P0503-D300-RPT-GEO-006
Rev. 0

B0810-C022

Central Area Guideway

65% Submittal: Combined Geotechnical Baseline Report

For Reference Only

December 31, 2008
Issued for Record

Disclaimer:
VTA makes no representation or warranty regarding the accuracy of the information provided in this document. VTA provides this information for reference purposes only. You may not rely on this information for any purpose, including but not limited to the preparation of any bids or proposals to VTA. By reviewing this information, you and your employees, agents, representatives, consultants, or contractors assume the risk of use or reliance on the information being provided and bear the sole responsibility for verifying any materials used or relied upon. Neither VTA nor its employees, agents, representatives, consultants or contractors shall be liable for any damages for the use of this information or the viewing or copying of such information.

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

See disclaimer on cover page.

For Reference Only

See disclaimer on cover page.

P0503-D300-RPT-GEO-006
Rev. 0

B0810-C022

Central Area Guideway

65% Submittal: Combined Geotechnical Baseline Report

For Reference Only

December 31, 2008
Issued for Record

HMM/Bechtel

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

65% SUBMITTAL: COMBINED GEOTECHNICAL BASELINE REPORT

FOR

65% ENGINEERING PHASE

Contract No. S03099

Submitted by: HMM/Bechtel SVRT, a Joint Venture

For David J. Young, P.E., G.E. 12/31/08 Date and Andrew Liu, P.E., G.E. 12/31/08 Date

Reviewed by:

For Alan Haldin 12/31/08 Date and Jimmy Thompson 12/31/08 Date

Approved by:

John Hawley P.E. 31 Oct 08 Date
Central Area Guideway Design Manager

Prepared by
HMM/Bechtel SVRT,
a Joint Venture

HMM/Bechtel

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

The Contract Document contained herein has been prepared by or under the direction of the following Registered Persons.

REGISTERED CIVIL ENGINEER
David Young, C51951

REGISTERED GEOTECHNICAL ENGINEER
Andrew Liu, GE2771

For Reference Only

See disclaimer on cover page.

This page left intentionally blank.

For Reference Only

See disclaimer on cover page.

TABLE OF CONTENTS

1.0 Introduction 1
2.0 Project Description 5
3.0 Site Conditions 7
3.1 Sources of Geotechnical Information 7
3.2 Regional Geology 7
3.3 Regional Groundwater Regime 7
3.4 Regional Seismicity 8
3.5 Site Geology 8
3.6 Hydrologic Setting 13
3.7 Anticipated Hydrogeological Conditions 13
3.7.1 Artesian Pressures 13
3.7.2 Flooding 13
3.7.3 Dissolved Gases 13
3.8 Soil Porosity and Permeability 14
3.9 Engineering Properties of Soil 15
3.10 Contaminated Soil and Groundwater 17
3.11 Anticipated Performance of Ground Improvement 18
3.11.1 Results from the Permeation Grout Trial 18
3.11.2 Conclusions from the Permeation Grout Trial 19
3.12 Related Tunnel Construction Experience 19
3.12.1 Utility Tunneling 19
3.12.2 San Francisco Public Utilities Commission (SFPUC) Bay Tunnel 20
3.12.3 Related Open Cut Excavations 23
3.12.4 Tieback Installation 24
3.12.5 Other Relevant Experience 24
4.0 Tunnels 27
4.1 General 27
4.2 Anticipated Ground Conditions 27
4.2.1 Ground Stability 28

For Reference Only

See disclaimer on cover page.

4.2.2 Gas Classification 30
4.2.3 Abrasivity 30
4.2.4 Cobbles and Boulders 30
4.2.5 Soil Clogging Potential 31
4.2.6 Soil Gradation and Fines Content 32
4.2.7 Potential Obstructions to Tunneling 37
4.3 Bored Tunnels 39
4.3.1 Design Considerations 39
4.3.2 Bored Tunnel Construction Considerations 40
4.4 Excavation Interfaces 52
4.4.1 Excavation Interface Between Slurry Walls and Bored Tunnel 52
4.5 Mined Cross Passages 53
4.5.1 Site Conditions and Anticipated Behavior 54
4.5.2 Design Considerations 54
4.5.3 Cross Passage Construction Considerations 63
5.0 Surface Excavations 70
5.1 General 70
5.2 Anticipated Subsurface Conditions 70
5.3 Basis of Design 75
5.3.1 Gas Classification 75
5.3.2 Groundwater Control 75
5.3.3 Excavation Support 76
5.4 Construction Considerations 76
5.4.1 Slurry Wall Construction 76
5.4.2 Tieback Installation 78
5.4.3 Excavation Methods 80
6.0 Settlement Considerations 82
6.1 Tunneling Induced Initial Settlement 82
6.2 Cross Passages 84
6.3 Cut and Cover Excavations 84

For Reference Only

See disclaimer on cover page.

6.4 Combined Effects of Tunneling and Cut and Cover Excavations 84
6.5 Risk of Building and Utility Damage 85
6.6 Physical Damage Categorization 85
6.7 Settlement Limits and Responsibility for Damages 85
7.0 Instrumentation and Monitoring 90
7.1 Bored Tunnel 90
7.1.1 Photographic Surveys 90
7.1.2 Monitoring 90
7.1.3 Settlement Monitoring Points 90
7.1.4 Ground Movement Monitoring 90
7.1.5 Slurry Wall Instrumentation 90
7.1.6 Ground Water Monitoring 90
7.1.7 Special Structures 90
7.1.8 Monitoring Arrays 90
7.1.9 Instrumentation Zones 91
7.1.10 Tunnel Deformation Monitoring 91
7.2 Structures 91
7.2.1 Photographic Surveys 91
7.2.2 Monitoring 91
7.2.3 Settlement Monitoring Points 91
7.2.4 Ground Movement Monitoring 91
7.2.5 Slurry Wall Instrumentation 91
7.2.6 Ground Water Monitoring 91
7.2.7 Special Structures 92
7.2.8 Monitoring Arrays 92
7.2.9 Instrumentation Zones 92
7.3 Trigger Levels 92
8.0 Spoil Disposal 94
9.0 Limitations 96

REFERENCES

For Reference Only

See disclaimer on cover page.

LIST OF TABLES

Table 2-1 Summary of Tunnel Drives 6
Table 2-2 Summary of Cut and Cover Structures 6
Table 3-2 Porosity Values 14
Table 3-3 Horizontal Permeabilities 14
Table 3-4 Summary of Occurrence of Gravelly Sands and Sandy Gravels in Borings at Tunnel Depth Horizon 15
Table 3-5 Generalized Geotechnical Soil Parameters 16
Table 3-6 Comparison of SVRT to Planned SFPUC Bay Tunnel 20
Table 3-7 Description of Internal Support System 23
Table 4-1 Baseline of Soils Anticipated in Bored Tunnels – Length 27
Table 4-2 Ground Stability Classification at Tunnel Depth 29
Table 4-3 Summary of Alignment Length Anticipated to Encounter Soil with Less than 20% Fines 37
Table 4-4 Potentially Limiting (Upper and Lower Bound) Face Pressures 42
Table 4-5 Definition of Foam Mix Design Terms 46
Table 4-6 Soil Type Classification System for Soil Conditioning Purposes 49
Table 4-7 Classification of Soils for Cross Passage Ground Treatment and Tunneling 54
Table 4-8 Ground Treatment Options 56
Table 4-9 Cross Passage Ground Improvement and Excavation Support Basis of Quantities 58
Table 4-10 Cross Passage Ground Improvement Bid Quantities 58
Table 4-11 Permeation Grouting & Vacuum Wellpoint Ground Treatment Options 59
Table 4-12 Cross Passage Ground Excavation Sequence Bid Quantities 63
Table 5-1 Summary of Minimum Wall Depths and Baseline Maximum Pumping Rates 75
Table 5-2 Summary of Anticipated Soil Units for Slurry Wall Excavation 77
Table 5-3 Summary of Anticipated Soil Units for Tie-back Excavation below 40 ft 78
Table 5-4 Summary of Anticipated Soil Units for Tie-back Excavation above 40 ft 79
Table 5-5 Summary of Anticipated Soil Units for Mass Excavation 80
Table 6-1 Damage Categories for Frame Buildings 87
Table 6-2 Damage Categories for Brick Bearing Wall Buildings 88

For Reference Only

See disclaimer on cover page.

LIST OF FIGURES

Figure 1-1 Project Location 3
Figure 3-1 Interpretive Geologic Profile – Summary Sheet 11
Figure 3-2 Generalized Distribution of Undrained Shear Strength 17
Figure 3-3 Geologic Profile and Soil Abrasivity Test (SAT) Results for SFPUC Bay Tunnel 21
Figure 4-1 Stability Ratio at Tunnel Horizon 28
Figure 4-3 Categories of Potential TBM Clogging for SVRT Cohesive Soil at Tunnel Depth 32
Figure 4-4 Frequency of Average Weighted Fines at Tunnel Face 33
Figure 4-5 Summary Gradation Plot, Stockton and Cinnabar/Lenzen 34
Figure 4-6 Summary Gradation Plot, Stockton and Harding 35
Figure 4-7 Summary Gradation Plot, Stockton and Taylor/Asbury 36
Figure 4-8 Earth Pressure Balance Machine (EPBM) Diagram 39
Figure 4-9 Selection of Target Face Pressure 42
Figure 5-1 Construction and Design Water Levels for West Portal. 70
Figure 5-2 Construction Water Levels for East Portal U Wall. 71
Figure 5-3 Construction Water Levels for East Portal Box Structure. 71
Figure 5-4 Construction Water Levels for Alum Rock Station North of STA 604+00. 72
Figure 5-5 Construction Water Levels for Alum Rock Station South of STA 604+00. 72
Figure 5-6 Construction Water Levels for DSJS and Crossover. 73
Figure 5-7 Construction Water Levels for Diridon/Arena Station. 73
Figure 5-8 Construction Water Levels for Mid-Tunnel Ventilation Structure STS. 74
Figure 5-9 Construction Water Levels for Mid-Tunnel Ventilation Structure FSS. 74
Figure 6-1 Predicting Ground Settlement above Single Bore 83
Figure 6-2 Settlement above Twin Bores by Super-position of Settlement curves 83
Figure 6-3 Locations of Slope Calculations 86

For Reference Only

See disclaimer on cover page.

