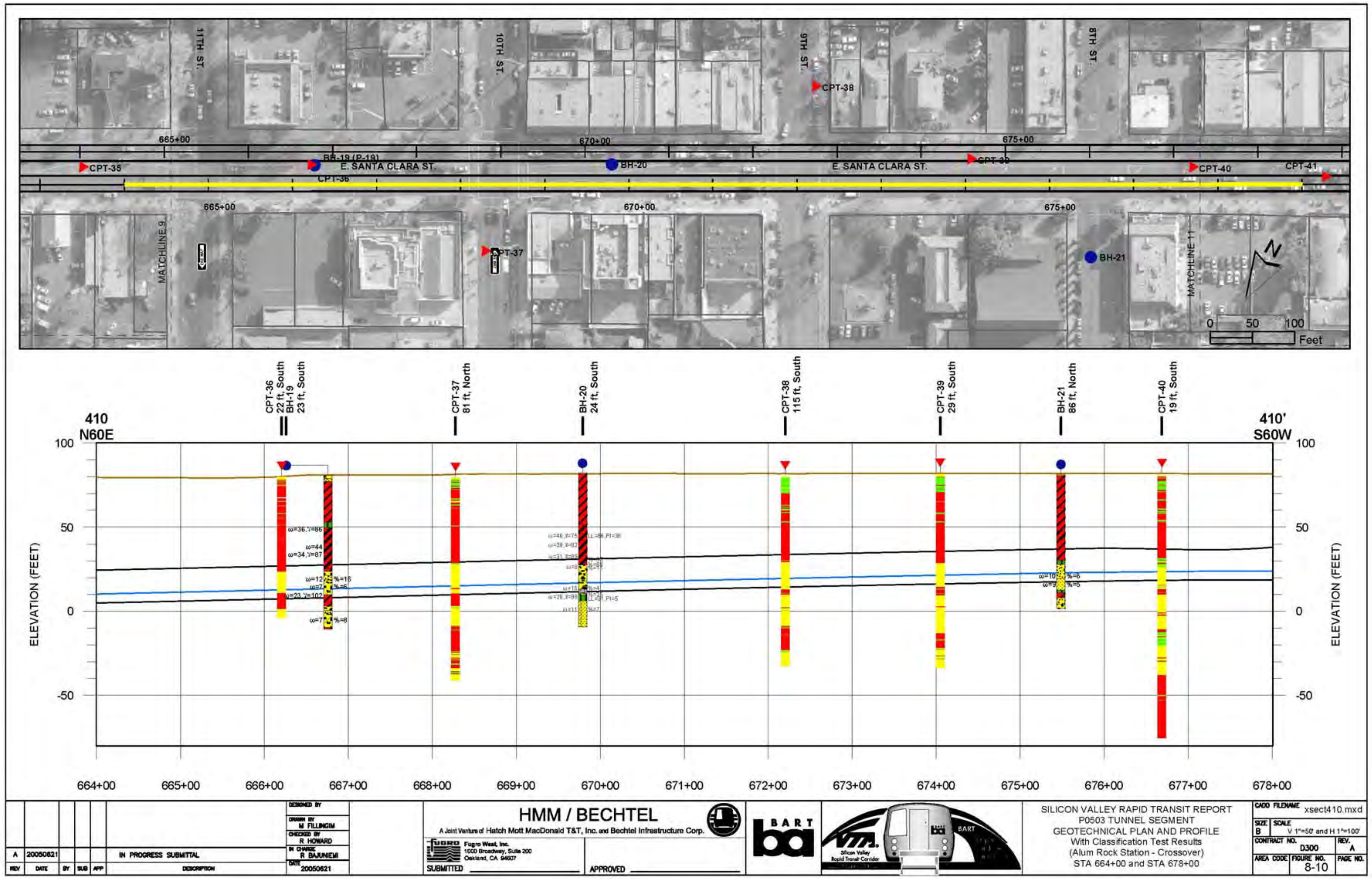


Figure 8-10. Geotechnical Plan and Profile with Classification Test Results: Station 664+00 to 678+00.

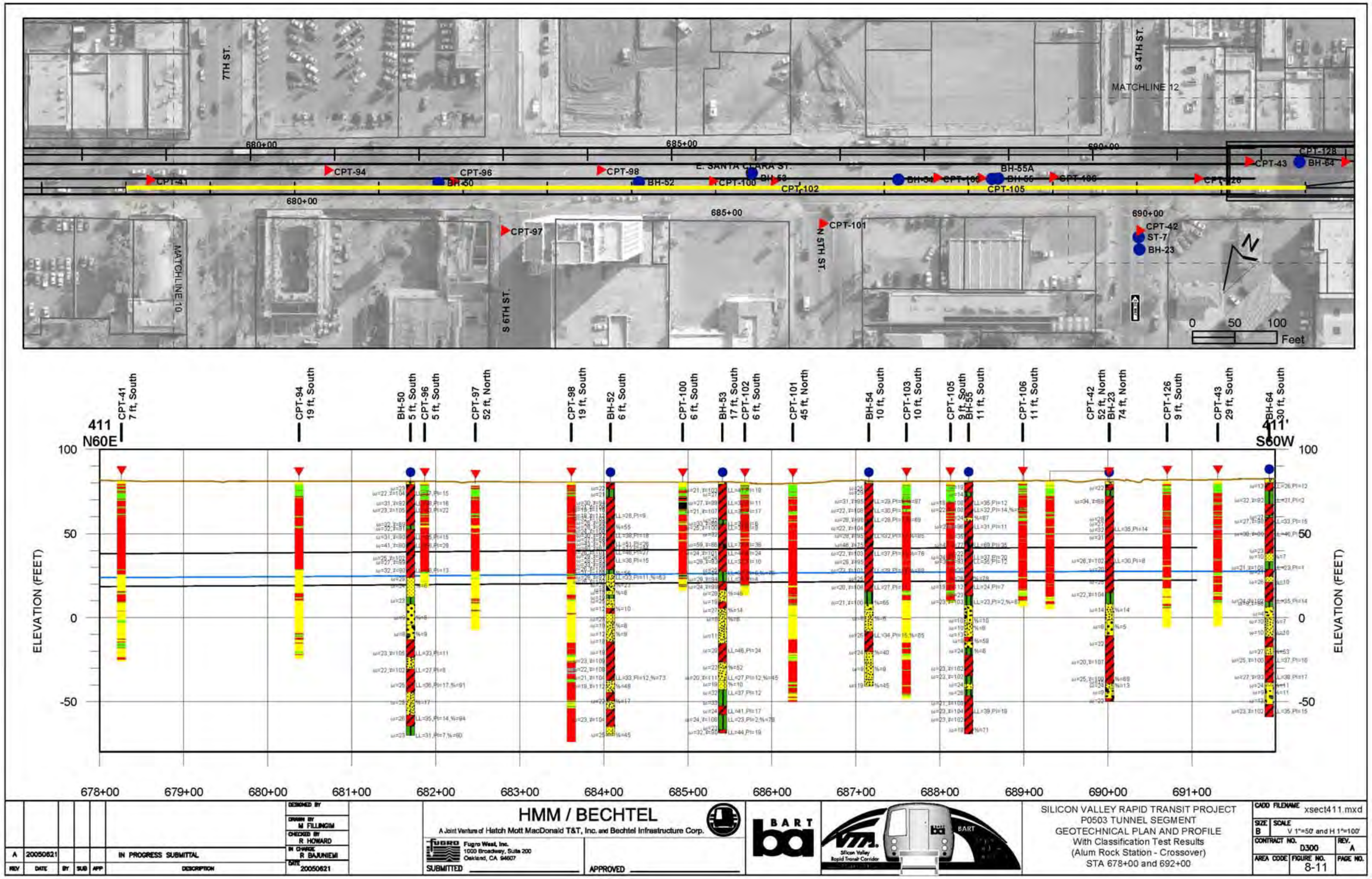


Figure 8-11. Geotechnical Plan and Profile with Classification Test Results: Station 678+00 to 692+00.

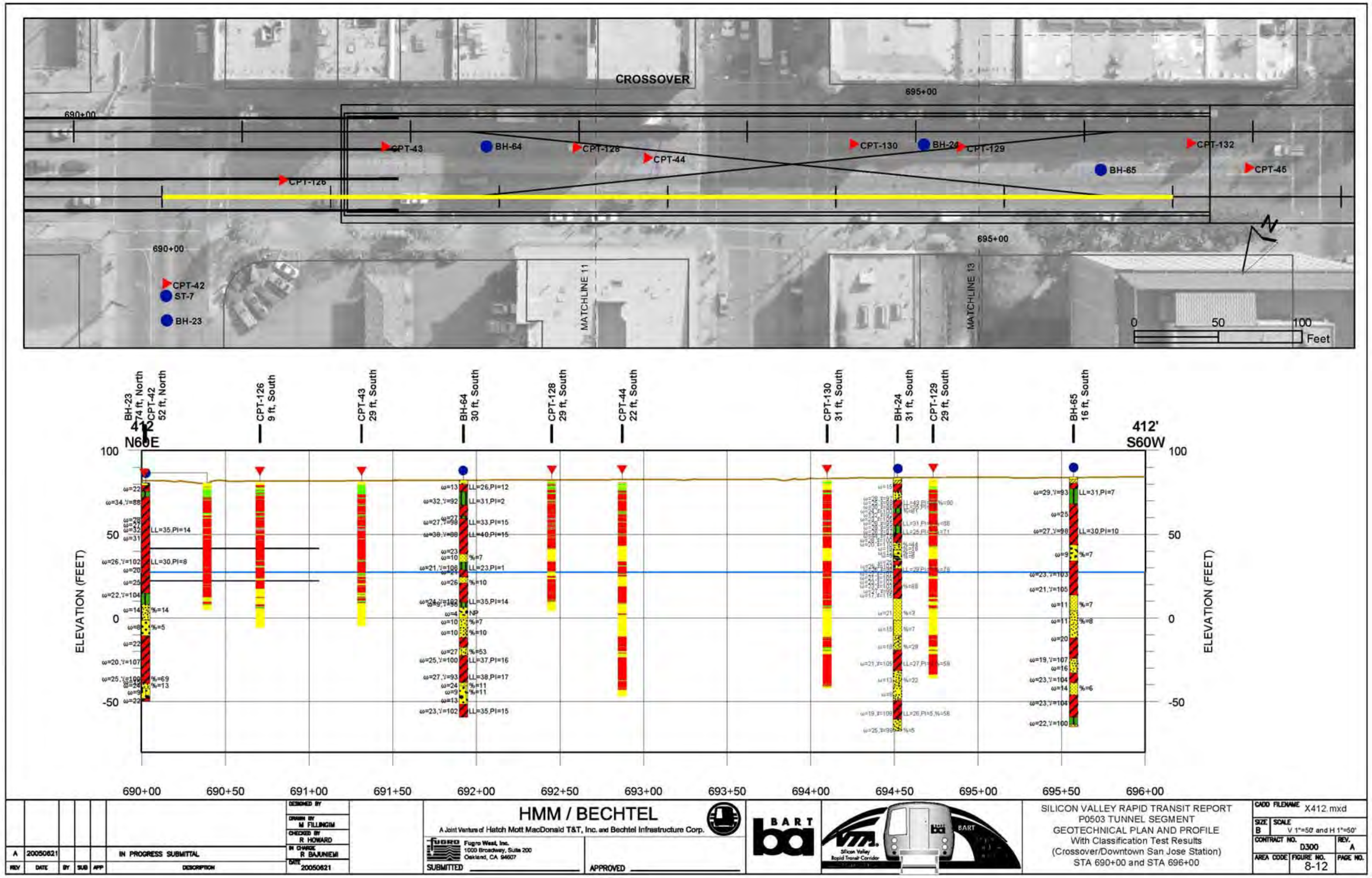


Figure 8-12. Geotechnical Plan and Profile with Classification Test Results: Station 690+00 to 696+00.

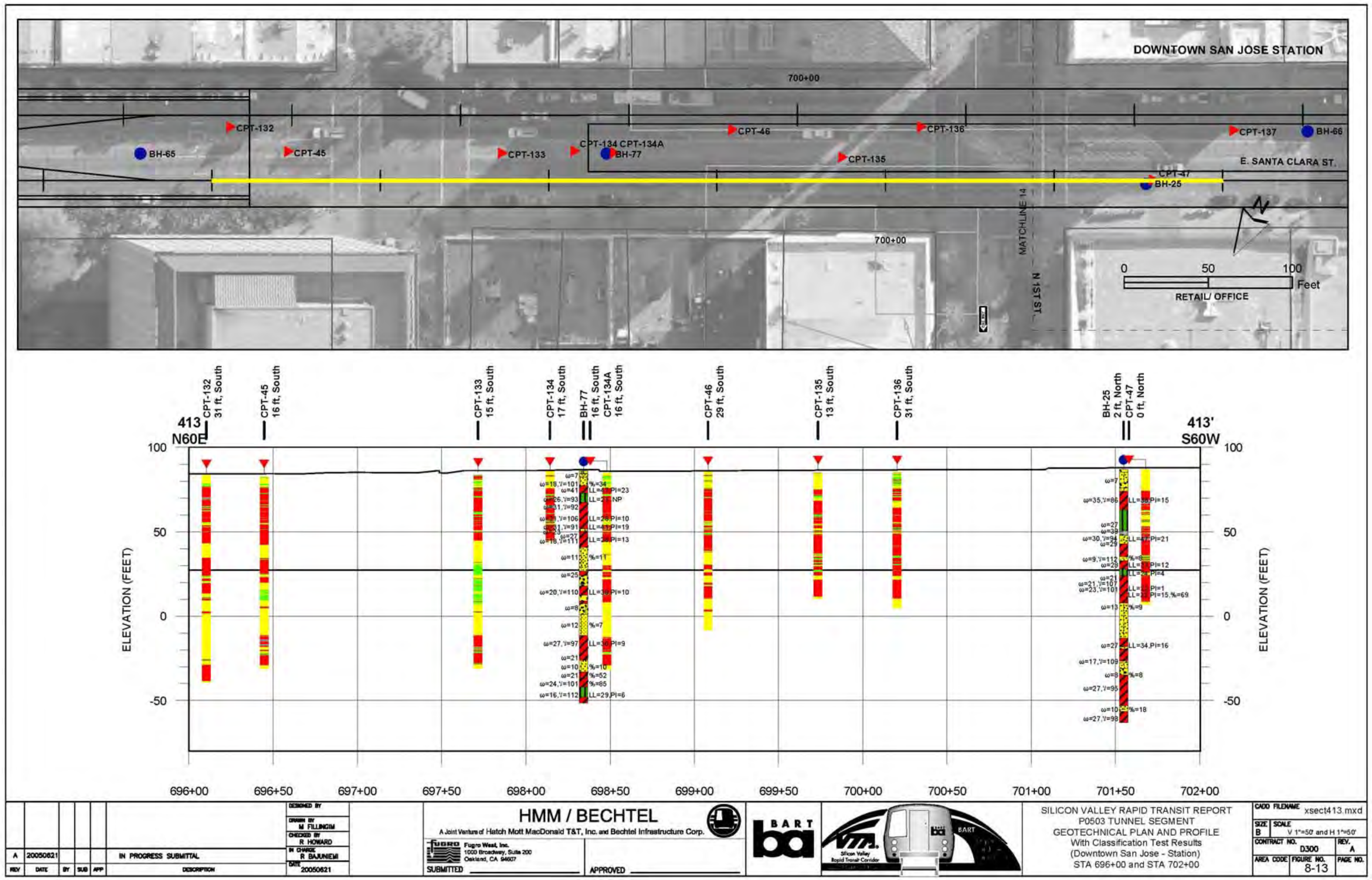


Figure 8-13. Geotechnical Plan and Profile with Classification Test Results: Station 696+00 to 702+00.

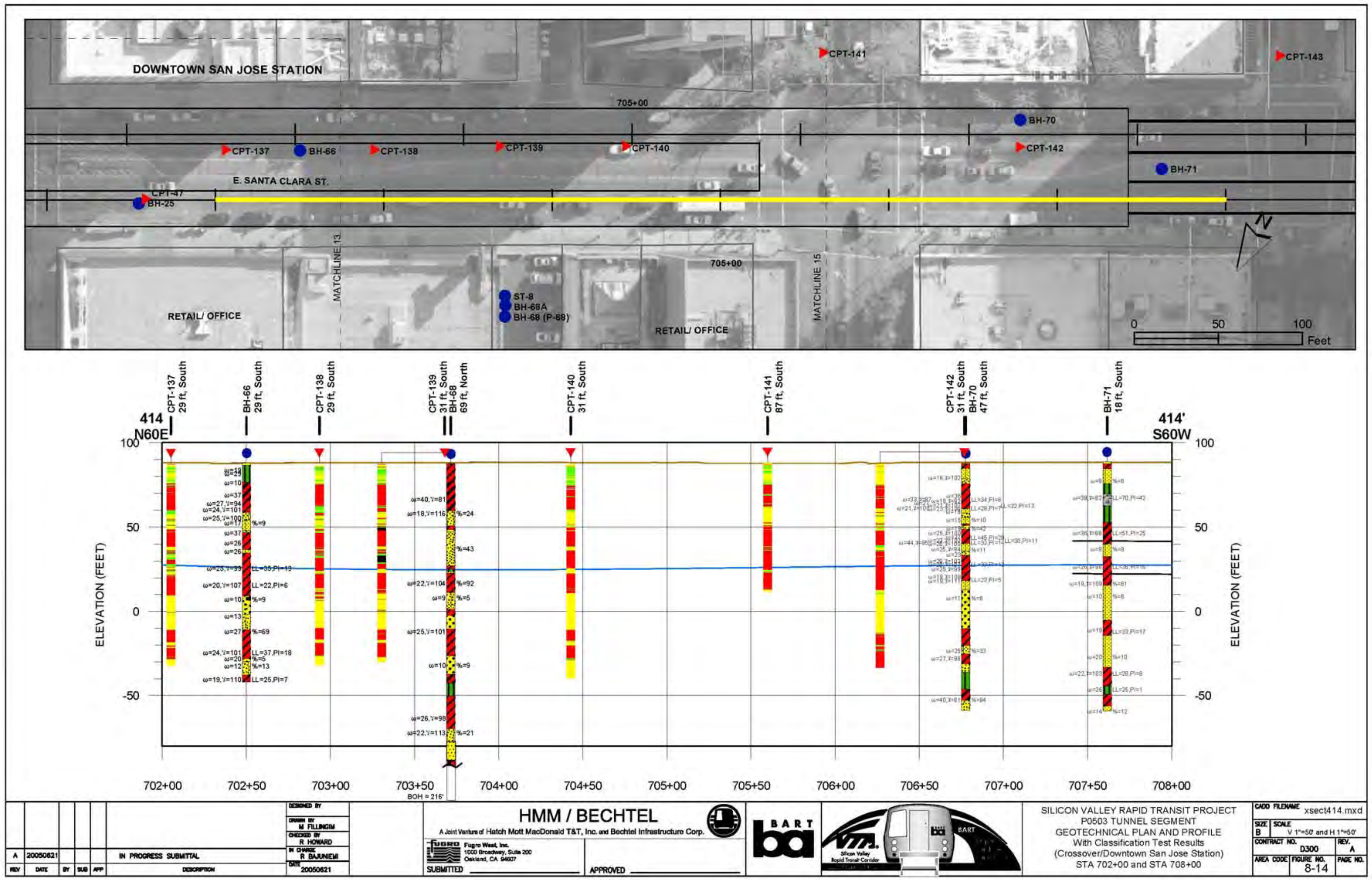


Figure 8-14. Geotechnical Plan and Profile with Classification Test Results: Station 702+00 to 708+00.

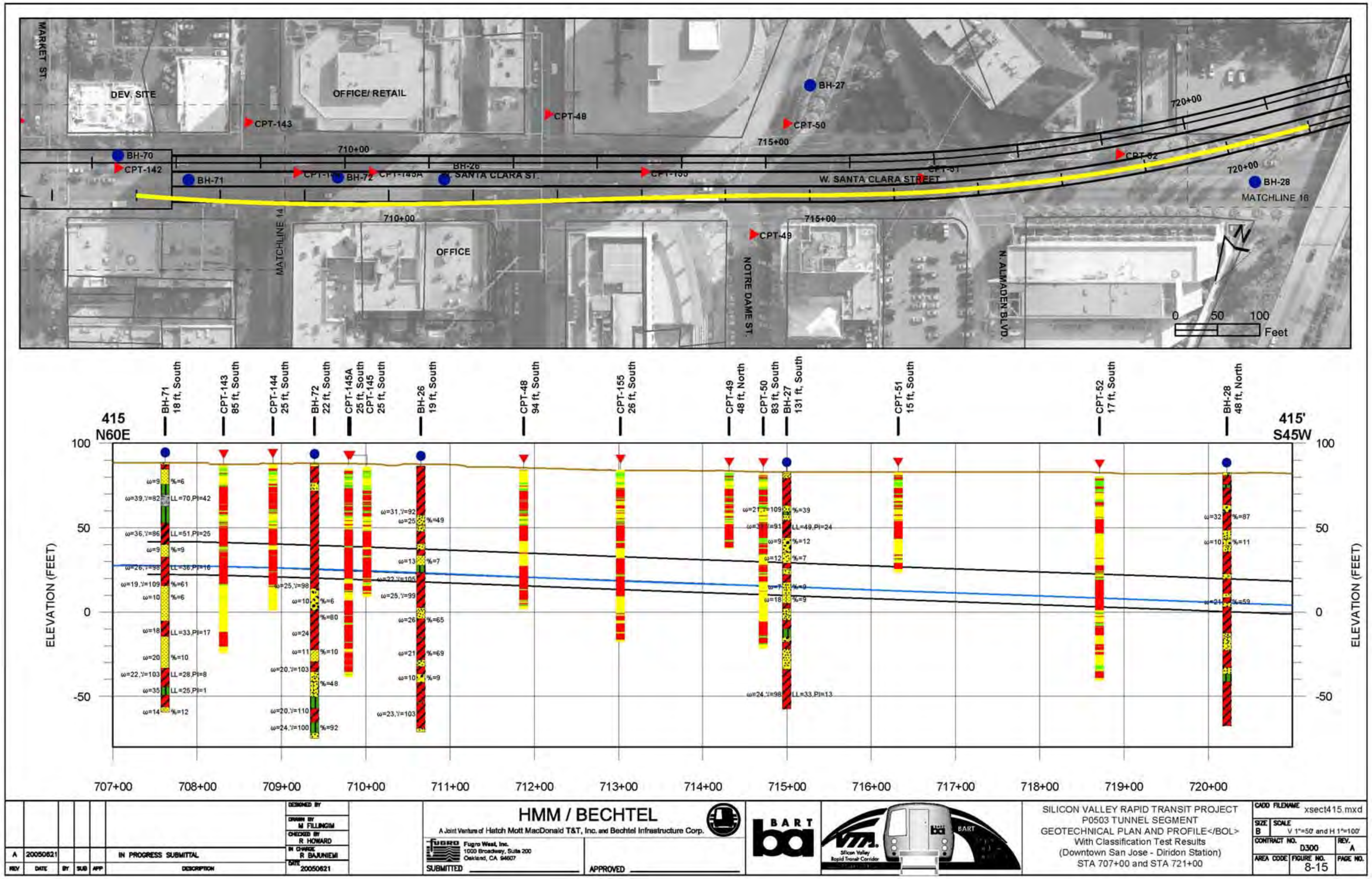


Figure 8-15. Geotechnical Plan and Profile with Classification Test Results: Station 707+00 to 721+00.

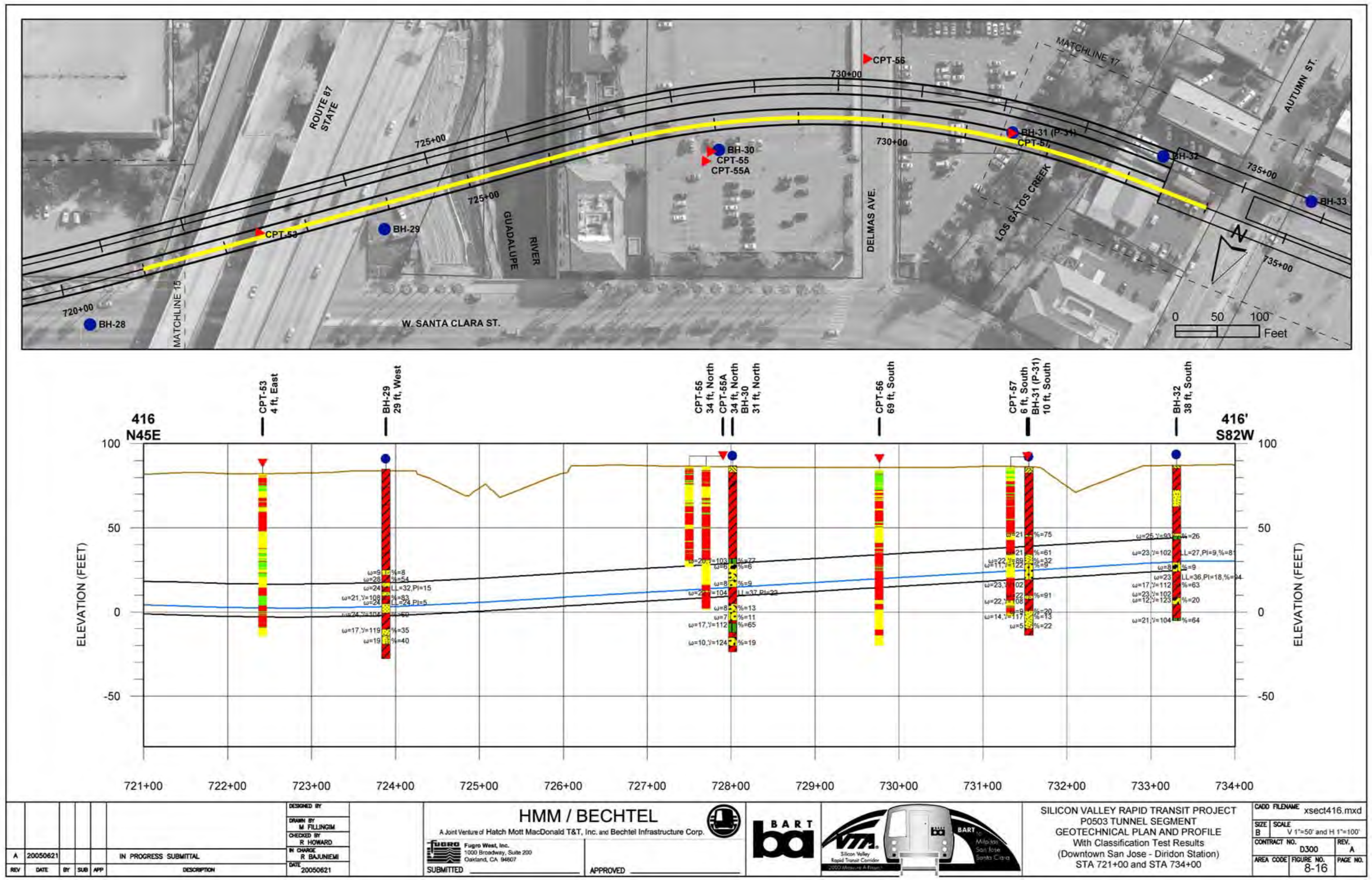


Figure 8-16. Geotechnical Plan and Profile with Classification Test Results: Station 721+00 to 734+00.

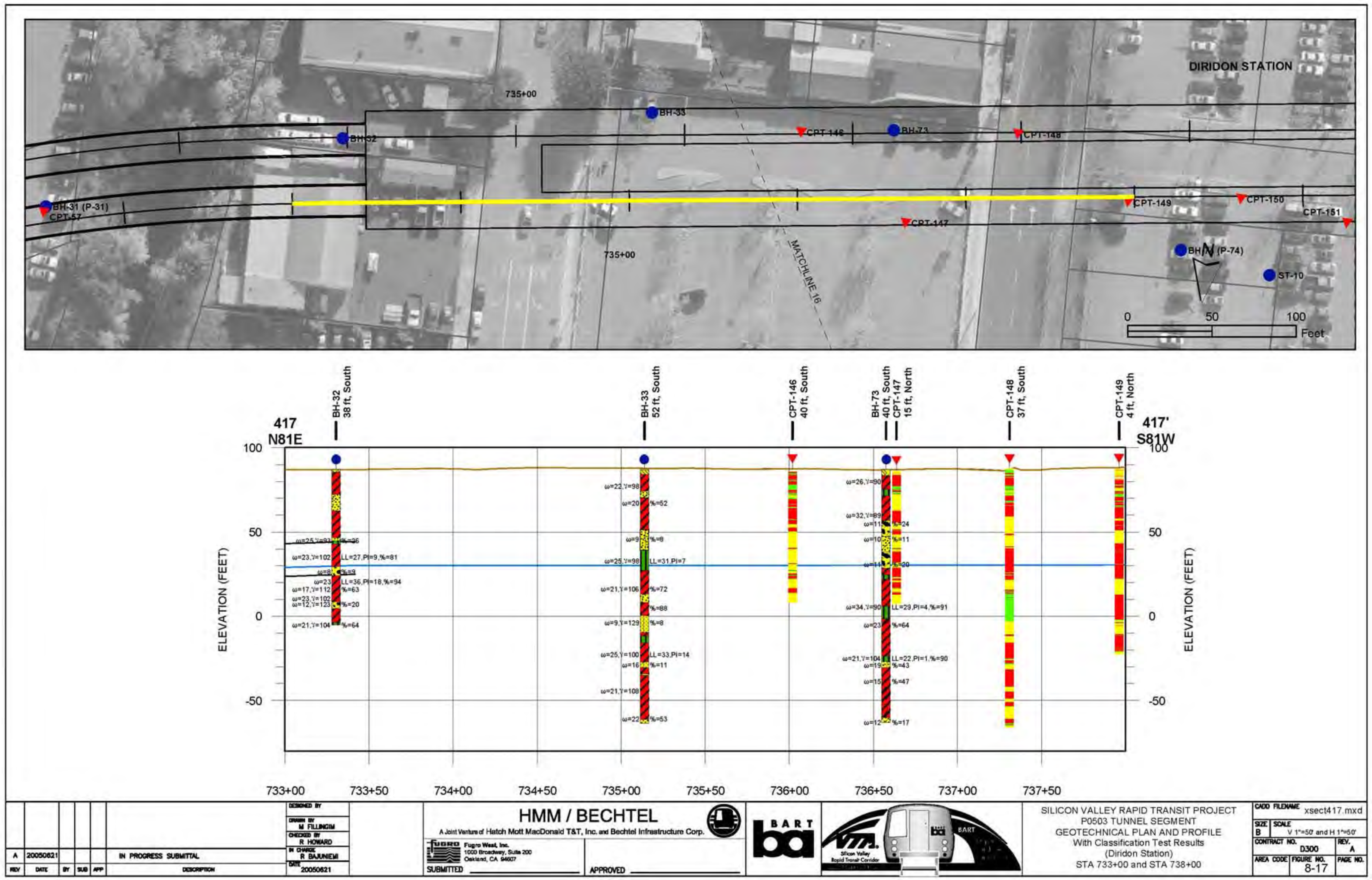


Figure 8-17. Geotechnical Plan and Profile with Classification Test Results: Station 733+00 to 738+00.

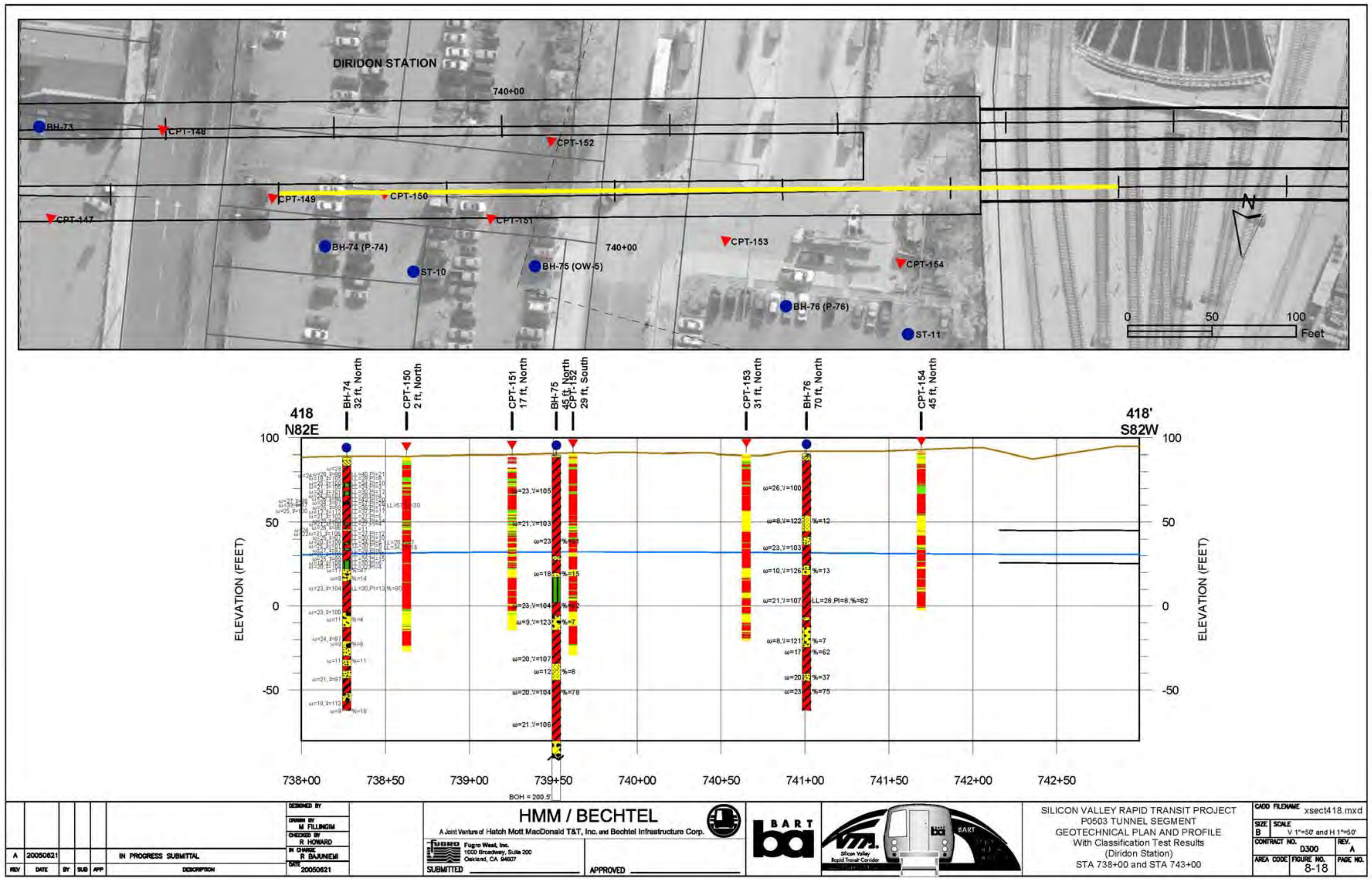


Figure 8-18. Geotechnical Plan and Profile with Classification Test Results: Station 738+00 to 743+00.

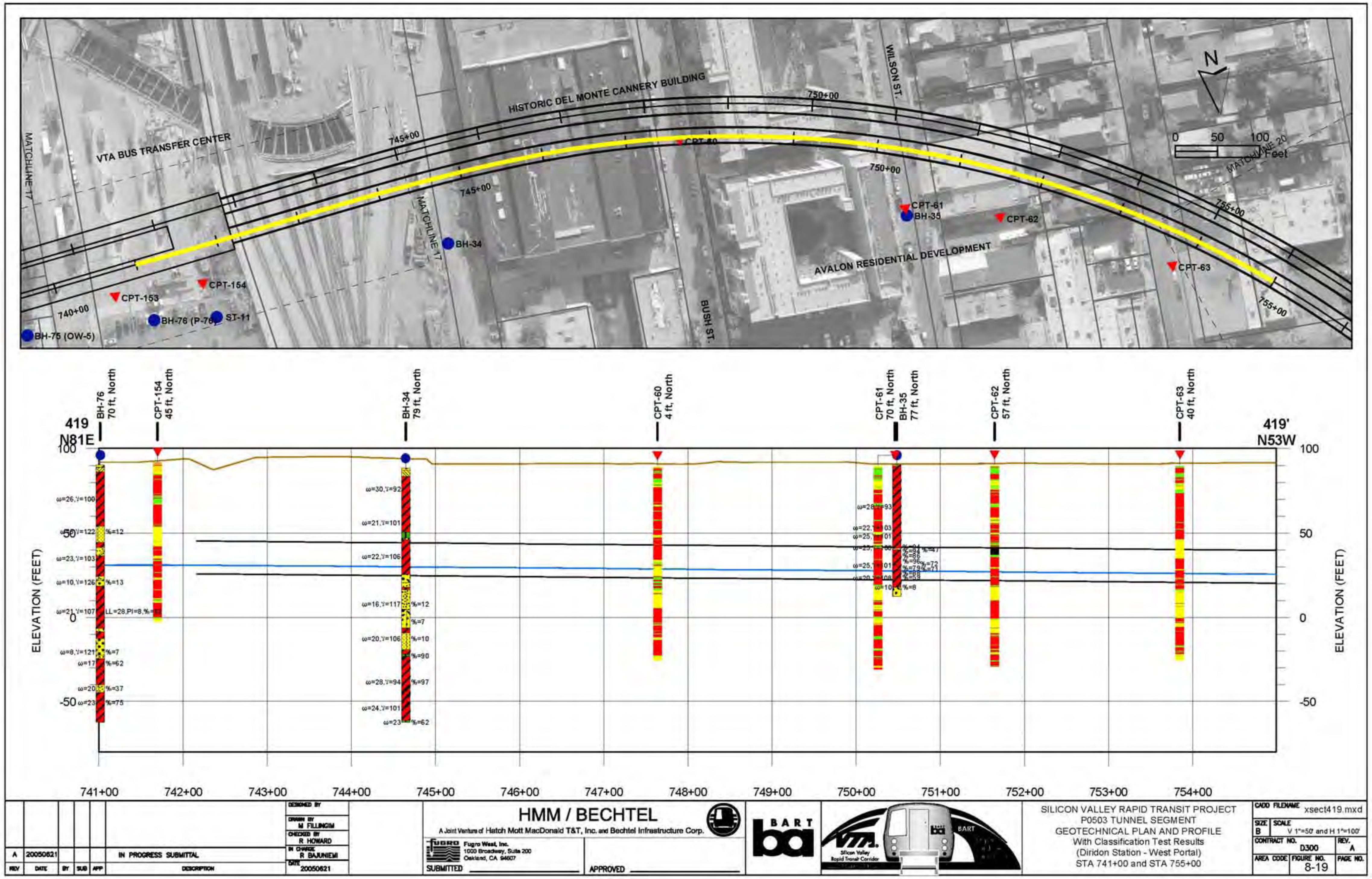


Figure 8-19. Geotechnical Plan and Profile with Classification Test Results: Station 741+00 to 755+00.

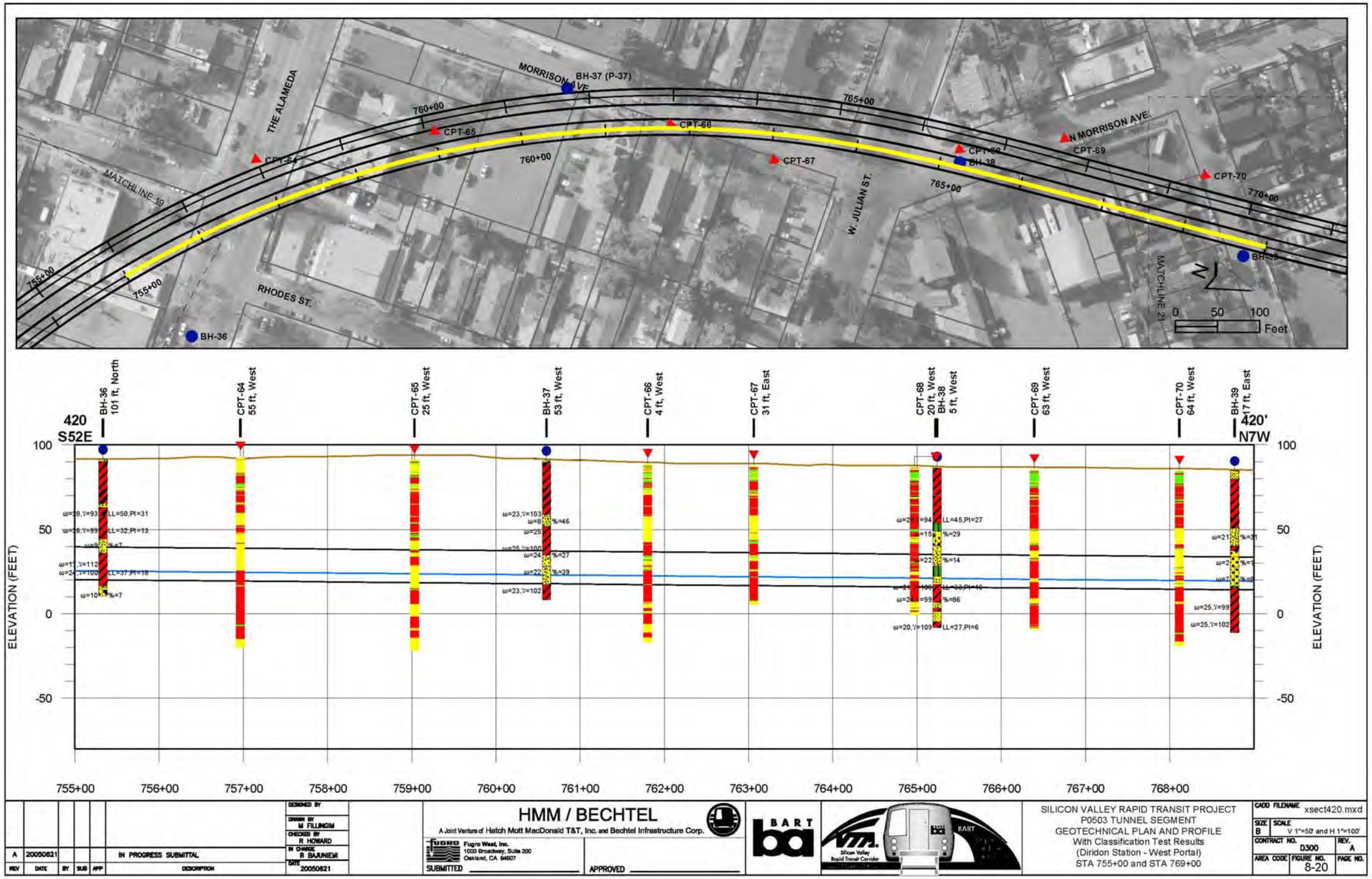


Figure 8-20. Geotechnical Plan and Profile with Classification Test Results: Station 755+00 to 769+00.

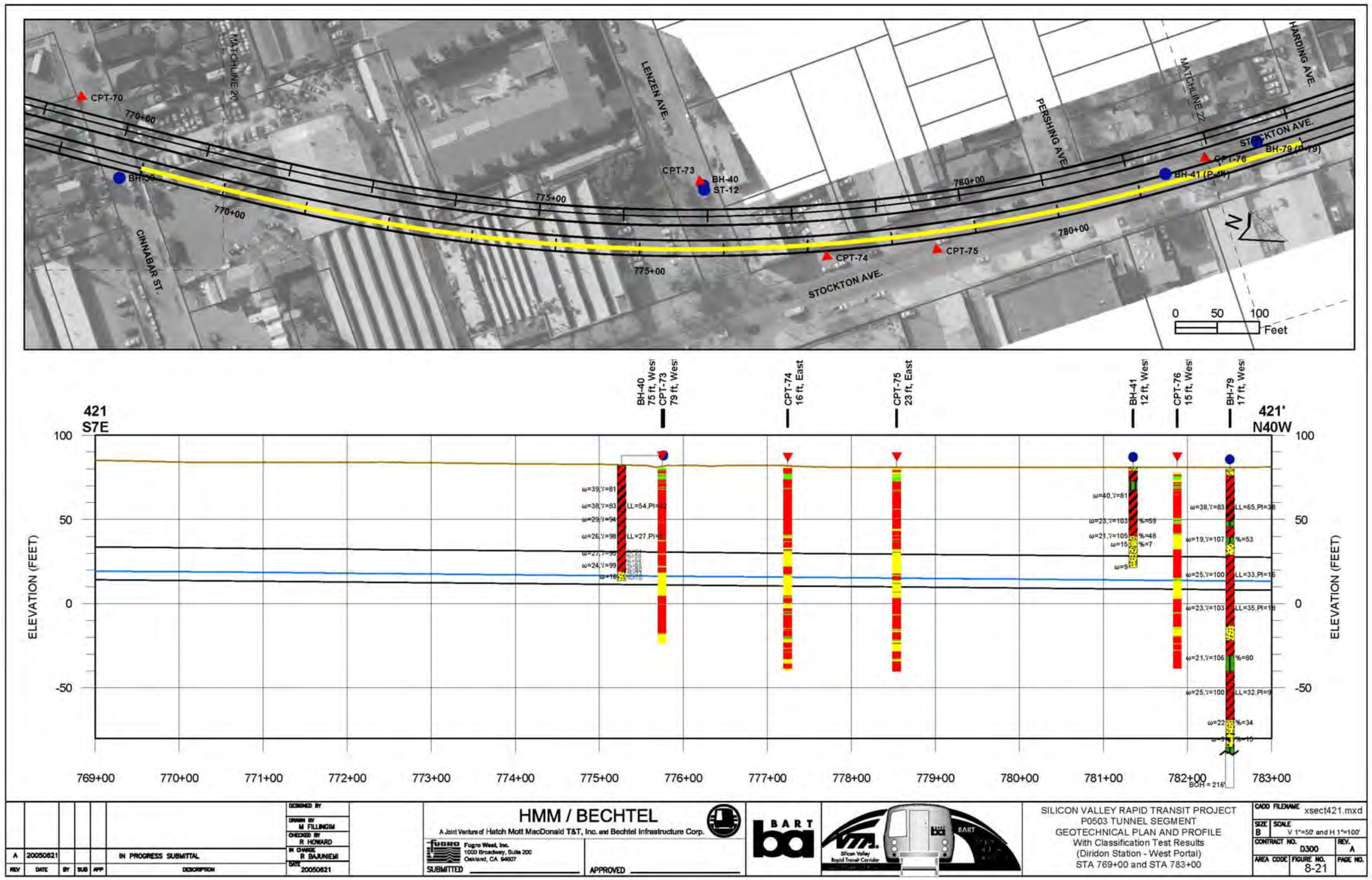


Figure 8-21. Geotechnical Plan and Profile with Classification Test Results: Station 769+00 to 783+00.

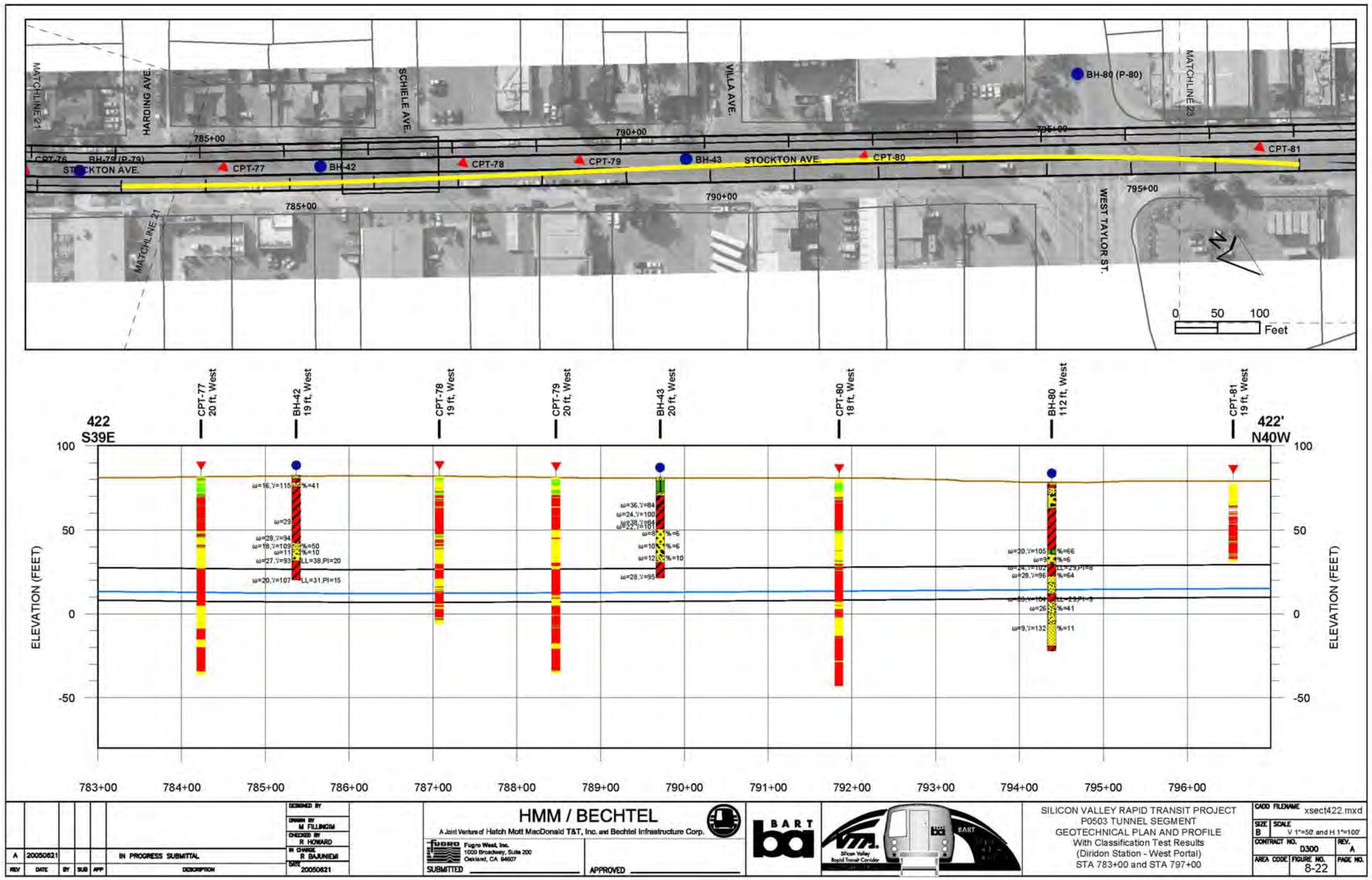


Figure 8-22. Geotechnical Plan and Profile with Classification Test Results: Station 783+00 to 797+00.