LIST OF DRAWINGS INCORPORATED IN GBR

D300-TR-C300 - Borehole Location Map
D300-TR-C301 - Interpretive Geological Profile - Study Section 1 - East Portal to Alum Rock
D300-TR-C302 - Interpretive Geological Profile - Study Section 3 - Alum Rock to Downtown
D300-TR-C303 - Interpretive Geological Profile - Study Section 5 - Downtown to Diridon
D300-TR-C304 - Interpretive Geological Profile - Study Section 7 - Diridon to West Portal
D300-TR-C305 - Interpretive Geological Profile - Study Section 2 - Alum Rock Station
D300-TR-C306 - Interpretive Geological Profile - Study Section 4 - Crossover / Downtown Station
D300-TR-C307 - Interpretive Geological Profile - Study Section 6 - Diridon Station
D300-TR-C308 - Cross Passage Geotechnical Plan and Profile Legend
C330-TR-C309 - Interpretive Soil Profile at CP 1
-through-
C330-TR-C341 - Interpretive Soil Profile at CP 33

APPENDICES

Appendix A – Guidelines for Interpretation of Boring and CPT Logs for Soil Layer Profiles
Appendix B – Cal/OSHA Underground Classification
Appendix C – Soil Conditioning Tests at BASF Lab; P0503-D300-TM-DE-070, Rev. A
Appendix D – Description of Preliminary Ground Classification
Appendix E – Description of Preliminary Face Pressure Calculations

For Reference Only

See disclaimer on cover page.

PREFACE

For the 65% Engineering Phase, a single combined Geotechnical Baseline Report (GBR) has been produced which covers all the construction contracts except trackwork. This has allowed a consistent approach to the generic sections of the GBR that are common to all contracts, and interpretation of ground conditions and, in particular, the geotechnical long sections. During Advanced Design, the combined GBR will be split into individual GBRs for each construction contract, as listed below, and the sections of the GBR that are unique to each contract will be developed in detail. At 65% design stage, these sections are in early draft form, and in some cases are incomplete. This is normal, as the development and completion of these sections will follow completion of detailed contract specifications, bid schedules and payment provisions during Advanced Design.

Central Area Guideway construction contract packages at 65% design stage covered by this combined GBR are as follows:

- C321-Alum Rock Station (support walls and excavation)
- C322-Downtown San Jose Station and Crossover (support walls and excavation)
- C323-Diridon / Arena Station (support walls and excavation)
- C330-Bored Tunnels, Cross Passages and Portals
- C380-East Mid-Tunnel Ventilation Structure (FSS)
- C390-West Mid-Tunnel Ventilation Structure (STS)

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

1.0 Introduction

The Santa Clara Valley Transportation Authority (VTA) is constructing the Silicon Valley Rapid Transit (SVRT) Project in San Jose, California. The Project will extend the Bay Area Rapid Transit (BART) system 16.3 miles from Warm Springs in Fremont to San Jose. The Central Area Guideway alignment includes 4.3 miles of twin-bored tunnels, three underground stations, two mid-tunnel ventilation shafts, an underground cross-over structure and two portal structures; see Figure 1-1.

This Geotechnical Baseline Report (GBR) provides a contractual baseline by describing the relationship between ground conditions and design and provides a basis for allocating risk in the Construction Contracts.

This report includes:

- A summary of the geological and geotechnical information obtained for the project.
- Interpretations of anticipated ground and groundwater conditions to be encountered, including geotechnical long sections.
- A summary of how these anticipated conditions have influenced the project design and are expected to impact construction.
- Discussion of other design and construction considerations that will impact construction.
- Baseline quantities for ground and groundwater conditions and physical obstructions.

Interpretation of subsurface information contained in this GBR has included interpolation between borings, extrapolation beyond borings, and assessment of laboratory test data. The actual conditions can be expected to vary from the interpreted conditions. The judgments assume the use of appropriate means, methods, and levels of workmanship. Ultimately, the behavior of the geologic materials present in the surface and subsurface excavations will be influenced by the contractor's means and methods, and levels of workmanship.

In its final form, the GBR will be made contract specific and will form part of the Contract Documents for each contract. This GBR is included in the Contract Documents. The GBR is be read in conjunction with the following Data Reports, which also from part of the Contract Documents for each contract, as appropriate:

- P0503-D300-RPT-GEO-002, Rev. 1; Geotechnical Data Report – Phase 1 35% Engineering Design Investigation
- P0503-D300-RPT-GEO-004, Rev. 1; Geotechnical Data Report – Phase 2 65% Engineering Design Investigations
- P0503-D300-RPT-GEO-005, Rev. 0; Pumping Test Data Report

For Reference Only

See disclaimer on cover page.

- P0503-D300-RPT-GEO-007, Rev. 0; Review of Geotechnical Data from Selected Projects
- P0503-D300-RPT-DE-020, Rev. 1; Hydrogeology Report
- P0503-D300-RPT-DE-051, Rev. A; Cross Passage Ground Treatment – Grout Trial: Data Report

These Geotechnical Data Reports (GDRs) contain factual geotechnical data resulting from the explorations and testing completed for this project, and are referred to collectively as the project GDRs.

This GBR is applicable to Contracts C321, C322, C323, C330, C380 and C390, and supersedes all previously prepared geotechnical design and interpretive reports and memoranda. A separate TBM Geotechnical Basis of Design Report has been prepared for Contracts C301 TBM Procurement and C303 Conveyor Procurement. GBRs prepared for other contracts are considered as reference documents for this contract. Precedence of contract documents is addressed in the contract conditions.

Certain elements of the work are based on requirements that cannot be varied. These include but are not limited to:

- Sequence of the work including beginning the tunnel drives from the West Portal, followed by the East Portal drives
- Number and location of cross passages
- Vertical and horizontal alignment of the portals, running tunnels, cross passages mid-tunnel ventilation shafts, and underground station excavations
- Use of two Owner-procured and novated earth pressure balance tunnel boring machines (EPBMs), including Owner-procured long tunnel conveyors
- Use of a single pass, precast concrete tunnel lining system, also Owner-furnished
- Soil conditioning while operating EPBMs
- Use of slurry walls for support of deep excavations, including internal bracing or tiebacks as specified
- Use of jet grouted blocks at break-in and break-out locations

A number of elements of the project are flexible within defined limits, and afford the Contractor latitude in carrying out the work, subject to approval of the VTA. These flexible elements of the work include, but are not limited to:

- Operating face pressures applied by the EPBM during tunneling.

For Reference Only

See disclaimer on cover page.

- Quantities and types of soil conditioners
- Frequency of cutterhead maintenance
- Cross passage pre-excavation ground improvement
- Design of dewatering systems at open cut excavations
- Method of open cut excavation

Some of the technical concepts, terms, and descriptions in the GBR may not be familiar to bidders. It is highly recommended that bidders engage a California registered geotechnical engineer or engineering geologist who is familiar with all topics of this report, to carefully review and explain this information so that a complete understanding of the information presented in the GBR can be developed prior to submitting a bid.

Certain drawings and figures contained in other documents in the Contract are referenced by the GBR as an aid to bidders in understanding the elements of the work. Such drawings are not reproduced in the GBR, so this GBR should be reviewed in conjunction with the Drawings and Specifications.