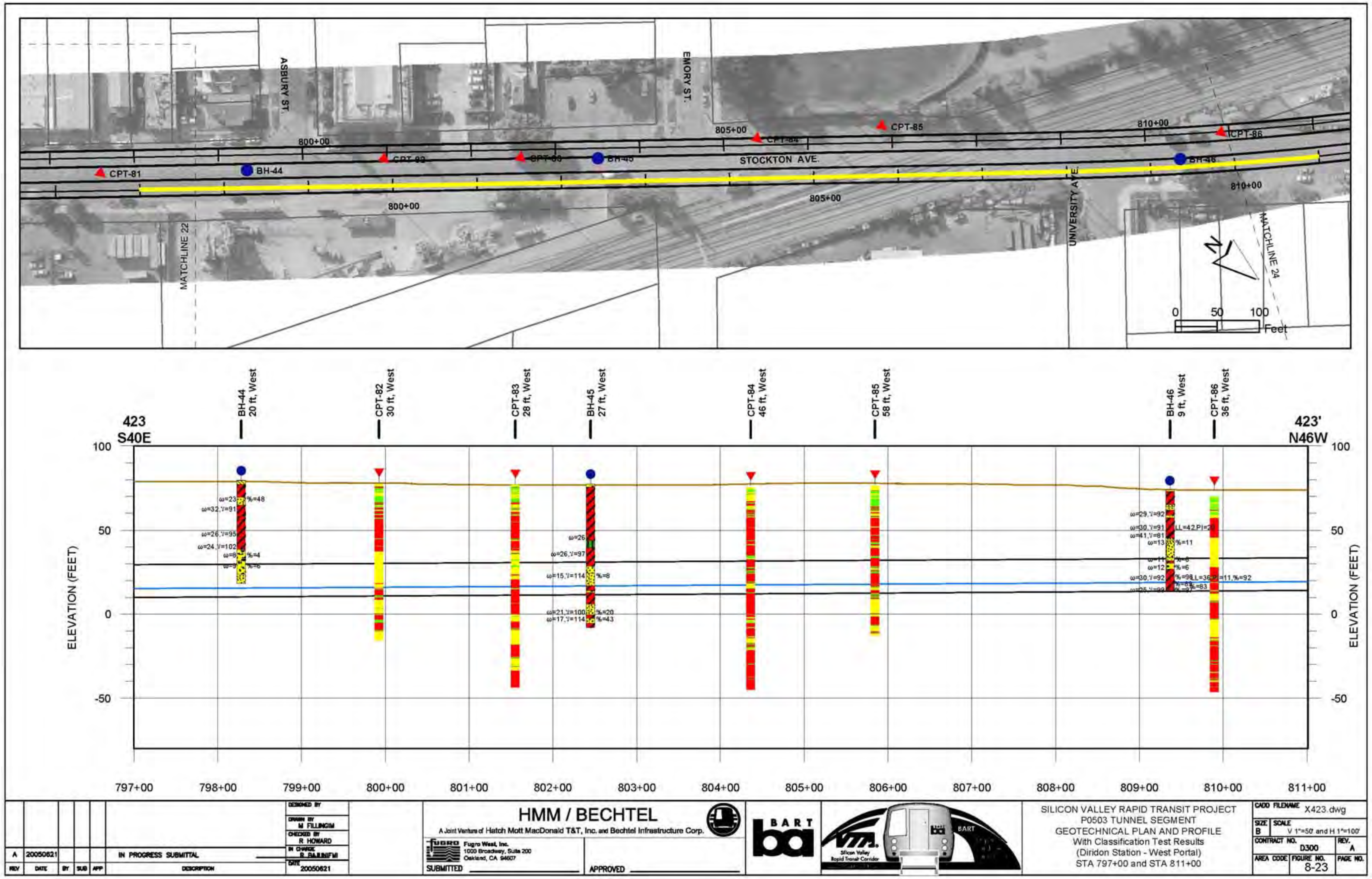


Figure 8-23. Geotechnical Plan and Profile with Classification Test Results: Station 797+00 to 811+00.

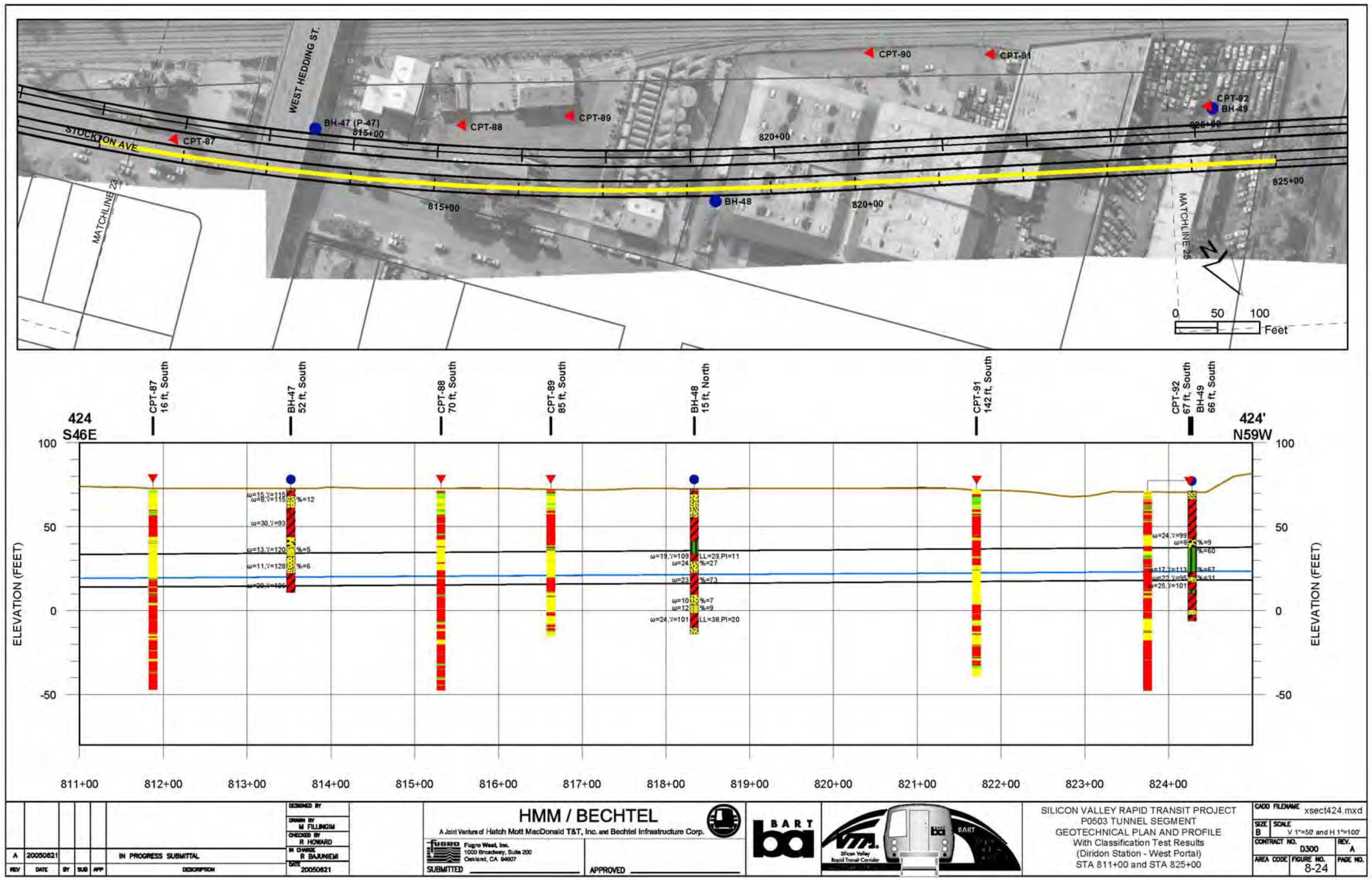


Figure 8-24. Geotechnical Plan and Profile with Classification Test Results: Station 811+00 to 825+00.

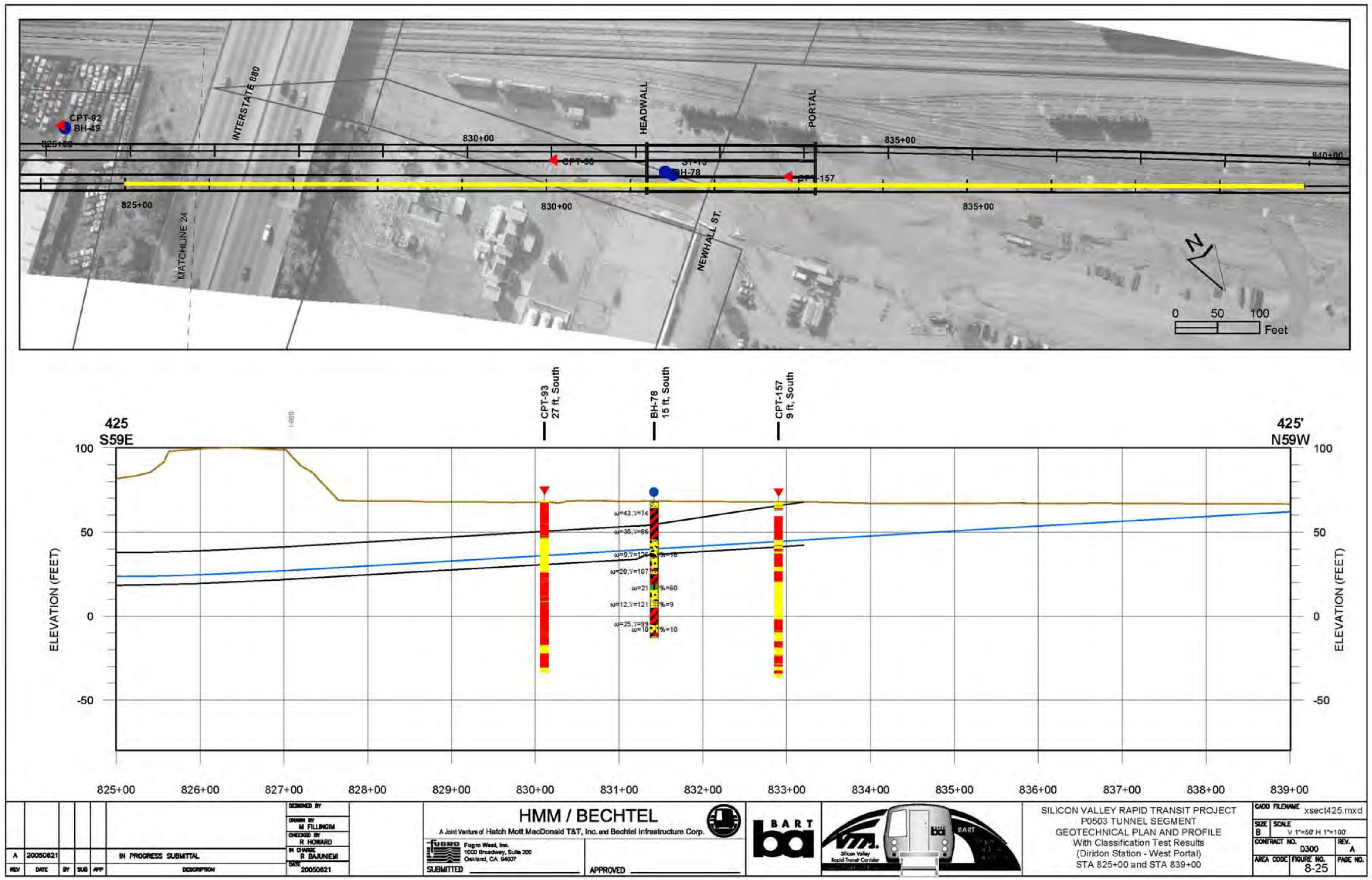


Figure 8-25. Geotechnical Plan and Profile with Classification Test Results: Station 825+00 to 839+00.

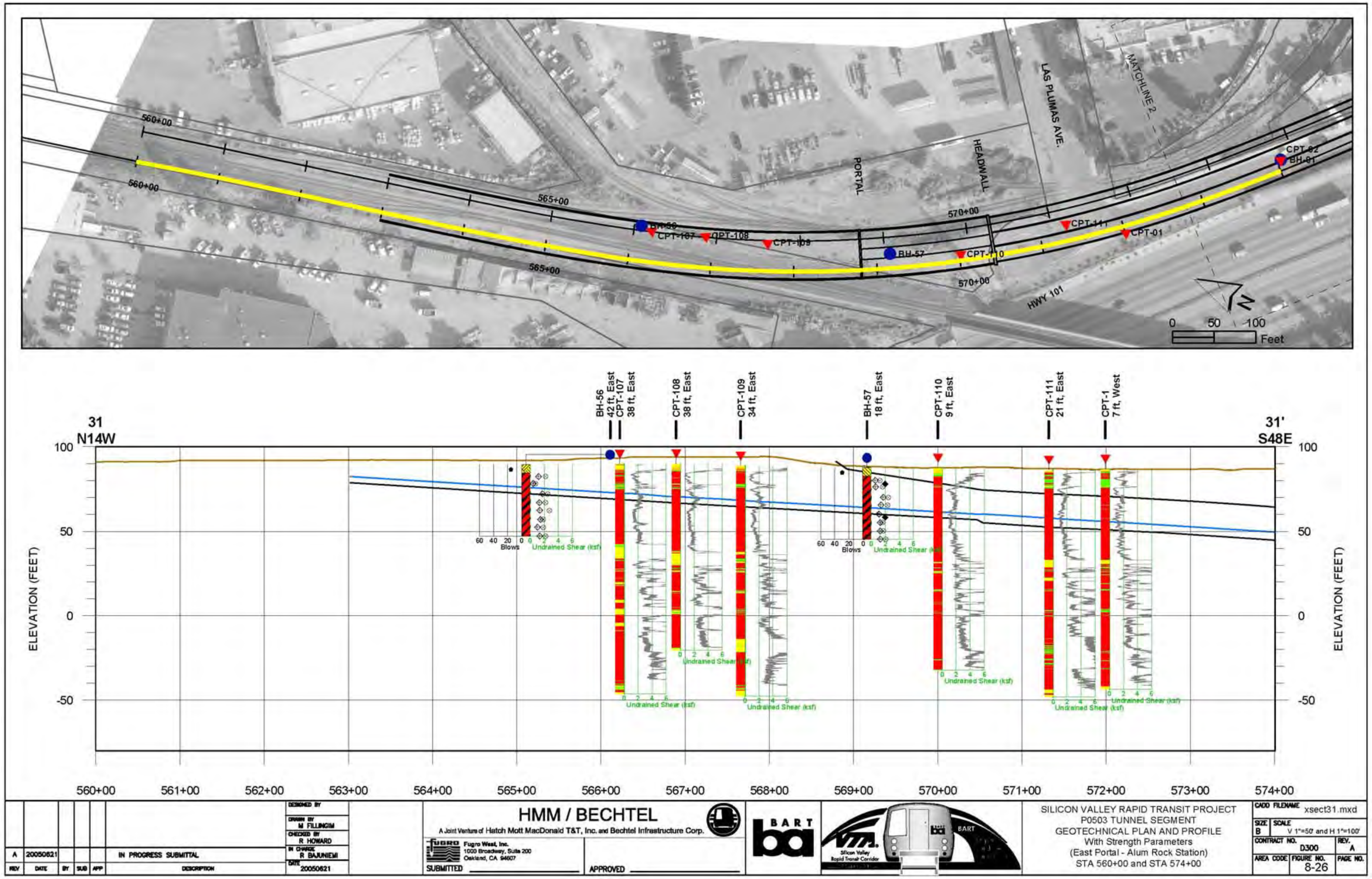


Figure 8-26. Geotechnical Plan and Profile with Strength Parameters: 560+00 to 574+00.

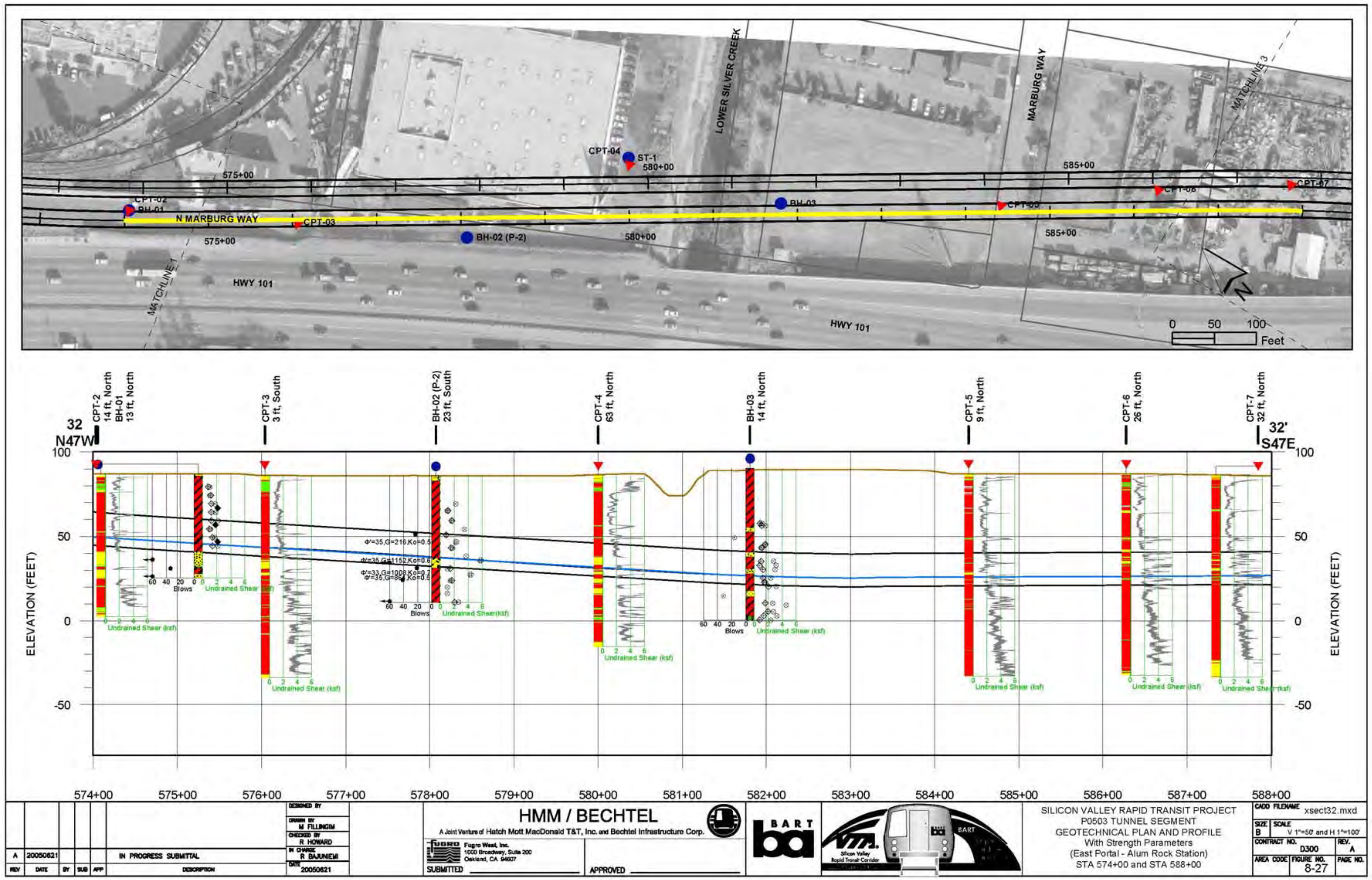


Figure 8-27. Geotechnical Plan and Profile with Strength Parameters: 574+00 to 588+00.

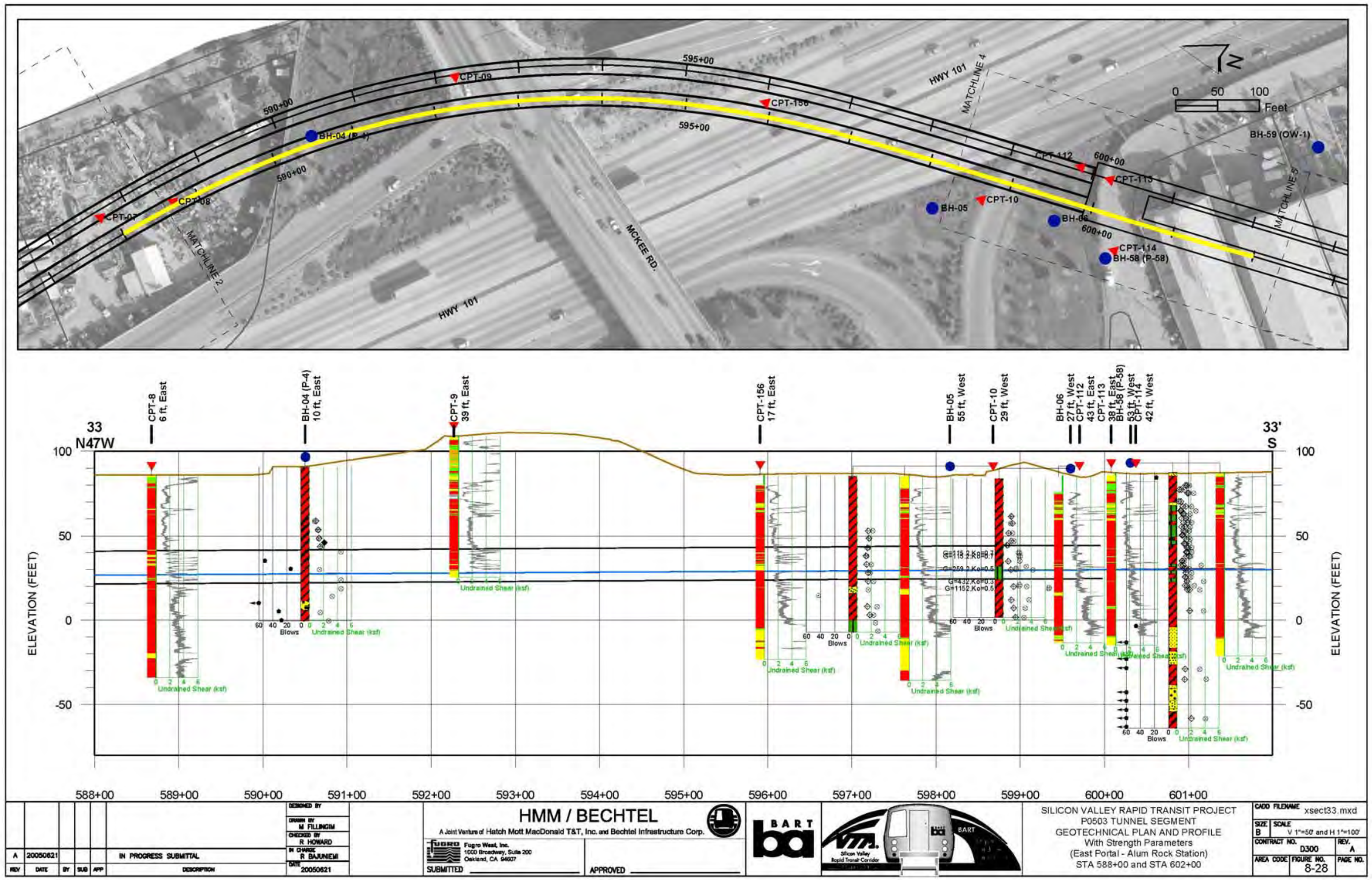


Figure 8-28. Geotechnical Plan and Profile with Strength Parameters: 588+00 to 602+00.

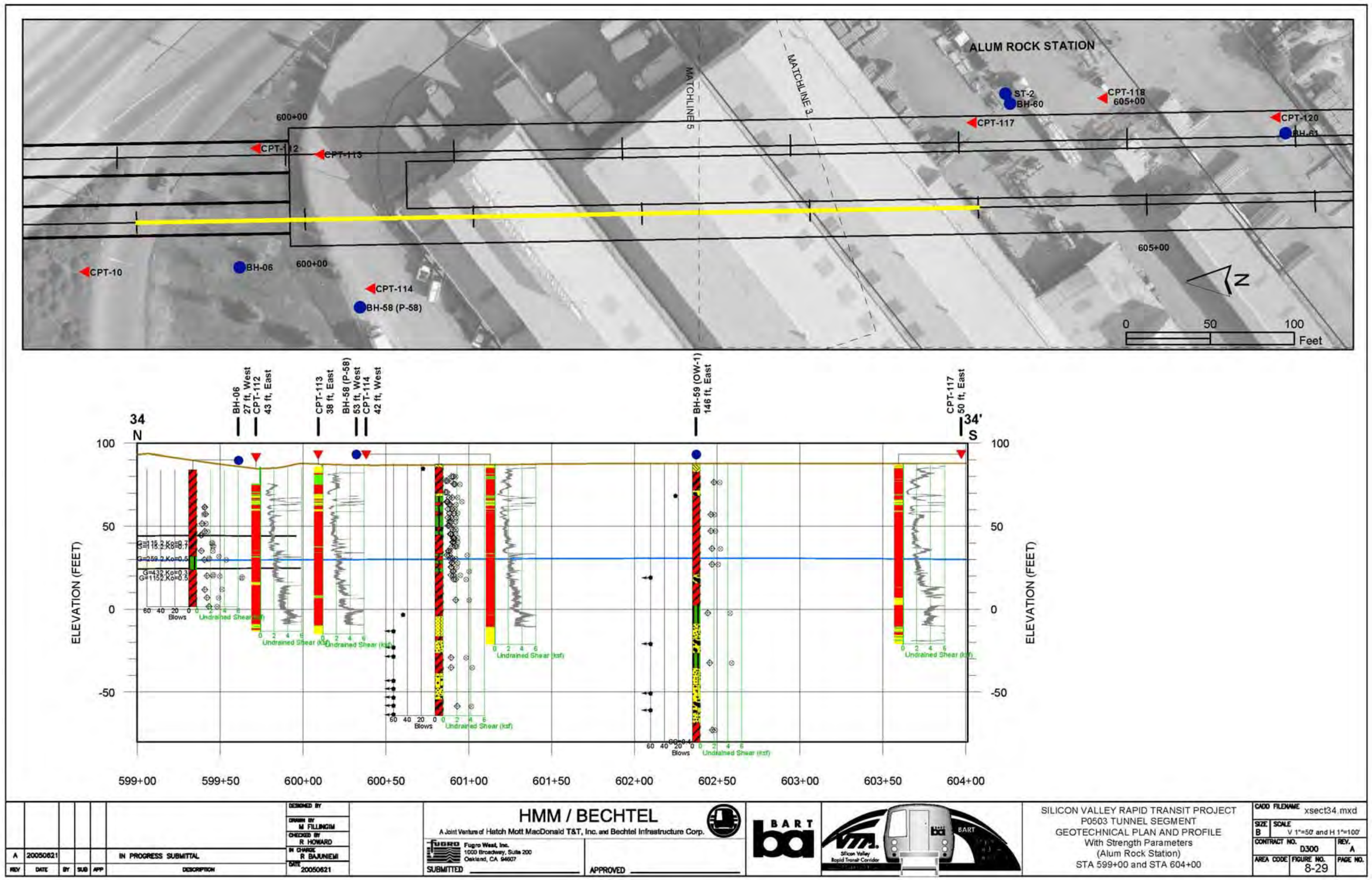


Figure 8-29. Geotechnical Plan and Profile with Strength Parameters: 599+00 to 604+00.

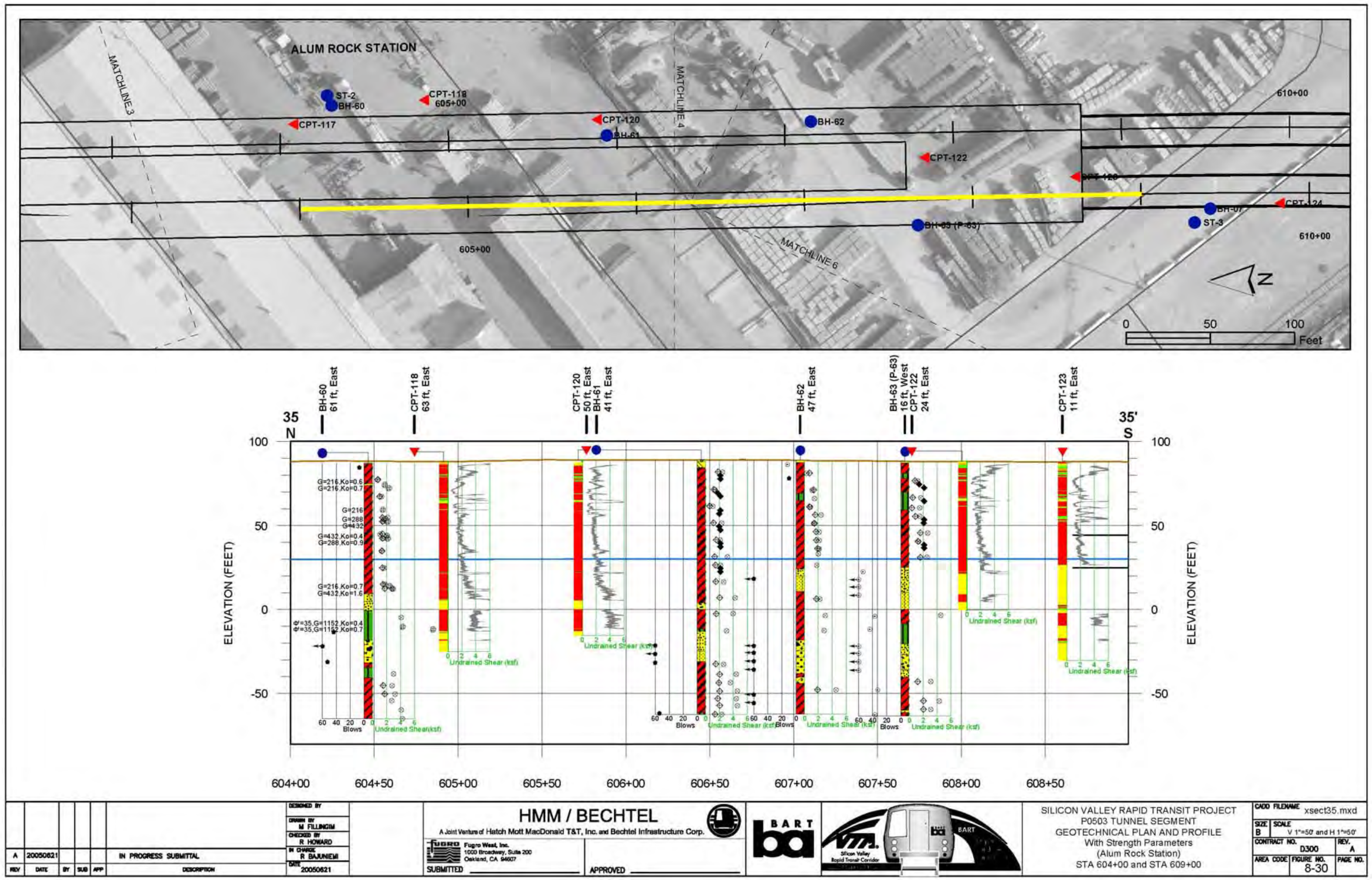


Figure 8-30. Geotechnical Plan and Profile with Strength Parameters: 604+00 to 609+00.

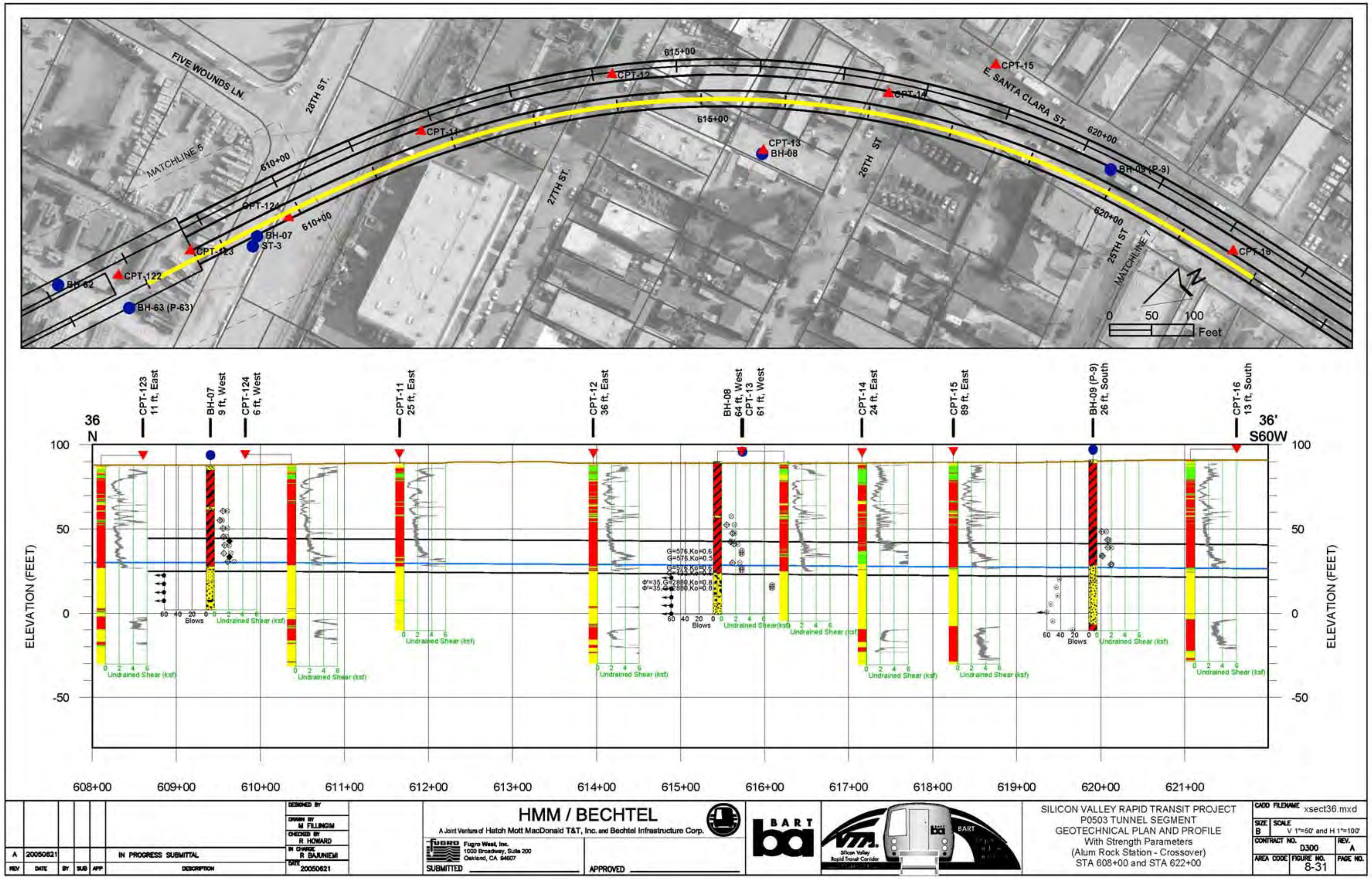


Figure 8-31. Geotechnical Plan and Profile with Strength Parameters: 608+00 to 622+00.

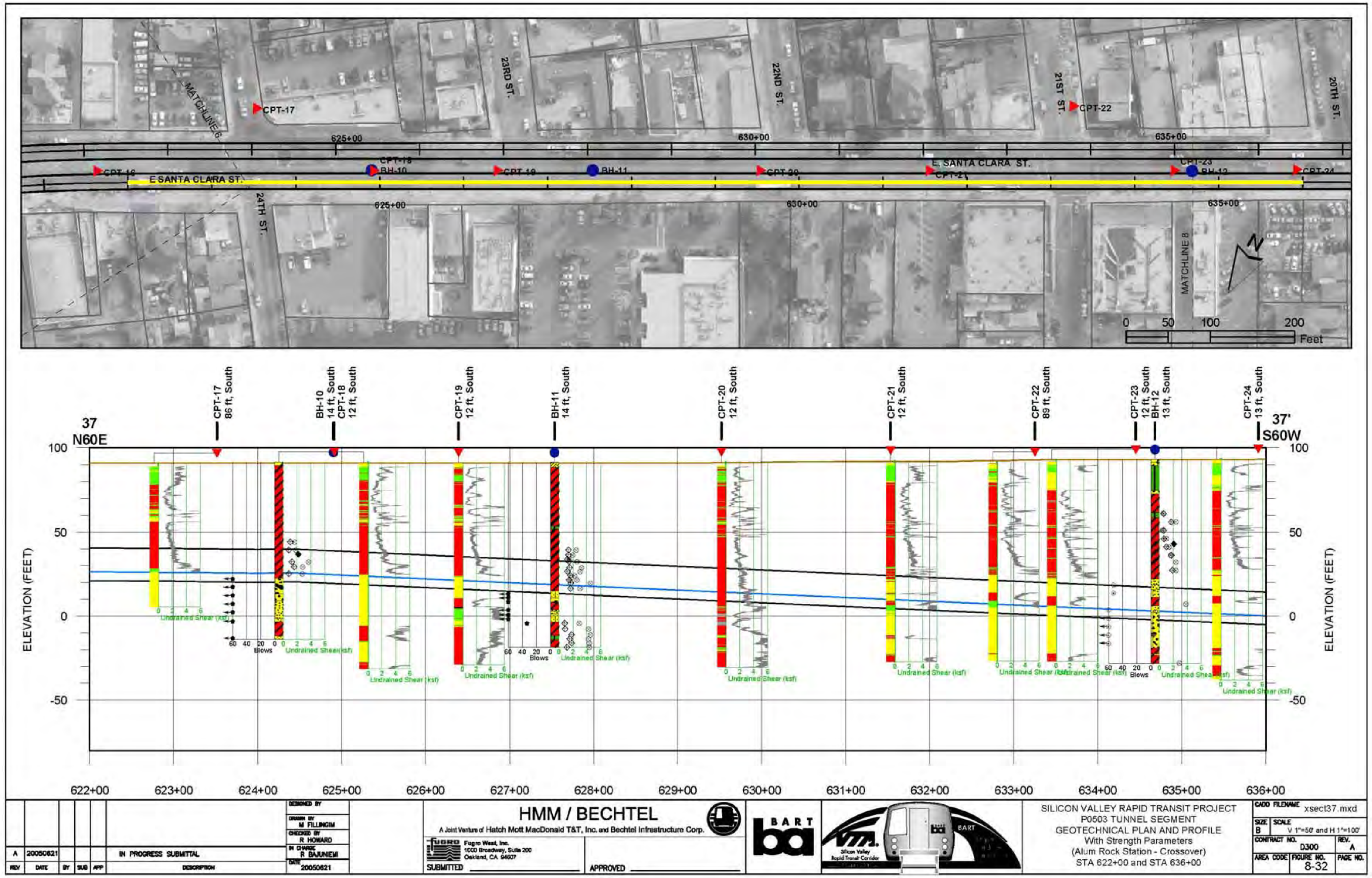


Figure 8-32. Geotechnical Plan and Profile with Strength Parameters: 622+00 to 636+00.

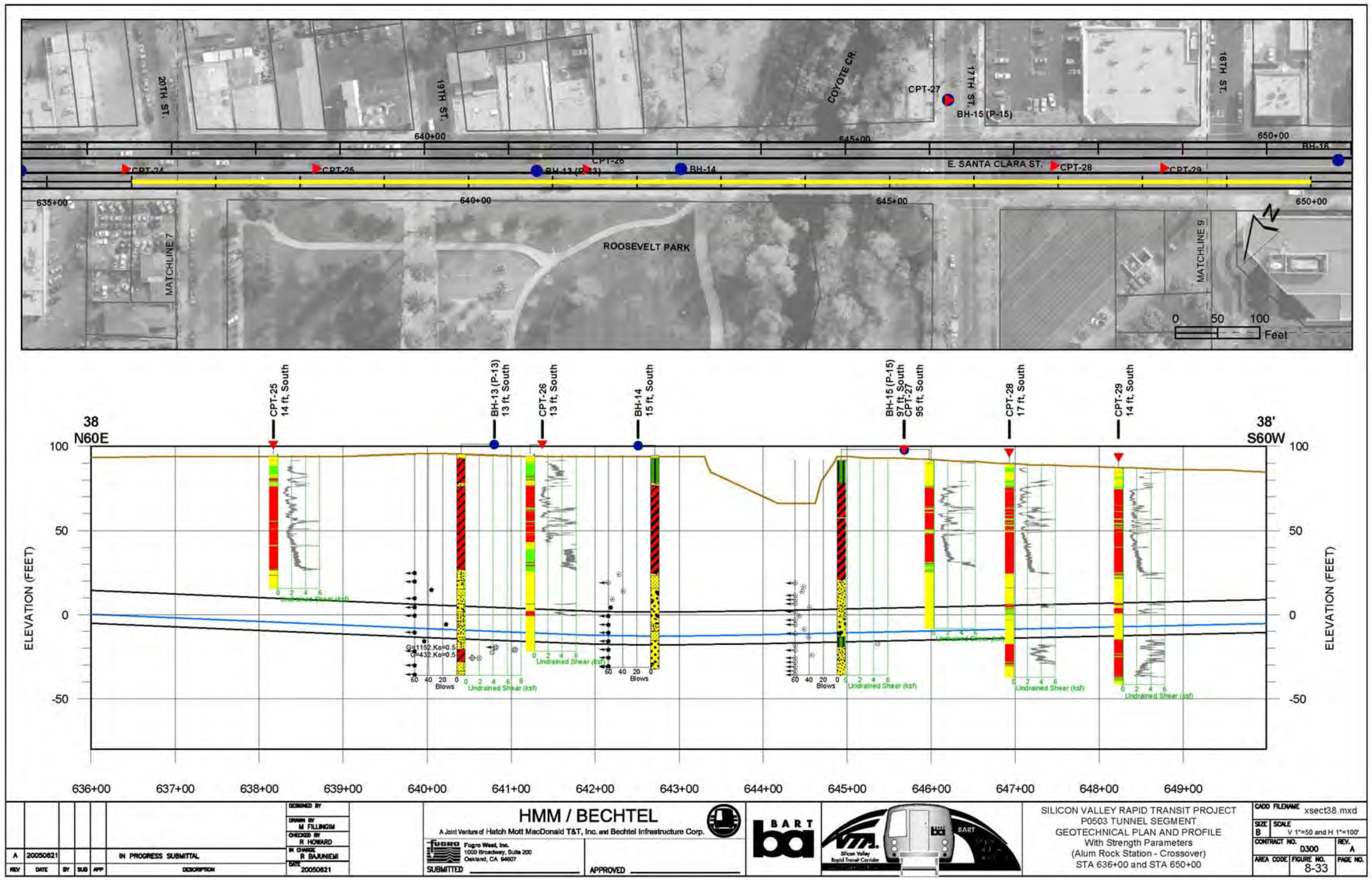


Figure 8-33. Geotechnical Plan and Profile with Strength Parameters: 636+00 to 650+00.

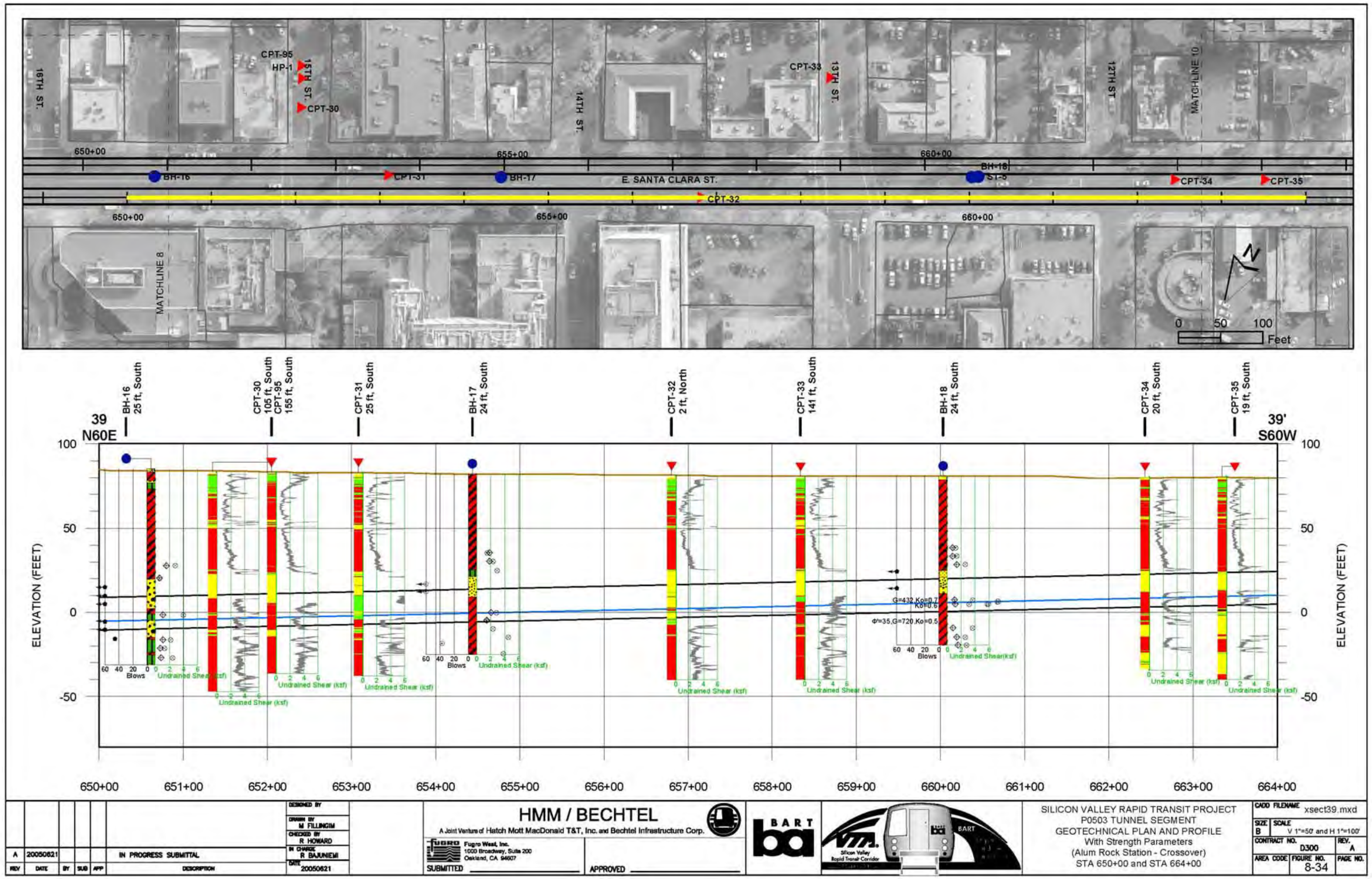


Figure 8-34. Geotechnical Plan and Profile with Strength Parameters: 650+00 to 664+00.