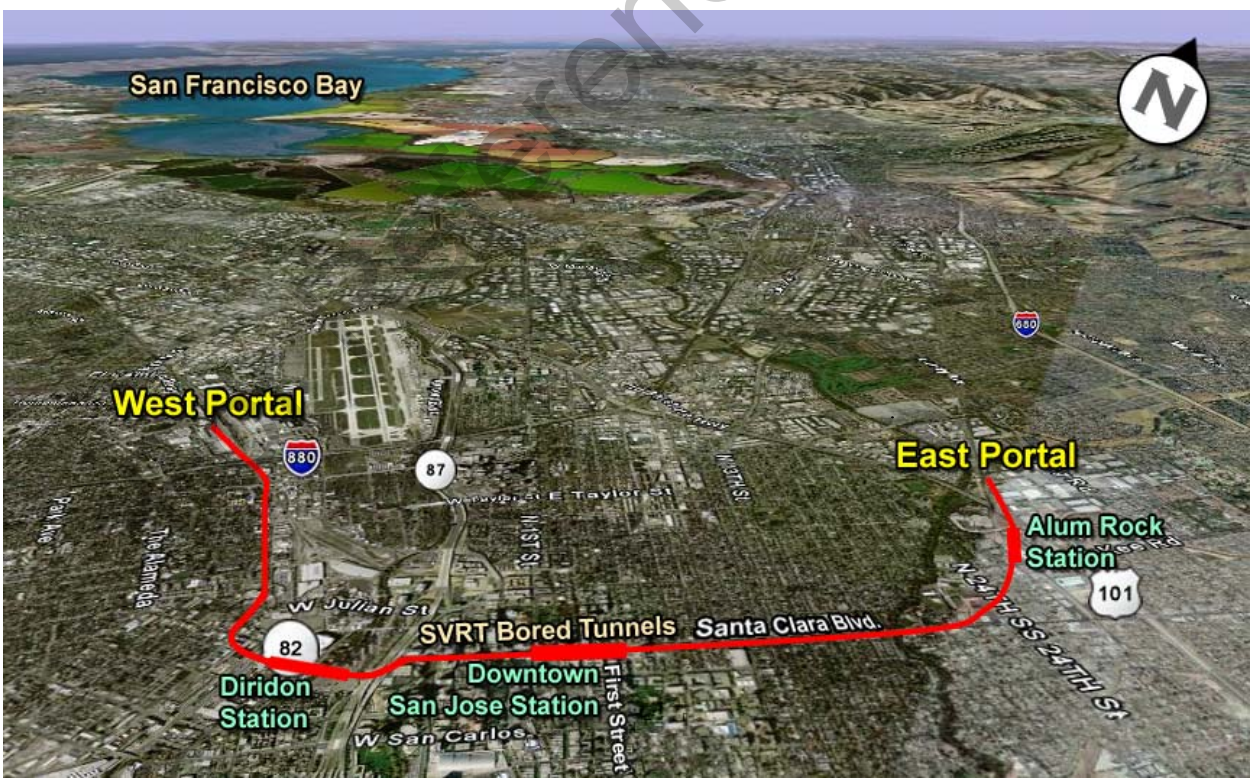


Figure 1-1 Project Location

Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

2.0 Project Description

The alignment of the 16 mile long SVRT project is comprised of three major segments: a Line Segment (Northern Area Guideway and Stations) which will be approximately 9.7 miles of at-grade, elevated and cut-and-cover track from Warm Springs to San Jose, a Tunnel Segment (also referred to herein as the Central Area Guideway and stations) at approximately 5.3 miles long through Downtown San Jose, and a Yard and Shops Segment (Western Area Guideway) which will be approximately 1.1 miles of maintenance facilities, including an at-grade station.

The Central Area Guideway comprises the following construction contracts:

- C330 – Bored Tunnels, Cross Passages and Portals, which includes 4.3 miles of twin-bored tunnel with 33 mined cross passages and 2 portals
- C321 – Alum Rock Station,
  C322 – Downtown San Jose Station and Crossover and
  C323 – Diridon/ Arena Station, each of which include support walls and excavation at each location
- C380 and C390 – East and West Mid-Tunnel Ventilation Structures, respectively, which include support walls, excavation and buildings.

In order to manage risk and accelerate the project construction schedule, the VTA will purchase two approximately 21 foot diameter EPBMs, trailing gear and conveyor systems and provide them through a novation process to the Contract C330 Contractor. The pre-cast concrete segmental tunnel lining system will also be procured by the VTA and supplied to the Contractor.

The EPBMs will be used to install a one-pass lining system consisting of 17 foot-10 inch internal diameter, 10 inch thick pre-cast concrete segmental rings. The tunnels will be driven from the portals toward the Downtown Station as shown on the Contract C330 Drawings. The EPBMs will be skidded through cut-and-cover excavations made in advance of tunneling at the Alum Rock and Diridon stations. They will tunnel through pre-excavated, shored and backfilled structures at two mid-tunnel ventilation shafts. The detailed alignment is shown on the Contract C330 Drawings and a summary is provided in Table 2-1.

The cross-passages will be hand excavated from within the twin bore tunnels. Pre-excavation ground improvement including jet-grouting from the surface or permeation grouting and vacuum well-points from the tunnels is required for the majority of cross passages.

The portals, ventilation shafts and station/ crossover excavations will be supported by slurry walls and braced with a combination of struts and tie-backs. A summary of the cut and cover structure sizes is provided in Table 2-2.

For Reference Only

See disclaimer on cover page.

Table 2-1 Summary of Tunnel Drives

Begin	Walk or Tunnel through	End	Approximate Tangent Alignment Length, miles	Approximate Curved Alignment Length, miles	Max. Grade	Min. Curve Radius	No. of Cross-Passages
West Portal	Tunnel through STS Vent Shaft; Walk through Diridon/ Arena Station	Downtown San Jose Station	S1: 0.9 S2: 0.9	S1: 1.3 S2: 1.3	-2.2 % +3.8 %	1000 ft	17
East Portal	Tunnel through FSS Vent Shaft; Walk through Alum Rock Station	Cross-over Structure	S1: 0.9 S2: 0.9	S1: 1.2 S2: 1.2	-3.1% +0.4%	1000 ft	16

Note: In this table, grade is shown with regard to tunnel drive direction with downward as a negative grade measurement and upward as a positive grade measurement.

Table 2-2 Summary of Cut and Cover Structures

Structure	Length Along Alignment, feet	Width, feet	Depth to Mud Slab, feet	Wall Depth Range, feet	Temporary Bracing
West Portal	1187	52 – 67	5 - 37	8-99	Tie-backs & Struts at Headwall
West Mid-Tunnel Vent Shaft [STS]	42	187 – 208	86 (Head-house 20 -38)	200	Struts (5 rows max)
Diridon/ Arena Station	887	60 – 130	62 – 64	142 – 149	Tie-backs
Downtown San Jose Station and Crossover	1380	66 – 100	60 – 62	141 – 149	S1 705+37 to S1 706+01 are tie-backs; struts elsewhere
East Mid-Tunnel Vent Shaft [FSS]	43	222	71 (Head house 20-30)	195	Struts (5 rows max)
Alum Rock Station	886	66 – 132	57 – 58	148 – 168	Tie-backs
East Portal	684	58 – 66	34	23 – 76	Tie-backs & Struts at Headwall

Notes:

1. Approximate dimensions are provided for summary and quick reference. Refer to Contract Drawings for precise dimensions. Station excavations include enlargements for station entrances that are not included in the overall dimensions summarized here.
2. The Wall Depth Ranges are specified as minimum depths from the Contract Drawings. Actual conditions may require increasing these depths as described in the Contract Drawings and Specifications

For Reference Only

See disclaimer on cover page.

3.0 Site Conditions

3.1 Sources of Geotechnical Information

Multiple phases of site investigation have been completed in the project vicinity. For complete information on the available geotechnical information, refer to the GDRs.

Several geotechnical reports and design memoranda have been prepared that evaluate the geotechnical site data and make interpretations and recommendations pertinent to design and construction of the SVRT project. These documents are available for reference upon request, but interpretations in this GBR take precedence.

3.2 Regional Geology

The Santa Clara Valley is an alluvium-filled groundwater basin that consists of gently sloping topography formed by coalescing alluvial fans, with natural levees along the principal stream channels that drain generally northward to San Francisco Bay. The ground surface ranges from Elevation 70 to 95 feet (NAVD88) along the alignment. The alluvial deposits are over 1,000 feet thick and are underlain by bedrock that outcrops in the mountain ranges to the northeast and southwest of the tunnel alignment.

The origins of the alluvium are described by the US Geological Survey (USGS) (Clark 1924) as being:

"…built up by deposits from streams that are very irregular in their flow and from time to time change their courses. For this reason the nature of the deposits change rapidly…"

The great and rapid fluctuations that occurred in streams flowing while basin infill deposition was occurring, which changed the flow from almost nothing to floods of several thousand cubic feet per second, resulted in:

"*…coarse material being borne much further downstream at some times than at others."*

"Therefore the coarse material borne far downstream toward the bay in times of flood will be buried beneath fine sediment later, when the streams are at only a moderate stage."

Shifting the stream channels produced lenses of gravel, and the later burial of old channels produced stringers and pipes of gravel. These became encased in finer sediments and now exist above one another and cross one another at various angles.

3.3 Regional Groundwater Regime

The hydro-geologic units recognized in the Santa Clara Valley are summarized below:

For Reference Only

See disclaimer on cover page.

- Surficial Aquifer: Holocene alluvial deposits of silt and silty sand. Absent in some locations. Thickness is typically less than 15 feet. At some locations, the water table will be below these sediments.
- Confining Layer: Holocene alluvial deposits composed of clays and silts, with channels of sand. The sand channels are most common near the Guadalupe River and Las Gatos Creek. The general thickness varies from 50 feet to 80 feet along the Central Area Guideway.
- Upper Aquifer: Holocene and possibly late Pleistocene alluvial deposits composed of mixtures of silty sand, sand, gravelly sand, and sandy gravel. Includes intersecting and coalescing channels of varying thickness and differing hydraulic conductivity. The top of this unit generally varies from 50 to 80 feet below ground surface. Thickness generally ranges from 10 to 40 feet.
- Major Aquitard: Pleistocene alluvial deposits composed primarily of clays and silts, but can include deposits of sand and silty sand. The depth to the top of this unit appears to range from about 80 feet to 150 feet along the Central Area Guideway, with depths at the three stations ranging from about 110 feet to 150 feet. The thickness may be approximately 100 feet, but this is uncertain because nearly all geotechnical borings are limited to 200 feet or less in depth.
- Lower Aquifer: Pleistocene and Pliocene sediments composed of sand and gravel zones, with intervening clay and silt layers. The top of this unit may be about 200 feet to 250 feet below ground surface, and the thickness may be about 800 feet or more. This is the primary zone of ground-water supply in Santa Clara Valley.
- Basement: The underlying non-water-bearing stratigraphy consists of portions of the Plio-Pliocene Age Santa Clara Formation and the Mesozoic age Franciscan Complex.