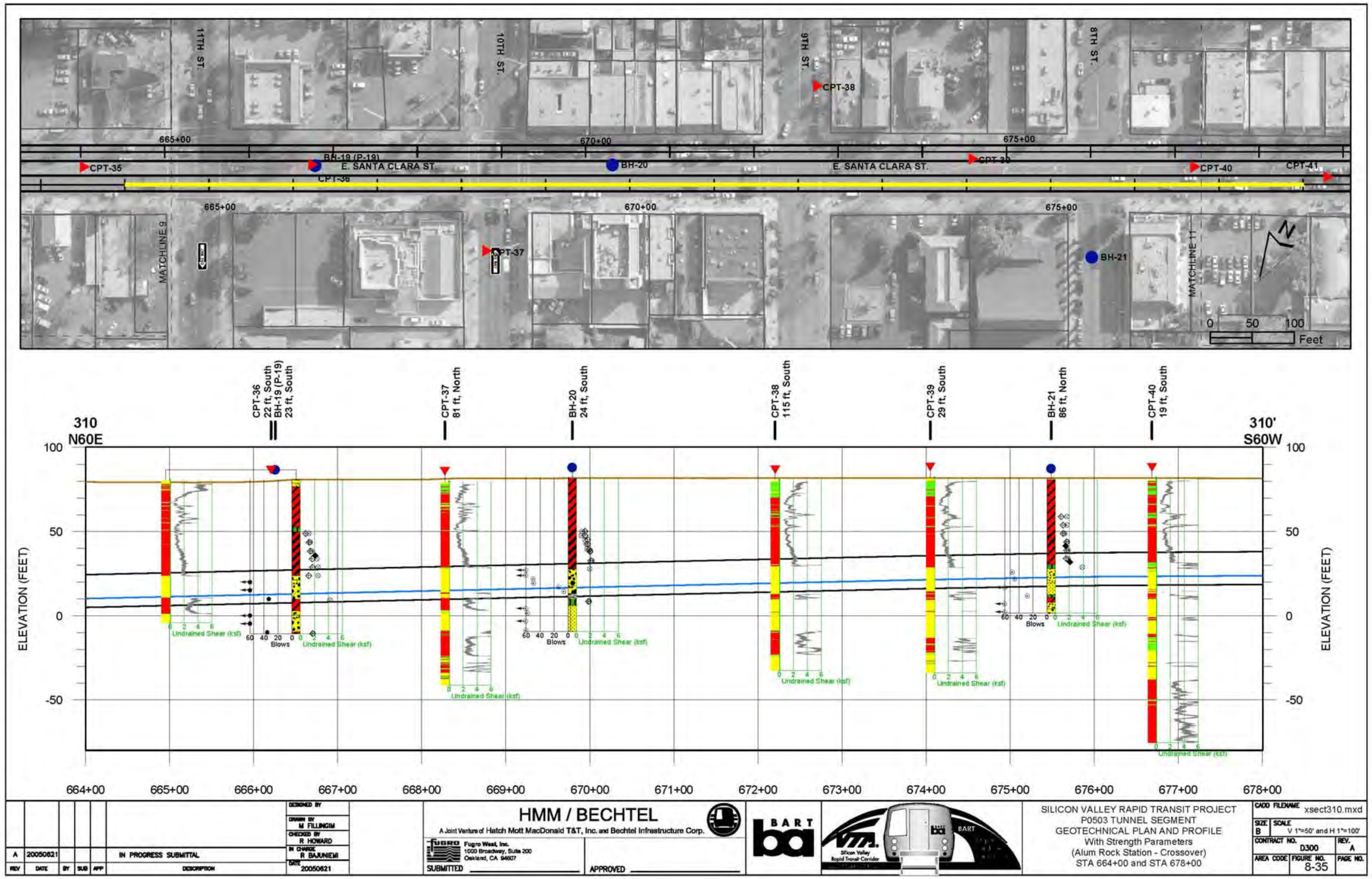


Figure 8-35. Geotechnical Plan and Profile with Strength Parameters: 664+00 to 678+00.

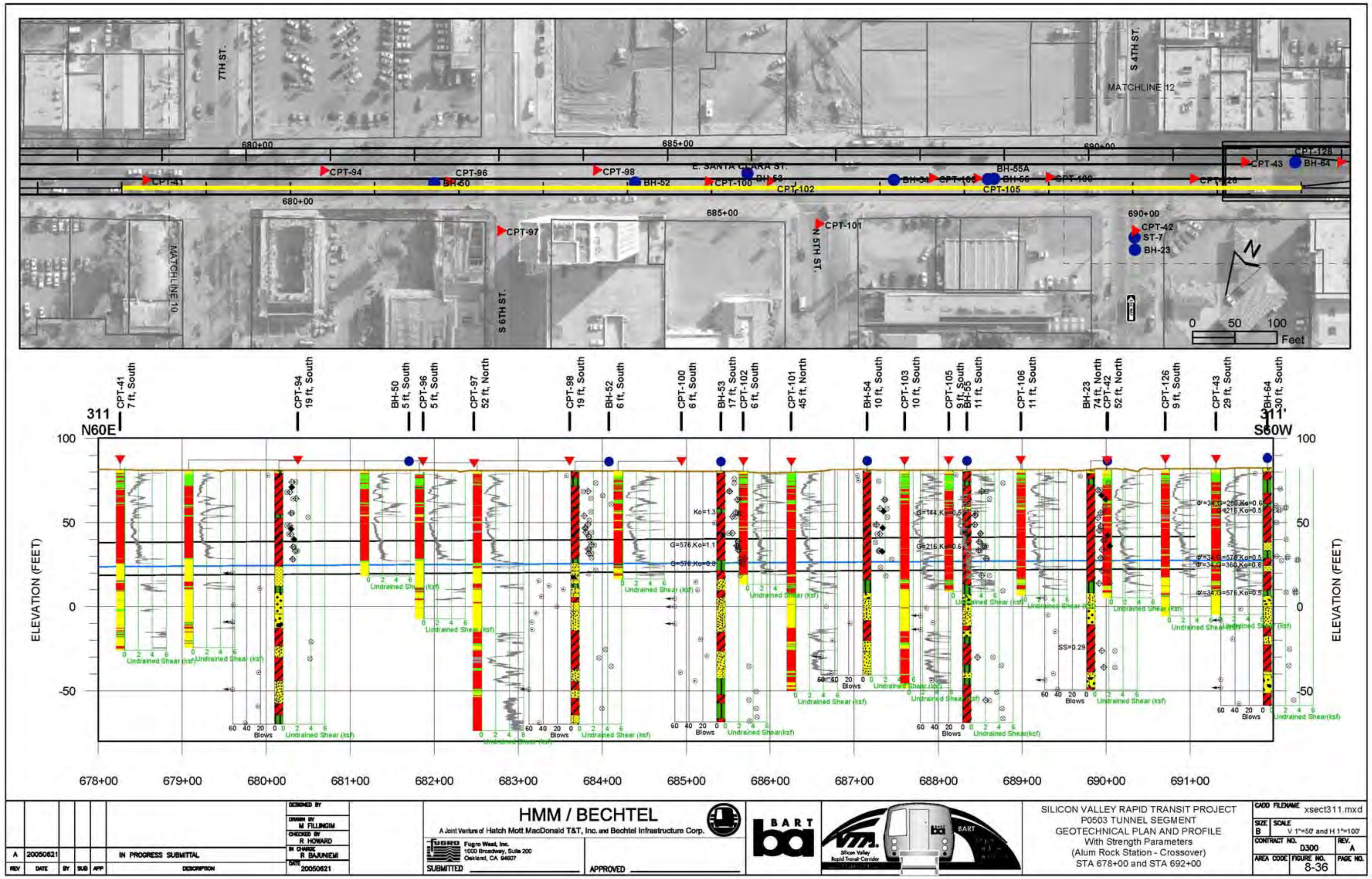


Figure 8-36. Geotechnical Plan and Profile with Strength Parameters: 678+00 to 692+00.

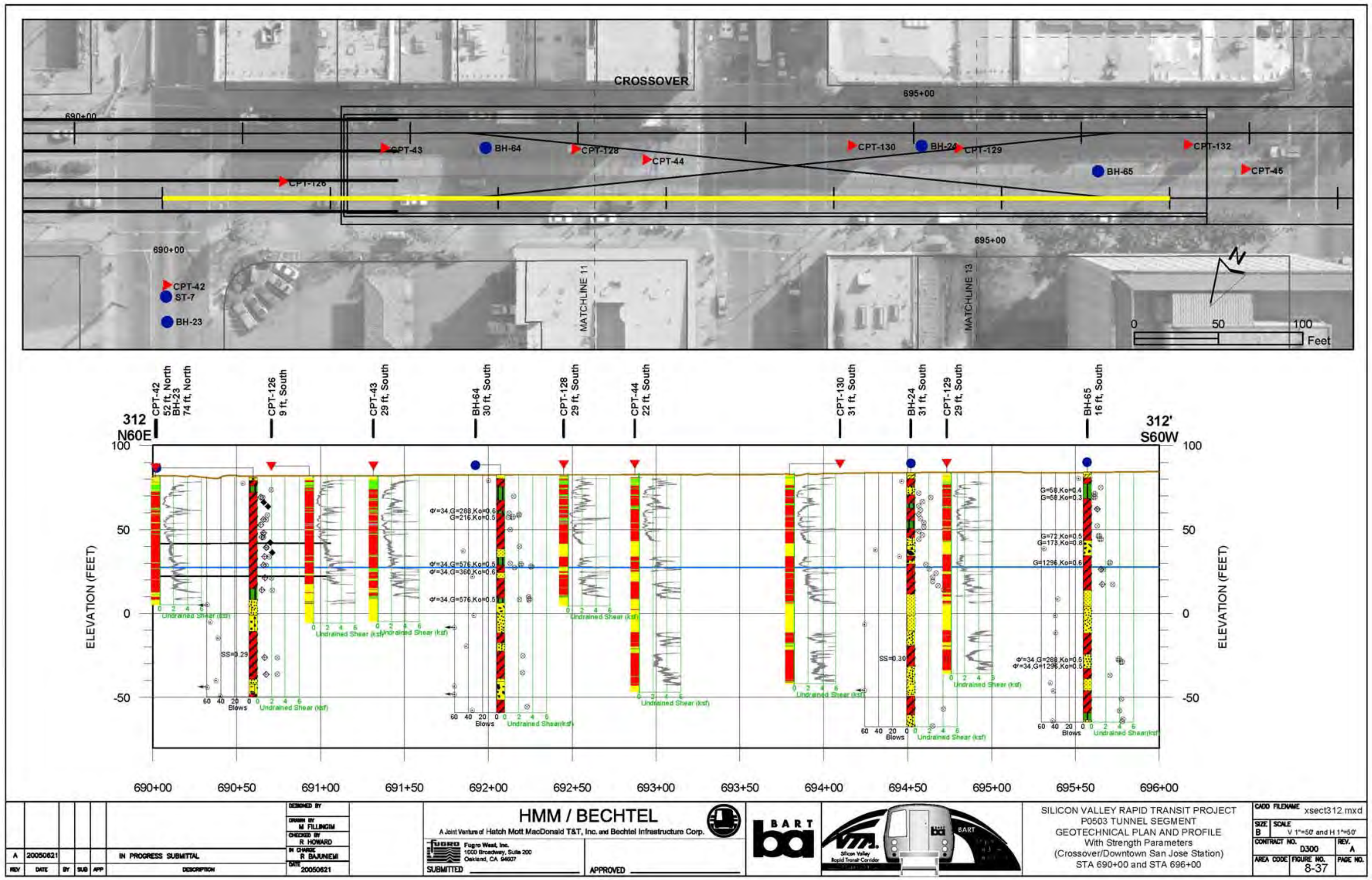


Figure 8-37. Geotechnical Plan and Profile with Strength Parameters: 690+00 to 696+00.

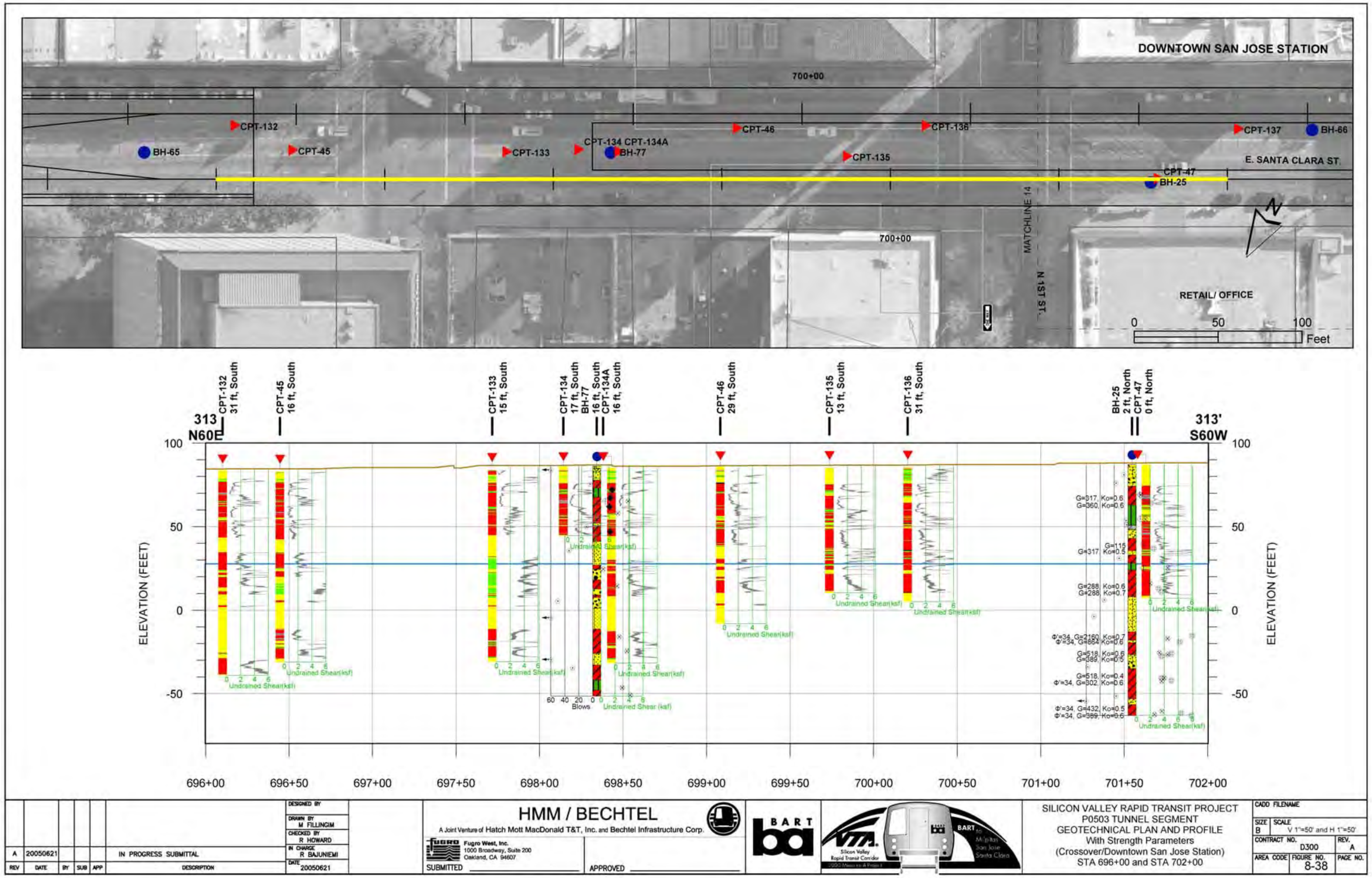


Figure 8-38. Geotechnical Plan and Profile with Strength Parameters: 696+00 to 702+00.

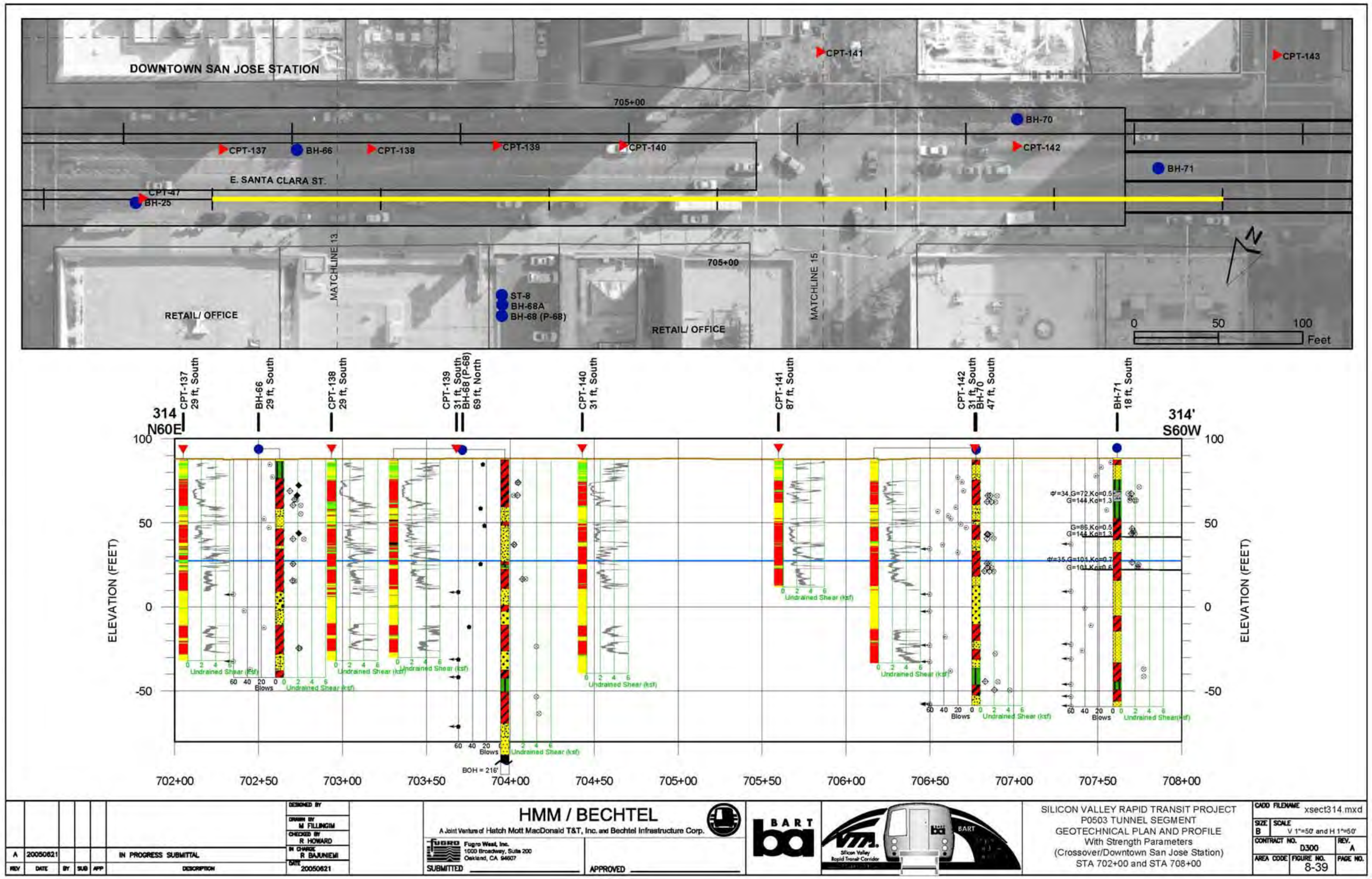


Figure 8-39. Geotechnical Plan and Profile with Strength Parameters: 702+00 to 708+00.

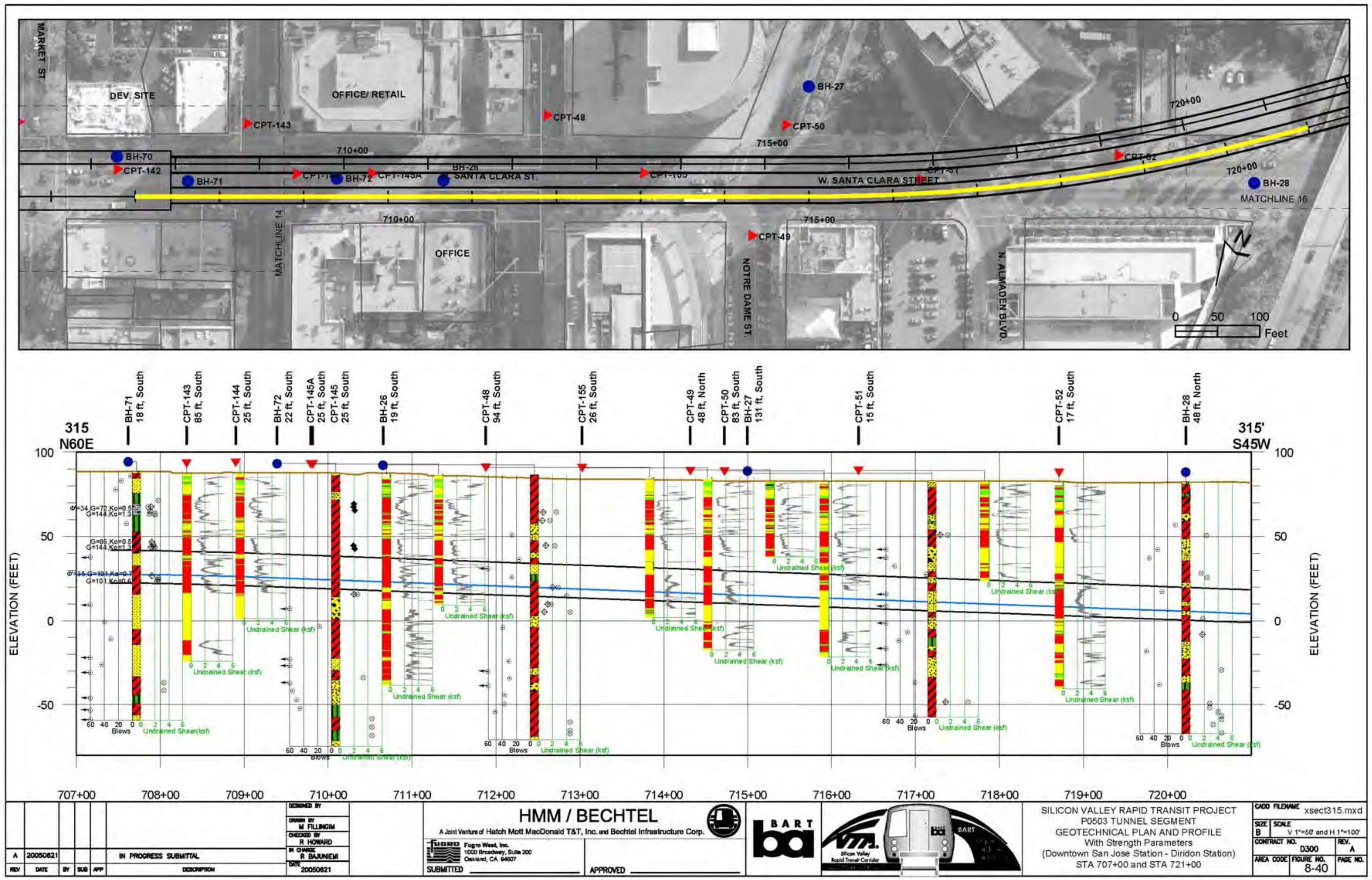


Figure 8-40. Geotechnical Plan and Profile with Strength Parameters: 707+00 to 721+00.

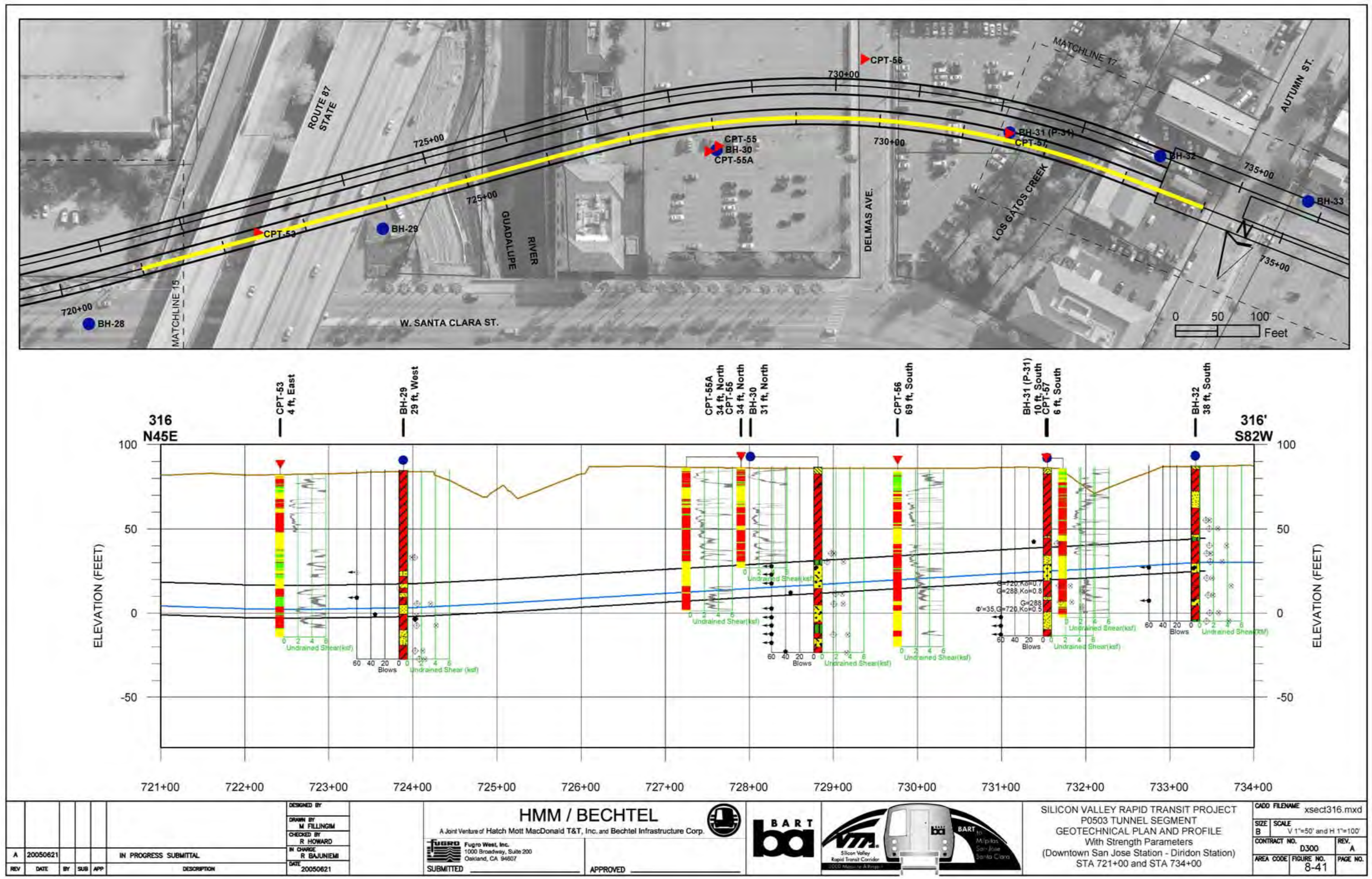


Figure 8-41. Geotechnical Plan and Profile with Strength Parameters: 721+00 to 734+00.

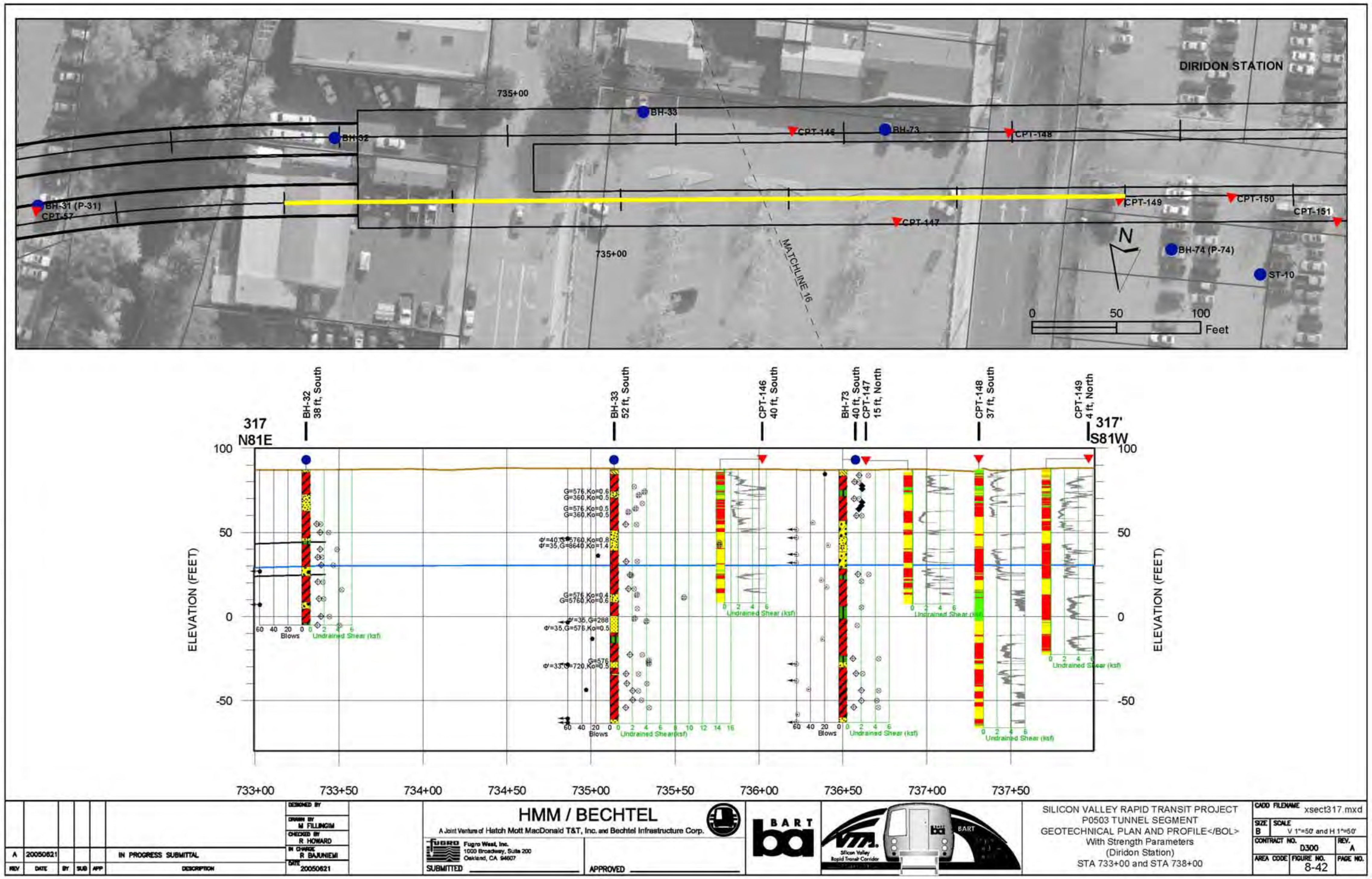


Figure 8-42. Geotechnical Plan and Profile with Strength Parameters: 733+00 to 738+00.

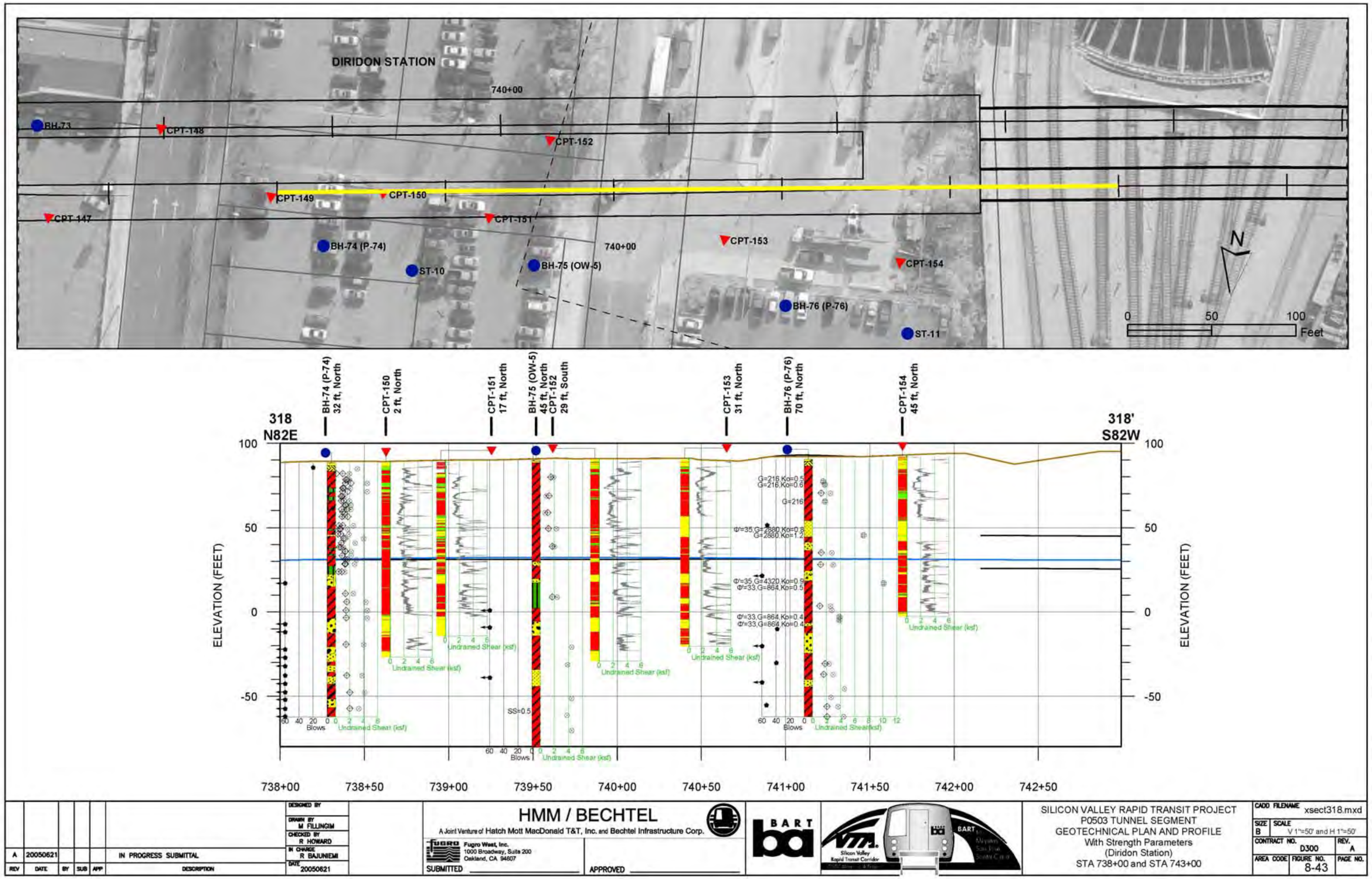


Figure 8-43. Geotechnical Plan and Profile with Strength Parameters: 738+00 to 743+00.

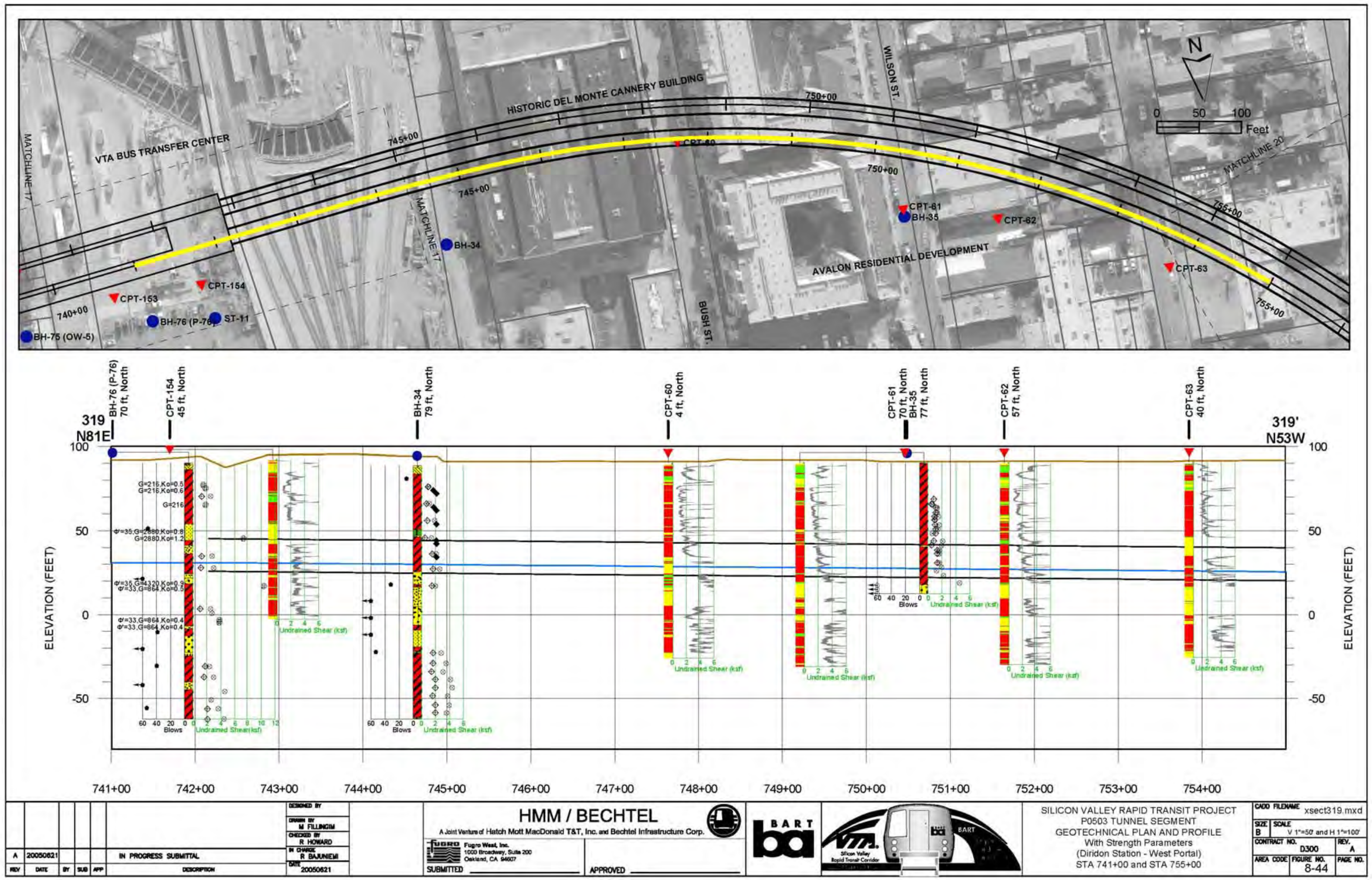


Figure 8-44. Geotechnical Plan and Profile with Strength Parameters: 741+00 to 755+00.

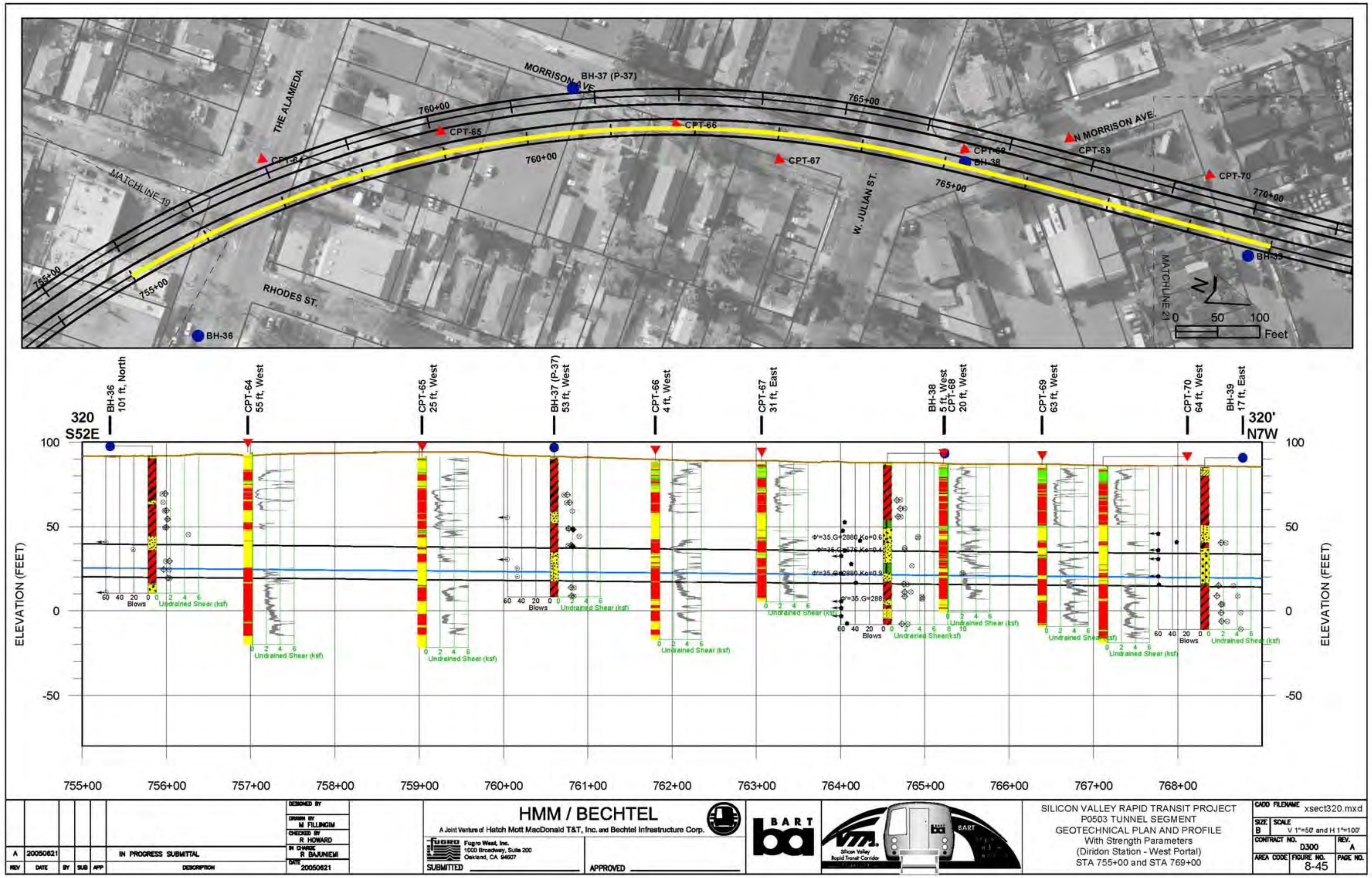


Figure 8-45. Geotechnical Plan and Profile with Strength Parameters: 755+00 to 769+00.

Figure 8-46. Geotechnical Plan and Profile with Strength Parameters: 769+00 to 783+00.

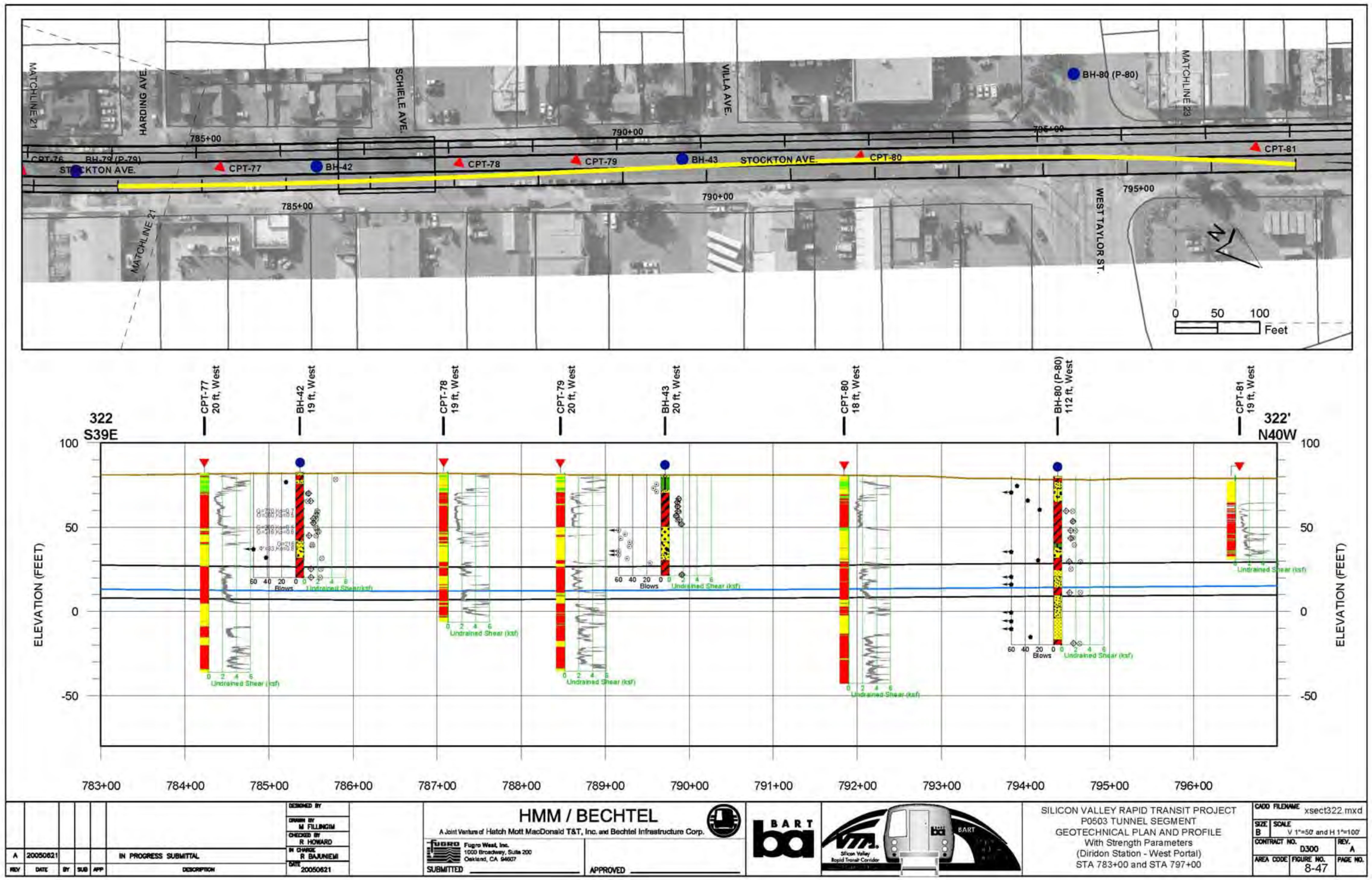


Figure 8-47. Geotechnical Plan and Profile with Strength Parameters: 783+00 to 797+00.

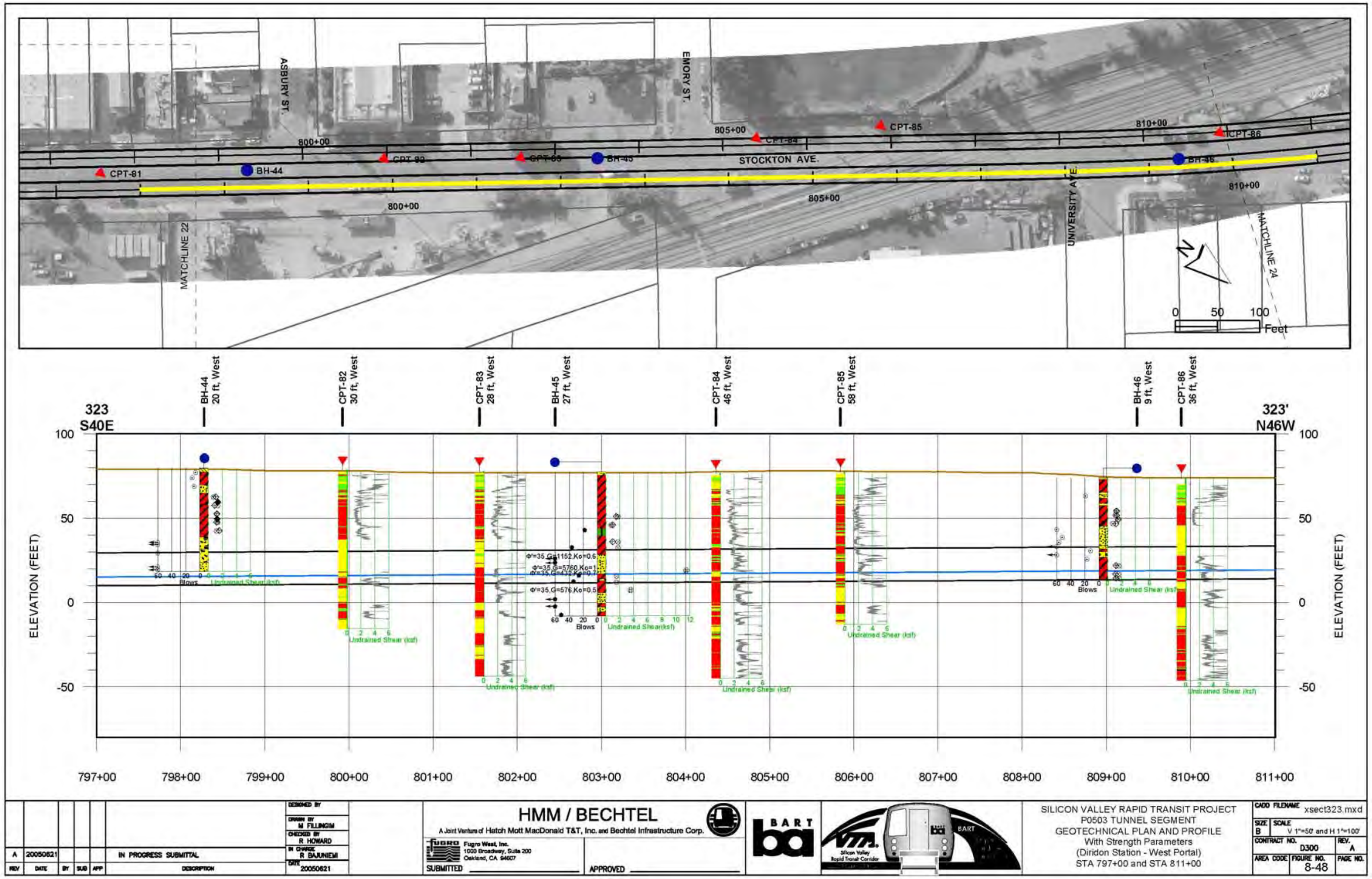


Figure 8-48. Geotechnical Plan and Profile with Strength Parameters: 797+00 to 811+00.

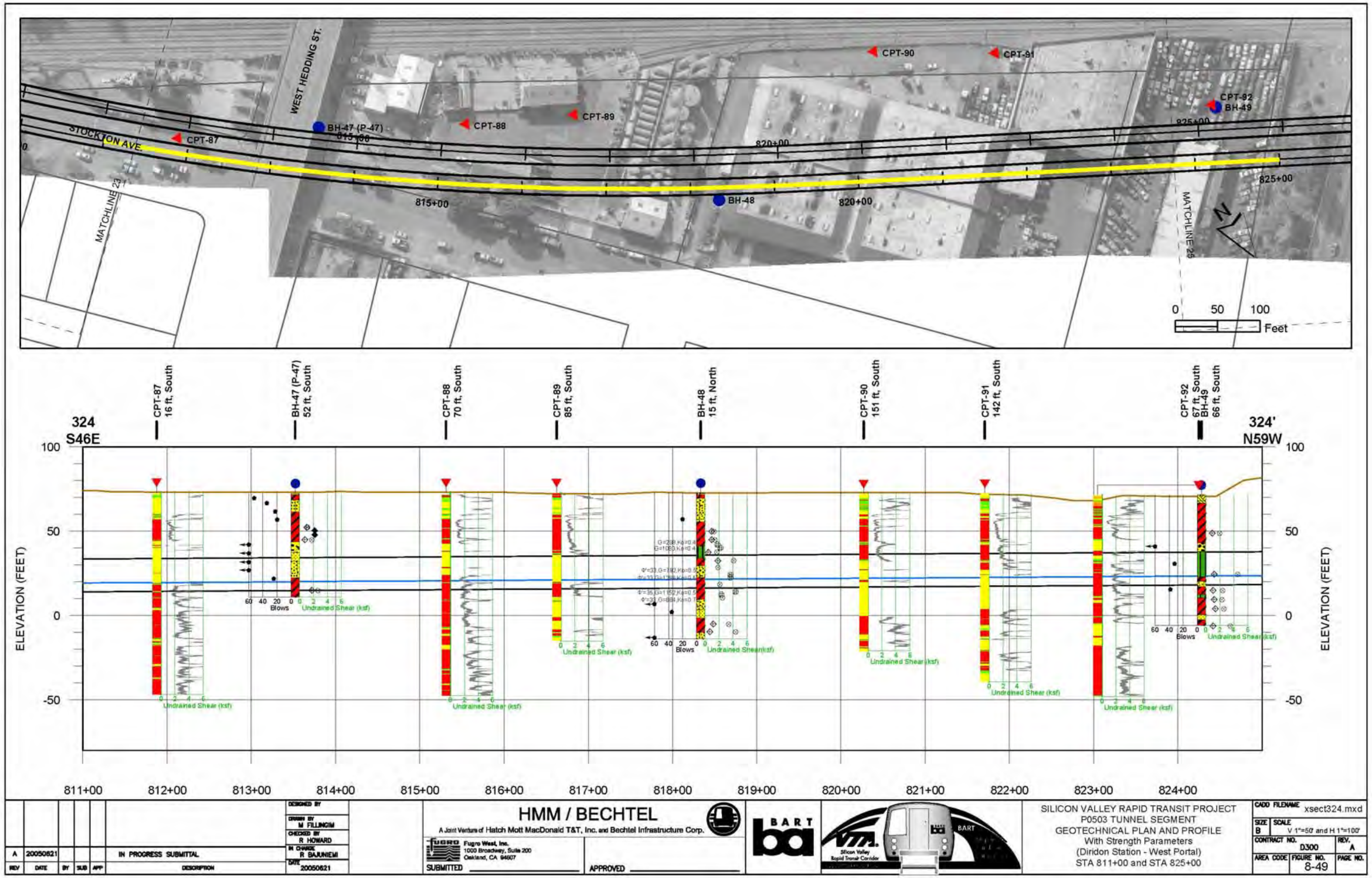


Figure 8-49. Geotechnical Plan and Profile with Strength Parameters: 811+00 to 825+00.

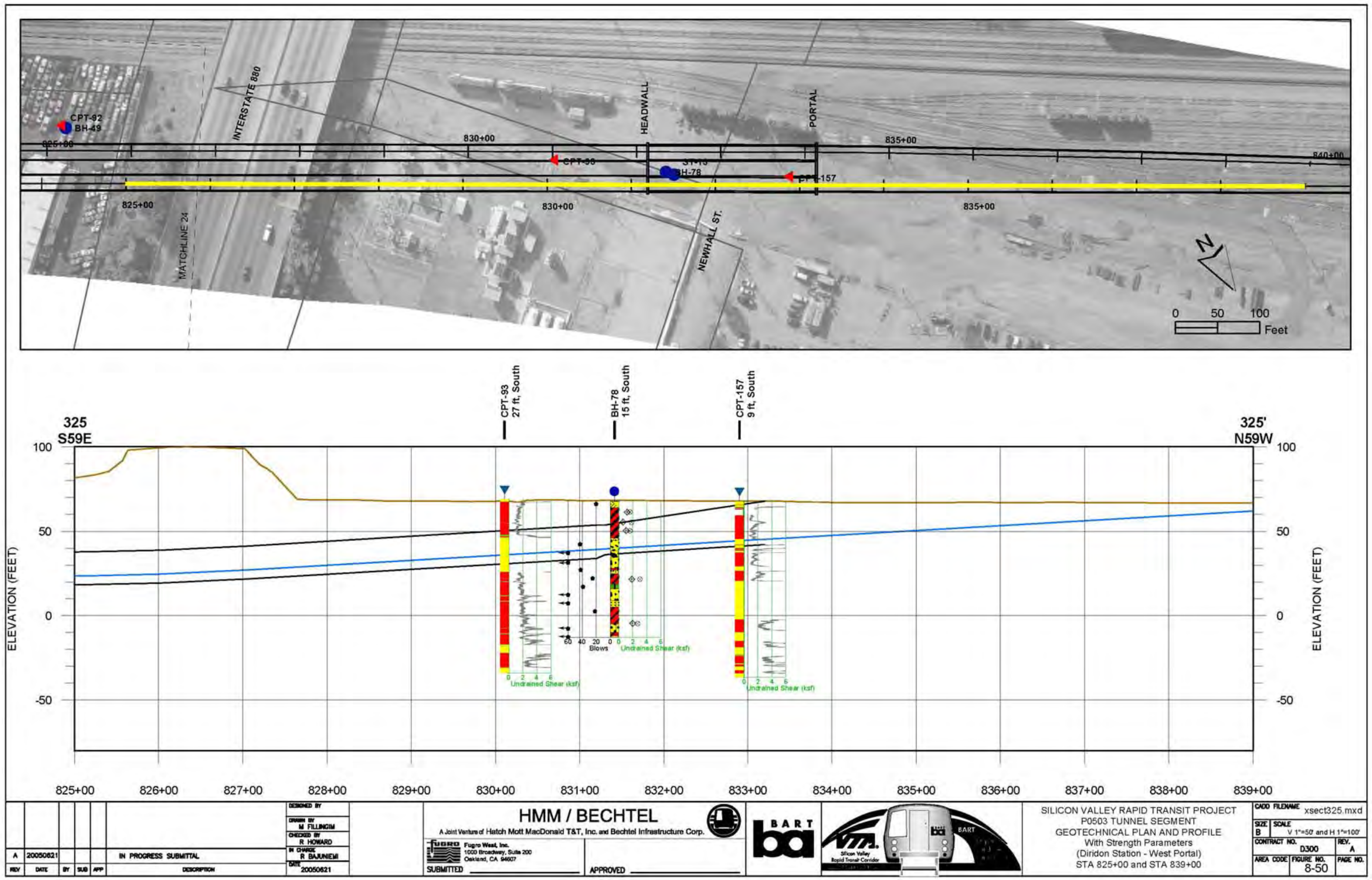


Figure 8-50. Geotechnical Plan and Profile with Strength Parameters: 825+00 to 839+00.

This page intentionally left blank.

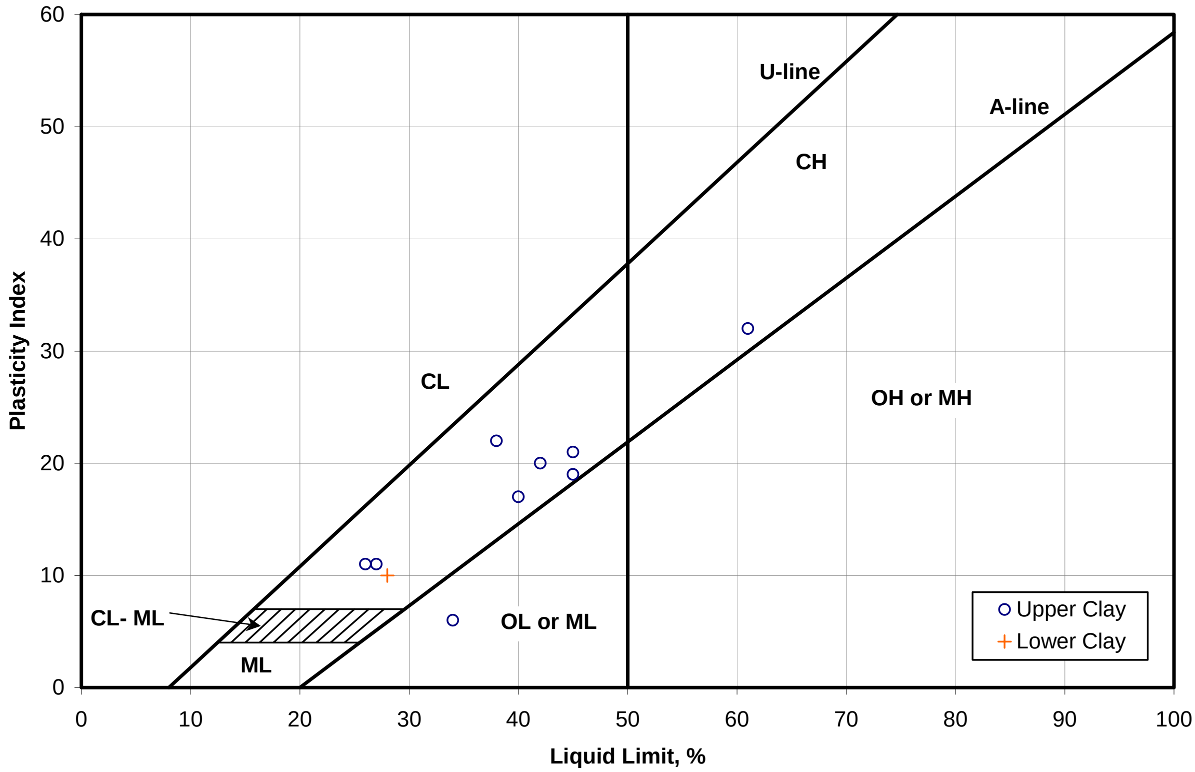


Figure 8-51. Plasticity Chart, Study Section 1: East Portal to Alum Rock Station.

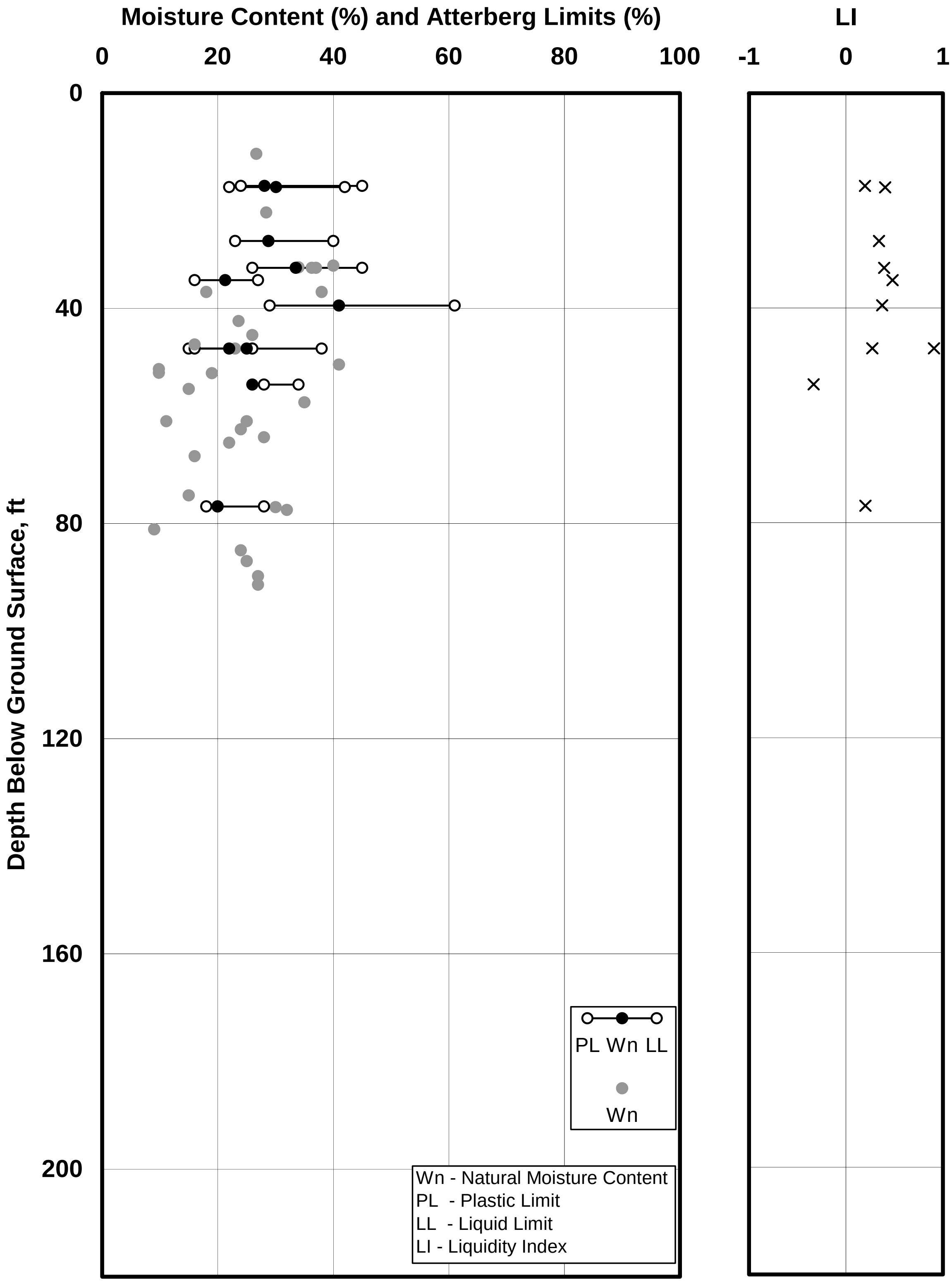


Figure 8-52. Moisture Content and Atterberg Limits, Study Section 1: East Portal to Alum Rock Station.

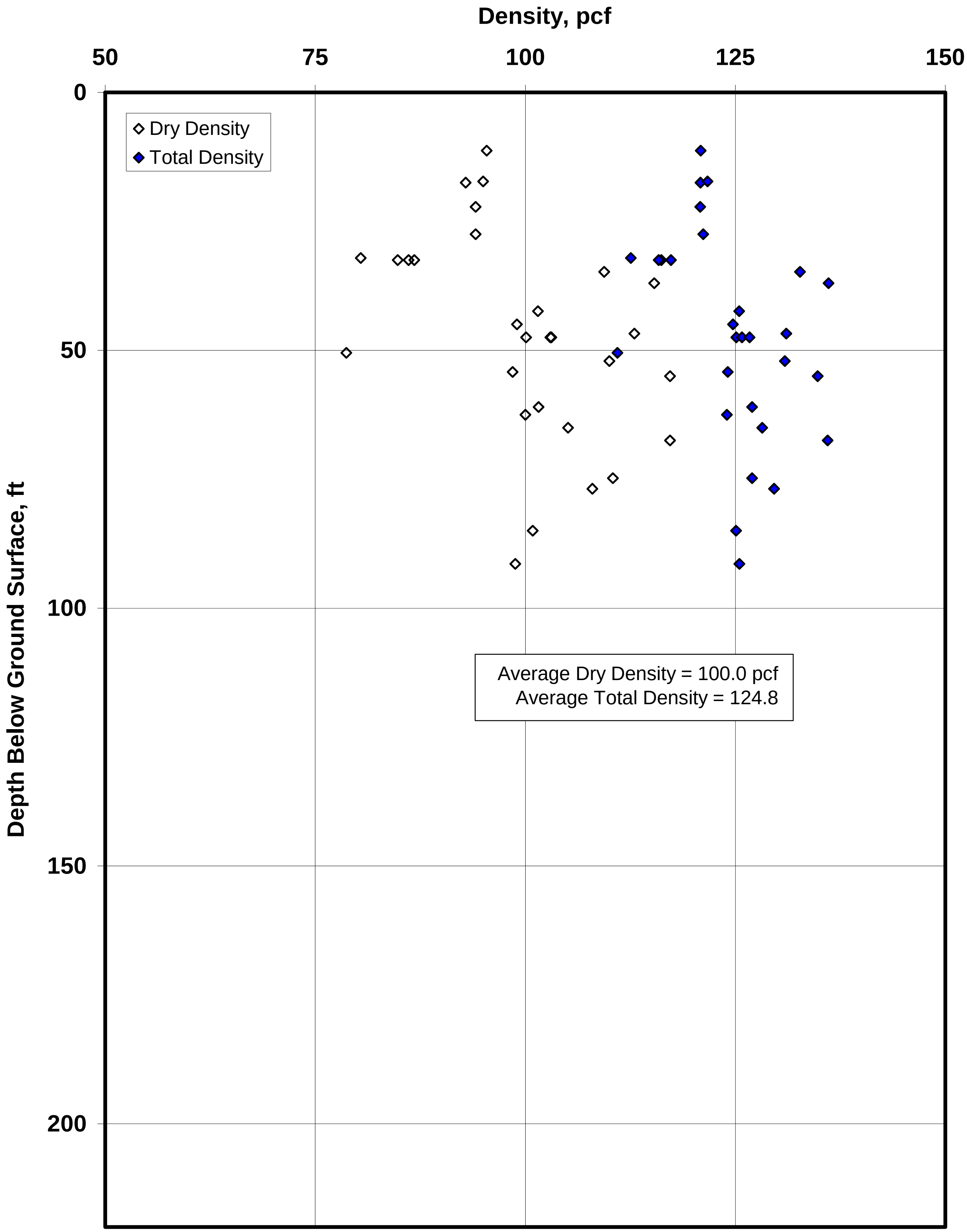


Figure 8-53. Total and Dry Densities, Study Section 1: East Portal to Alum Rock Station.

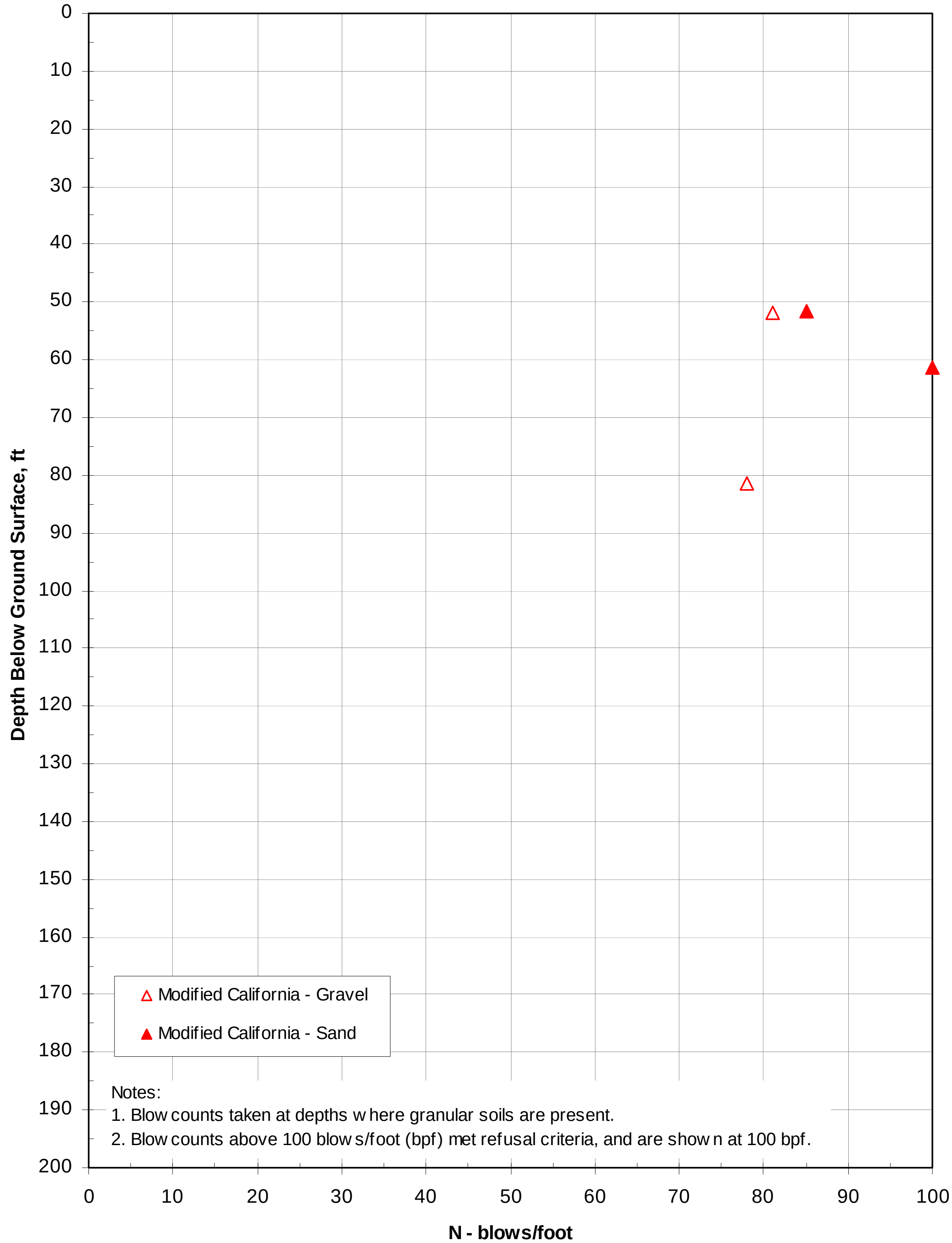


Figure 8-54. Uncorrected SPTs and Modified California Blow Counts, Study Section 1: East Portal to Alum Rock Station.

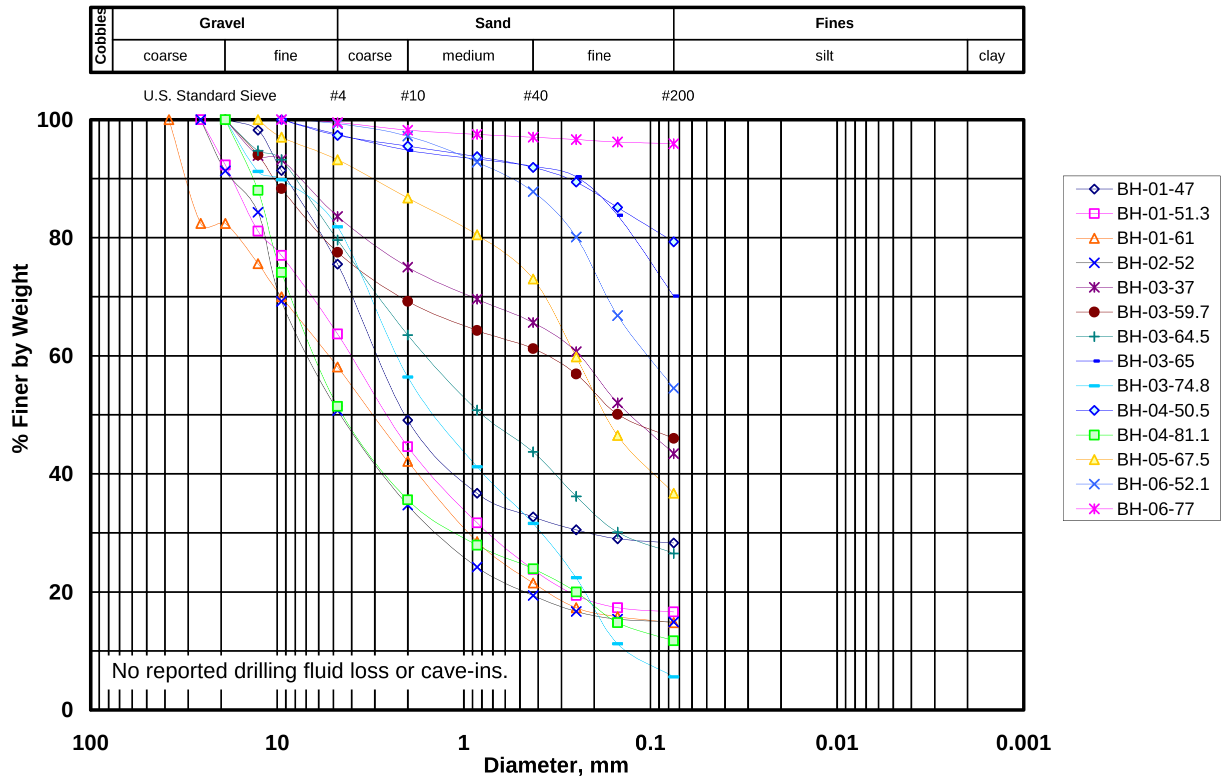


Figure 8-55. Grain Size Distribution, Study Section 1: East Portal to Alum Rock Station.

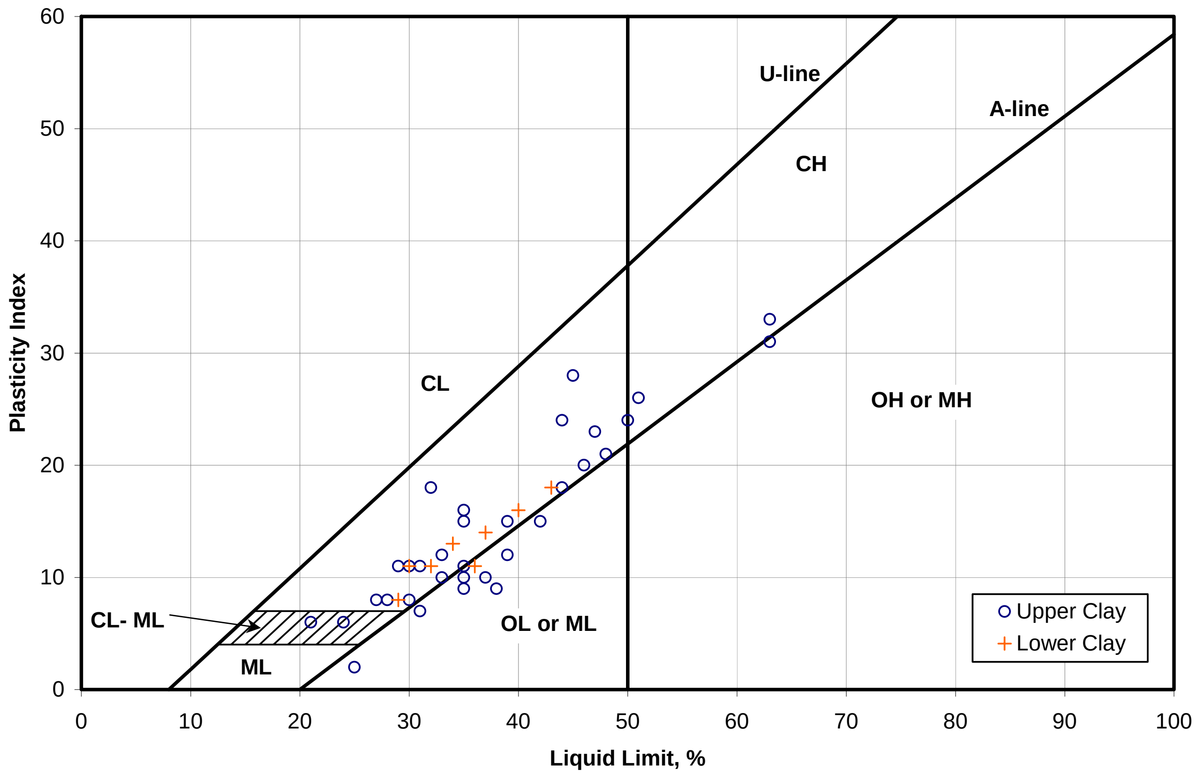


Figure 8-56. Plasticity Chart, Study Section 2: Alum Rock Station.

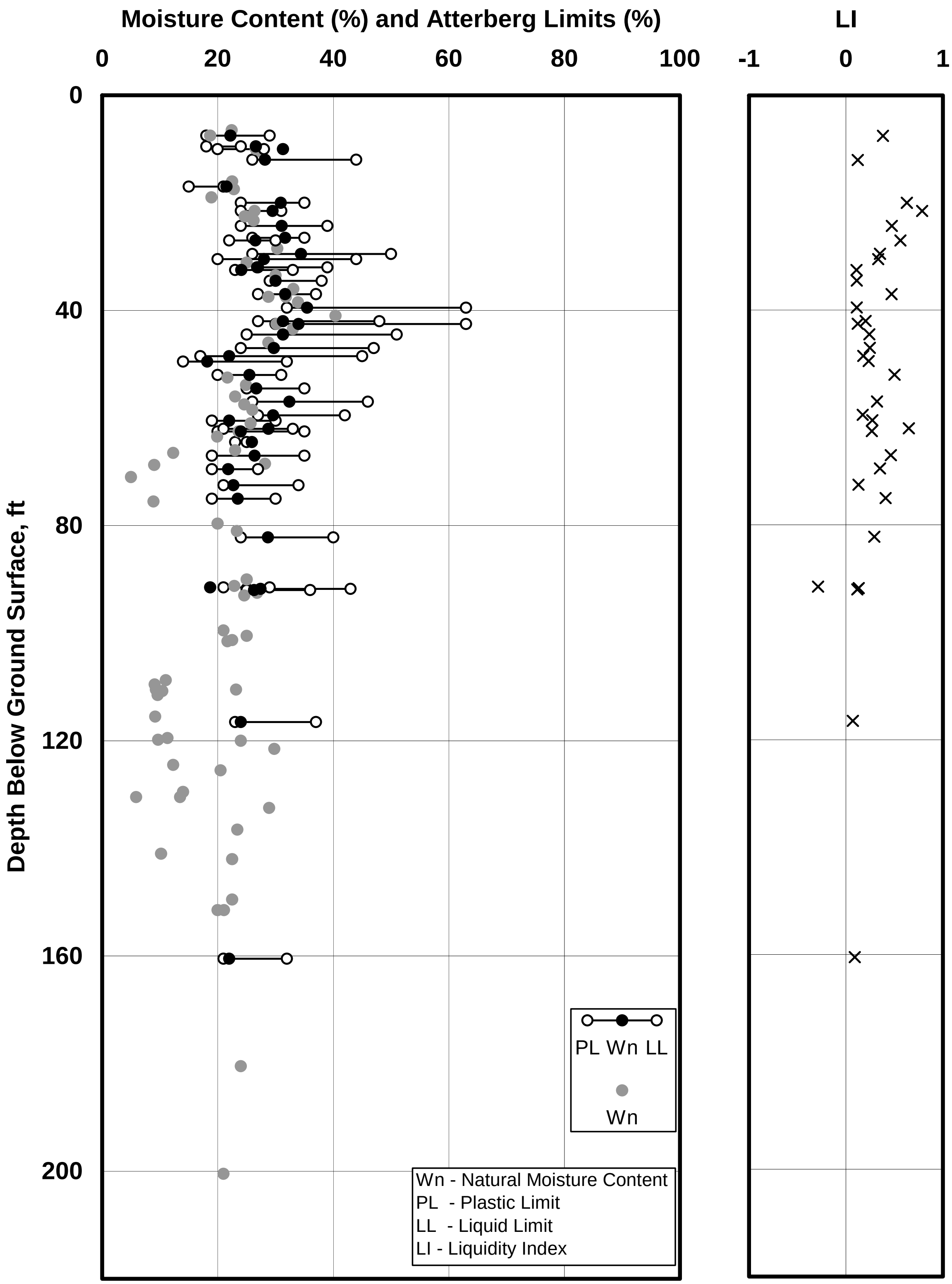


Figure 8-57. Moisture Content and Atterberg Limits, Study Section 2: Alum Rock Station.

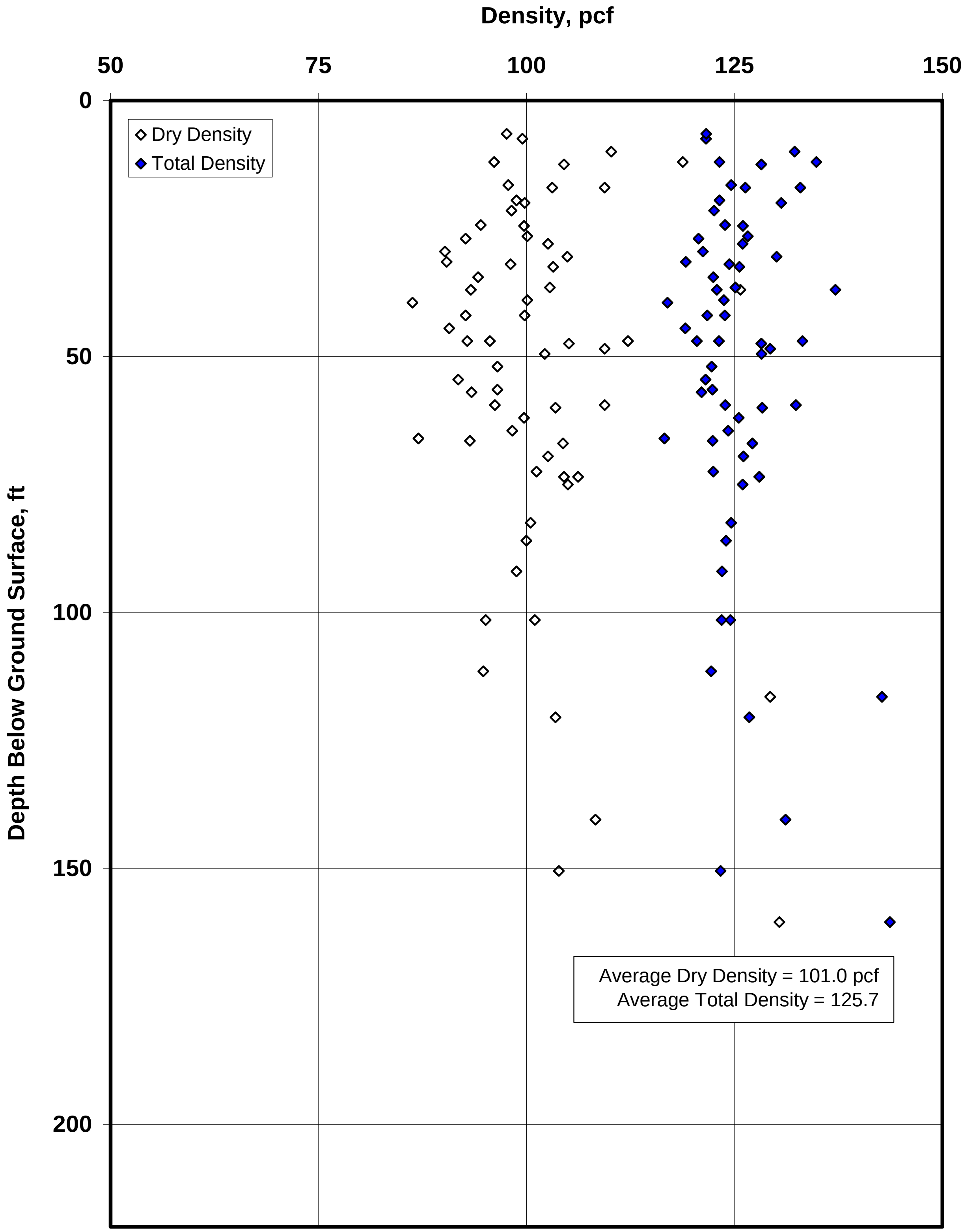


Figure 8-58. Total and Dry Densities, Section 2: Alum Rock Station.

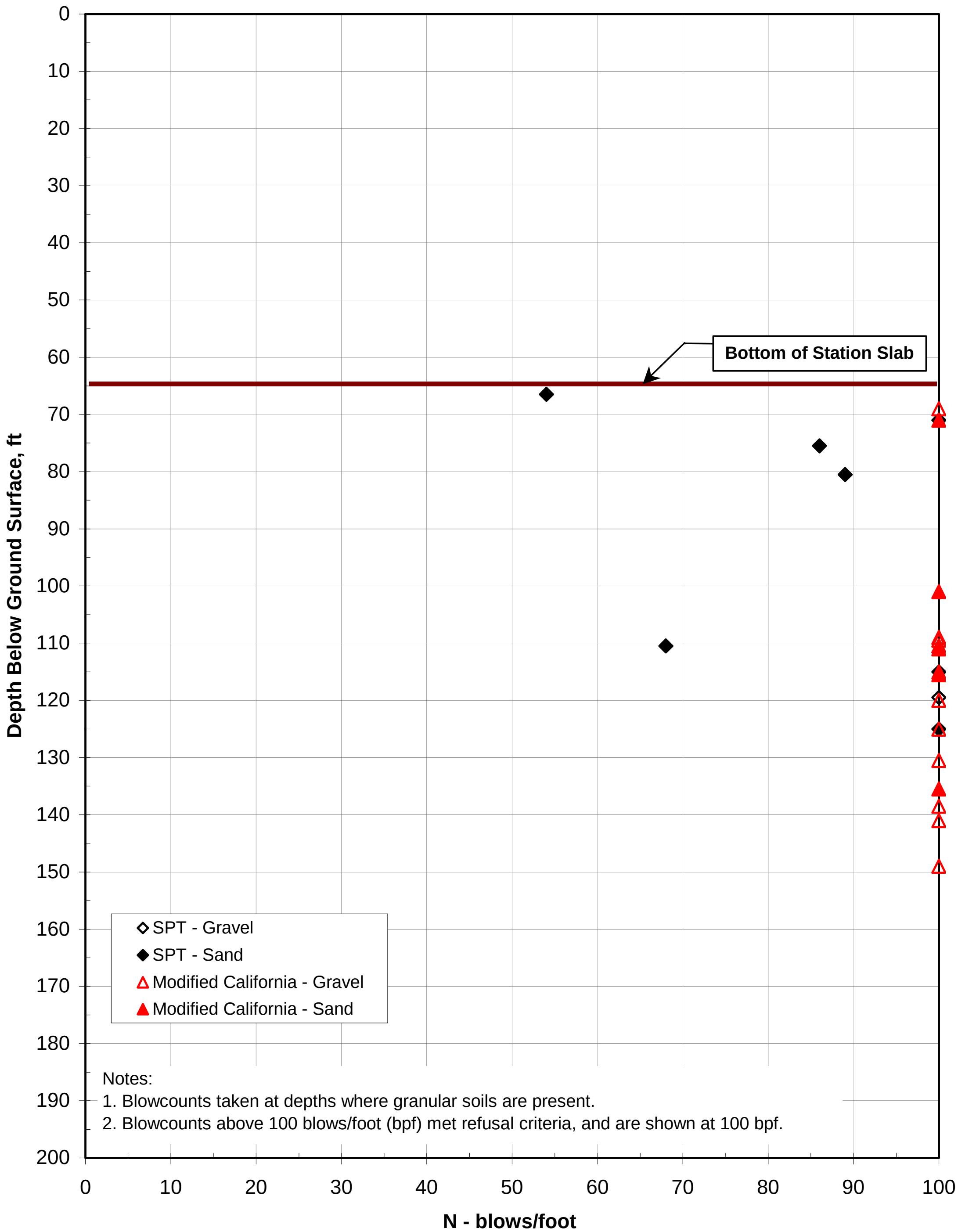


Figure 8-59. Uncorrected SPTs and Modified California Blow Counts, Study Section 2: Alum Rock Station.

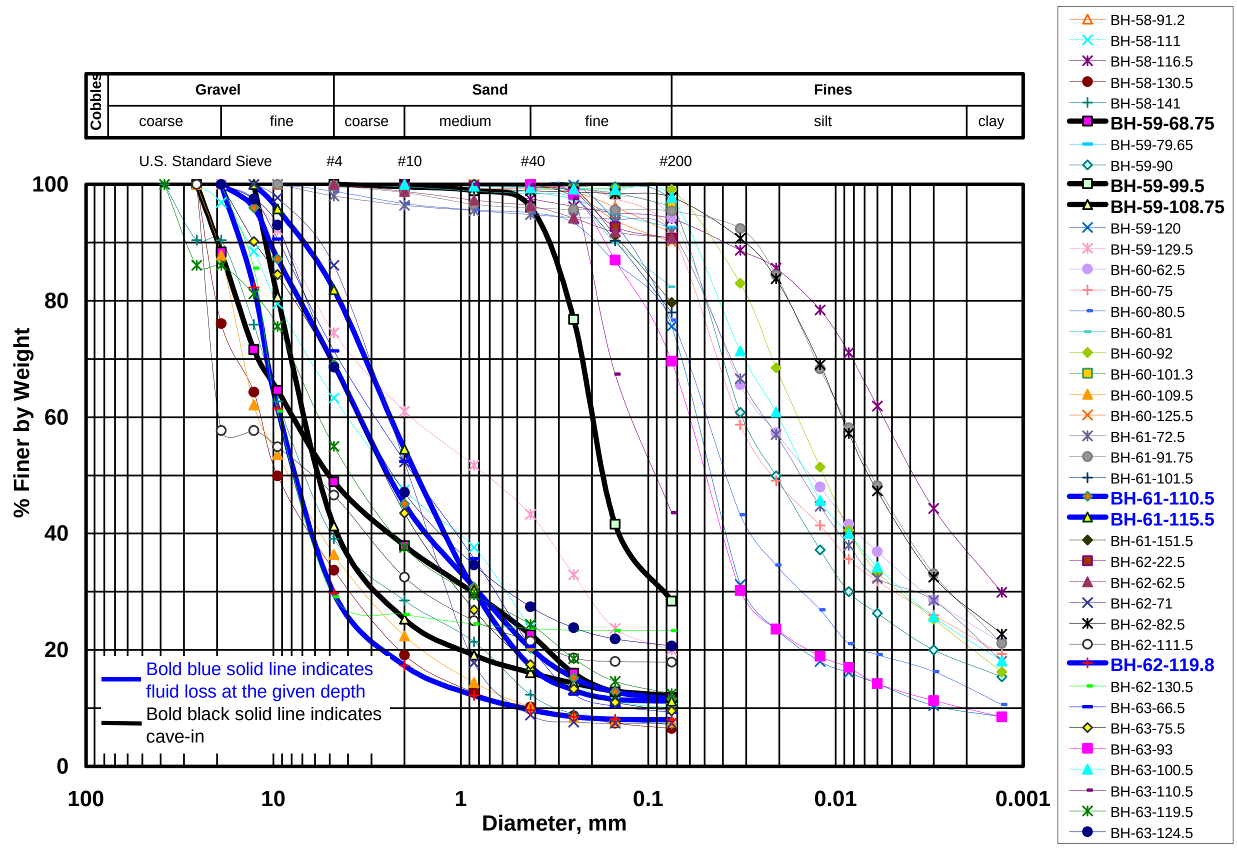


Figure 8-60. Grain Size Distribution, Study Section 2: Alum Rock Station.

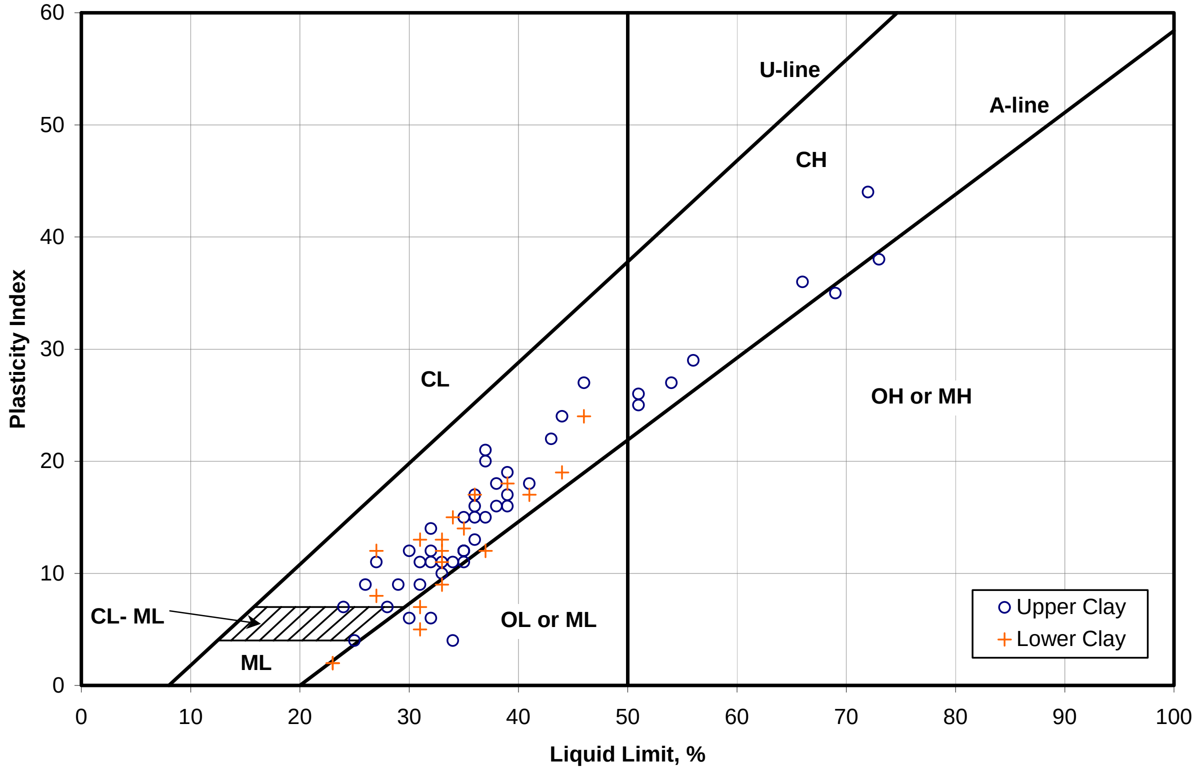


Figure 8-61. Plasticity Chart, Study Section 3: Alum Rock Station to Crossover.