A notable feature of the groundwater basin is that the lower aquifer was over-pumped for decades, which induced land subsidence and salt-water intrusion. Regional initiatives to counteract over-pumping, including groundwater recharge, have curtailed land subsidence since 1968 and the advance of saltwater intrusion, primarily since 1985. In most recent years, pressures in the lower aquifer have recovered to the extent that wells in the basin interior again became artesian (see below) on an intermittent basis.

3.4 Regional Seismicity

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. Additional information is provided in the GDRs.

3.5 Site Geology

The Unified Soil Classification System (USCS) as defined by ASTM D 2487 was used to describe the soils encountered in borings and anticipated along the alignment. Soil samples from borings were visually classified. Some intervals of the borings were confirmed by classification testing including grain size distribution and Atterberg Limit testing. For engineering analyses, excavations and supporting systems, the soils at the

For Reference Only

See disclaimer on cover page.

site were subdivided into soil Units 1 and 2, which are defined on Figure 3-1. An interpretive geotechnical and geologic profile has been developed based on the information in the GDRs and is provided on GBR Drawings C301 through C341 of this report. Boring locations are shown on GBR Drawing C300. Information is summarized in Figure 3-1. The procedure adopted for interpreting soil layering on the interpretive profiles is provided in Appendix A.

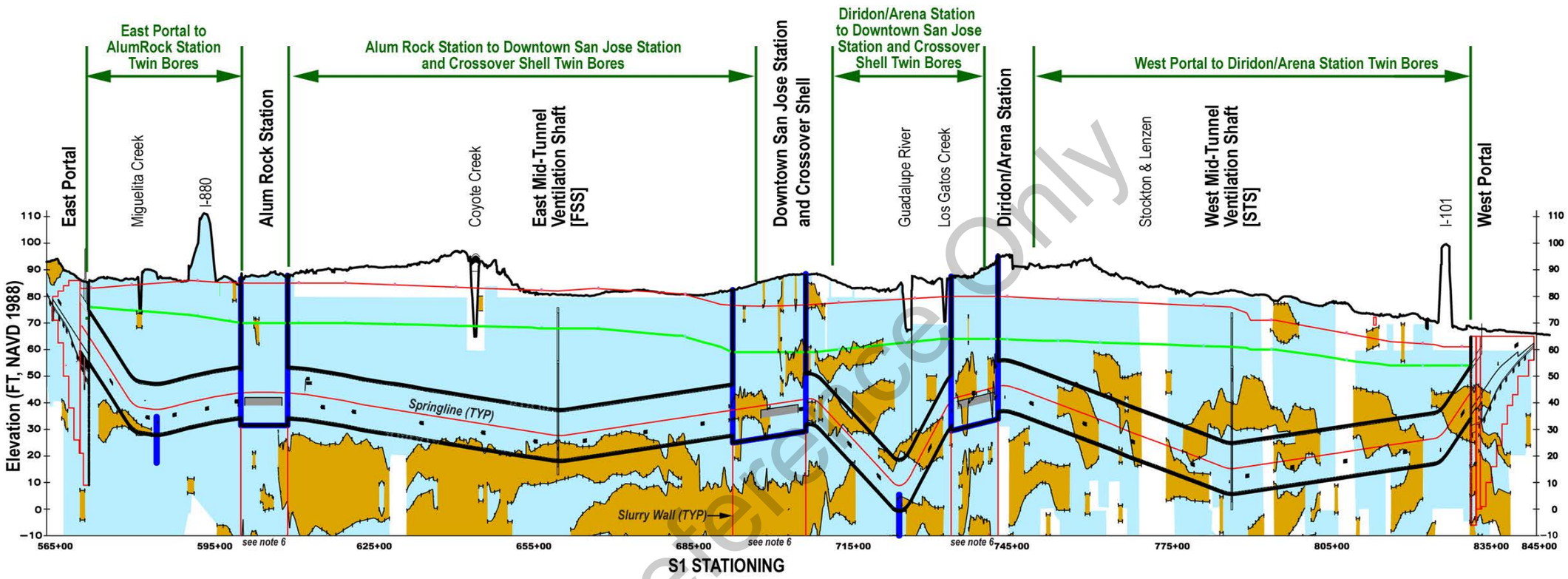
For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.



LIMITATIONS:

THE INFORMATION ON THIS PROFILE IS SIMPLIFIED AND SHOULD NOT BE RELIED UPON FOR BIDDING OR CONSTRUCTION. REFER TO DRAWINGS D300-S-TR-C300 THROUGH D300-S-TR-C307 FOR MORE RELIABLE INFORMATION.

SVRT GENERALIZED SOIL CLASSIFICATION

UNIT	PRIMARY STRATUM DESCRIPTION	INCLUDED IN THIS LAYER (SEE NOTE 1)
1	SILTS, CLAYS AND ORGANIC SOILS. SOIL BEHAVIOR TYPE (SBT) ZONES 1 THROUGH 4, AND 9, AS CLASSIFIED BY CPTS USING ROBERTSON, 1990.	• MAN-MADE FILL CONSISTING OF ASPHALT CONCRETE AND AGGREGATE BASE ENCOUNTERED MOST OFTEN WITHIN FIVE FEET BELOW GROUND SURFACE. • THIN LAYERS OF COARSE-GRAINED SOIL AS DESCRIBED IN NOTE 2
2	SANDS AND GRAVELS, AND SBT ZONES 6 THROUGH 8, CLASSIFIED BY CPTS USING ROBERTSON, 1990.	• COBBLES • THIN LAYERS OF FINE-GRAINED SOIL AS DESCRIBED IN NOTE 2

DESIGN WATER LEVELS:

MAXIMUM PIEZOMETRIC LEVEL FOR TBM TUNNEL CONSTRUCTION.

MINIMUM PIEZOMETRIC LEVEL FOR TBM TUNNEL CONSTRUCTION.

NOTES:

1. THE SBT ZONE 5 - 'SAND MIXTURES - SILTY SAND AND SANDY SILT' CONSISTS OF BOTH FINE-GRAINED AND COARSE-GRAINED SOILS. THEREFORE, IT MAY BE INCLUDED IN EITHER SOIL UNIT.
2. LAYER THICKNESS: DUE TO THE VARIABILITY IN SOIL PROFILE ON THE PROPOSED TUNNEL ALIGNMENT, FIVE FEET WAS SELECTED AS A THRESHOLD THICKNESS FOR THIN SOIL LAYERS.
3. MEASURED WATER LEVELS SHOWN ON THESE SUBSURFACE PROFILES REPRESENT THE PIEZOMETRIC LEVELS EXPECTED AT THE DEPTH OF THE TBM TUNNEL. THE WATER LEVELS SHOWN ARE NOT APPLICABLE FOR THE DESIGN OF OTHER SVRT PROJECT EXCAVATIONS AND STRUCTURES.
4. ALIGNMENT SHOWN ON PROFILE CORRESPONDS TO APRIL 2008 QUARTERLY UPDATE ALIGNMENT SUBMITTAL.
5. HYDROGEOLOGICAL INFORMATION FROM HMMB HYDROGEOLOGY REPORT P0503-D300-RPT-DE-020 MAY 2008.
6. BOTTOM OF SLURRY WALLS TRUNCATED. REFER TO GBR DRAWINGS D300-S-TR-C305 TO C307 FOR FURTHER DETAILS.

THIS FIGURE IS PRELIMINARY AND WILL BE REVISED BASED ON ADDITIONAL GEOTECHNICAL INVESTIGATION RESULTS AND REVISED MATERIAL DESCRIPTIONS.

Figure 3-1 Interpretive Geologic Profile – Summary Sheet

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

3.6 Hydrologic Setting

The 100 year flood is used as a basis of design for project facilities sensitive to flooding. Refer to the Surface Water Hydrology Report (HMM/Bechtel, 2004) for additional information. A 100 year flood would inundate all project areas with stormwater depths up to an estimated one foot above existing grade.

3.7 Anticipated Hydrogeological Conditions

The expected seasonal fluctuation in hydrostatic pressures to be encountered by the work is between 4 and 18 feet depending on location along the alignment and is represented by the difference between anticipated maximum and minimum piezometric levels during construction, as shown on GBR Drawings C301 to C304. The low point along the bored tunnel alignment is beneath the Guadalupe River where the axis level is about Elevation +9 feet. The maximum hydrostatic head above axis during construction is about 70 feet at the low point.

3.7.1 Artesian Pressures

Groundwater pressures at construction depth will be influenced by the hydrostatic pressure in both the surficial aquifer which is unconfined, and the upper aquifer which is confined. In some instances, confined groundwater pressures are artesian, meaning pressure heads exceed ground surface level periodically, during seasonal fluctuations.

3.7.2 Flooding

Some of the local drainages above the tunnel are prone to flooding during winter storms. Flooding will affect hydrostatic pressure in the surficial aquifer, but the flood prone areas coincide with the deeper portions of the alignment where pressure at tunnel depth is governed by pressures in the upper aquifer. Seasonal flooding is not expected to influence the hydrostatic pressures exerted on the TBMs.