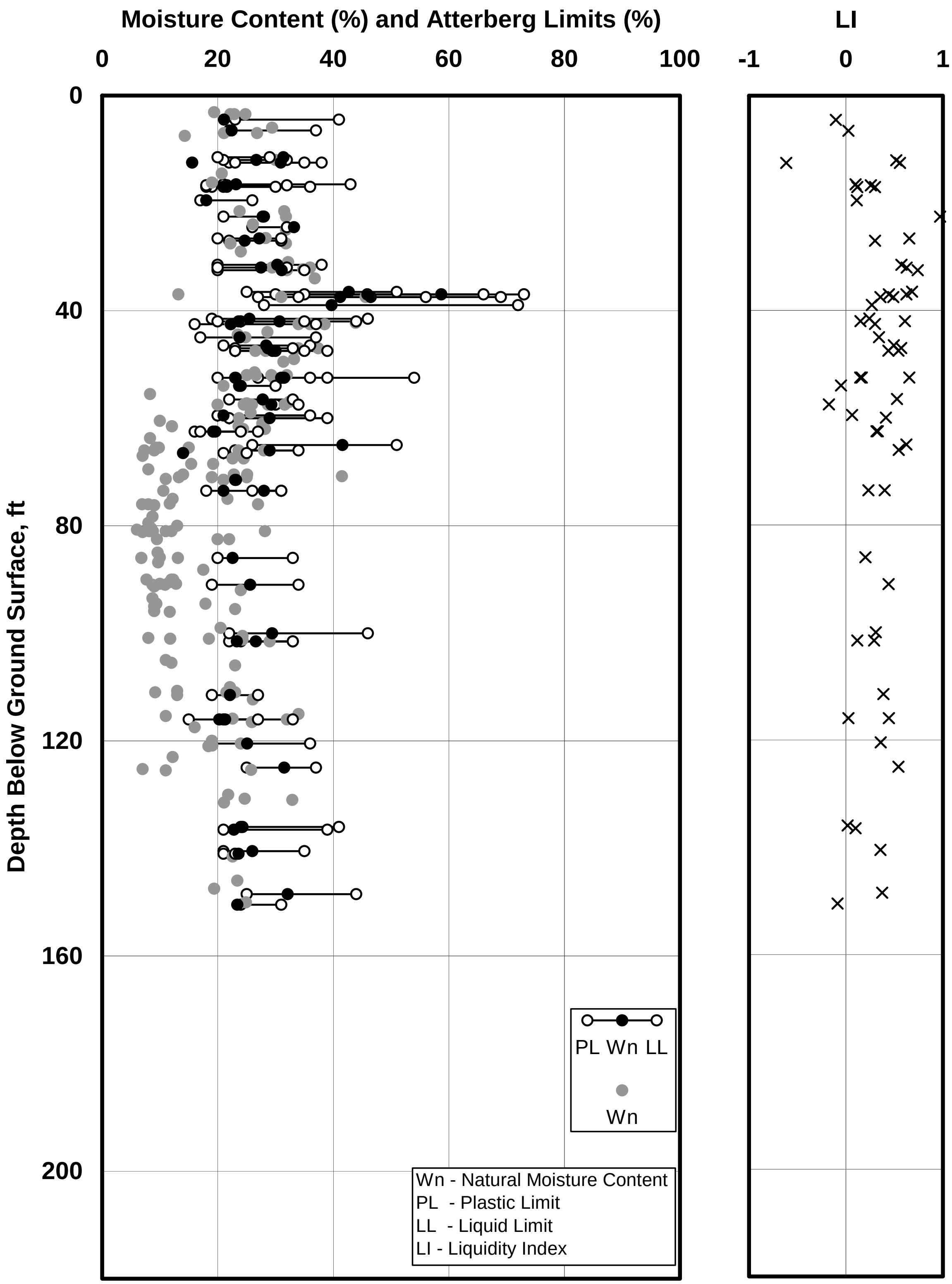


Figure 8-62. Moisture Content and Atterberg Limits, Study Section 3: Alum Rock Station to Crossover/Downtown San Jose Station.

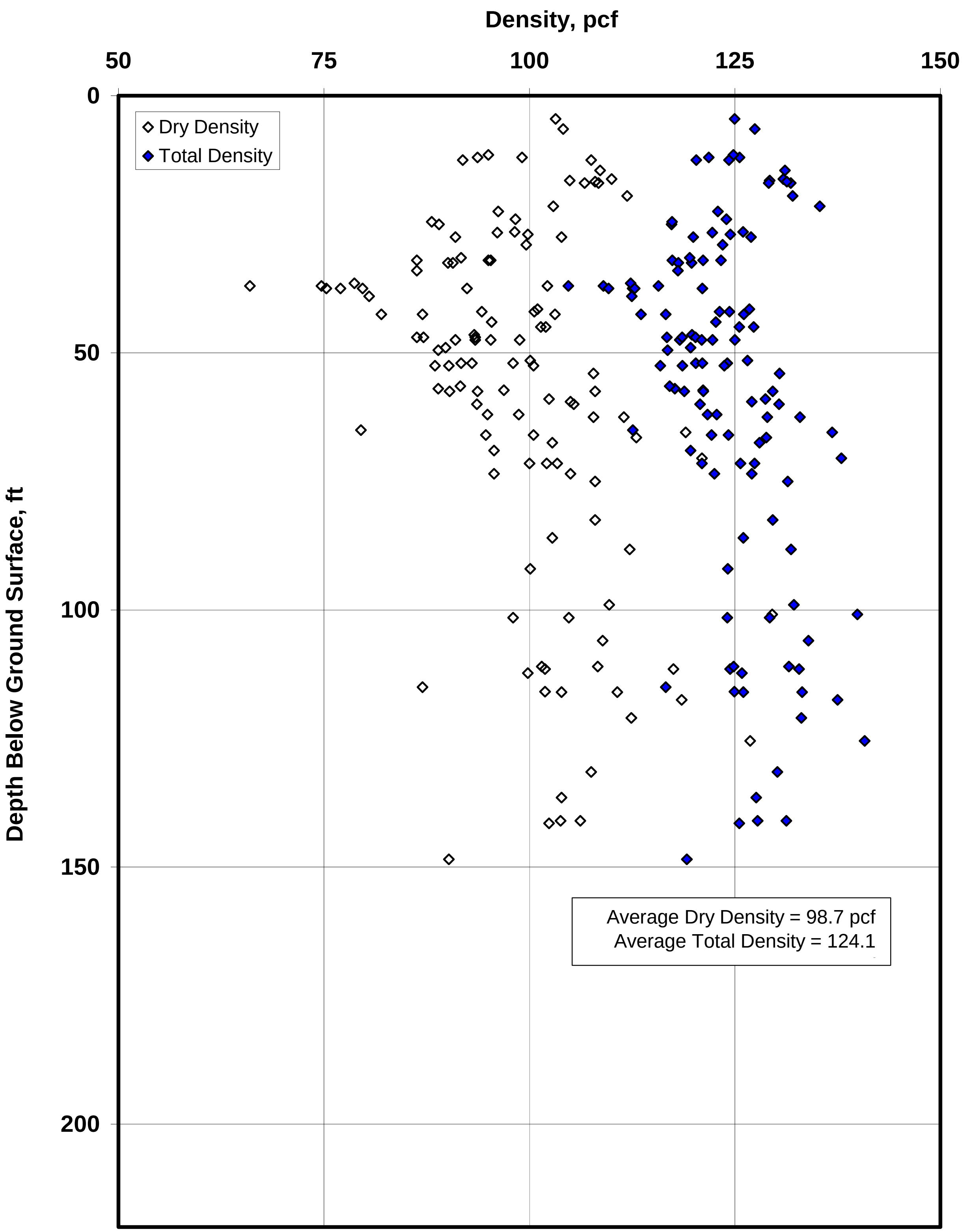


Figure 8-63. Total and Dry Densities, Study Section 3: Alum Rock Station to Crossover/Downtown San Jose Station.

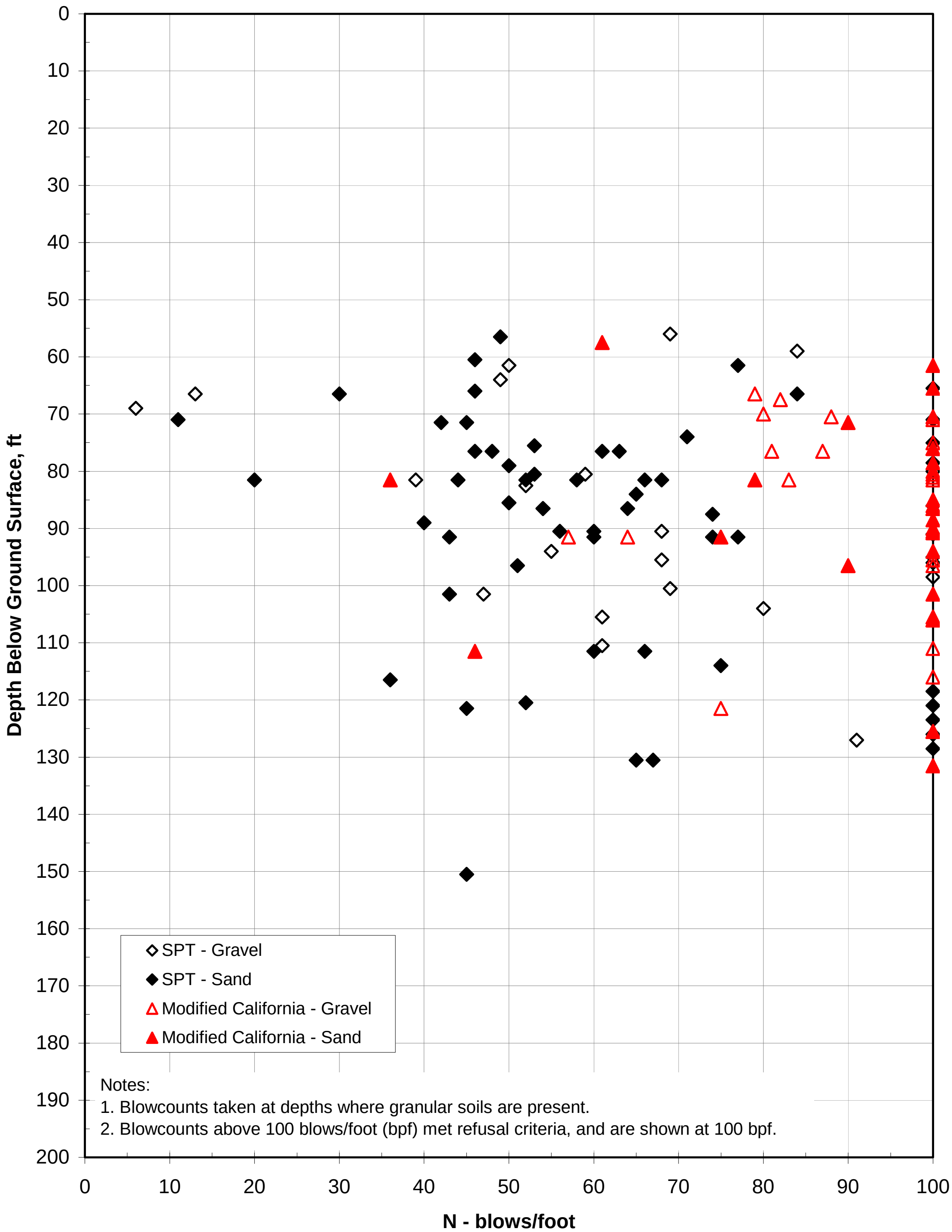


Figure 8-64. Uncorrected SPTs and Modified California Blow Counts, Study Section 3: Alum Rock Station to Crossover.

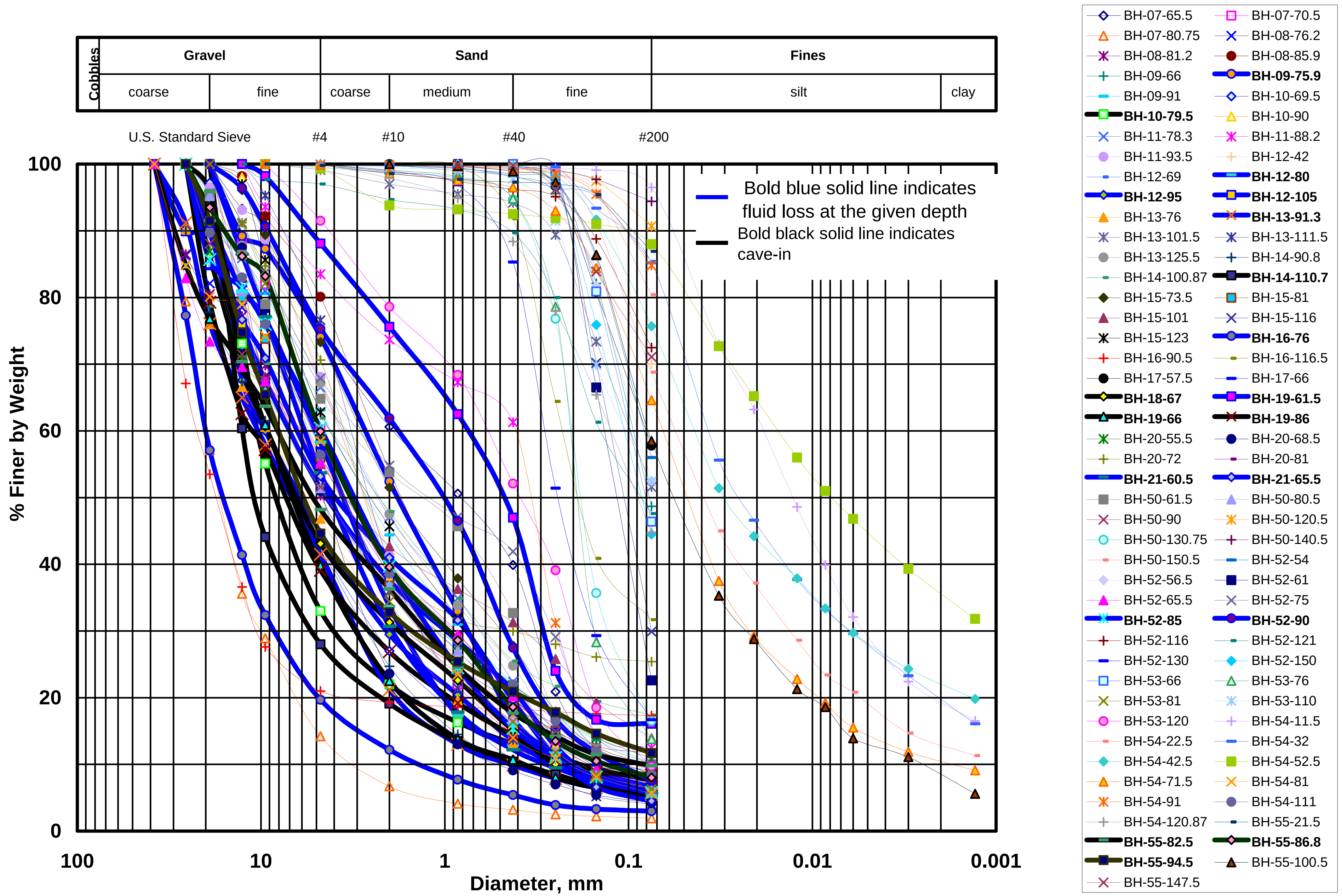


Figure 8-65. Grain Size Distribution, Study Section 3: Alum Rock Station to Crossover/Downtown San Jose Station.

This page intentionally left blank.

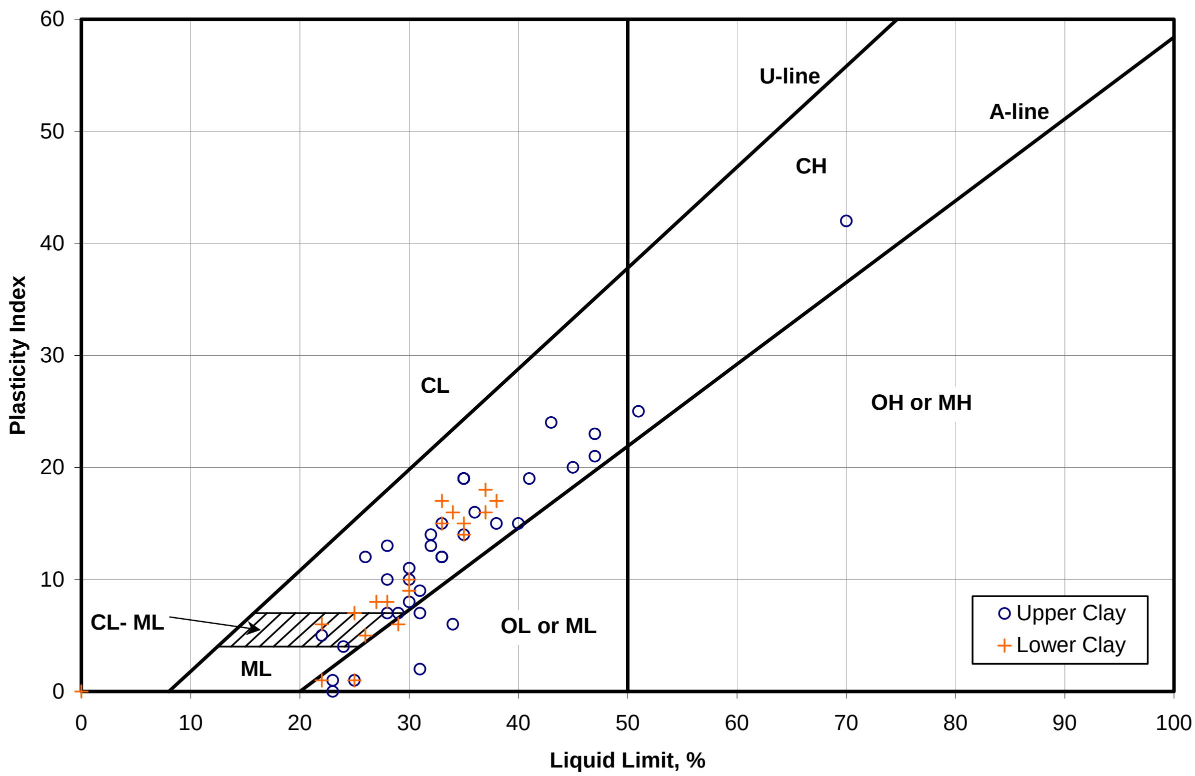


Figure 8-66. Plasticity Chart, Study Section 4: Crossover and Downtown San Jose Station.

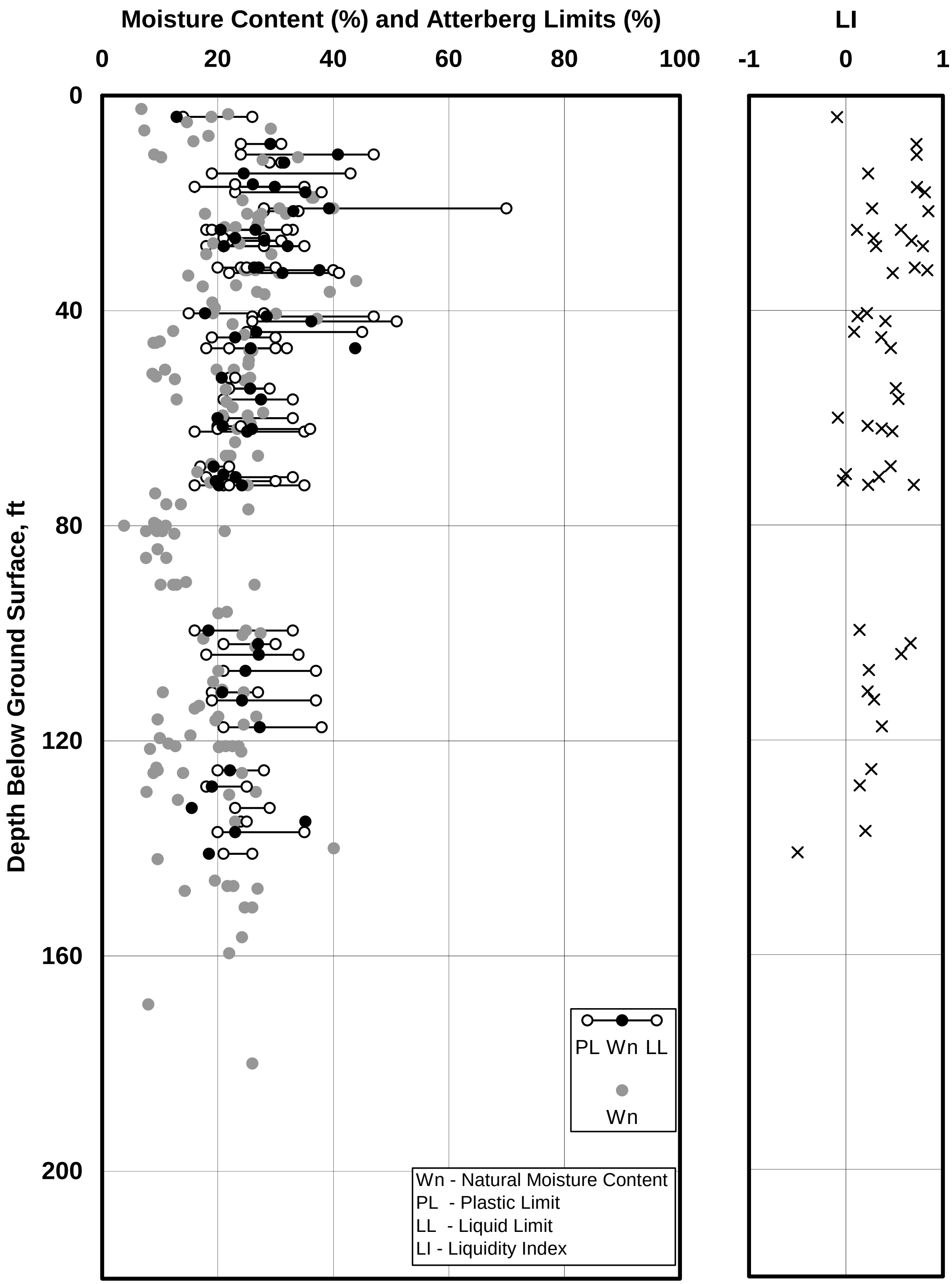


Figure 8-67. Moisture Content and Atterberg Limits, Study Section 4: Crossover/Downtown San Jose Station.

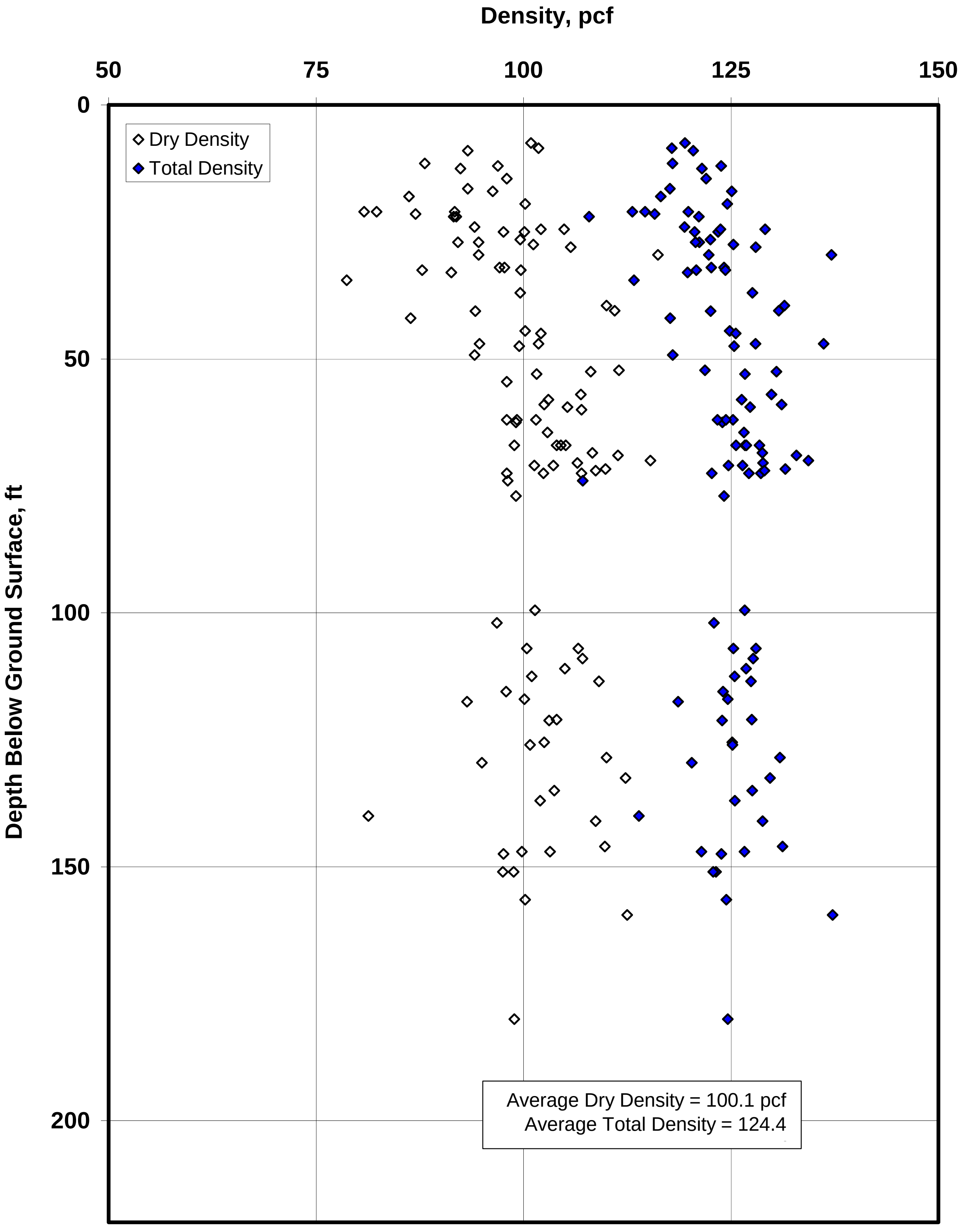


Figure 8-68. Total and Dry Densities, Study Section 4: Crossover/Downtown San Jose Station.

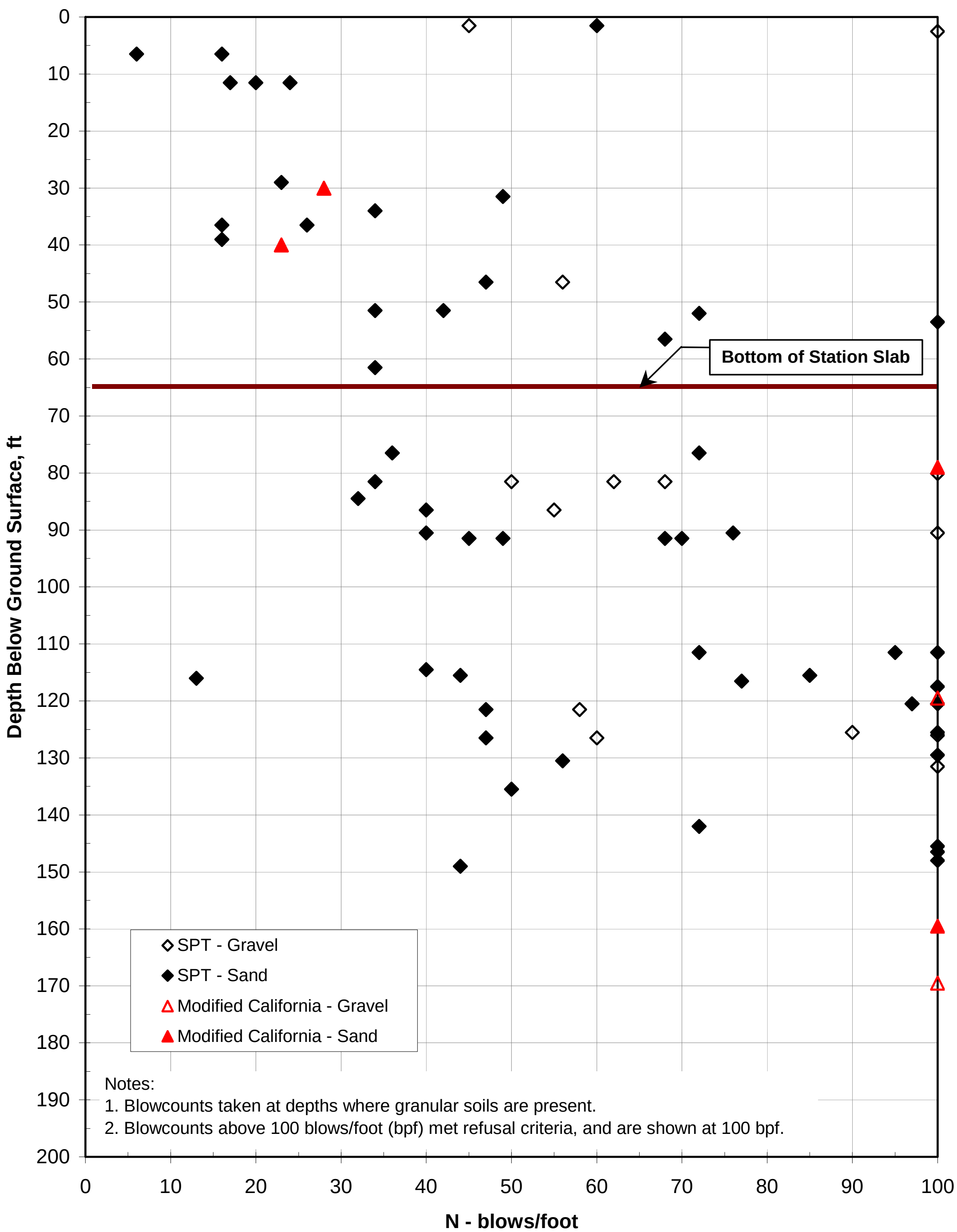
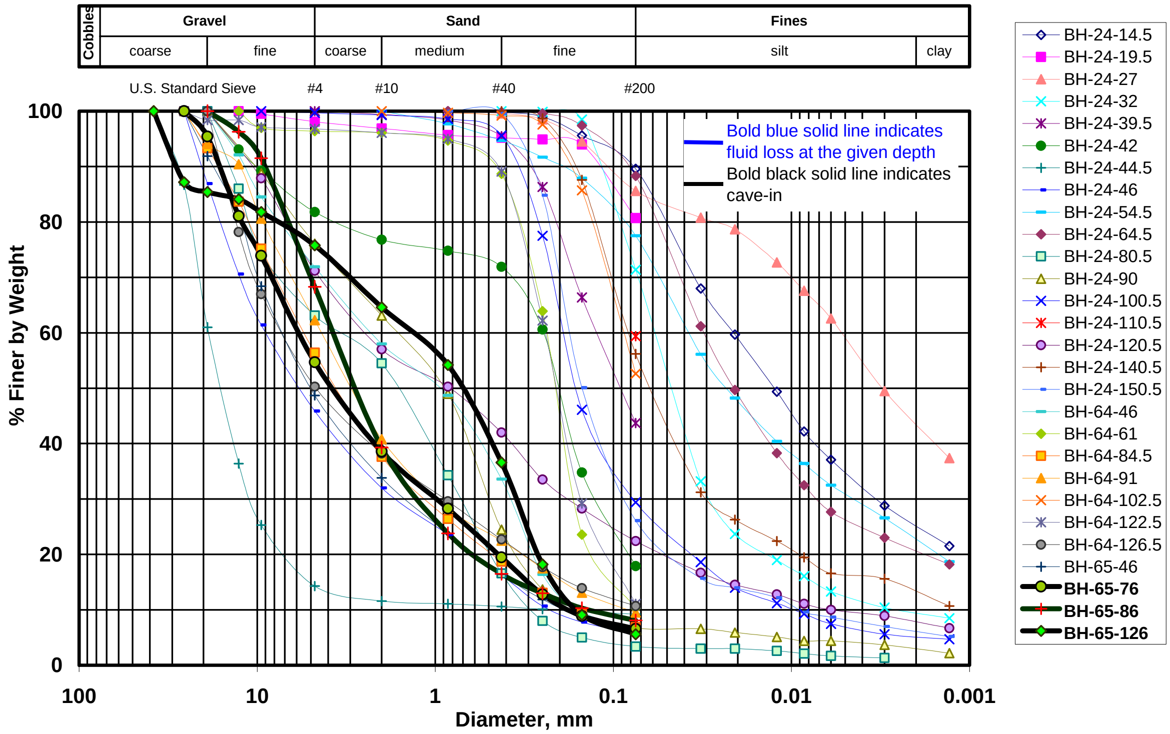


Figure 8-69. Uncorrected SPTs and Modified California Blow Counts, Study Section 4: Crossover and Downtown San Jose Station.

Crossover:



Downtown San Jose Station:

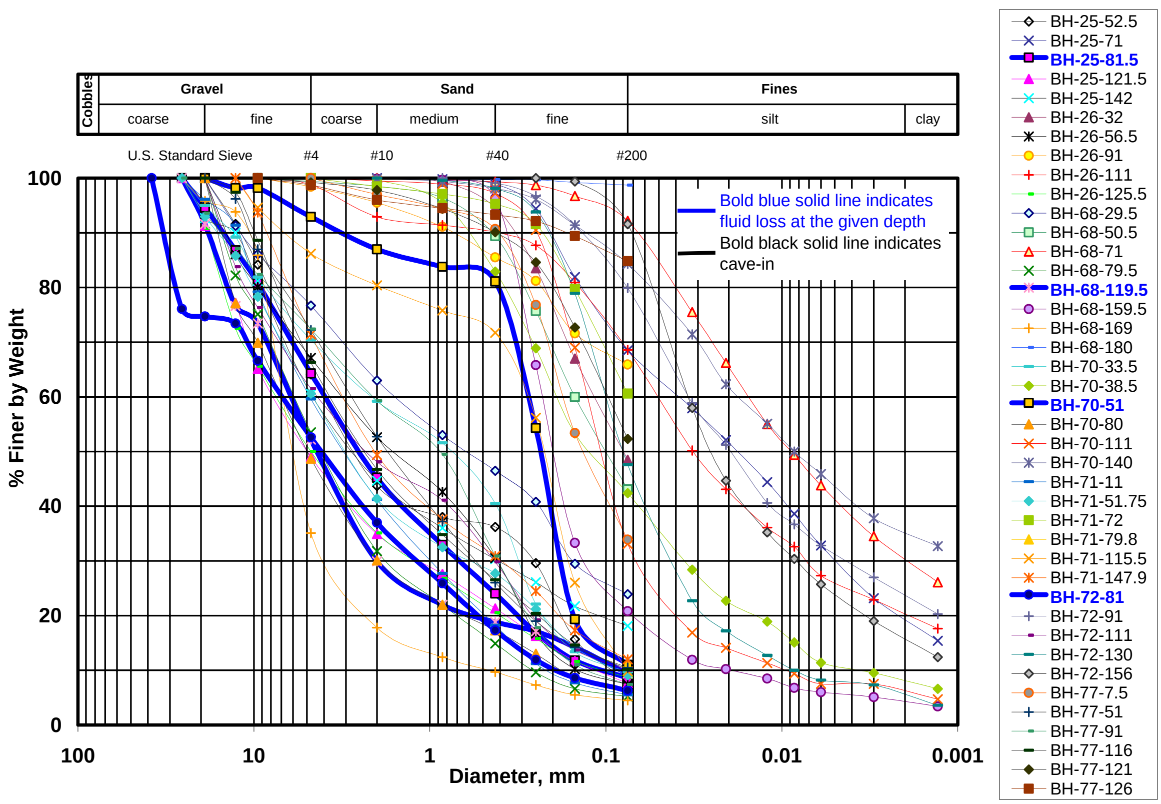


Figure 8-70. Grain Size Distribution, Study Section 4: Crossover/Downtown San Jose Station.

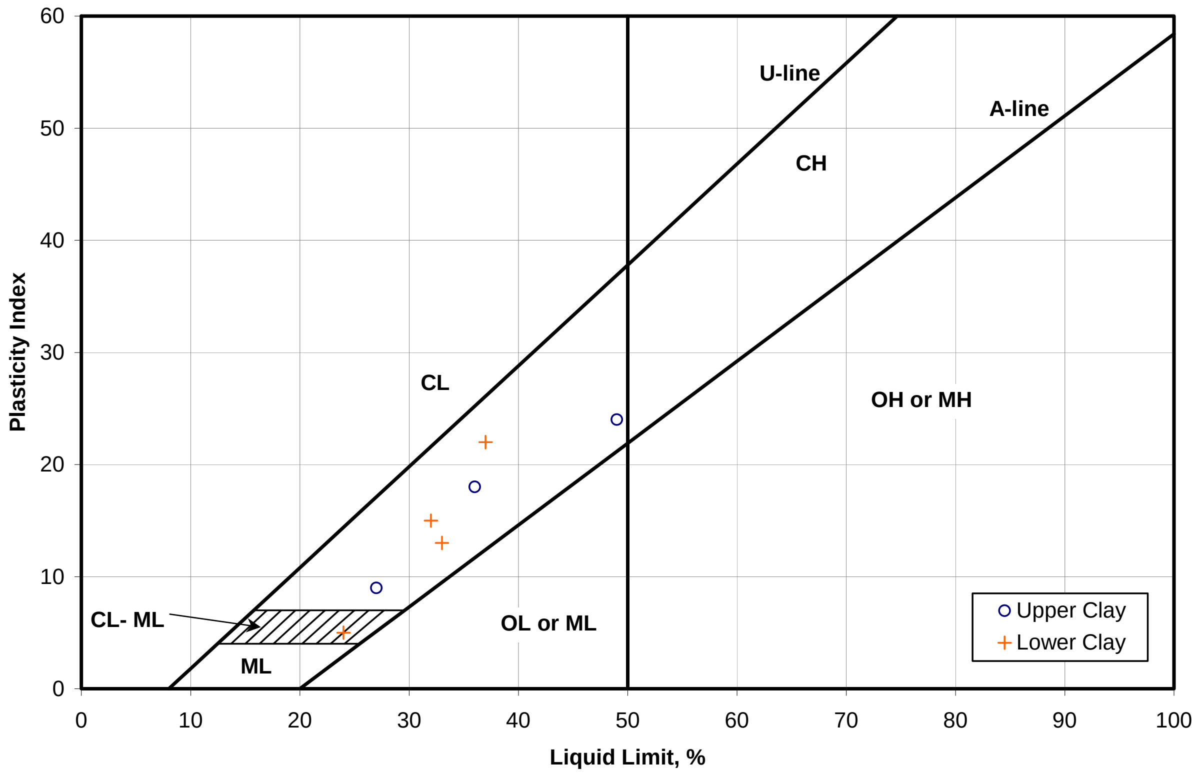


Figure 8-71. Plasticity Chart, Study Section 5: Downtown San Jose Station to Diridon/Arena Station.

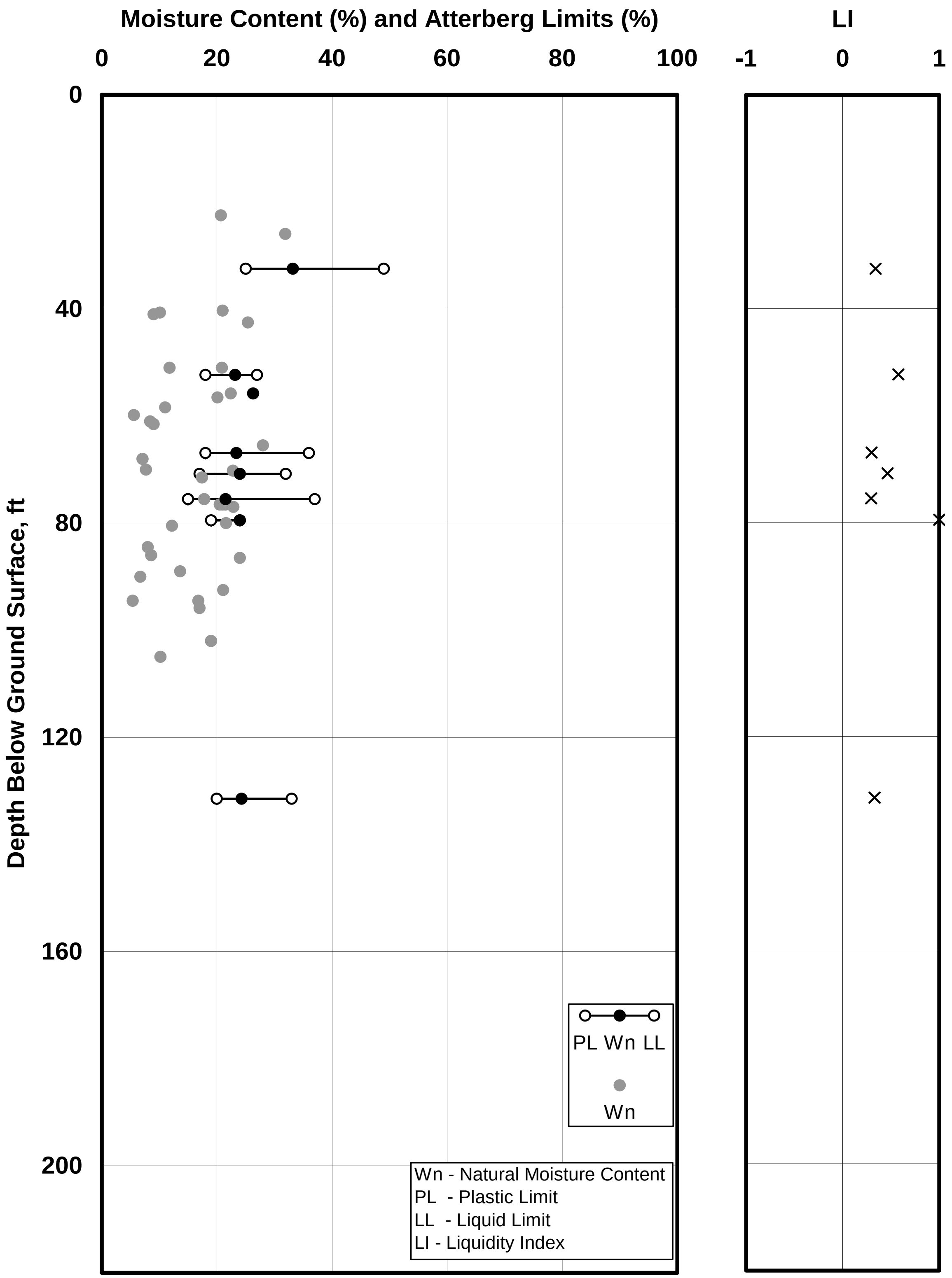


Figure 8-72. Moisture Content and Atterberg Limits, Study Section 5: Downtown San Jose Station to Diridon/Arena Station.

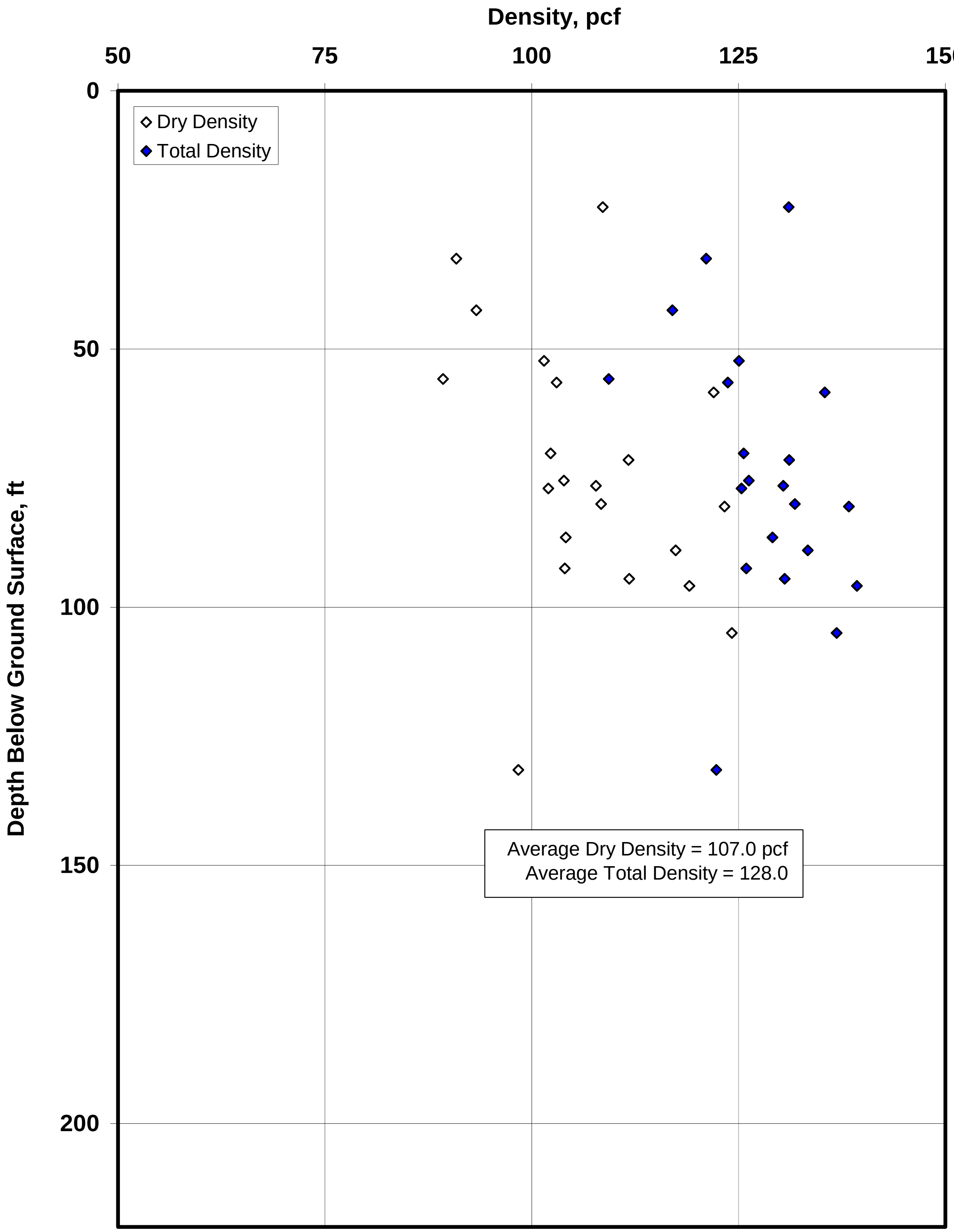


Figure 8-73. Total and Dry Densities, Study Section 5: Downtown San Jose Station to Diridon/Arena Station.

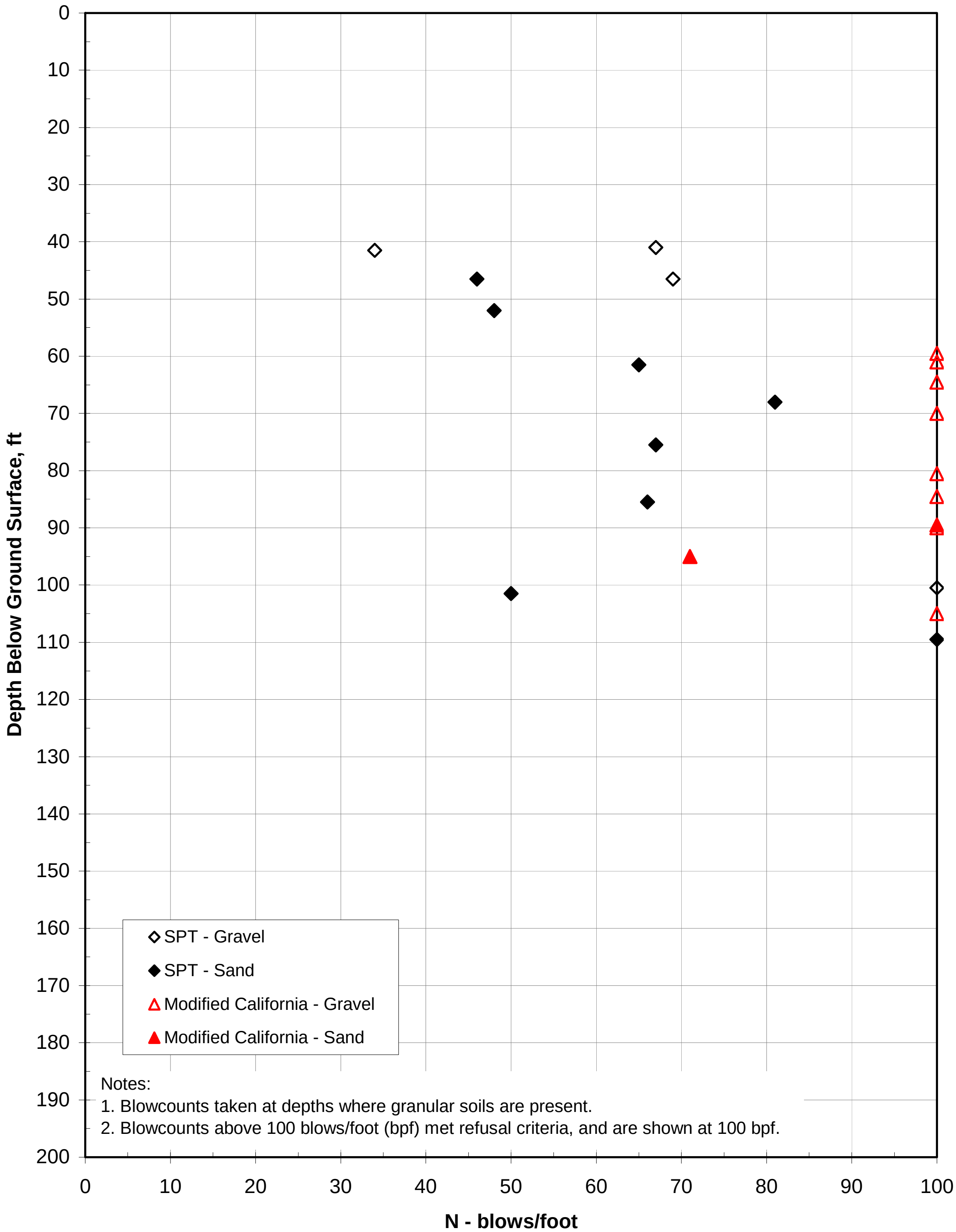


Figure 8-74. Uncorrected SPTs and Modified California Blow Counts, Study Section 5: Downtown San Jose Station to Diridon/Arena Station.