3.7.3 Dissolved Gases

Minor amounts of nitrogen, carbon dioxide, argon and methane have been detected in samples of groundwater from monitoring wells after gas was released from investigation holes during the site investigation program, as discussed in the Hydrogeology Report (HMM/Bechtel 2008), and GDRs. Based on the encountered concentrations and the limited amount of groundwater expected to enter the tunnel in the spoil and through leakage, the effects of groundwater degassing on the tunnel atmosphere during construction is expected to be manageable with ventilation. Gas sensors are required on the TBM with an interlock to automatically shut down if the tunnel atmosphere approaches a hazardous condition.

For Reference Only

See disclaimer on cover page.

3.8 Soil Porosity and Permeability

Porosity varies significantly in soil depending on particle size distribution and degree of compaction or consolidation. Porosity can be estimated using data available in the GDRs. The estimated range of soil porosity is summarized according to USCS classification in Table 3-2.

Table 3-2 Porosity Values

Soil Type	Porosity, n	
	Min	Max
ML	0.29	0.52
SM	0.23	0.47
SP	0.27	0.33
SW	0.29	0.42
GW	0.39	0.50
SW-GW	0.34	0.46
SW-SM	0.26	0.45
SP-SM	0.25	0.41

The horizontal permeability of soils was assessed in a variety of different ways, including pumping tests, slug tests, laboratory tests, and correlations with particle size distribution in the Hydrogeology Report (HMM/Bechtel, 2008a). Generalized values for horizontal permeability for some soil types are provided in Table 3-3. Horizontal permeabilities are typically higher than vertical permeabilities. The Hydrogeology Report (HMM/Bechtel, 2008a) recommends K_h:K_v as 1:1 for sands and silty sands, and 2:1 for clays. Both vertical and horizontal permeabilities are discussed in greater detail in the Hydrogeology Report (HMM/Bechtel, 2008a).

Table 3-3 Horizontal Permeabilities

Material	Permeability	
	Min	Max
Gravelly sands and sandy gravels	4×10^{-2} cm/s	5.1×10^{-1} cm/s
Silty sands	3.2×10^{-5} cm/s	1.5×10^{-1} cm/s
Clays	2×10^{-6} cm/s	3.4×10^{-6} cm/s

Since the permeability of gravelly sands and sandy-gravels are of concern for face stability, the occurrence of this material in the site investigation borings is summarized in Table 3-4.

For Reference Only

See disclaimer on cover page.

Table 3-4 Summary of Occurrence of Gravelly Sands and Sandy Gravels in Borings at Tunnel Depth Horizon

Property of Gravelly Sands and Sandy Gravels	Parameter	Comments
Average Bed Thickness	5 ft	30 out of 56 boreholes had gravel present within the tunnel face.
Maximum Bed Thickness	20 ft	Occurs at tunnel face in BH-39, Stockton and Cinnabar, See Drawing C303.

3.9 Engineering Properties of Soil

The Contractor's design responsibilities include the following:

- Construction dewatering system
- Cross passage initial support
- Initial support grouting plan
- Tiebacks
  - a. Bond length
  - b. Hole diameter
  - c. Secondary pressure grouting cycles
- TBM launch frame
- Temporary works

The use of practical geotechnical parameters from the most relevant borings and CPTs near the structure is encouraged. For generalized geotechnical design parameters, use the recommended values in Table 3-5 and Figure 3-2.

For Reference Only

See disclaimer on cover page.

Table 3-5 Generalized Geotechnical Soil Parameters

Index Property	Remarks
Over-Consolidation Ratio (OCR)	Depth < 40ft: Assume ~ 3 Depth > 40ft: Assume ~ 2.3
Plasticity Index (PI)	Use location specific PI from Atterberg tests, or assume the following PI: Fat Clay (CH) ~ 30 Lean Clays (CL) ~ 10 to 20 Silts (ML) ~5 Sands ~ 0
Unit Weight (ρg)	Depth < ~ 35ft: Assume ~ 122 pcf Depth > ~ 35ft: Assume ~ 126 pcf
Moisture Content (wc)	Depth < ~ 35ft: Assume ~ 27.4% Depth > ~ 35ft: Assume ~ 23.7%
Average Porosity (n)	Depth < ~ 35ft: Assume ~ 0.421 Depth > ~ 35ft: Assume ~ 0.382
Undrained Poisson's Ratio (υ)	Clays, Sands and Gravels ~ 0.49
Drained Poisson's Ratio (υ)	Sands and Gravels ~ 0.3
Concrete Adhesion and Friction (Slurry Walls)	Adhesion of clays to concrete: 900 psf Friction coefficient between concrete and granular soils = 0.4 using 2/3 of an effective friction angle ϕ' of 33 ^{o}
Effective Stress Friction Angle (ϕ')	Clays (OC, PI <30) ~ 32° Clays (PI ≥30) ~ 28° Silty Sand ~ 33° Dense Sand and Gravel ~ 37°
Critical State Friction Angle (ϕ'_{crit})	Silty Sand ~ 30° Dense Sand and Gravel ~ 30°
Dilation Angle (ψ)	Silty Sand ~ 2° Dense Sand and Gravel ~ 6°
Specific Gravity (Gs)	2.65
Lateral At-Rest Pressure Coefficient (Ko)	Depth < ~20ft: Assume ~ 1.0 Depth > ~20ft: Assume ~ 0.6 for design of tunnel lining Assume ~ 0.7 for design of stations
Unit Weight (ρg)	Depth < ~ 35ft: Assume ~ 122 pcf Depth > ~ 35ft: Assume ~ 126 pcf

For Reference Only

See disclaimer on cover page.

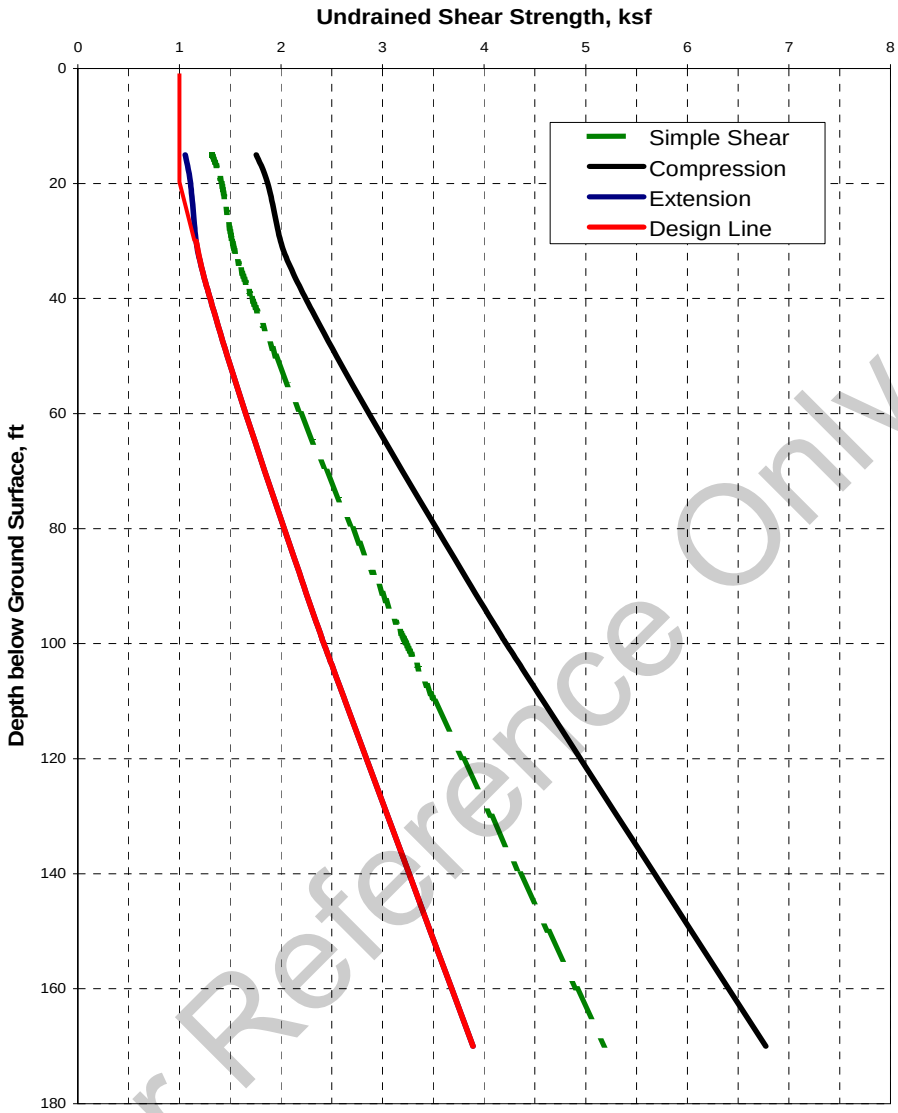


Figure 3-2 Generalized Distribution of Undrained Shear Strength

For Reference Only

3.10 Contaminated Soil and Groundwater

The SVRT project will involve construction within an urban area where soil and groundwater contamination are concerns due to past land uses and undocumented releases. Types of contaminants found in soil and/or groundwater within a quarter mile of the alignment include: gasoline, diesel, benzene, MTBE, BTEX, solvents including chlorinated solvent, Stoddard Solvent from a dry cleaner, PCBs, insecticides, lead, halogenated volatile organic compounds, heater fuel, waste oil, TCA, DCA, VC, Metals, Volatile Organic Compounds, PAHs, fuels, and mineral oil. These have been found at shallow depth for the most part, and none of these contaminants have been detected at tunnel depth, though testing to date has been very limited. Several of these contaminants

See disclaimer on cover page.

are considered Dense, Non-Aqueous Phase Liquids (DNAPLs) which can pass downward through groundwater to tunnel depth. For additional information, refer to the GDRs.