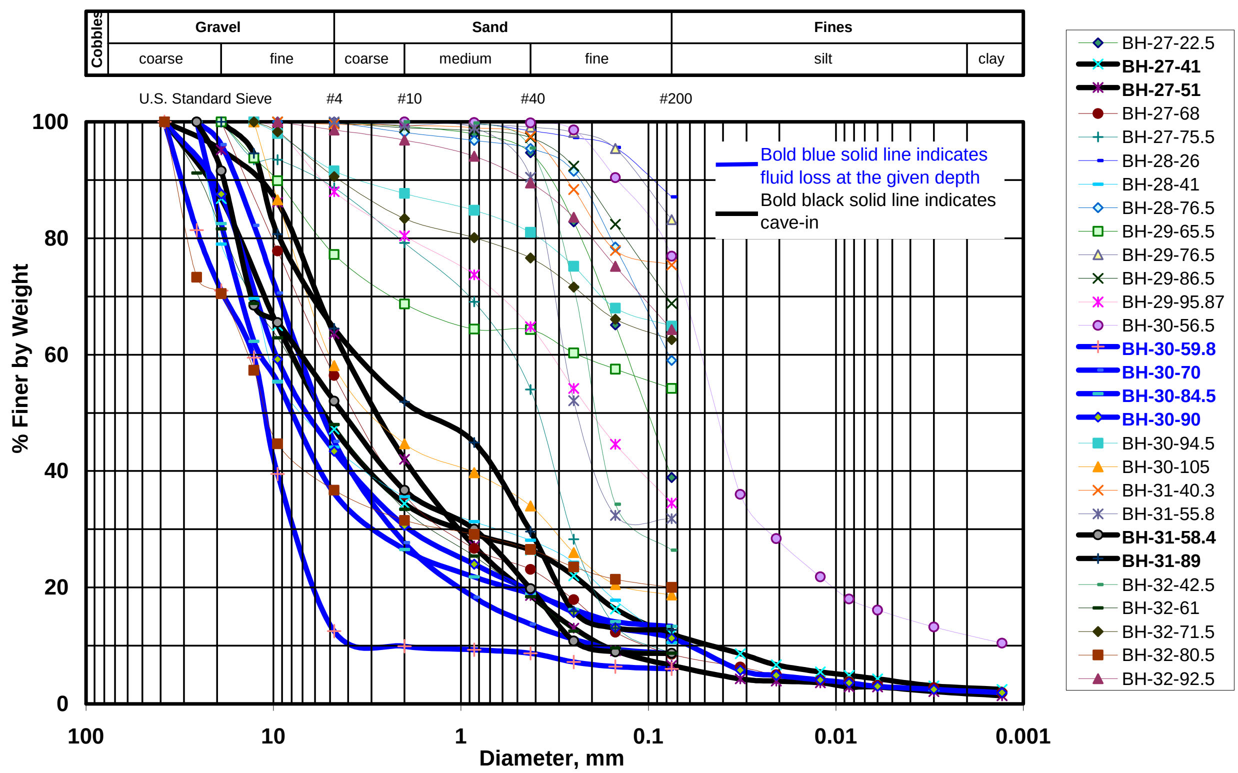


Figure 8-75. Grain Size Distribution, Study Section 5: Downtown San Jose Station to Diridon/Arena Station.

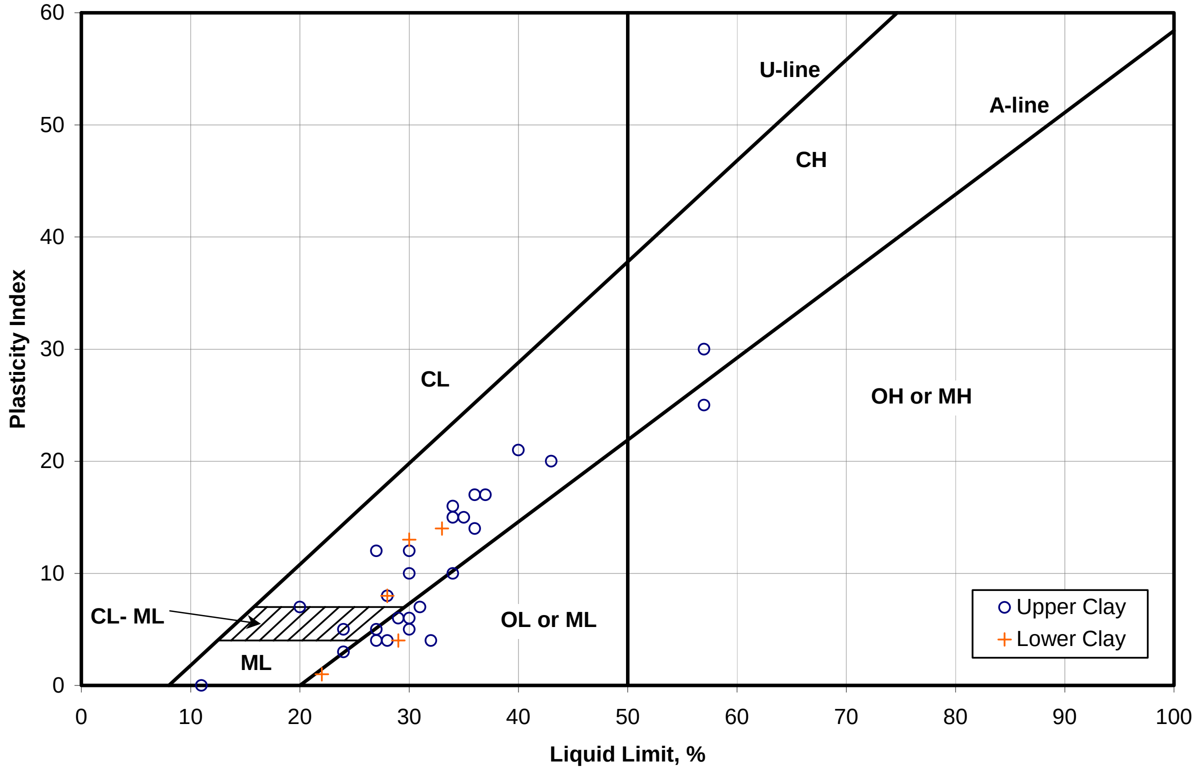


Figure 8-76. Plasticity Chart, Study Section 6: Diridon/Arena Station.

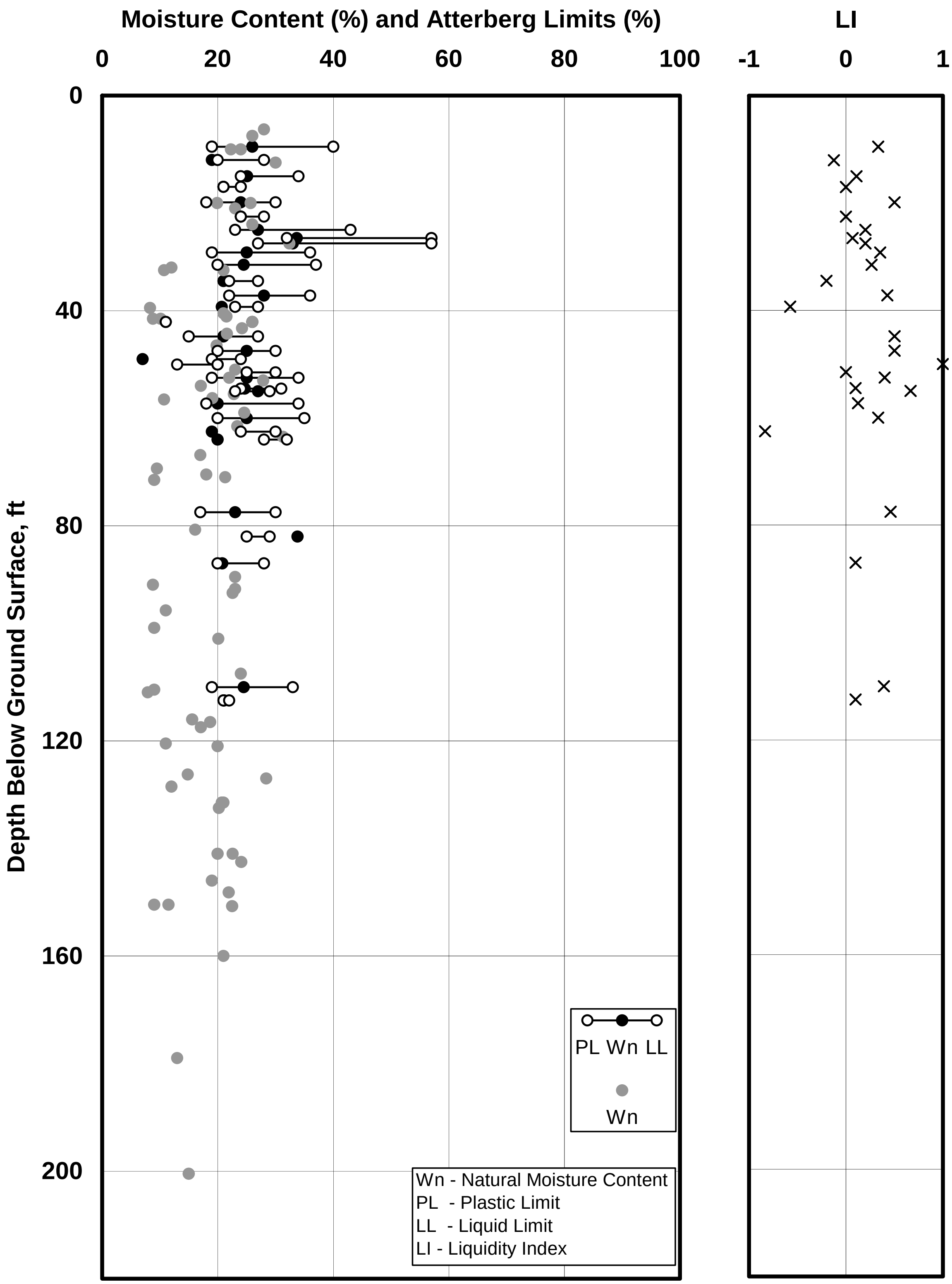


Figure 8-77. Moisture Content and Atterberg Limits, Study Section 6: Diridon/Arena Station.

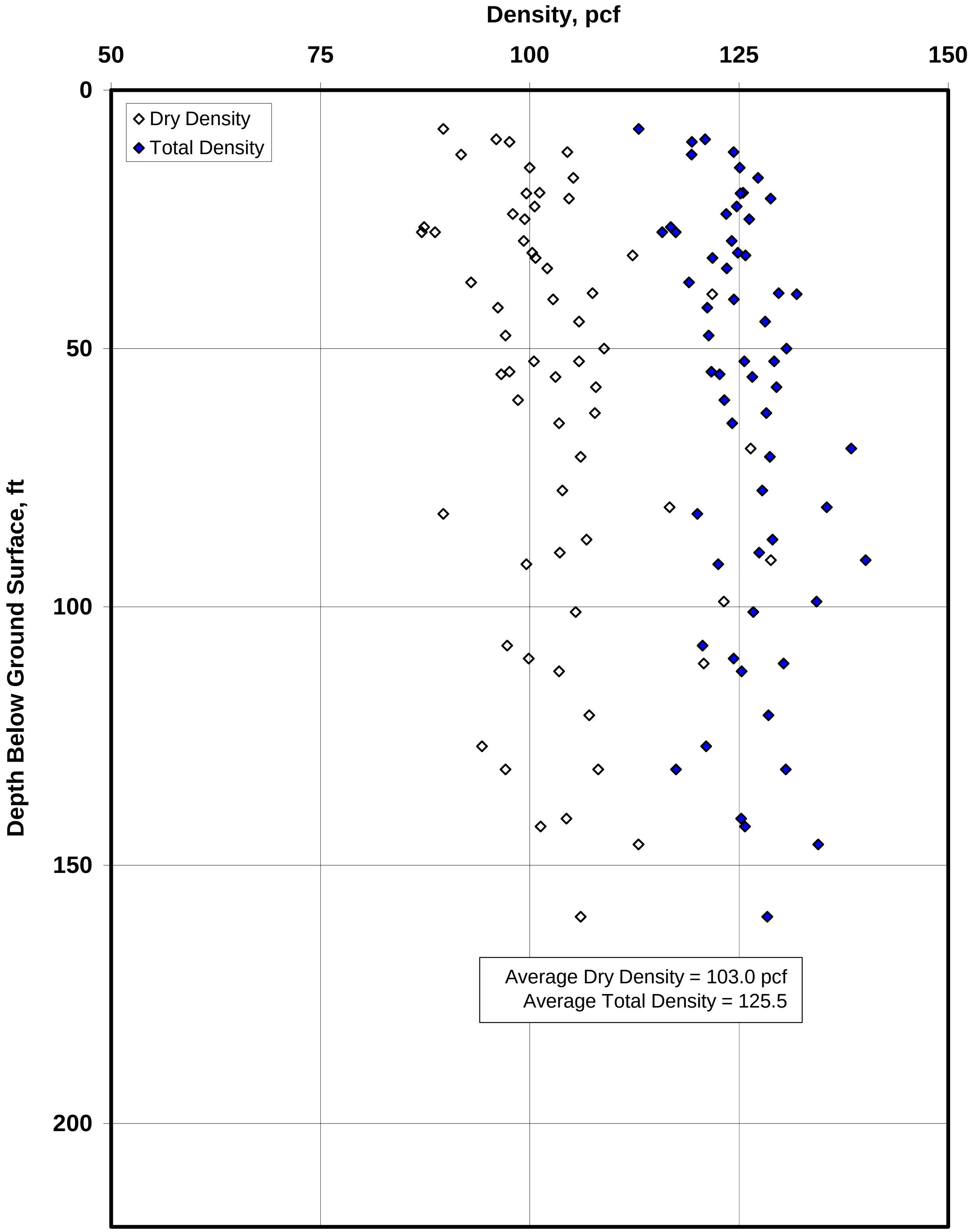


Figure 8-78. Total and Dry Densities, Study Section 6: Diridon/Arena Station.

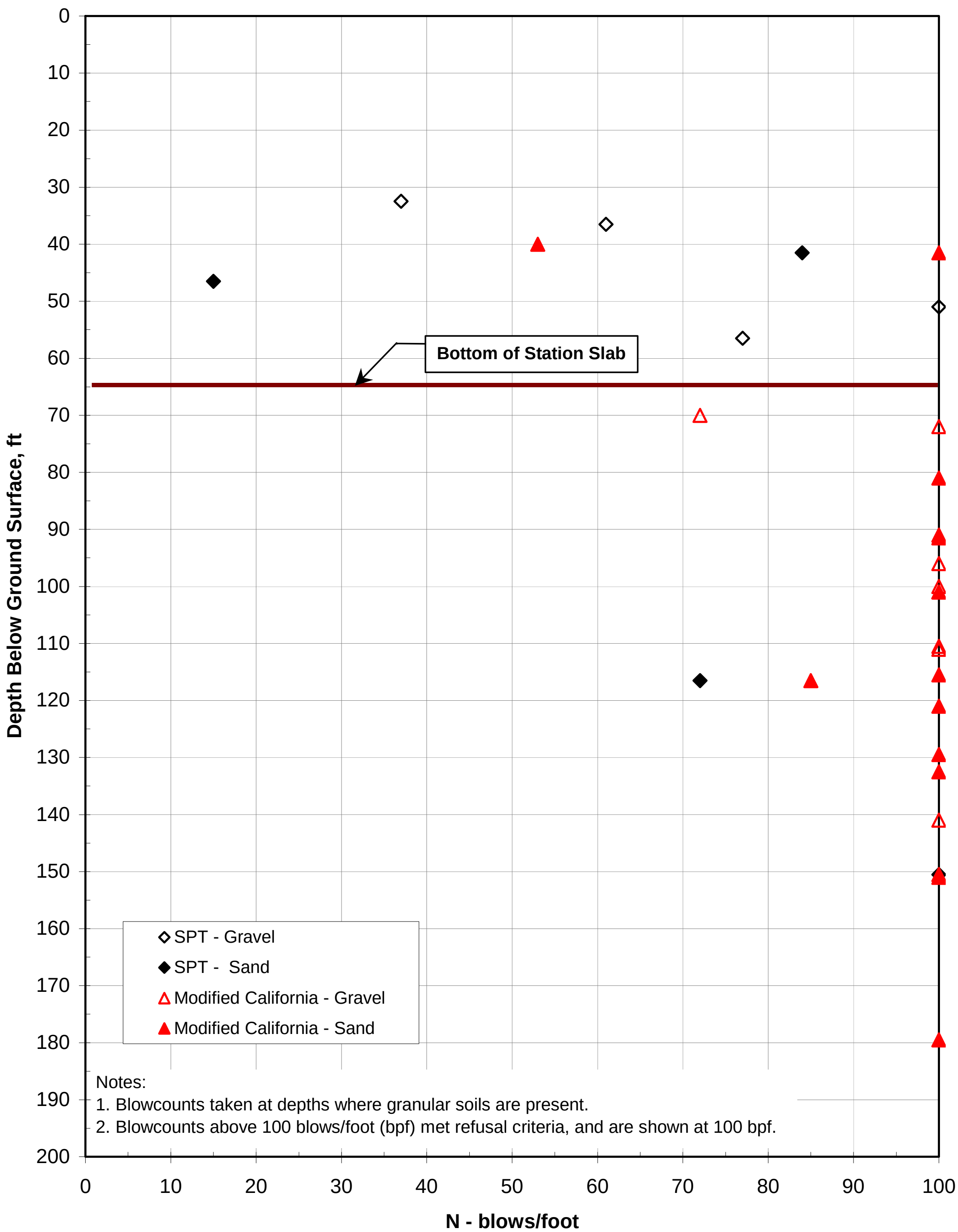


Figure 8-79. Uncorrected SPTs and Modified California Blow Counts, Study Section 6: Diridon/Arena Station.

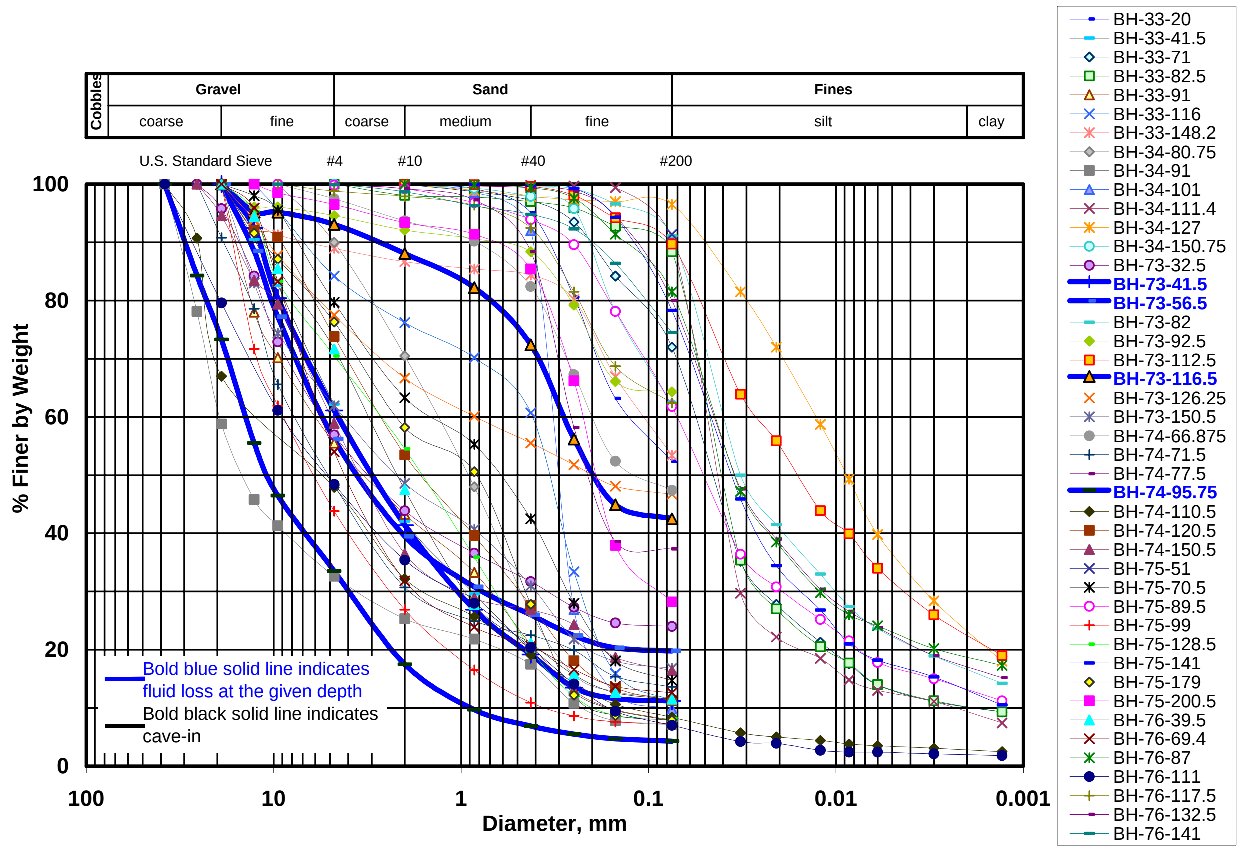


Figure 8-80. Grain Size Distribution, Study Section 6: Diridon/Arena Station.

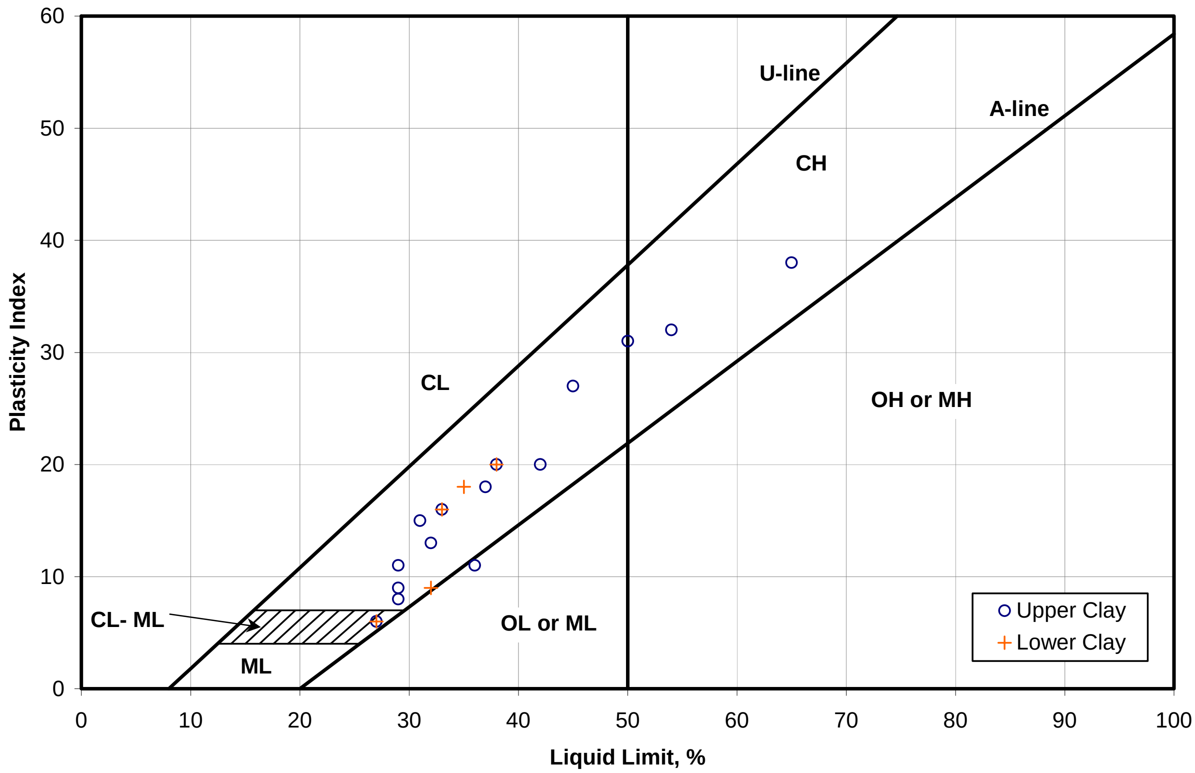


Figure 8-81. Plasticity Chart, Study Section 7: Diridon/Arena Station to West Portal.

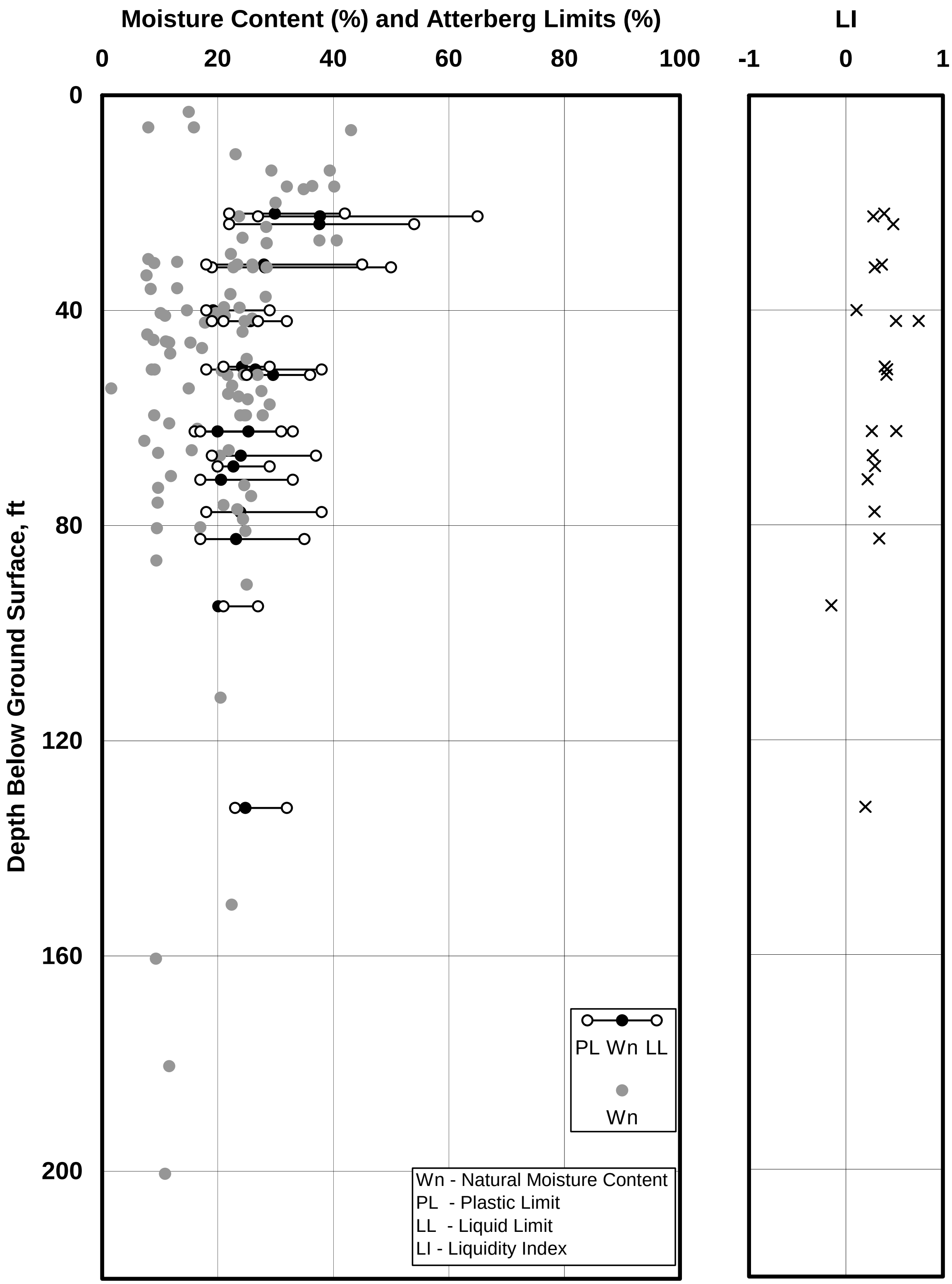


Figure 8-82. Moisture Content and Atterberg Limits, Study Section 7: Diridon/Arena Station to West Portal.

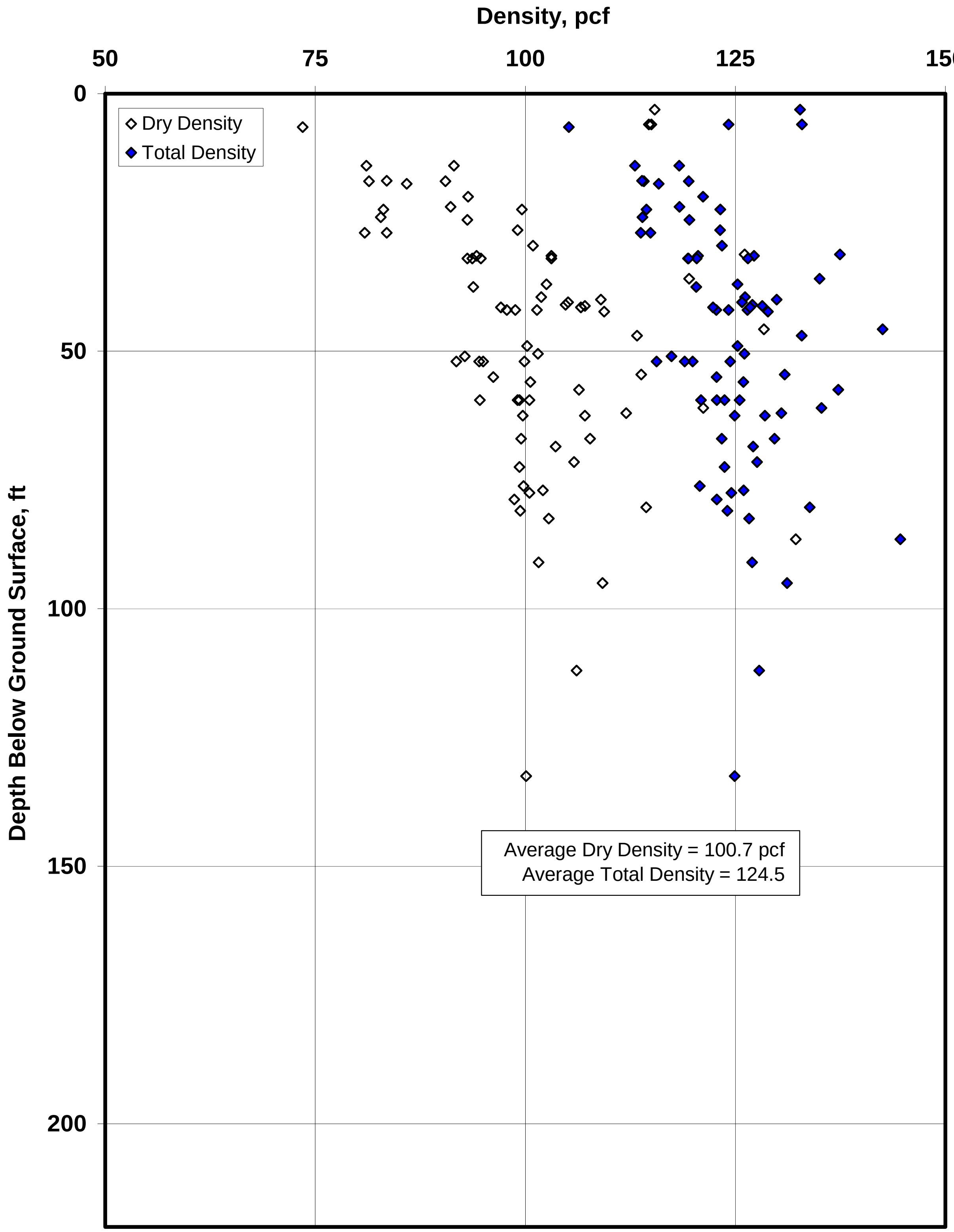


Figure 8-83. Total and Dry Densities, Study Section 7: Diridon/Arena Station to West Portal.

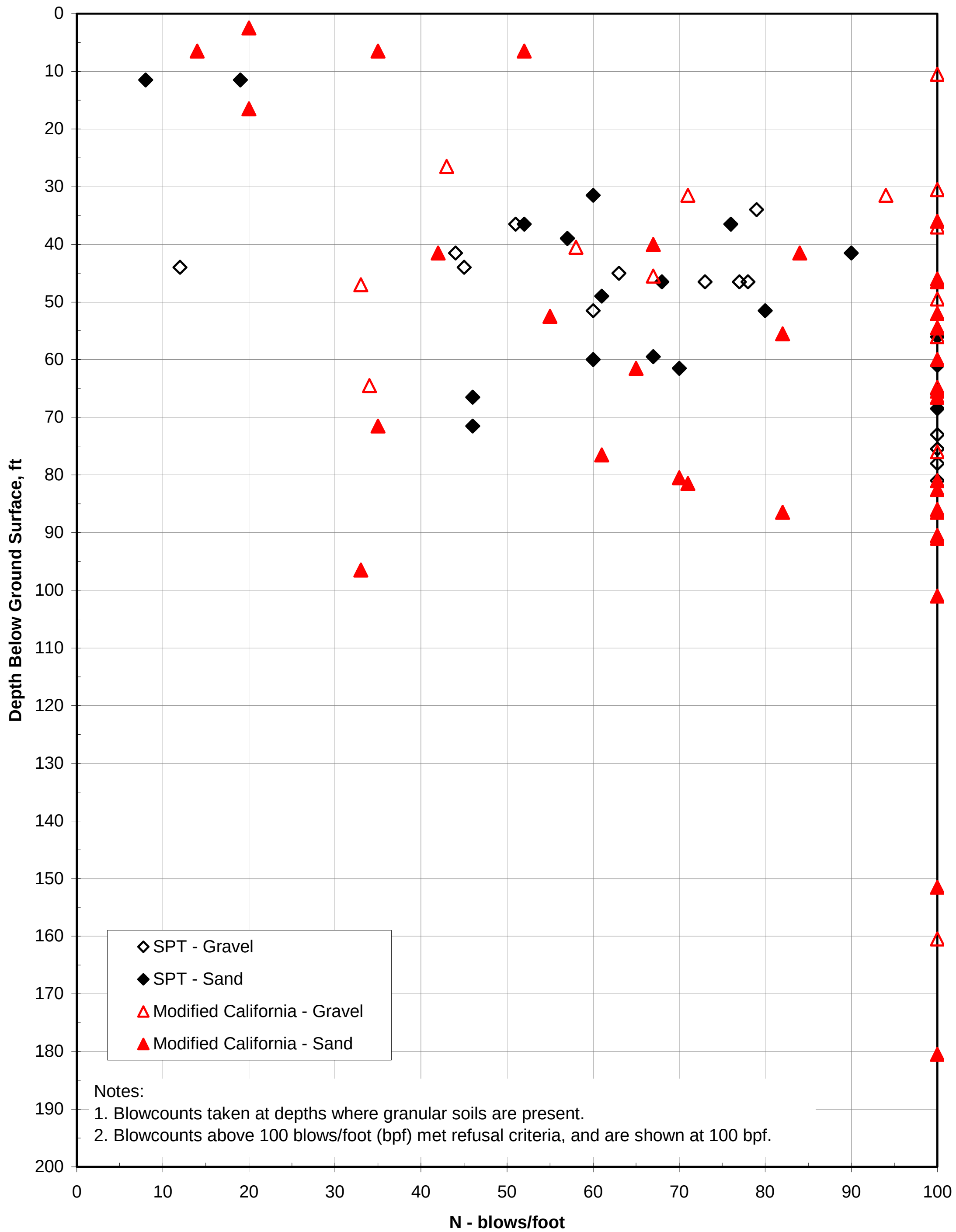


Figure 8-84. Uncorrected SPTs and Modified California Blow Counts, Study Section 7: Diridon/Arena Station to West Portal.

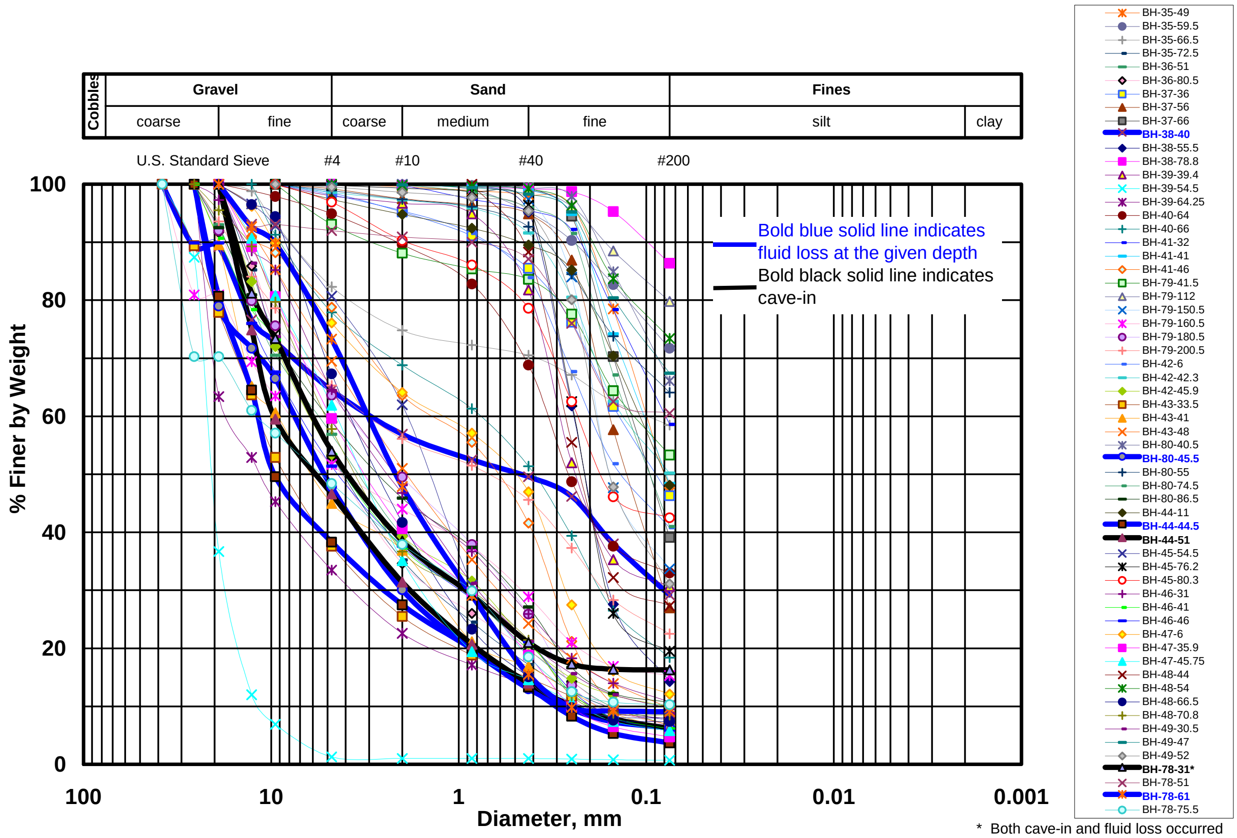


Figure 8-85. Grain Size Distribution, Study Section 7: Diridon/Arena Station to West Portal.

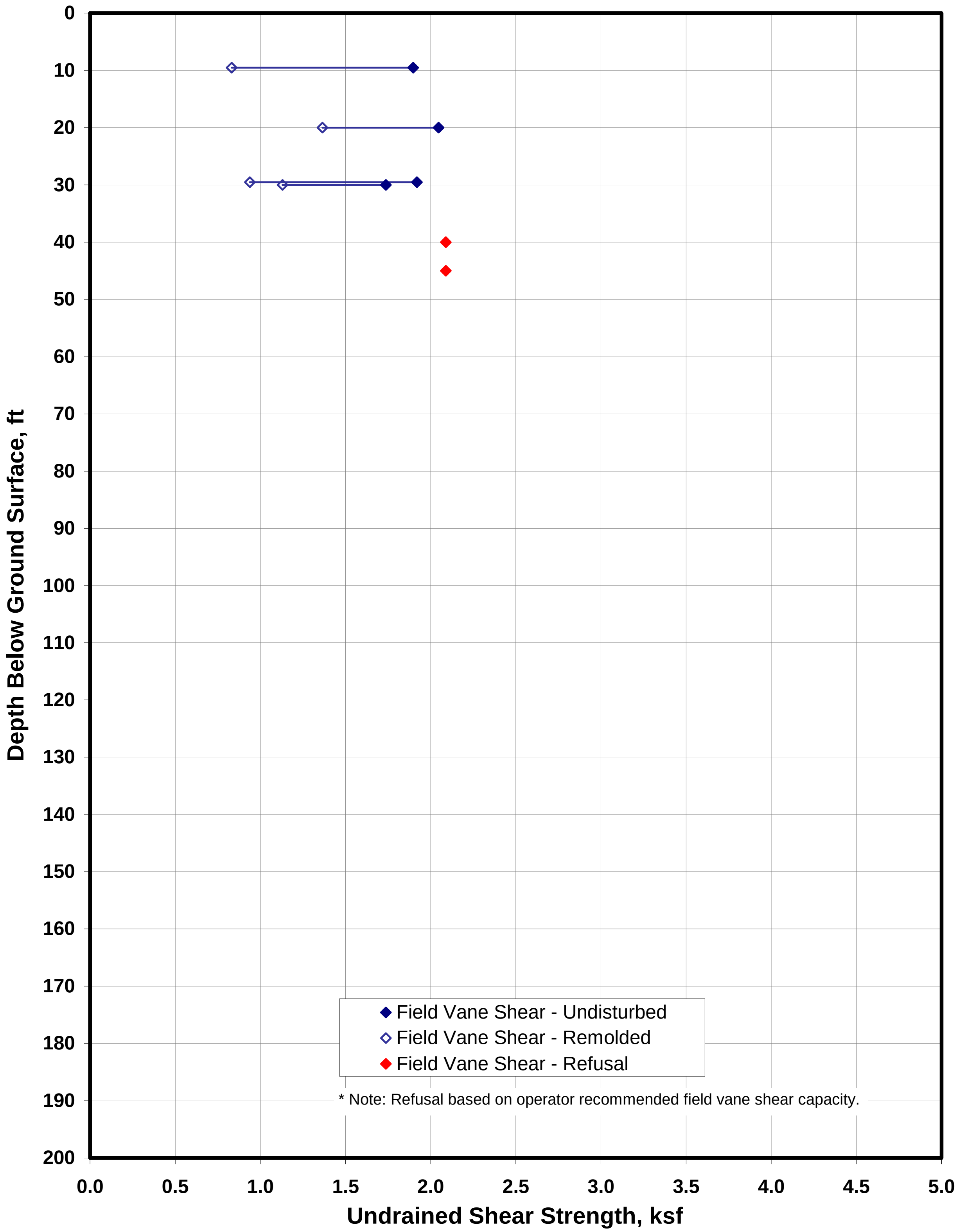


Figure 8-86. Undrained Shear Strength from Vane Shear Tests, Study Section 1: East Portal to Alum Rock Station.

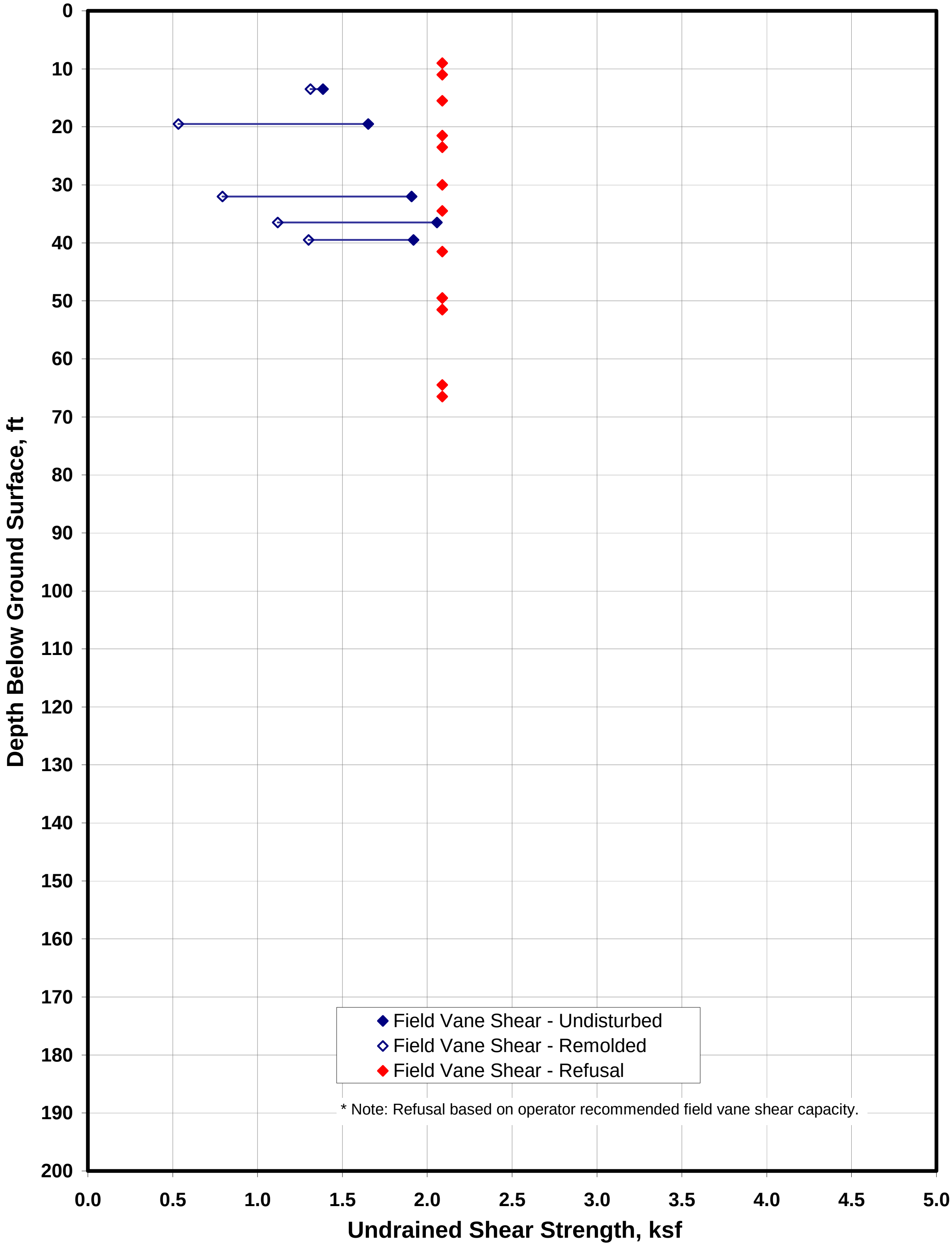


Figure 8-87. Undrained Shear Strength from Vane Shear Tests, Study Section 2: Alum Rock Station.

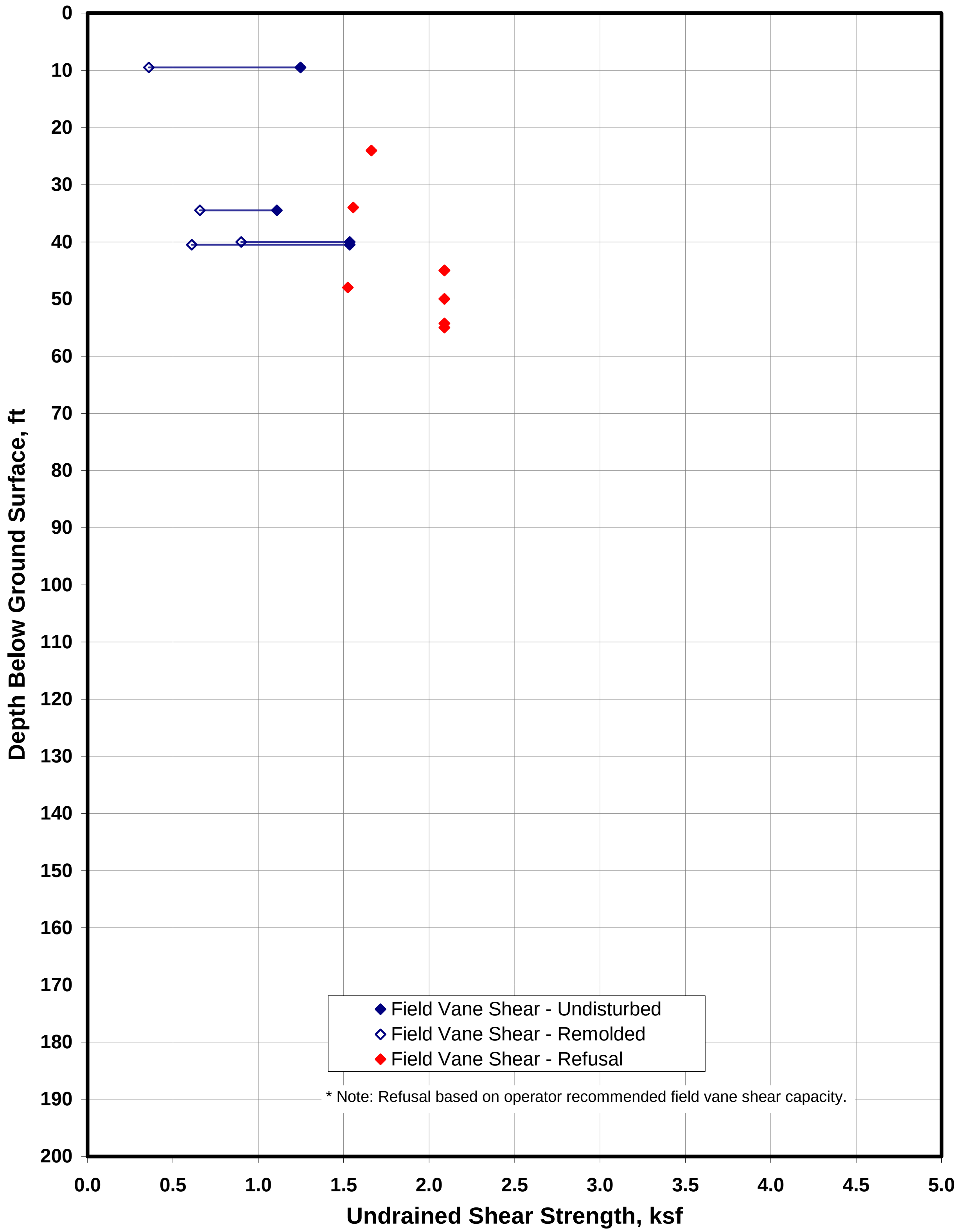


Figure 8-88. Undrained Shear Strength from Vane Shear Tests, Study Section 3: Alum Rock Station to Crossover.