Encountering any of these hazardous materials must be addressed in the Contractor's safety plan. If any of these materials are encountered at hazardous levels, it will be an unforeseen physical condition.

3.11 Anticipated Performance of Ground Improvement

Ground improvement by permeation grouting is intended to reduce the permeability of granular soils prior to excavation for cross-passages. In combination with depressurization using vacuum wellpoints, the ground improvement is designed to modify soils from a flowing or running condition to a slow raveling or firm condition as described in the Tunneling Classification system.

A grouting trial was performed in Newhall Yard, during March through May 2008, to confirm the appropriateness of including permeation grouting in the design. Data collected during this trial is summarized in the GDRs and interpretations of the data is included in the Grout Trial Interpretive Report (HMM/Bechtel, 2008b).

3.11.1 Results from the Permeation Grout Trial

The grout trial confirmed that granular soils, representative of those along the alignment, can be permeated with grouts, with the exceptions of silty or clayey sands and gravels (SM, SC, GM, GC) and sandy silt (ML).

The following are grout trial observations of the difficulties and limitations in using permeation grouting:

- Maintaining borehole stability during the drilling process
- Preventing contamination of the formation by the annular grout used for grouting the sleeve port grout pipes
- The high level of effort that will be required to develop effective permeation of soils from within the bored tunnels
- The apparent reduction of in-situ permeabilities; premature refusal of permeation grouting on approximately 35% of injection ports; and poor recovery of gravelly soils
- Grout communication along highly permeable formations which would reduce the grouting effect on the annulus around the cross-passages
- The two foot injection port spacing was not adequate as noted by the limited recovery of grouted soils in verification holes

For Reference Only

See disclaimer on cover page.

- Injection of both ultrafine cement and sodium silicate grouts in the same hole resulted in a less than optimum permeation by the sodium silicate grouts

3.11.2 Conclusions from the Permeation Grout Trial

The level of effort required to reduce the permeability of substantial bodies of gravels and some sands with permeation grouting from within the tunnels, will be high, requiring multiple holes and several stages and phases of grouting to build the necessary confinement. This will include adoption of specialist techniques, including pre-drilling with downstage grouting, for the installation of sleeve port grout pipes. Therefore, permeation will require practitioners with state of the art equipment and techniques.

The indicative and indirect assessment of Apparent Lugeon Values, calculated during grouting of the injection ports, suggests that coefficients of permeability (k) in some soils were reduced by up to two orders of magnitude by the weak cement grout, used as drilling fluid and annular grout, entering the formation. However, windows existed in the block on completion of grouting resulting in destabilizing movement of water to the verification holes.

To maintain borehole stability, drilling should be performed by stabilizing the borehole walls using a casing method or through the use of a drilling mud such as bentonite slurry.

The grout trial enabled assessment of various grout mixes to be made. Effective ultrafine cement grout mixes were developed using a bleed reducing welan gum. Grout mixes need to be accelerated and engineered to build confinement.

The trial confirms that grout travel will be non-uniform. The trial also confirms the need for a conservative design of grout holes, which does not include optimistic assessment of grout travel. Sleeve port spacing should be one foot to compensate for vertical and lateral variability of soils and to optimize grout sequencing and staging. This supports the design models indicating that installation and permeation grouting would be required semi-concurrently from both running tunnels, at any cross passage where permeation grouting is employed.

3.12 Related Tunnel Construction Experience

The information contained herein comes primarily from undocumented discussion with personnel directly involved with design and construction of the projects mentioned. References are cited where applicable.

3.12.1 Utility Tunneling

Utility tunneling in the San Jose area has generally been with smaller diameter tunneling shields and at shallower depths (10 to 40 feet below ground surface).

For Reference Only

See disclaimer on cover page.

Past experiences have shown a high presence of lean clays and wet sands. Potentially problematic soils encountered in past projects also include clean sands, sticky clays, and gravel layers. Flowing sands were encountered on the San Jose Interceptor Sewer crossing beneath the Hetch-Hetchy water pipelines.

For pipe-jacking tunnel projects, test pits excavated to shallower depths of 10 to 20 feet encountered debris, timber, and boulders.

3.12.2 San Francisco Public Utilities Commission (SFPUC) Bay Tunnel

The SFPUC Bay Tunnel is a water supply tunnel extending from Menlo Park to Newark in the San Francisco Bay Area. Tunneling will progress from the Ravenswood Shaft in Menlo Park east to the Newark Shaft, and the tunnel will serve to supply water to the peninsula and San Francisco. The project alignment is located approximately fifteen miles north of the SVRT alignment. Construction is scheduled to begin some time after 2009. A brief comparison of the Bay Tunnel to the SVRT project is provided in Table 3-6 .

Table 3-6 Comparison of SVRT to Planned SFPUC Bay Tunnel

	Bay Tunnel	SVRT
Length, feet (ft)	26,192 – single bore	22,800 – twin bores
Excavated Diameter, ft	14-16	22
Invert Elevation, ft (ground cover, ft bgs*)	El. -103.5 (110 ft bgs) to El. -70.7 (80 ft bgs)	El. 0 (85 ft bgs) to El 55 (34 ft bgs)

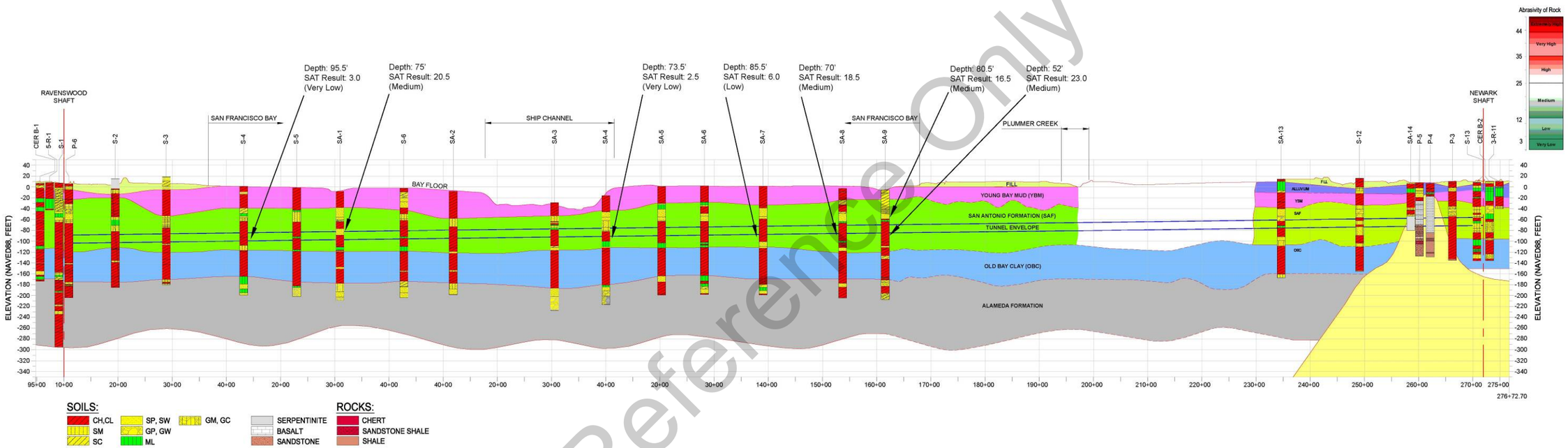
*below ground surface (bgs)

The majority of the tunneling of the Bay Tunnel will be through the San Antonio Formation as shown in Figure 3-3. This formation is approximately 70 to 90 feet thick and was formed from a sequence of terrestrial alluvial fan deposits and lies beneath the Young Bay Mud. The gravels, sands, silts, and clays of the alluvial fan were deposited from meandering streams and channels and filled by layers of fine-grained silt and clay deposits from flood plain sediments. Sources of the sediment are likely the hills of the south bay area, but mineralogy of these sediments is unknown.

Soil Abrasivity Test (SAT) results of samples taken along the alignment indicate the soil abrasivity at the Bay Tunnel, summarized in Figure 3-3, indicate most clays and silts are rated “very low” to “low”, while sandy soils are rated “medium” (Jacobs, 2007). No SAT results are available for the gravels. The soil abrasivity on the SVRT Project is expected to be similar to the Bay Tunnel.

For Reference Only

See disclaimer on cover page.



Note: 1. All borings within 500 feet of alignment.
2. The Soil Abrasion Test (SAT) is conducted according to the standard procedure for AVS abrasion test originally developed for rock as detailed in Nilsen's "Abrasivity of Soils in TBM tunneling" (2006).

Source: Jacobs Associates for SFPUC. Bay Division Pipelines Reliability Upgrade (CUW36801) – Bay Tunnel Project – Draft Geotechnical baseline Report. November 30, 2007.

Figure 3-3 Geologic Profile and Soil Abrasivity Test (SAT) Results for SFPUC Bay Tunnel

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

3.12.3 Related Open Cut Excavations

There have been numerous open cut excavations in the South Bay Area; many in the downtown San Jose area, ranging from generally 20 feet to the deepest extending 40 feet below ground surface. Excavation walls have successfully included the use of both slurry walls and Deep Soil Mix (DSM) walls. Excavation supports have also successfully included the use of both internal bracing and soil anchor tiebacks.

Two areas of concern for excavations are high water table and settlement.

3.12.3.1 Vasona Line Light Rail Transit Tunnel

A recent past project in the vicinity of the SVRT alignment was the cut-and-cover Vasona Line Light Rail Transit tunnel (Doig, 2004). The tunnel is located near the Diridon Station in San Jose (also the location of SVRT's proposed Diridon/Arena Station) and is approximately 900 feet long and 22 feet deep. The project's ground conditions included fine-grained soils, coarse-grained sand, and fine gravels to a depth of 55 feet, and clay below that level. The groundwater level was found to be at a depth of 17 feet.

The excavation support was 2.5 foot diameter soil-mix columns placed 2 feet on center with W24x82 reinforcing extending 22 feet below bottom of the excavation. The support included two levels of struts to resist approximately 17 psi of pressure on the walls. The two-level support system information is detailed in Table 3-7 .

Table 3-7 Description of Internal Support System

	Waler Size	Strut	
		Size	Preloading
Surface Level	W18x76	18 in. diameter, ¼ in. thickness	67 lbs
Mid-depth Level	W18x119		135 lbs

Two issues noted during construction were the alignment of the soil-mix wall columns and the quality of the waterproofing. The contractor initially had difficulty with the alignment of the columns, which encroached on the permanent structure clearance. This was addressed by moving the walls out three inches. Another problem was the waterproofing with Paraseal. It was not determined whether the Paraseal leaked because of deficiencies in the system, problems with installation, or construction effects. Later on, partially successful chemical grout injection program was implemented during the spring of 2003.

For Reference Only

See disclaimer on cover page.

3.12.4 Tieback Installation

Tiebacks have been installed in downtown San Jose with zones of flowing sands. While depths of projects have generally been to 20 feet depths, they have also been used on deeper excavations such as the Knight Ridder Building (located at 50 W San Fernando St.), at depths of ___ feet. Inclinometers indicated horizontal ground movement of approximately 0.16% - 0.23% of excavation depth.

Author's Note: This section to be expanded with specific examples.

3.12.5 Other Relevant Experience

3.12.5.1 Dewatering

Four major excavations in San Jose are described below. Other major excavations requiring dewatering have been completed in San Jose, but details were not available.

San Jose City Hall. The excavation for the San Jose City Hall, near the Downtown San Jose Station, was approximately 650 feet by 340 feet in length and width, and extended to 66 feet elevation (NAVD88). A deep dewatering system consisting of 34 wells was installed in the Upper Aquifer at a perimeter spacing of about 50 feet. In December 2002, the pumping rate ranged between 1,700 and 2,500 gpm, and drawdown was approximately 6 to 8 feet (URS 2002). [Based on well spacing, drawdown, and well discharge, the hydraulic conductivity was back calculated by HMM/Bechtel utilizing techniques presented in the Naval Facilities (NAVFAC) Manual P-418 (U.S. Departments of the Army, the Air Force, and the Navy 1983) and is estimated as 0.2 cm/s for the Upper Aquifer.]

San Jose International Airport. Two pumping tests were conducted at the San Jose International Airport for expansion of the concourse buildings (URS 2004). One of the pumping tests was conducted in a sand and gravel layer at about 50 to 70 feet depth (possibly corresponding to the Upper Aquifer), and the other test was conducted in a sand and gravel later at about 25 to 40 feet depth. For the well completed in the lower layer, the maximum pumping rate was 175 gpm while the well completed in the shallow layer had a maximum pumping rate of 26 gpm. Reported estimates of hydraulic conductivity from the two pumping tests are on the order of 0.2 cm/s.

333 W. San Carlos Street (360 Park Avenue), Riverpark Development (SCVWD Site #17-D07s). Shallow dewatering wells and sump for a garage excavation at an apartment building extracted about 10 gpm in 1985 and 1986. After a solvent plume was discovered, a pumping test at 250 gpm was conducted in December 1986 using well E-1 (screened 58 to 98 feet depth), probably screened in the Upper Aquifer. Drawdown at

For Reference Only

See disclaimer on cover page.

monitoring well B-1, located 77 feet from E-1, indicated a drawdown of about 1 foot after 24 hours. The average flow rate of E-1 during remediation was 330 gpm (Beta Associates 1998).

211 W. Santa Clara Street, San Jose Mercury News (SCCDEH Site #08-M03f). Twenty-two temporary dewatering wells were planned for an excavation to 30 feet depth at the property between N. Almaden Avenue and Notre Dame Avenue. The dewatering subcontractor expected flow rates of up to 440 gpm in the 1st week, decreasing to 220 to 330 gpm in the 2nd week, and then steady at about 66-100 gpm (McLaren Hart 1999).

3.12.5.2 SR-87 Widening Project

In the vicinity of the SVRT alignment is the SR-87 High-Occupancy Vehicle (HOV) aerial structure near the Guadalupe River. Construction of the aerial structure required six 100 plus feet deep Cast in Drilled Hole (CIDH) piles.

Construction of these CIDH piles for the overhead structure proved to be highly problematic. Artesian water pressures in the sand and gravel layers of soil that were not balanced with bentonite slurry caused instability. These artesian pressures are anticipated at the Guadalupe River and are reflected on the piezometric surfaces shown on the interpretive profiles. Seasonal artesian pressures may also be anticipated in the granular soils in the vicinity of Coyote Creek.

At one pile, which had been excavated to approximately 100 feet bgs, artesian pressures caused water to flow out onto the ground. The contractor poured a 6 foot concrete plug at the bottom of the hole but this proved to be unsuccessful in stopping water flow. Ultimately, a welded casing was added so that 10 feet of casing protruded above the channel invert to stabilize the rising water. The amount of ground loss sustained at this pile is unknown.

At three other piles, the contractor also experienced cave-ins at sand and gravel layers. These occurred even after excavation design depth had been reached and the steel reinforcing cage placed. One cave-in caused a three foot diameter, six foot deep sinkhole on the sidewalk of Santa Clara Street. To improve the ground conditions to meet the necessary pile capacity, the contractor had to use compaction grouting around the external perimeter of the casing.

Author's Note: Additional details regarding a deep pumping station installed by VTA near the I-880/Coleman Ave. interchange and the Guadalupe River microtunnel.

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

4.0 Tunnels

4.1 General

For the running tunnels and cross passages, this section describes:

- the anticipated conditions during tunneling;
- how the anticipated ground conditions have influenced the design; and
- how they are expected to influence construction.

For the bored tunnels, the percentage of each soil unit that is anticipated to be encountered along the alignments is shown in Table 4-1. The percentages in this table were estimated based on the tunnel length excluding the length of stations, the crossover, and areas where there was insufficient geotechnical data to interpret soil type between borings.

Table 4-1 Baseline of Soils Anticipated in Bored Tunnels – Length

Soil Unit	Tunnel Segment				Total
	East Portal to Alum Rock Station	Alum Rock Station to Crossover Shell	Downtown San Jose Station to Diridon/Arena Station	Diridon/Arena Station to West Portal	
	Percent of Alignment Length[1]				
1	80%	48%	16%	36%	43%
Mixed 1 and 2, ≥ 7 ft thickness of 1[2]	20%	52%	70%	42%	46%
Mixed 1 and 2, < 7 ft thickness of 1[2]	0%	0%	15%	17%	8%
2	0%	0%	0%	6%	2%

Note:

1. The breakdown of percentages on tunnel length basis is illustrated on GBR Drawings C301 to C304 for the S1 alignment. These percentages are also considered representative of the S2 alignment.
2. The mixed ground is subdivided to facilitate assessment of clogging potential discussed later. Either single or multiple layers may make up the 7' thickness.

These percentages were estimated based on the bored tunnel length, excluding the length of stations, the crossover, and areas where there was insufficient geotechnical data to interpret soil type between borings.

4.2 Anticipated Ground Conditions

An assessment of anticipated ground behavior in response to tunneling is presented in this section. Assessments are provided for overall ground stability, EPB face pressures,