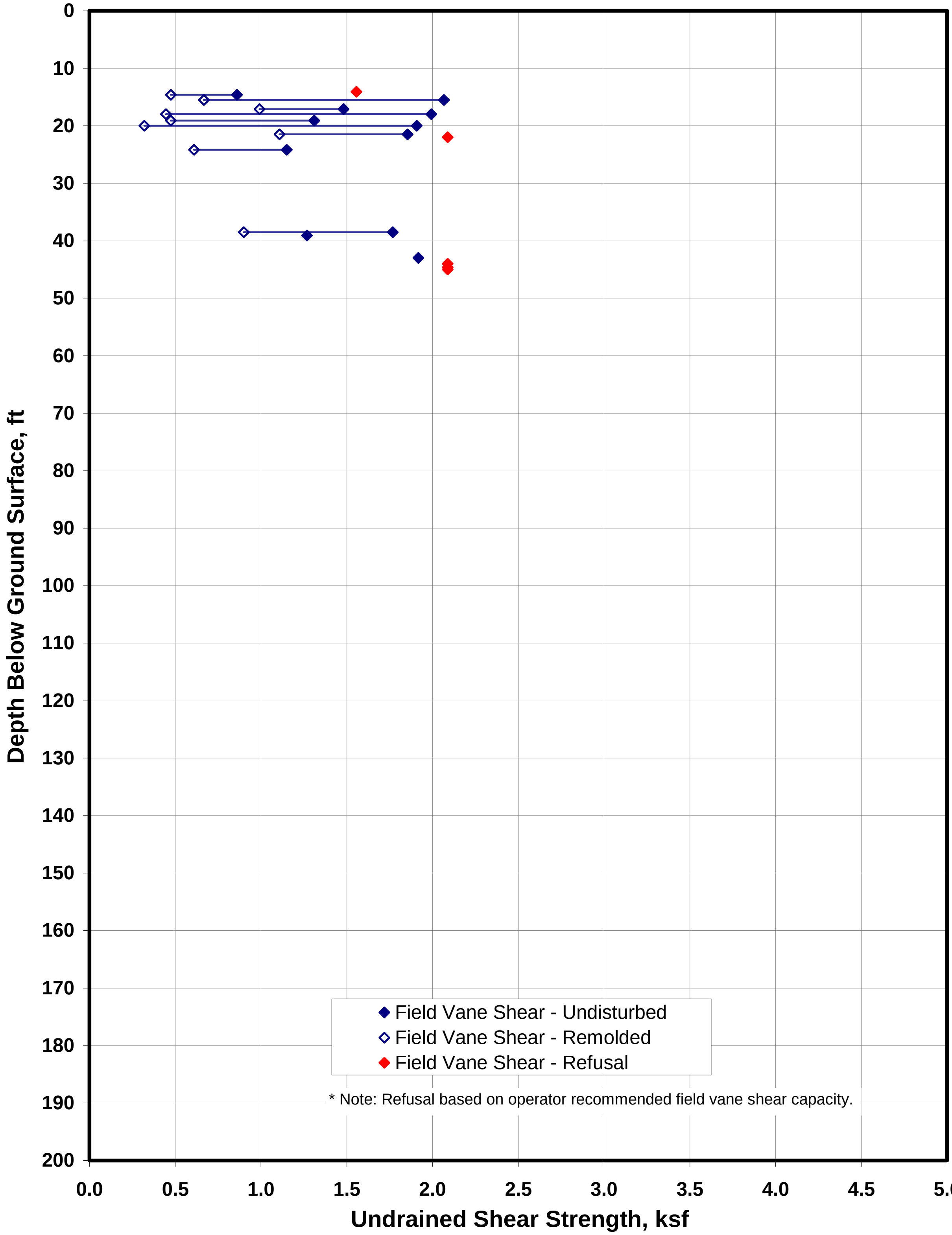


Figure 8-89. Undrained Shear Strength from Vane Shear Tests, Study Section 4: Crossover and Downtown San Jose Station.

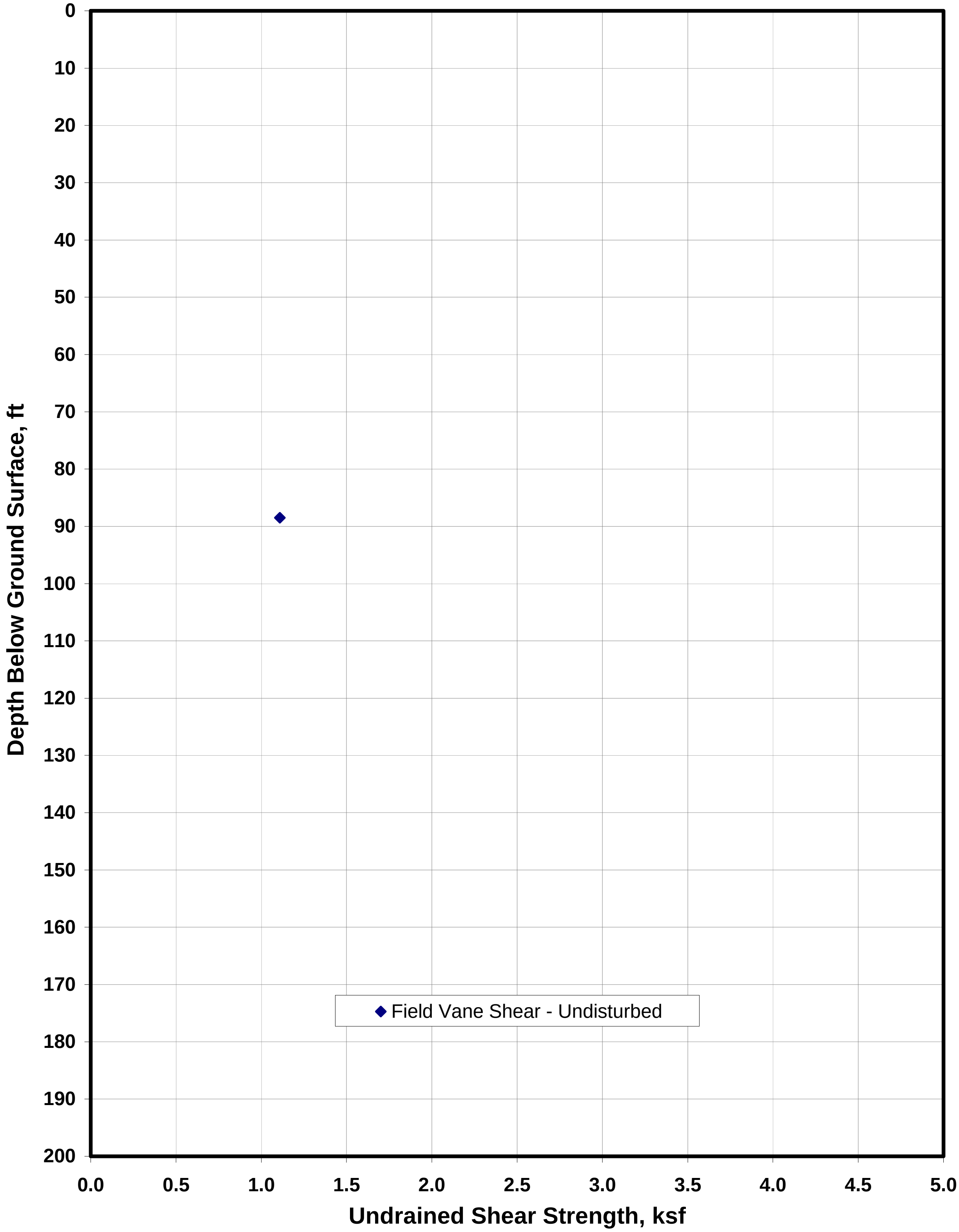


Figure 8-90. Undrained Shear Strength from Vane Shear Tests, Study Section 5: Downtown San Jose Station to Diridon/Arena Station.

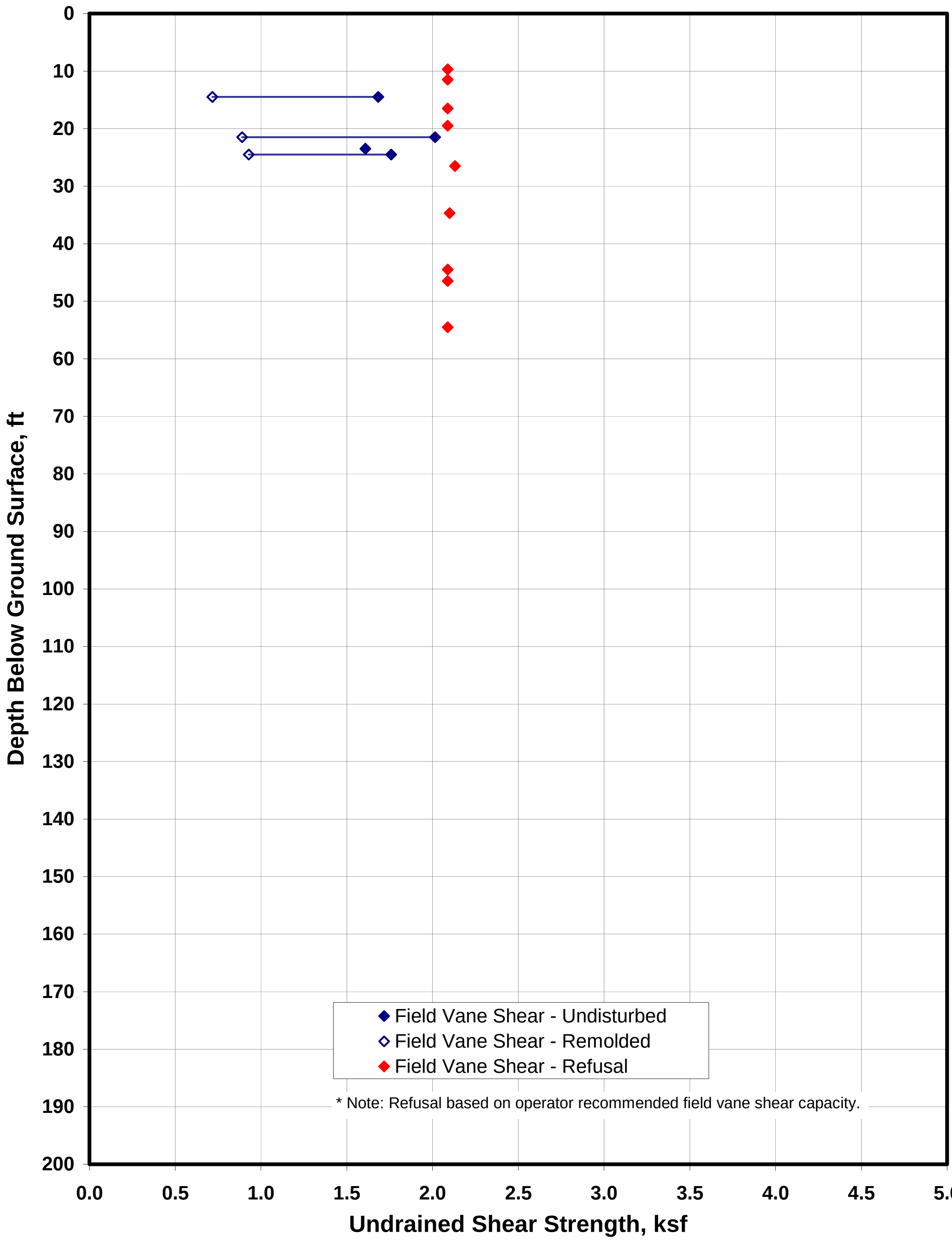


Figure 8-91. Undrained Shear Strength from Vane Shear Tests, Study Section 6: Diridon/Arena Station.

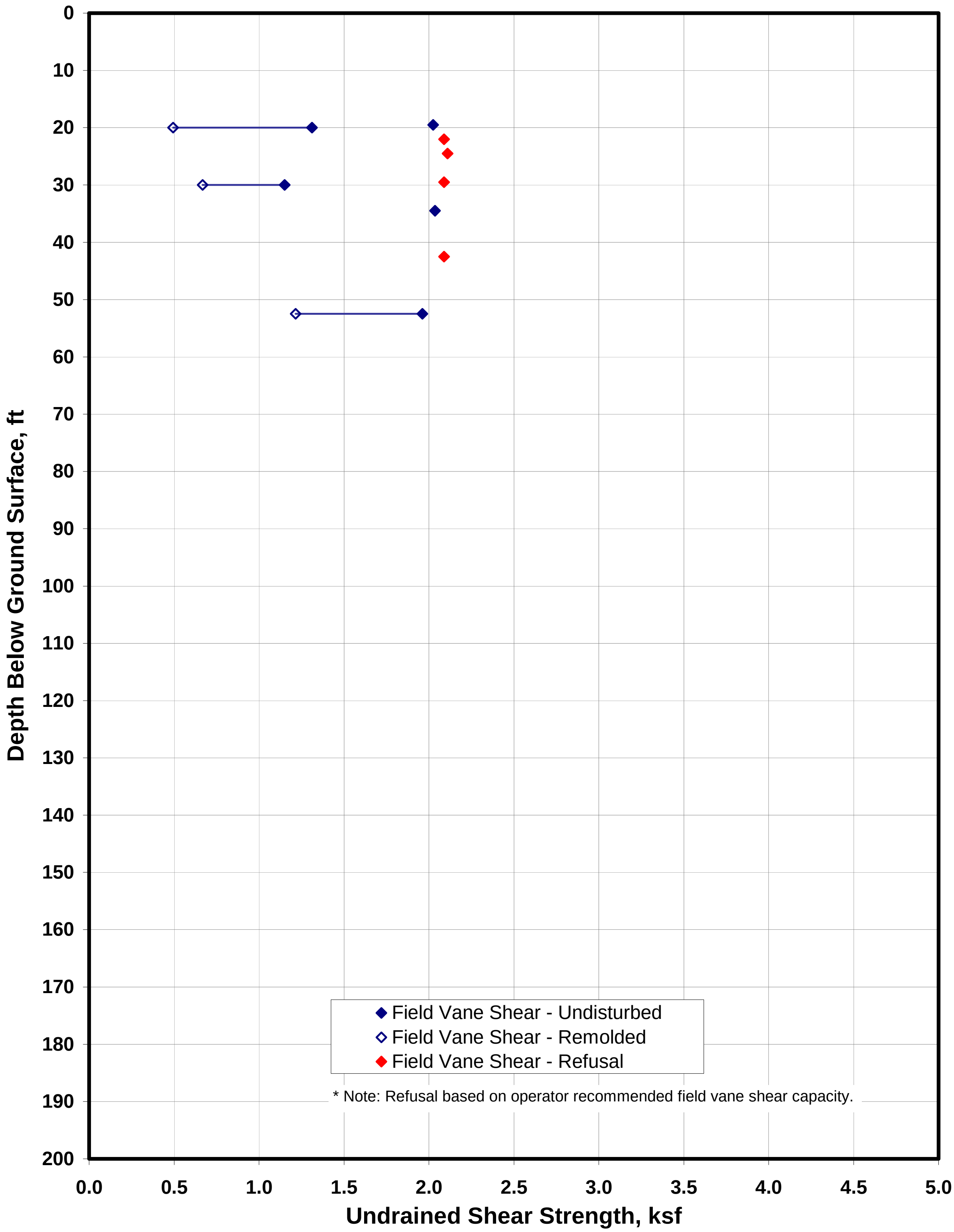


Figure 8-92. Undrained Shear Strength from Vane Shear Tests, Study Section 7: Diridon/Arena Station to West Portal.

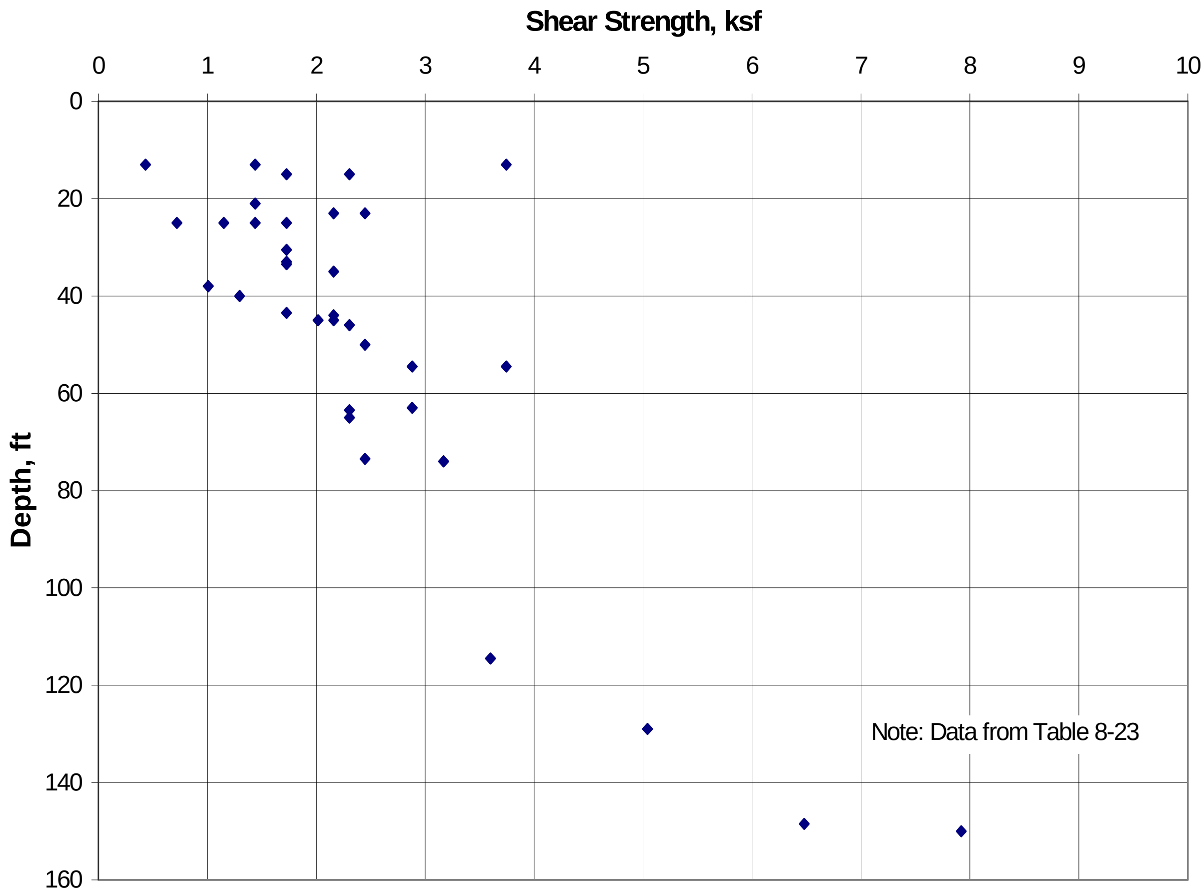


Figure 8-93. Undrained Shear Strength from Pressuremeter Tests.

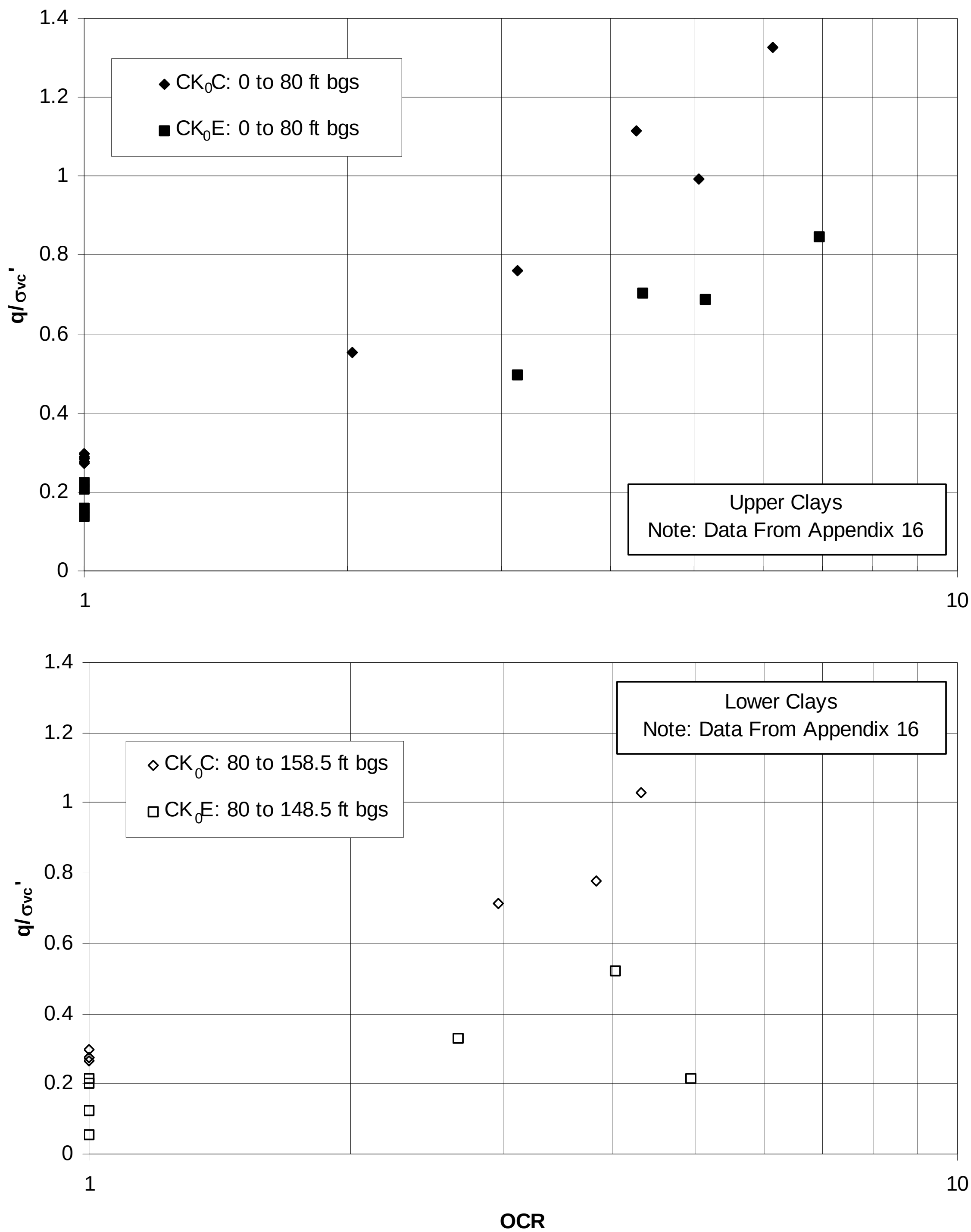


Figure 8-94. Laboratory-Derived Relationship between Over-Consolidation Ratio (OCR) and Normalized Undrained Shear Strength (q/σ_{vc}') from Triaxial Tests.

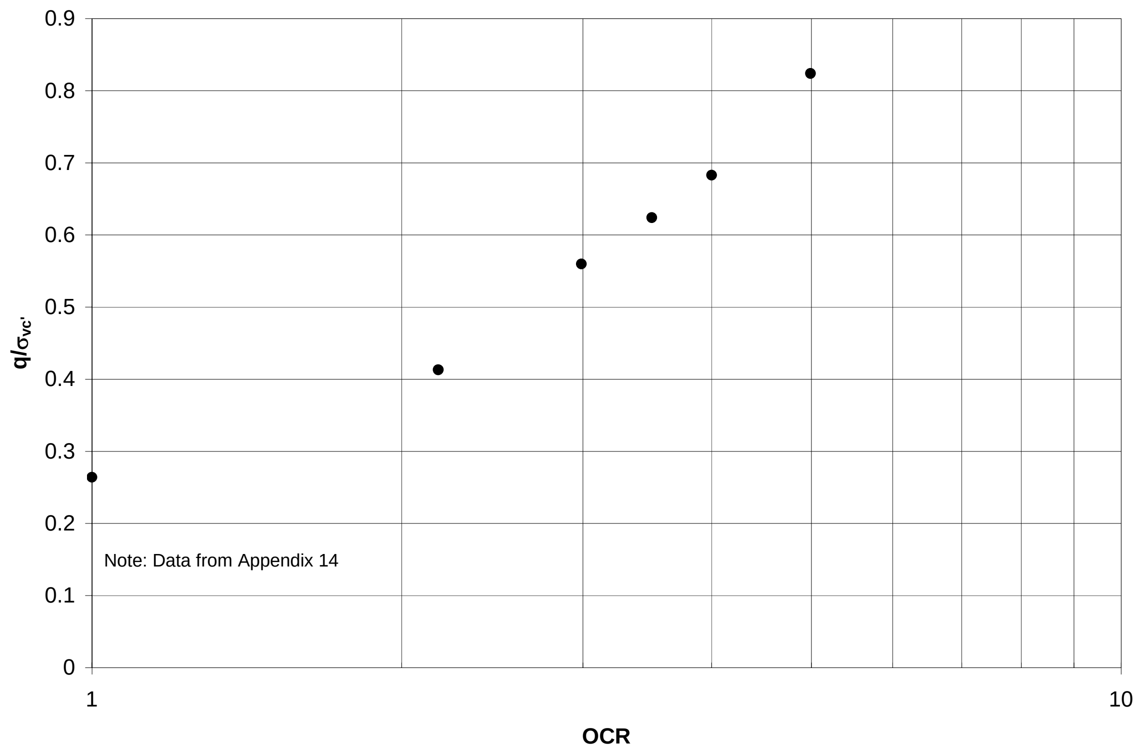


Figure 8-95. Laboratory-Derived Relationship between Over-Consolidation Ratio (OCR) and Normalized Undrained Shear Strength (q/σ_{vc}') from Simple Shear Tests.

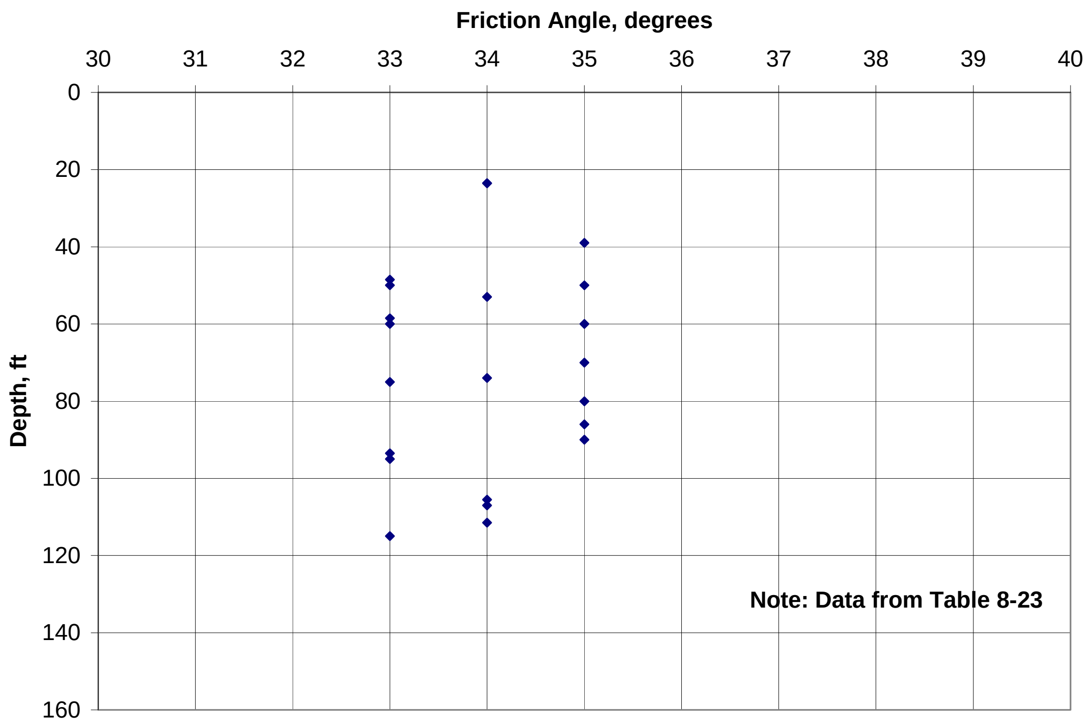
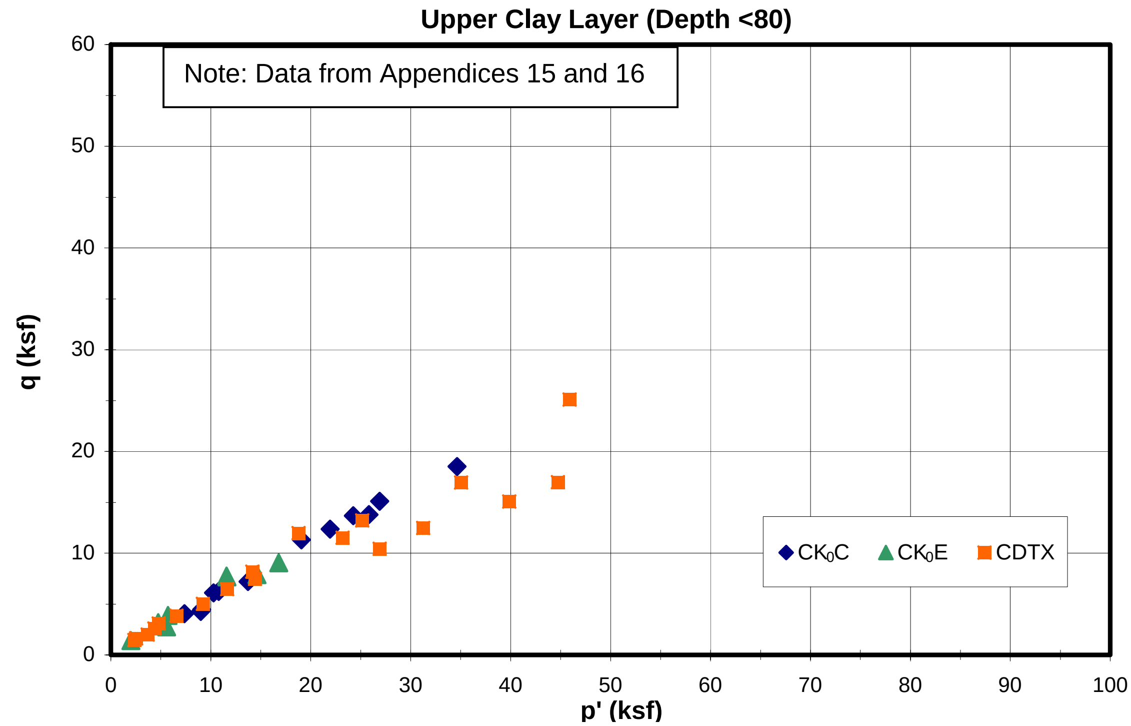


Figure 8-96. Friction Angle of Granular Materials from Pressuremeter Tests.



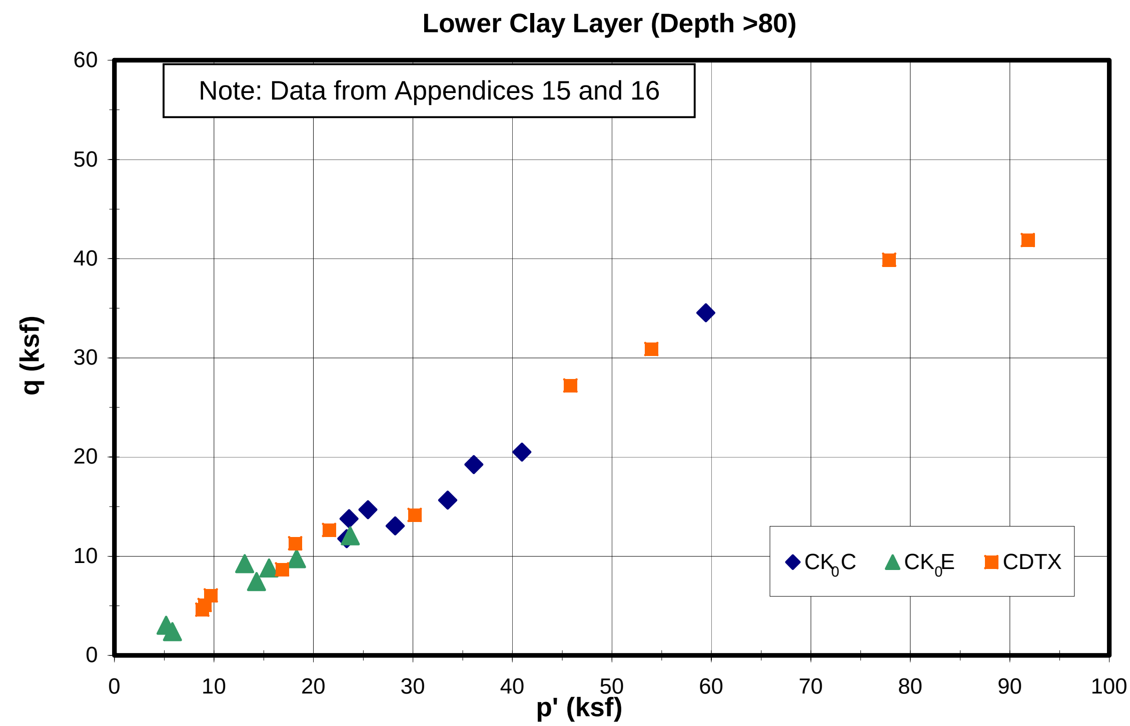


Figure 8-97. Effective Strength Parameters: Summary of Consolidated Undrained Triaxial Compression and Extension, and Drained Triaxial Tests.

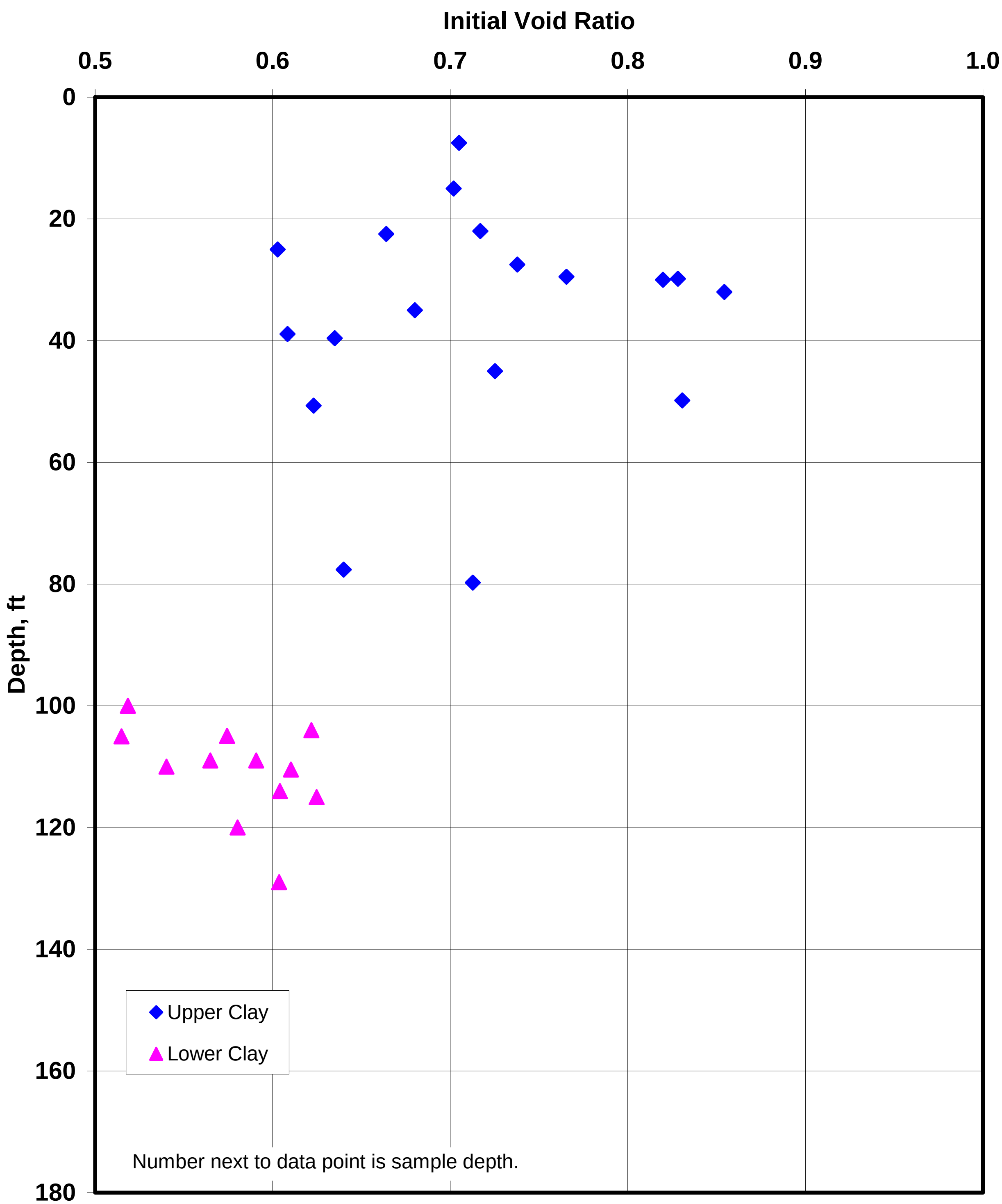
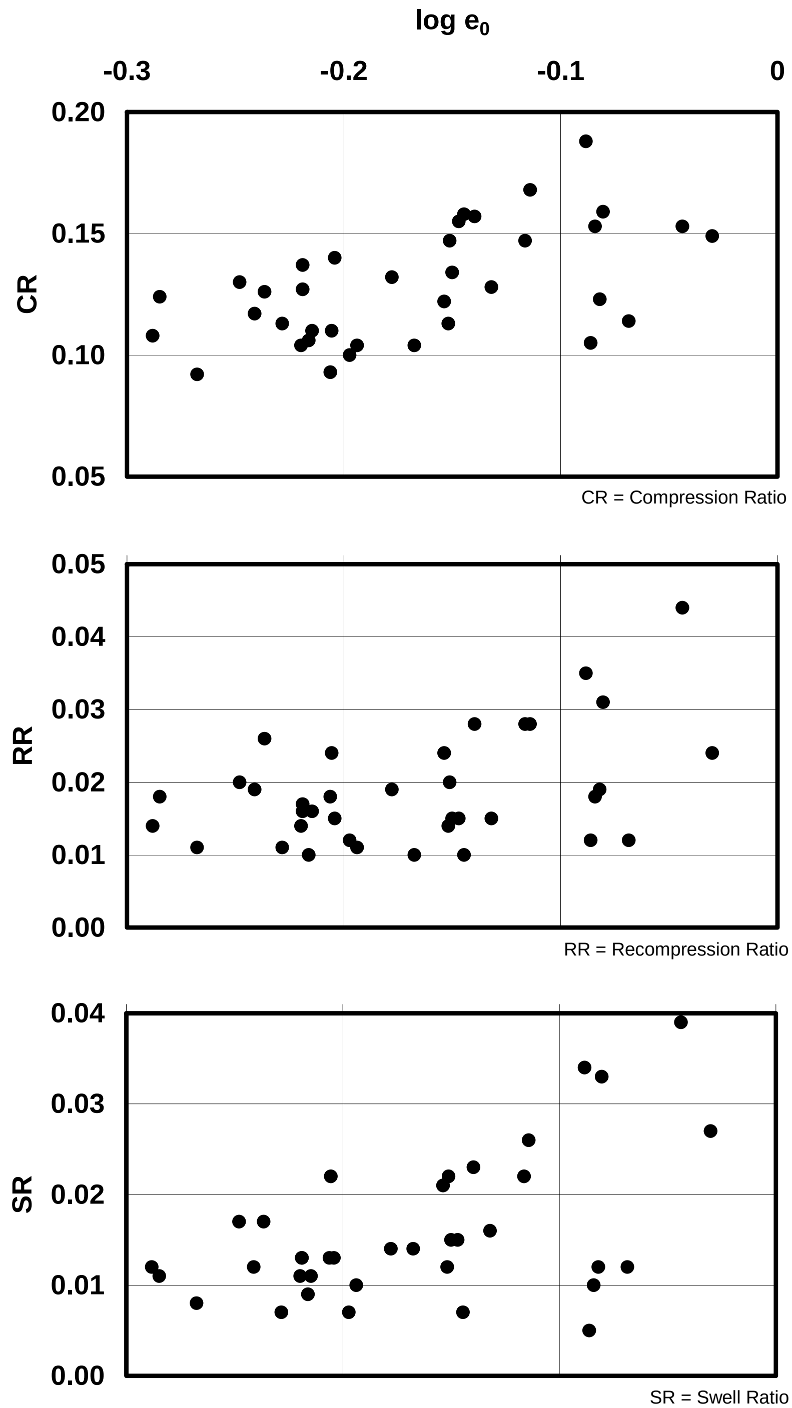


Figure 8-98. Laboratory Strain-Controlled Consolidation Tests: Void Ratio-Depth Relationship.



Note: Data from Appendix 13

Figure 8-99. Laboratory Strain-Controlled Consolidation Tests: Compression Ratios as a Function of Initial Void Ratio e_0.

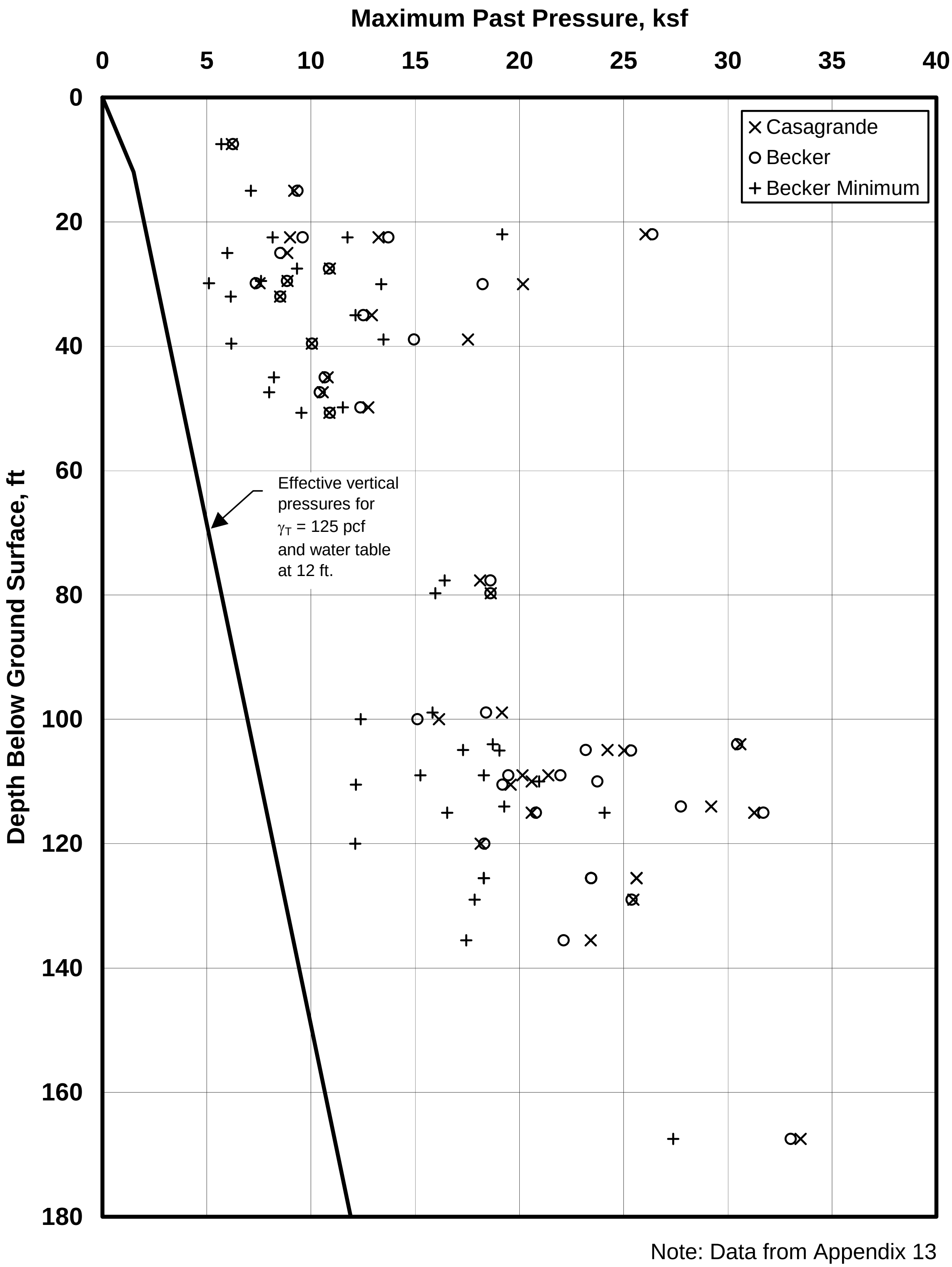


Figure 8-100. Laboratory Strain-Controlled Consolidation Tests: Maximum Past Pressures Estimated by Several Methods.

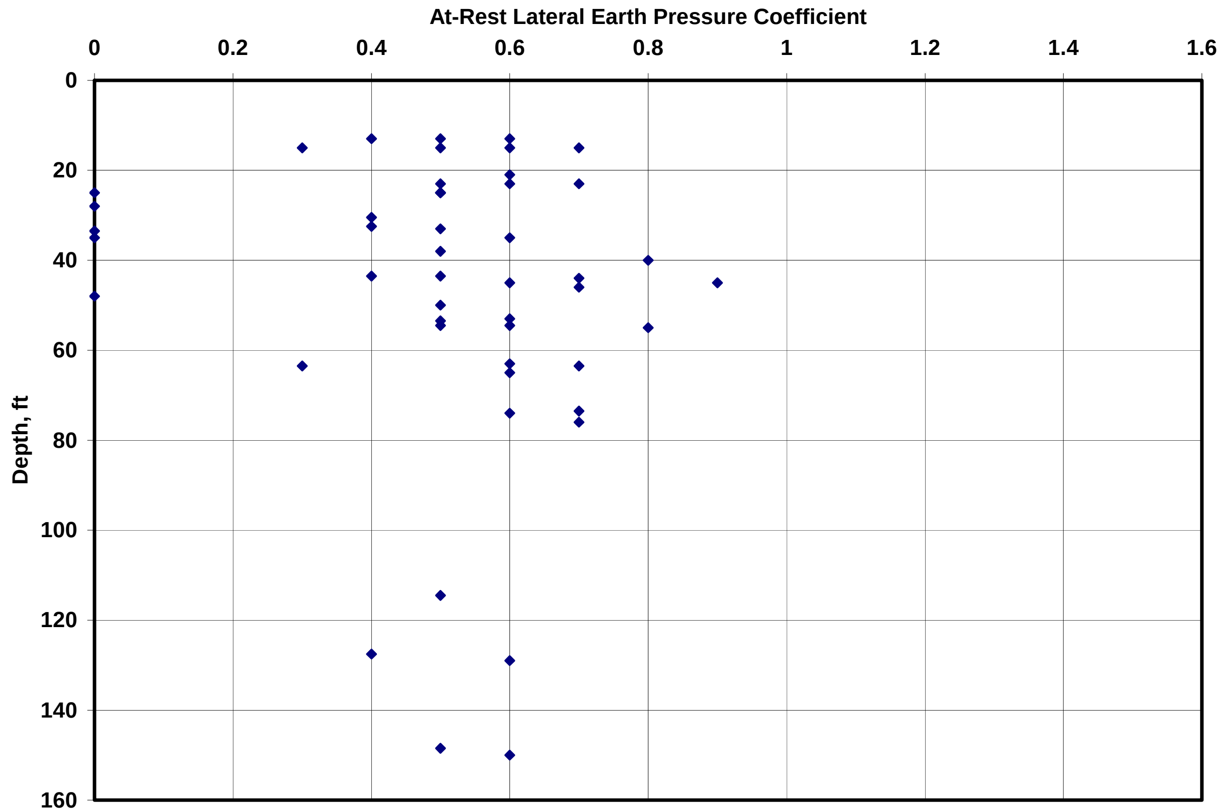


Figure 8-101. Variation with Depth of At-Rest Earth Pressure Coefficient (K_0) Derived from Pressuremeter Tests: Clay Soils.

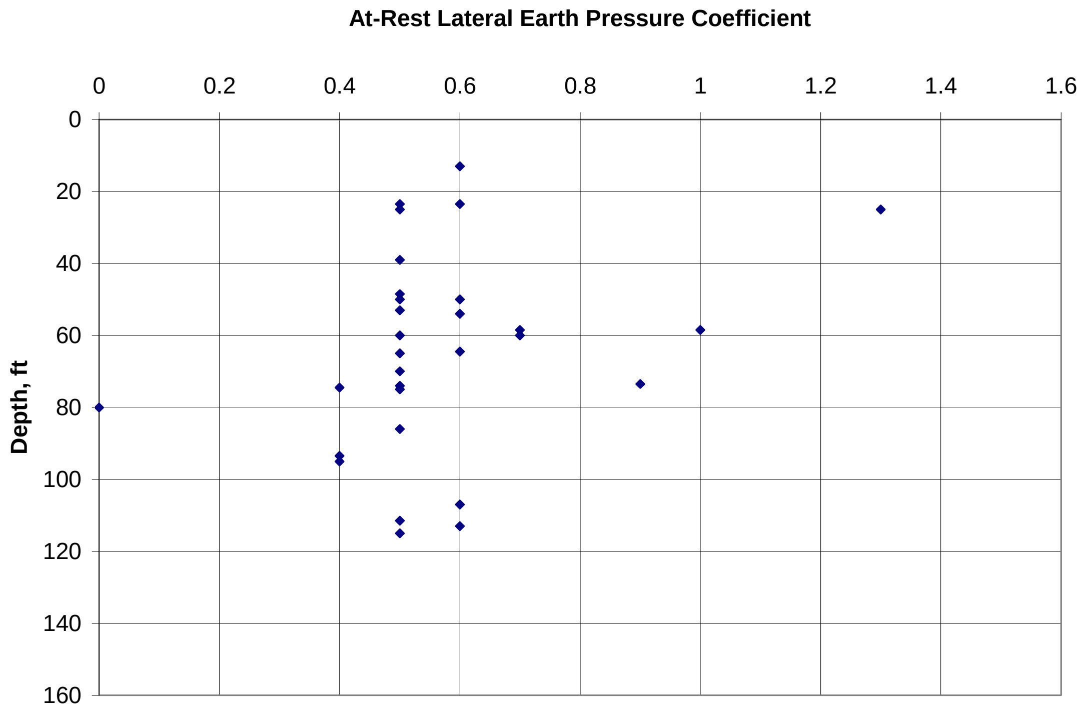


Figure 8-102. Variation with Depth of At-Rest Earth Pressure Coefficient (K_0) Derived from Pressuremeter Tests: Granular Soils.

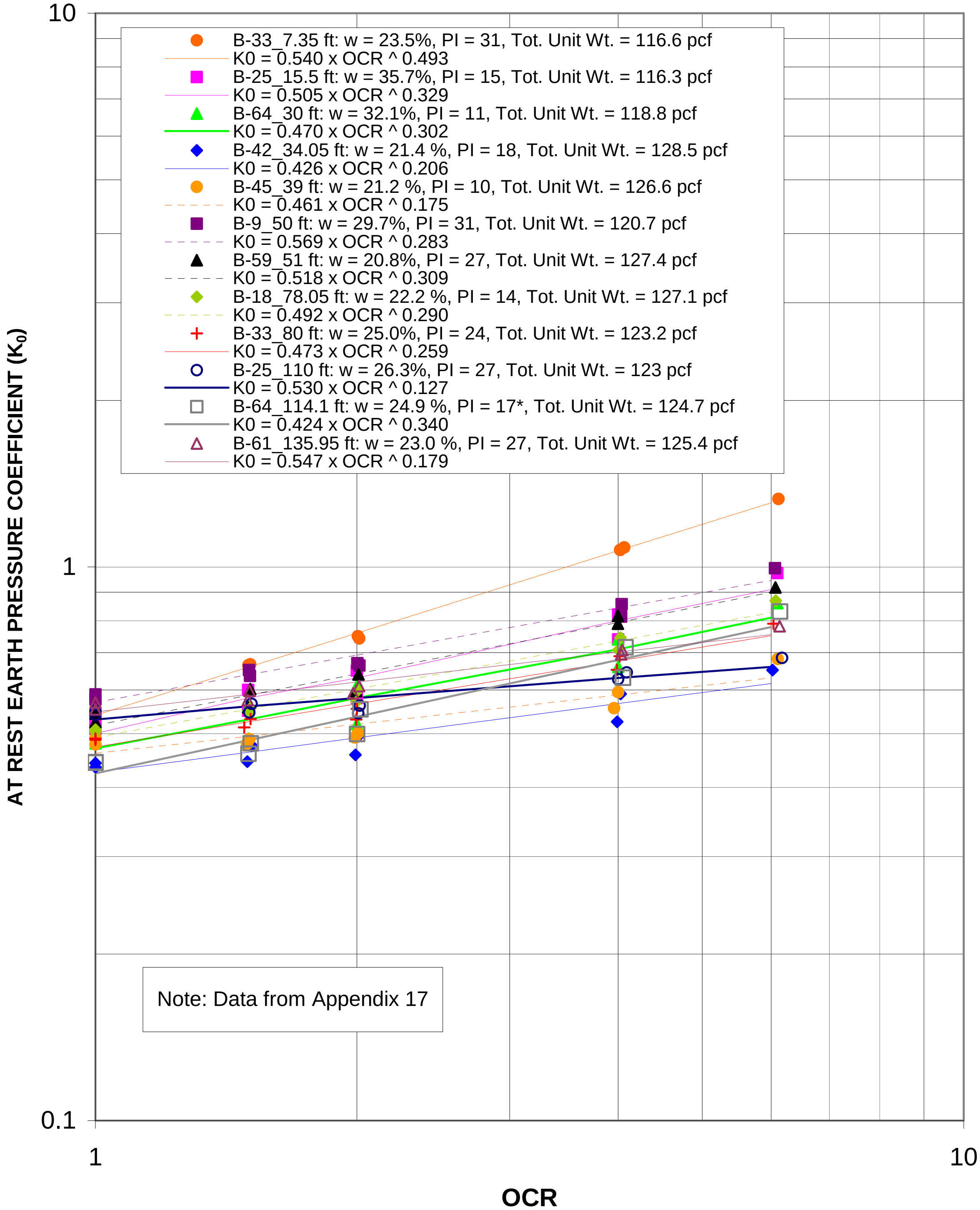


Figure 8-103. Laboratory-Derived Relationship between Over-Consolidation Ratio (OCR) and At-Rest Earth Pressure Coefficient (K_0).

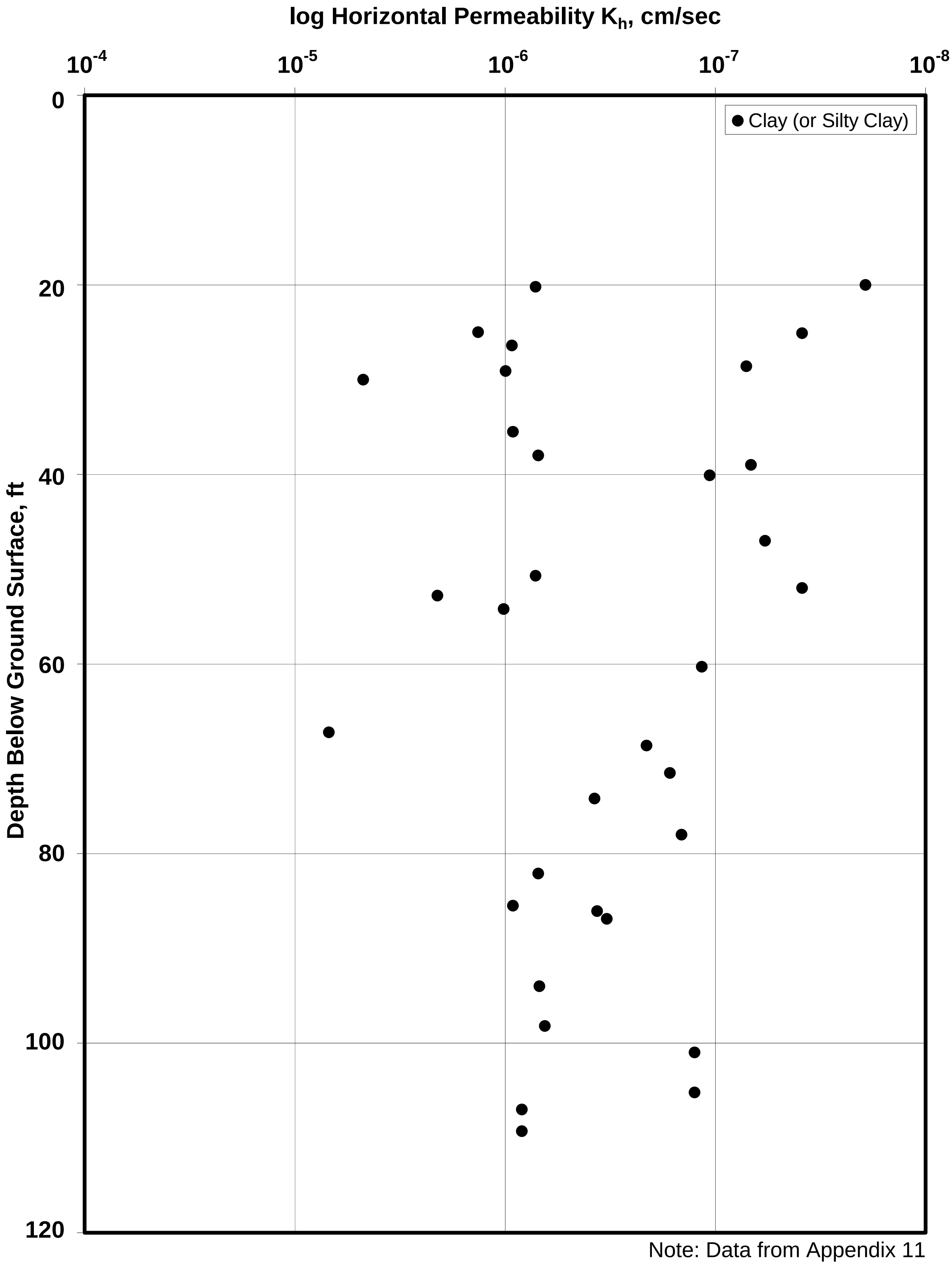


Figure 8-104. Coefficient of Horizontal Hydraulic Permeability from Cone Dissipation Test Results.

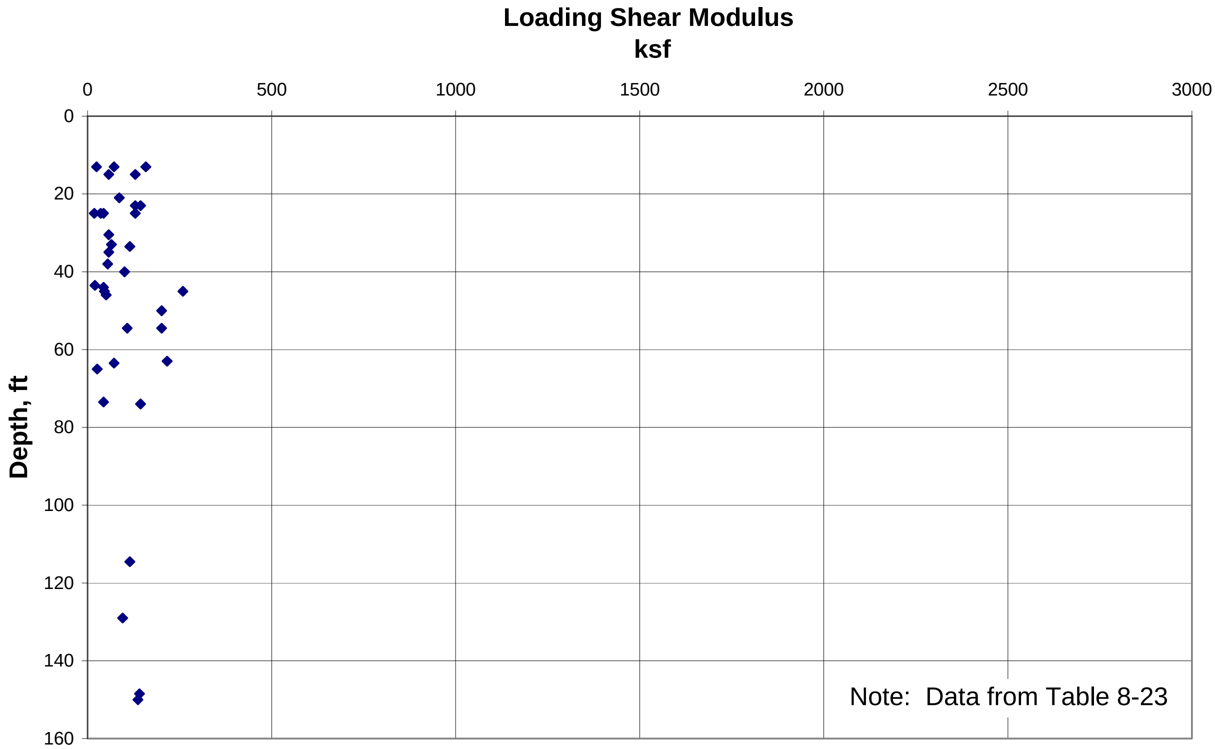


Figure 8-105. Initial Tangent Shear Modulus from Pressuremeter Tests: Clay Soils.

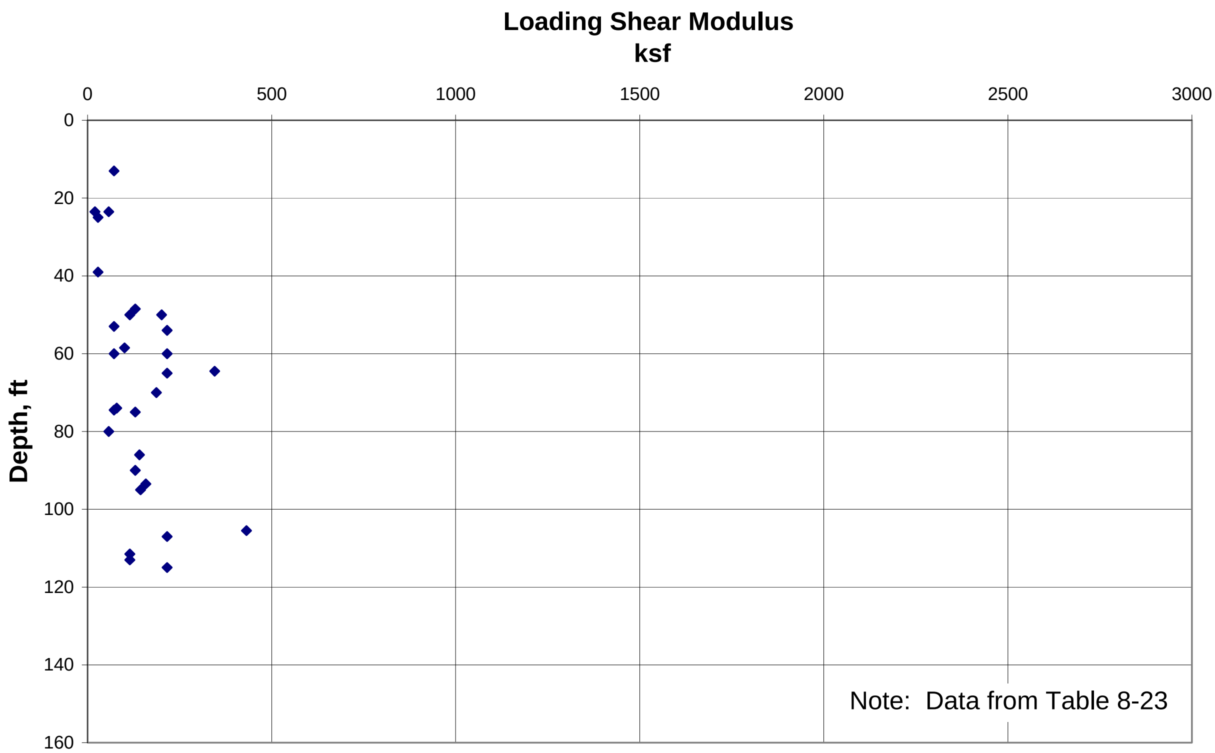


Figure 8-106. Initial Tangent Shear Modulus from Pressuremeter Tests: Granular Soils.

Figure 8-107. Secant Shear Modulus from Pressuremeter Tests: Clay Soils.

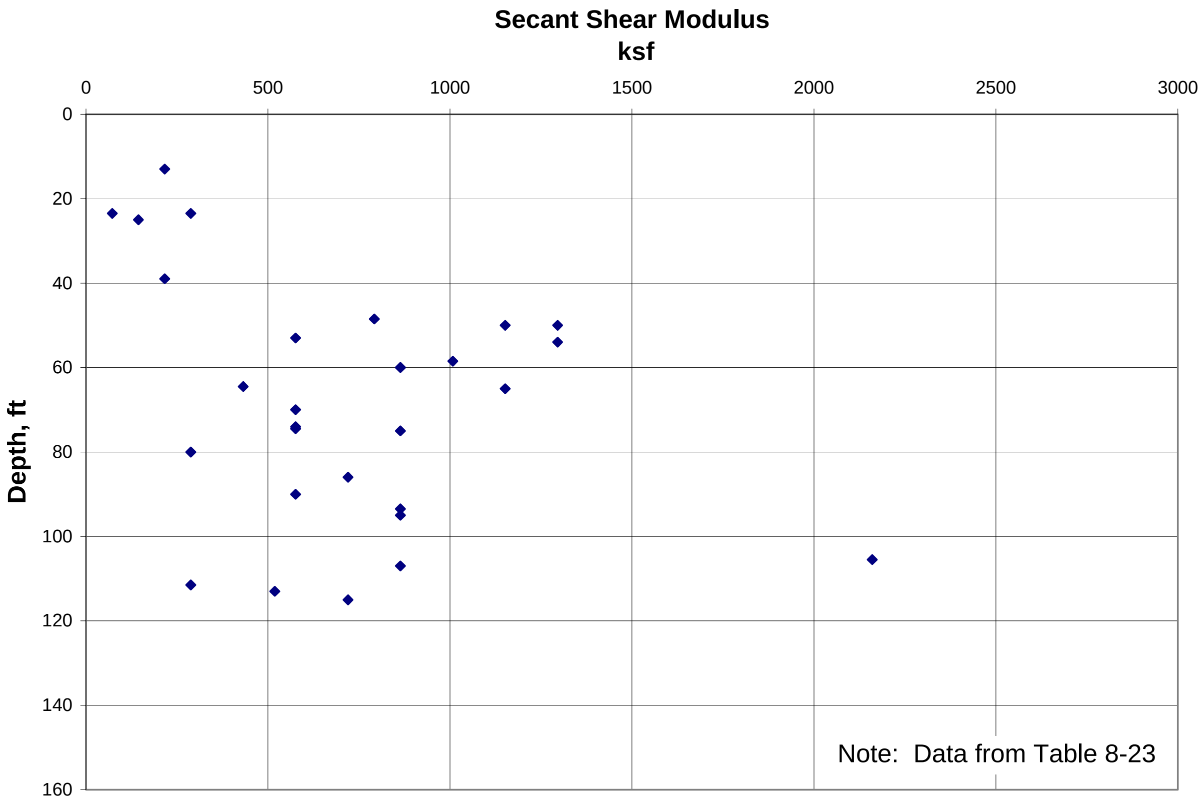


Figure 8-108. Secant Shear Modulus from Pressuremeter Tests: Granular Soils.

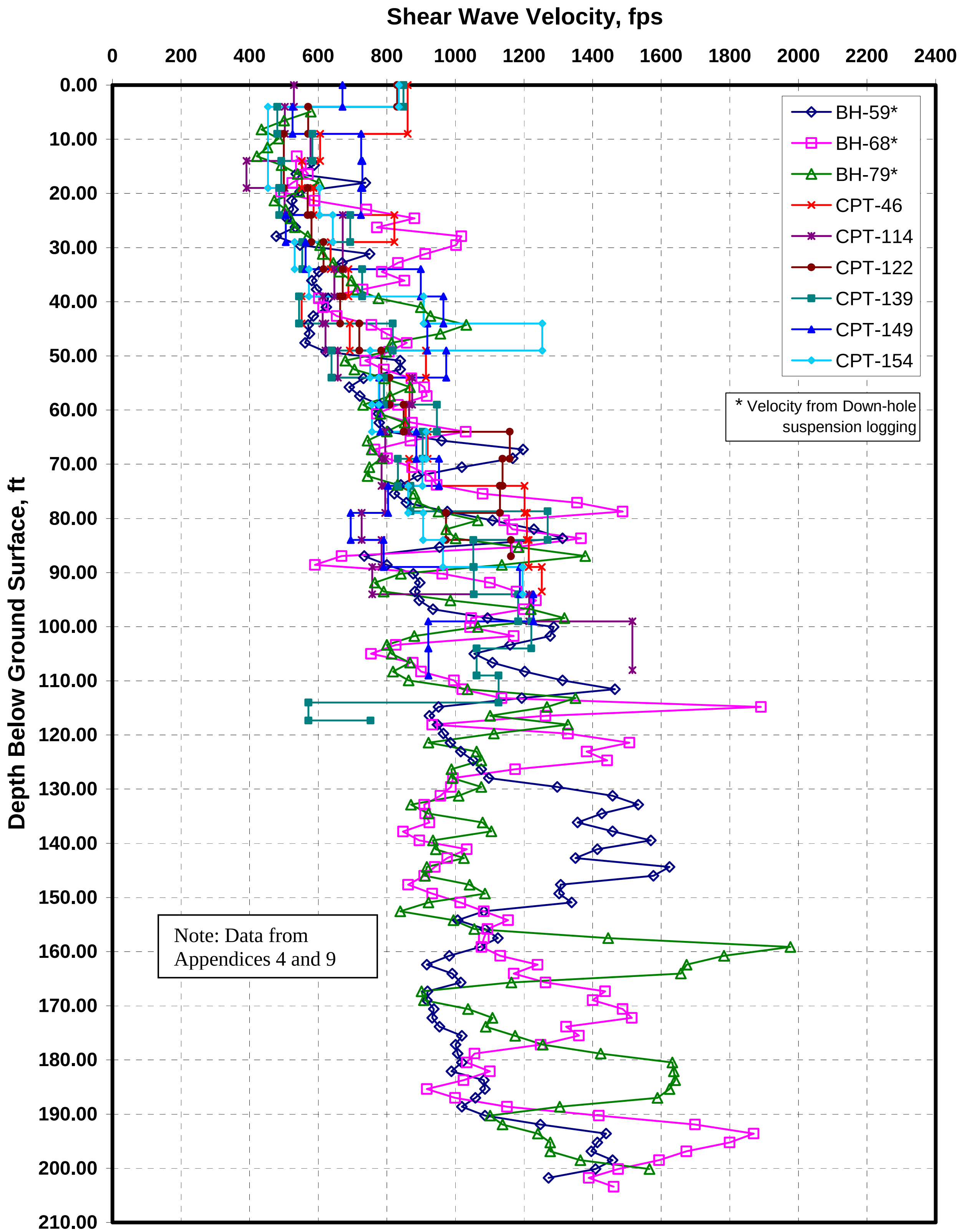


Figure 8-109. Shear Wave Velocities from Suspension Logging and Seismic Cone Soundings.

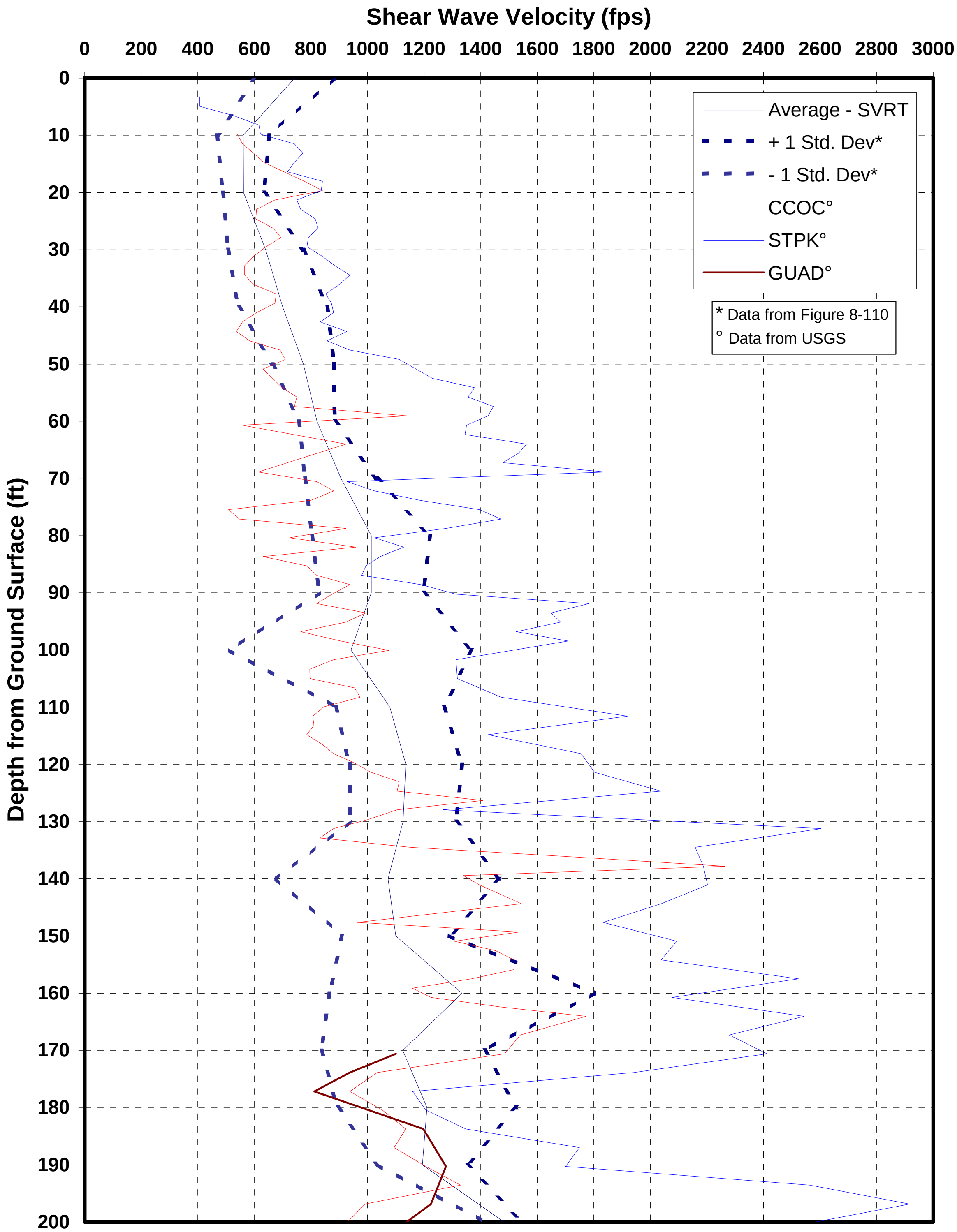


Figure 8-110. Comparison between Shear Wave Velocities from Suspension Logging, Seismic Cone, and Data from USGS.

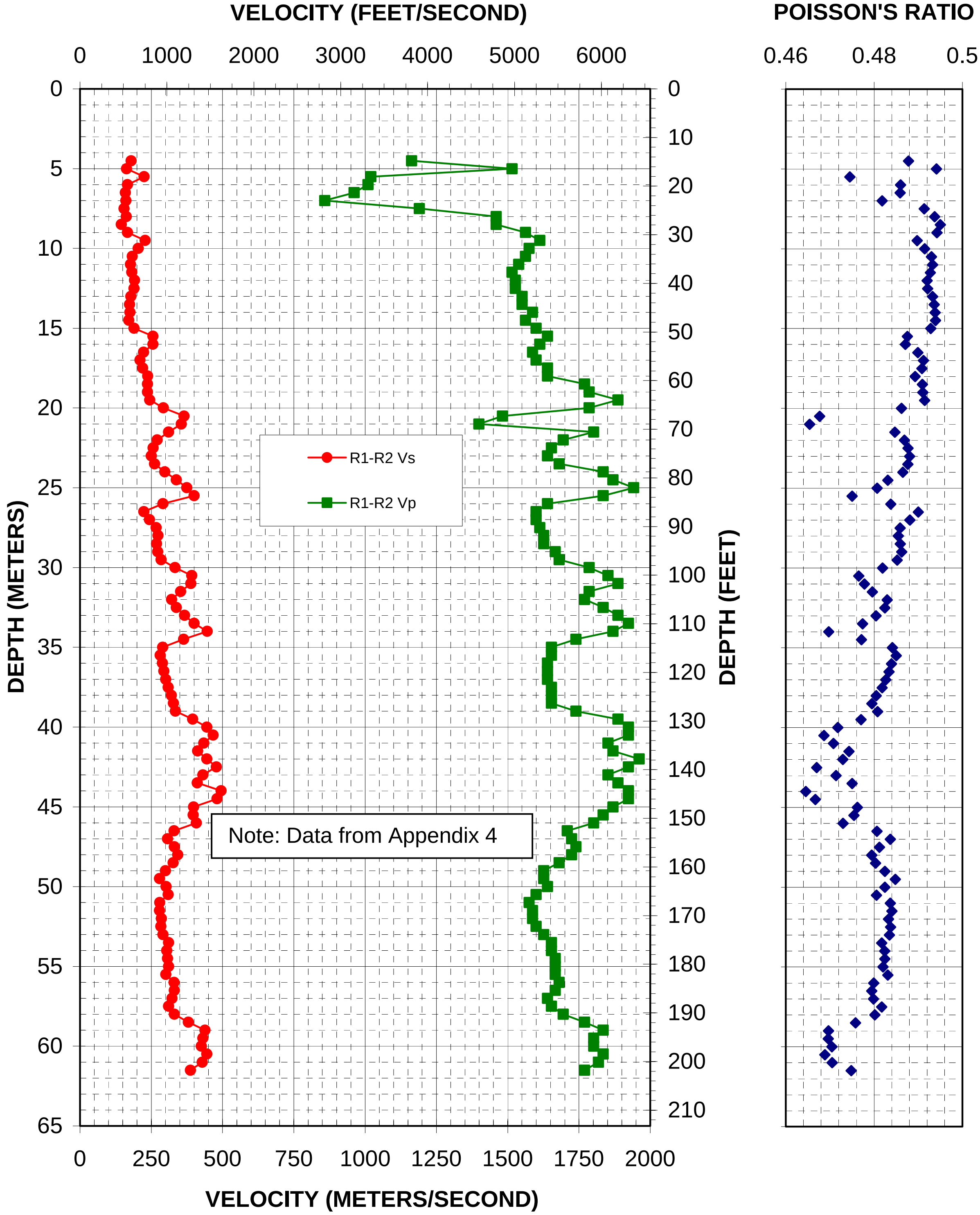


Figure 8-111. Suspension P and S Wave Velocities and Poisson's Ratio: Borehole 59.

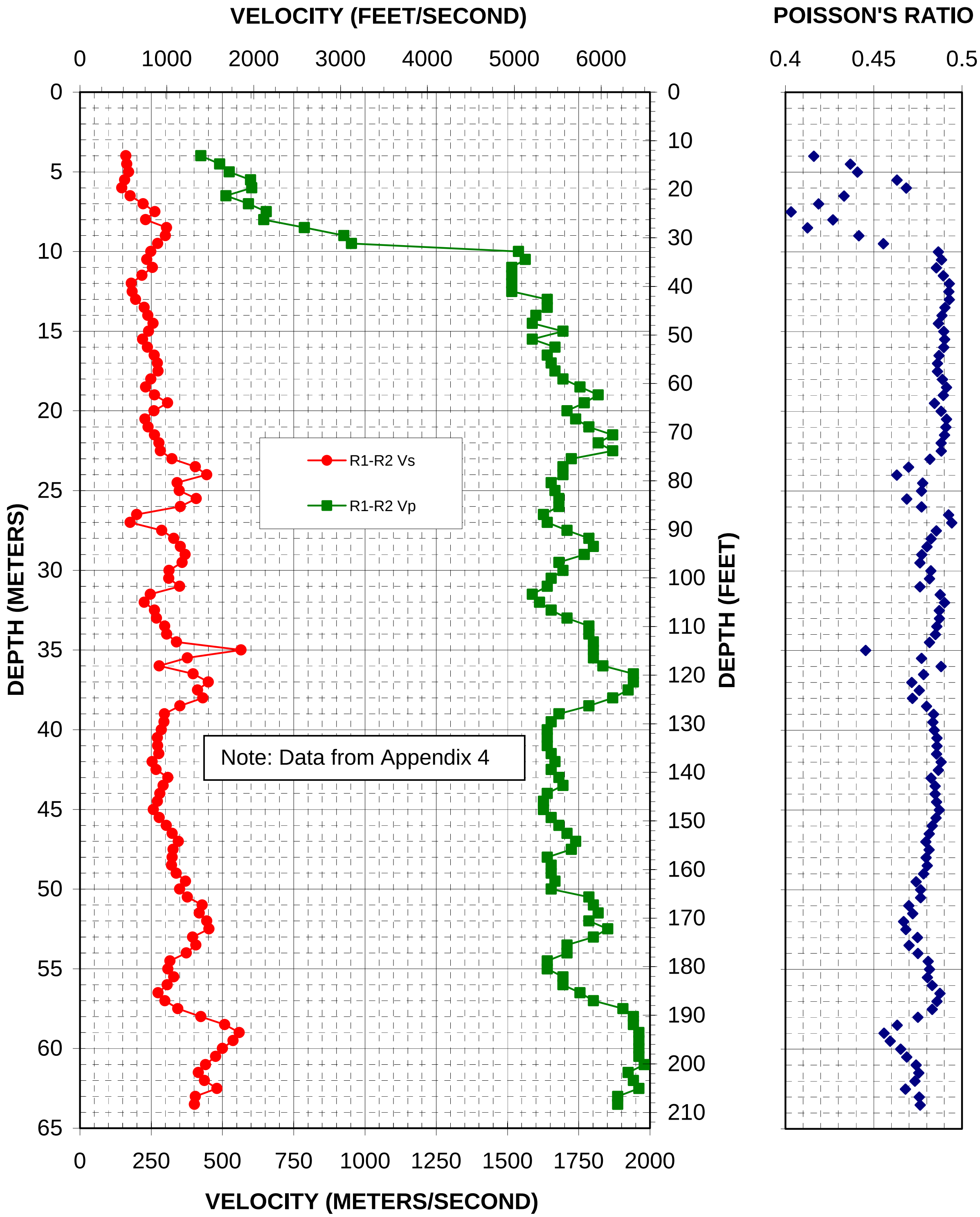


Figure 8-112. Suspension P and S Wave Velocities and Poisson's Ratio: Borehole 68.

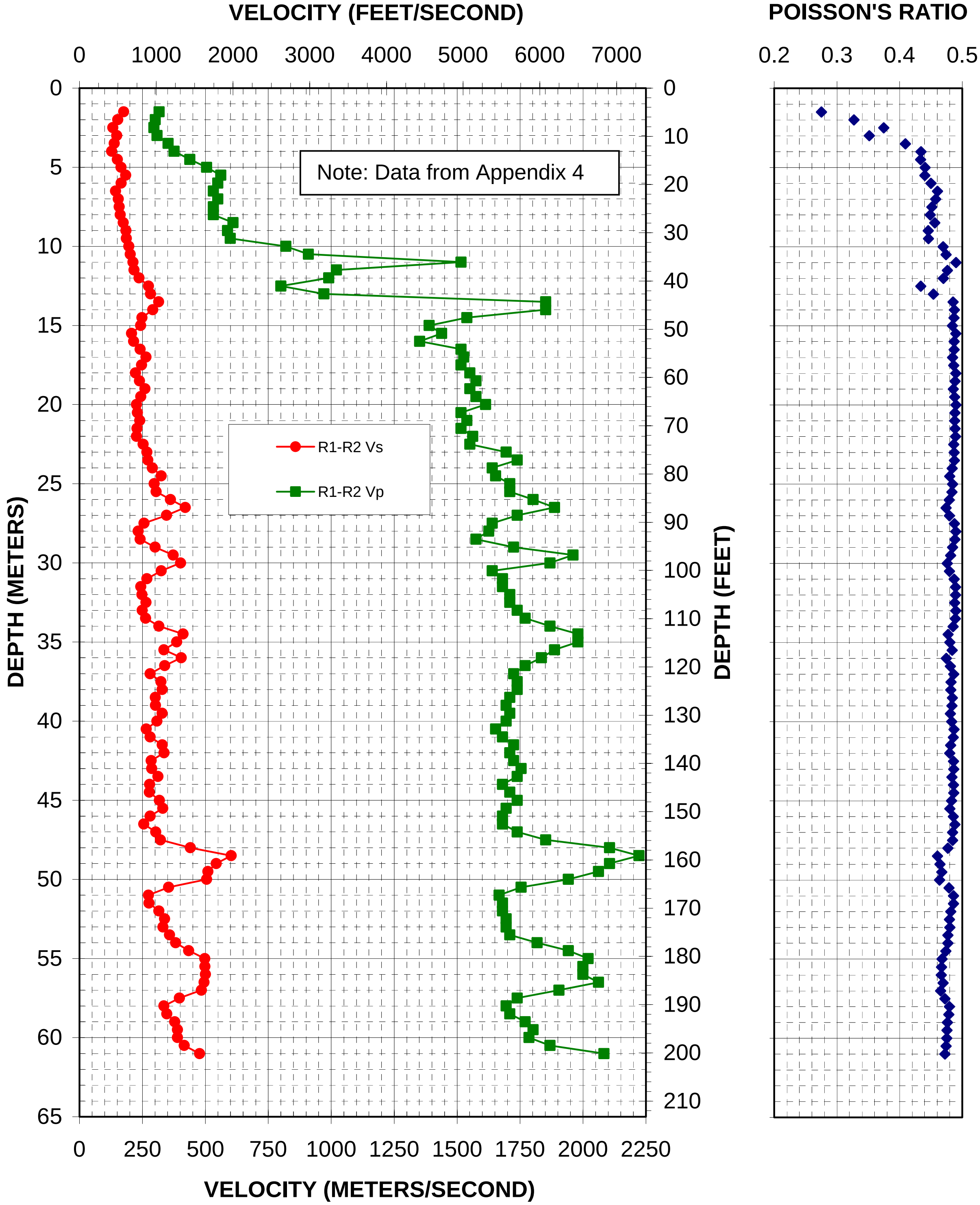


Figure 8-113. Suspension P and S Wave Velocities and Poisson's Ratio: Borehole 79.

9.0 References

American Society for Testing and Materials: Annual Book of ASTM Standards.

California Division of Mines and Geology (2000). "Seismic Hazard Evaluation of the San Jose East 7.5-Minute Quadrangle, Santa Clara County, California," *Open-File Report 2000-010*.

California Division of Mines and Geology (2002). "Seismic Hazard Zone Report for the San Jose West 7.5-Minute Quadrangle, Santa Clara County, California," *Seismic Hazard Zone Report 058*.

California Department of Water Resources (2005). Update to DWR Bulletin 195 (in progress), J. Goodridge (consultant).

City of San Jose (2004). Electronic files of elevation data around downtown San Jose transmitted by Roland Gong to Tina Takagi (HMM/Bechtel), December 7, 2004.

Clark, William O. (1924). Ground Water in Santa Clara Valley, California. U.S. Geological Water-Supply Paper 519.

County of Santa Clara, Department of Public Works (1966). "Drainage Manual".

Geomatrix Consultants (2004). "Summary Discussion for Silver Creek Fault, Version 1.0, Southeast San Francisco Bay Area, California," Report prepared for Valley Transit Authority, San Jose, California, September.

HMM/Bechtel (2004a). "Review of Available Geotechnical Data-Bart to San Jose Project, Segments 3 and 4," *Report No P0503-D300-RPT-GEO-001*, issued to VTA, San Jose, California.

HMM/Bechtel (2004b). Silicon Valley Rapid Transit Project: "Proposed Geotechnical Investigation Plan: Phases 1 and 2," *Report No P0503-D300-PLN-GEO-001, Rev. A*, issued to VTA, San Jose, California.

HMM/Bechtel (2004c). "Field Manual for Borehole Exploration," Report No 1637.001 prepared for VTA, San Jose, California.

HMM/Bechtel (2004d). " Ground Water Diversion Measures," Internal Project Memorandum, December 20.

HMM/Bechtel (2005a). "Changes to Proposed Geotechnical Investigation Plan, Phases 1 and 2," Internal Project Memorandum, September 2005.

HMM/Bechtel (2005b). "Silicon Valley Rapid Transit Project, Tunnel Segment: Report on Seismic Design Ground Motions," *Report No P0503-D300-RPT-DE012*, issued to VTA, San Jose, California.

HMM/Bechtel (2005c). "Site Class C Seismic Design Ground Motions for the Tunnel Section," Internal Report, May 6.

HMM/Bechtel (2005d). "Tunnel Segment Hydrogeology Report," *Report No P0503-D300-RPT-DE-020, Rev. A*, issued to VTA, San Jose.

HMM/Bechtel (2005e). "Observed Upward Ground Water Flow," Internal Project Memorandum, May 9.

HMM/Bechtel (2005f). Handwritten notes based on visual observation of extruded samples from four station borings, performed jointly by Parikh Consultants and Kleinfelder, and from eight tunnel borings, performed by P. Preedanont. Notes stored in Project Files.

Iwamura, T. I. (1995). "Hydrogeology of the Santa Clara and Coyote Valley Groundwater Basins, California." in *Sangines*, Recent Geologic Studies in the San Francisco Bay Area, Pacific Section of the Society of Economic Paleontologists and Mineralogists, Eugenia M., David W. Anderson, and Anna V. Buising, (eds.), v. 76, p. 173-192.

International Conference of Building Code Officials (1997)

Knudsen, K.L., Sowers, J.M., Witter, R.C., Wentworth, C.M., and Helley, E., J. (2000). "Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine County San Francisco Bay Region, California: A Digital Data Base", U.S. Geological Survey, *Open File Report 00_444*, Scale 1:275,000.

Ladd, C.C. and Foott, R. (1974). "New Design Procedure for Stability of Soft Clays", *Journal of the Geotechnical Engineering Division*, 100 (GT7), 763-786.

Moran, J., E., Bryan Hudson, Gail, F. E., and Roald, L. (2003). " California aquifer Susceptibility: A Contamination Vulnerability Assessment for the Santa Clara and San Mateo County Ground Water Basins, " *UCRL-TR-201929*, Lawrence Livermore National Laboratory, Livermore, California.

National Earthquake Hazard Reduction Program (NEHRP) (1997). "Recommended provisions for seismic regulations for new buildings," Federal Emergency Management Agency, *Report FEMA 302*, Washington, D. C.

Oppenheimer, D. H., Bakun, W. H. and Lindh, A., G. (1990). "Slip Partitioning of the Calaveras Fault, California, and Prospects for Future Earthquakes," *Journal of the Geophysical Research*, v. 95.

Robertson, P., K. (1990). "Soil Classification Using the Cone Penetration Test," *Canadian Geotechnical Journal*, v. 27.

Robertson, P. K., Campanella, R. G., Gillespie, D. and Greig, J. (1986). "Use of Piezocone Data," *Proceedings of the ASCE Specialty Conference In-situ 1986: Use of In-situ Tests in Geotechnical Engineering*, Blacksburg, 1263-80.

Santa Clara Valley Water District (2002). "Groundwater Conditions 2001," July 2002.

Santa Clara Valley Water District (2005). "Groundwater Cleanup Oversight Programs - View Files".

URS (2001). "Field Exploration for the Major Investment Project (MIS)," *Report to Earth Tech, Inc.*.

URS (2002). " Preliminary Exploration Report, Silicon Valley Rapid Transit Corridor, Alternative 11-BART on UPPR Alignment, Alameda County and Santa Clara County, California," *Report to Earth Tech, Inc*.

URS Corporation (2003). "Geotechnical Exploration Findings and Recommendations Report, Santa Clara Valley Transportation Authority (VTA), Bart Extension to Milpitas, San Jose and Santa Clara, 10% Conceptual Engineering Vol. ll: Tunnel and Underground Stations Segment," *Report to Earth Tech, Inc.*, October.

U.S. Army Corps of Engineers (1970). "Laboratory Soils Testing ", EM 1110-2-1906, Nov. 30, 1970.

U.S. Army Corps of Engineers (1986). "Laboratory Soils Testing," Engineering Manual, *EM 1110-2-1906*, Change 2.

U.S. Geological Survey (1999). "Land Subsidence in the United States," Circular 1182.

U.S. Geological Survey (2004a). Database files transmitted by mail, from Loren Metzger to Matthew Waterman (HMM/Bechtel), July.

U.S. Geological Survey (2004b). Shear-wave velocity profiles transmitted by email, from Carl Wentworth to Andrew Liu (HMM/Bechtel), September.

Wagner, D.L., Bortugno, E.J., and McJunkin, R.D. (1990). "Geologic Map of the San Francisco-San Jose Quadrangle, California," *California Division of Mines and Geology. Regional Geologic Map Series*, 1:250,000 scale.

Working Group on California Earthquake Probabilities (WG02), "Earthquake Probabilities in the San Francisco Bay Region: 2002-2031," *U. S. Geologic Survey Open File Report 03-214*.

REFERENCES CONSULTED BUT NOT QUOTED IN TEXT

California Department of Water Resources (1967). "Evaluation of Ground Water Resources, South Bay, Appendix A: Geology," *Bulletin No. 118-1*, August.

California Department of Water Resources (1975). "Evaluation of Ground Water Resources: South San Francisco Bay, Volume III: Northern Santa Clara County Area," *Bulletin No. 118-1*, December.

California Department of Water Resources (2003). "California's Ground Water: Bulletin No. 118. 2003 update," October.

California Regional Water Quality Control Board, San Francisco Region (2003). "A Comprehensive Groundwater Protection Evaluation for the South San Francisco Bay Basins," May.

California Geological Survey (2004). Database files transmitted by e-mail, from Anne Rosinski and Terilee McGuire, to Matthew Waterman (HMM/Bechtel), July 29, 2004. (Includes water level data for San Jose East and San Jose West 7.5-minute topographic quadrangles.)

Earthquake Engineering Research Institute (1989). "The Basics of Seismic Risk Analysis," *Earthquake Spectra*, 5(4), 675-702.

FEMA (1998). "Flood Insurance Study, City of San Jose California, Santa Clara County", Volumes 1 and 2," Revised August 17, 1998 NOTE: The original study was completed in 1979 and revised in 1988 and 1998. The 1988 and 1998 revisions did <u>not</u> include effects of any of the flood control projects presently being constructed for the Guadalupe River and for Lower Silver Creek.

FEMA (1998). "Flood Insurance Study, Santa Clara County California, Unincorporated Areas, Volumes 1 and 2," revised August 17, 1998 NOTE: The original study was completed in 1979 and revised in 1988 and 1998. The 1988 and 1998 revisions did <u>not</u> include effects of any of the flood control projects presently being constructed for the Guadalupe River and for Lower Silver Creek.

HMM/Bechtel (2004). "Water Level Trends," Technical Memorandum, December.

Santa Clara Valley Transportation Authority (2004). "SVRT - BART Extension to Milpitas, San Jose and Santa Clara, Draft Environmental Impact Statement / Environmental Impact Report and Draft 4(f) Evaluation," March.

Santa Clara Valley Water District (2004). Database files (water levels and water quality) transmitted by e-mail, from Ellen Fostersmith to Matthew Waterman (HMM/Bechtel), July 27, 2004. (Includes data for wells within ½ mile of tunnel alignment for conceptual design.)

Santa Clara Valley Water District (2004). Data file (CCOC water levels) transmitted by e-mail, from Roberta Novelli to Cliff Farrell (Bechtel), September.

Santa Clara Valley Water District (1989). "Lower Silver Creek (Reach 1 & 2) Design Report," December.

Santa Clara Valley Water District (1997). "Waterways Management Model," computer print out dated Tuesday, April 22, 1997, 4:56 P. M.

U. S. Army Corps of Engineers (1977). "Hydrologic Analysis of the Guadalupe River Basin," *Hydrologic Engineering Office Report*, Sacramento, CA, June.

U. S. Army Corps of Engineers (1991). "General Design Memorandum, Guadalupe River California," Sacramento District, January.

U. S Army Corps of Engineers (2001). "Final General Re-evaluation and Environmental Report for Proposed Modifications, Guadalupe River Project, Downtown San Jose, California," Sacramento District, February.

U.S. Geological Survey (2004). URL: http://waterdata.usgs.ca/ca/nwis/gw accessed September 9, 2004 (search criteria: County = Santa Clara).

U.S. Geological Survey (2004). "Documentation of the Santa Clara Valley Regional Ground Water / Surface-Water Flow Model, Santa Clara County, California," *USGS Scientific Investigations Report*, 2004-5231.

U.S. Geological Survey (2002). "Santa Clara Valley Water District Multi-Aquifer Monitoring Well Site, Coyote Creek Outdoor Classroom, San Jose, California," *Open File Report 02-369*.