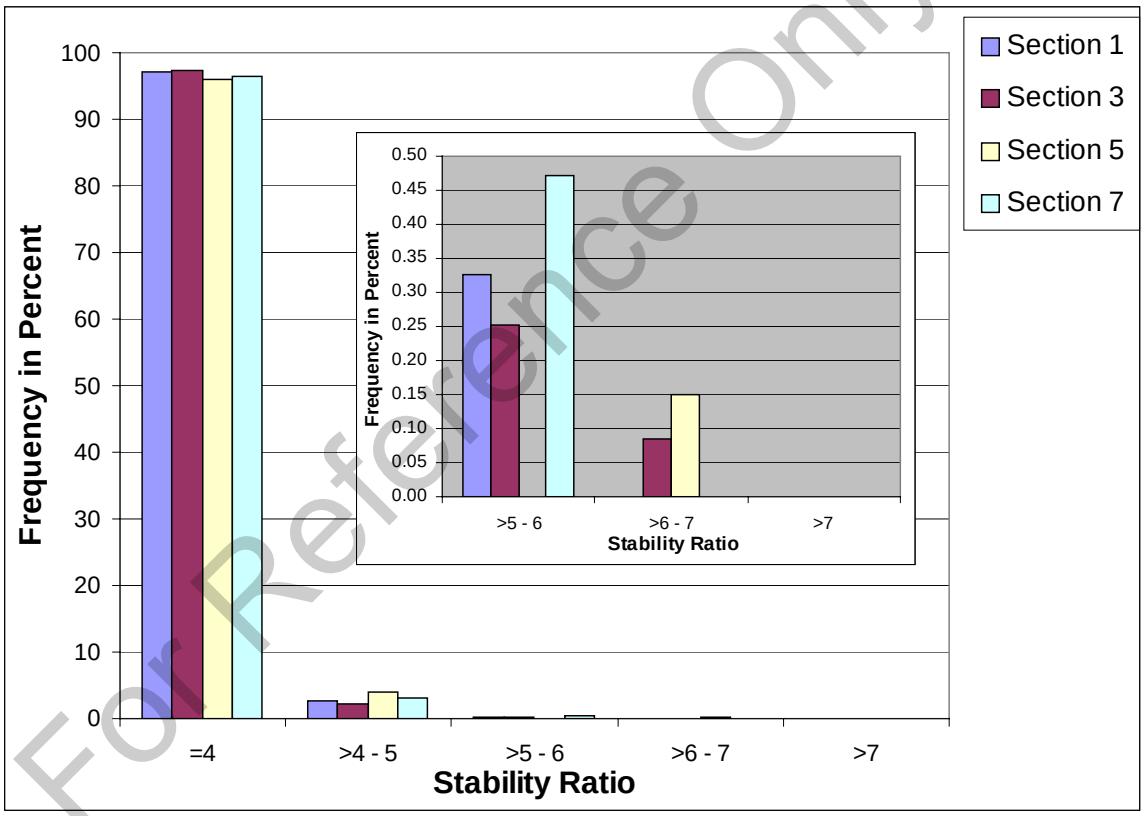
For Reference Only

See disclaimer on cover page.

Cal/OSHA tunnel classification, abrasivity, clay adhesion, soil gradation and fines content, hydraulic conductivity and cross passage construction.

4.2.1 Ground Stability

An evaluation of ground stability under atmospheric face pressures was performed following the concepts described in Appendix D and Appendix E for cohesive soil and for non-cohesive granular soil. Stability ratio for cohesive soils exposed to atmospheric face pressures along the alignment at tunnel depth vary from about 0.3 to 8.0. A histogram of stability ratios for each one foot layer of cohesive soil in the tunnel horizon defined as from 20 feet about to 20 feet below tunnel depth, from cone penetrometer test data, is provided in Figure 4-1.



For Reference Only

Figure 4-1 Stability Ratio at Tunnel Horizon
Note: Insert shows stability ratios greater than five

Heuer (1974) and Peck (1969) both indicate that for stability ratios of 4 or less, tunneling can be carried out without unusual difficulties, but that for ratios greater than 5, clays are likely to invade the annular void created by the cutterhead overcut, before the void can be filled by annulus grouting. Values between 4 and 5 are borderline conditions where layer thickness and time factors must be considered. The vast majority (about 94 to 97 percent) of cohesive soils in the tunnel horizon have stability ratios of 4 or less. Overall, 0.4 percent of the

See disclaimer on cover page.

cohesive soil in the tunnel horizon has a stability ratio about 5 and 2.6 to 4.6 percent between 4 and 5.

Most of the granular soils encountered by the EPBMs will have a tendency to flow if not for the exertion of a stabilizing face pressure. Due to the variability of the alluvial soils, it is not expected to be possible to reliably predict when flowing soils will be encountered so the EPBM must operate at face pressures necessary to control flowing soils at all times, as discussed in Section 4.3.2.

If sandy soils are present in the shield overcut area, they are likely to converge and fill the overcut outside the shield body before annulus grout can be injected. The capability for pressurized injection of bentonite or high density slurry in the shield overcut to form a filter-cake is required to help control settlement due to loss of overcut around the shield body, as discussed in Section 6.0.

The Soil Units were generally classified in terms of anticipated ground behavior if excavated at atmospheric pressure without pre-excavation ground improvement. The results of this analysis are included in Table 4-2.

Table 4-2 Ground Stability Classification at Tunnel Depth

Soil Unit	Stability Ratio of Cohesive Soils at atmospheric pressure	Relative Density of Granular Soils	Uniformity Coefficient U[1]	D_{10} Size[1], mm	Tunneling Classification
1	About 95% ≤4 About 4% > 4 and < 6 About 1% ≥ 6	Medium dense to very dense, in lenses and layers less than 5 feet thick	> 2.6	-	Generally Firm to Slow Raveling, with localized zones that may be Swelling or Squeezing. Localized granular soils are Flowing to Cohesive Running
2	NA	Loose to very dense	10.5	1.40	Generally Flowing

Note:

1. Uniformity coefficient is defined as D_{60} divided by D_{10}. D_{60} is the particle size that is larger than 60% of the material. D_{10} is the particle size that is larger than 10% of the material.

[1] Unit stand-up time is defined as the stand-up time of material exposed in a one foot wide strip, with stand-up time in a one foot square being 50% longer than the "unit stand-up time"

[2] For purposes of applying Deere's classification system, SPT blow counts less than 10 are considered loose and greater than 30 are considered dense

For Reference Only

See disclaimer on cover page.

4.2.2 Gas Classification

The tunnel alignment drawings, geotechnical data and information on soil and groundwater contamination were provided to Cal/OSHA along with a request for an underground classification in accordance with Title 8, Tunnel Safety Orders of the California Code of Regulations. The tunnels have received an underground classification of Potentially Gassy with Special Conditions from Cal/OSHA, a copy of which is included in Appendix B.

4.2.3 Abrasivity

Abrasivity is important as it relates to wear on excavation equipment. Abrasivity is related to strength of the intact material, and size, angularity and hardness of its constituents.

The GDRs contain mineralogy test results for the SVRT soils. Quartz content is reported in two different ways. In one way, the quartz content of the sandstone fragments is included and in another way, it is excluded. If the quartz content of the sandstone fragments is excluded, the quartz contents range up to about 60% by volume. The coarser grained soils have a higher percentage of lithic (i.e. rock) fragments like sandstone and a correspondingly lower percentage of mineral grains like quartz, feldspar and mica, however, there is insufficient test data to quantify the maximum quartz content including the quartz content of lithic fragments, so for baseline purposes, assume this value to be XX percent. The sand and gravel clasts are generally equant[3] and angular to subrounded, generally consisting of fine-grained sandstones and mineral grains (quartz and feldspar) and other lithic fragments. Refer to the GDRs for the test results. Approximately 28 percent of the soil volume to be excavated is expected to consist of sand and gravel sized particles that will be abrasive.

The Soil Abrasion Test (SAT) measures the abrasiveness of soil to the type of hardened steel used for rock TBM disc cutters. Seven test results from the Bay Tunnel located about 14 miles to the north of the SVRT alignment indicated medium abrasivity for sands and very low to medium abrasivity for clays however, there are no test results available for gravels. The soils on the SVRT alignment are expected to be more abrasive than the soils tested on the Bay Tunnel alignment because the SVRT alignment is closer to the source of sediments than the Bay Tunnel alignment and grain angularity and therefore abrasiveness is expected to increase closer to the source.

4.2.4 Cobbles and Boulders

In accordance with ASTM standards, cobbles[4] are particles that range from >3 to 12 inches in maximum dimension and boulders are greater than 12 inches in maximum dimension. The largest size particle sampled during the site

[3] Equant: having the same or nearly the same diameter or width dimension in all directions (i.e. cubic or spherical).
[4] As defined by ASTM D-2487 in English units; metric conversion does not apply.

For Reference Only
See disclaimer on cover page.

investigations was five inches in diameter (boring MW-4B @ 37ft to 42ft and boring PZ-6D @ 121ft to 128ft). However, the nature of the drilling and sampling methods used makes it difficult to retrieve larger particles. As a result, these limited instances are not representative of their occurrence. A more representative basis is the occurrence of larger particle sizes as noted in well drillers' logs, which reflect the drilling of holes up to 24 inches in diameter. Based on a review of the logs of 342 water wells drilled from 40 to 70 feet deep, drill rig chatter, a good indicator of the existence of cobbles and boulders, was noted in the logs for 1.2% of the hole lengths drilled.

Based on the above, for baseline purposes, assume that 1.2% of the tunnel length excavated will consist of cobble-size material up to 12 inches in maximum dimension, and that no particles greater than 12 inches in maximum dimension (i.e., boulders) will be encountered. Cobbles can be expected to occur randomly in any soil layer, or nested when associated with gravel layers.

4.2.5 Soil Clogging Potential

Soil is expected to adhere to metal surfaces under certain conditions of plasticity and water content. Such "stickiness" can cause clogging and consequent problems and delays with moving mechanical parts. This will take place in the EPBM cutterhead, the cutterhead arms, the excavation chamber bulkhead, the screw conveyor, and other spoil handling equipment. The EPBMs are required to be designed to minimize the potential for clogging. Soil conditioners intended to reduce clogging potential will also be employed.

The potential for clay clogging problems has been assessed for SVRT using a method published by Thewes (2004) that uses in-situ moisture content and Atterberg Limit test results. Thewes (2004) highlights clogging potential based on empirical relationships from slurry TBM case histories, which are considered to be relevant to EPBMs as well. This method is considered to be a more realistic prediction of clogging potential than the Atterberg Adhesion Limit test method, though some Adhesion limit test data is also included in the GDRs. The Thewes paper describes three categories of clay clogging potential, high, medium, and low. Soils with high clogging potential can lead to substantial problems during excavation and may required daily cleaning works. Both high and medium clogging potential are considered to require use of an anti-clay soil conditioner.

Thewes' criteria for clogging potential were applied to those cohesive soil samples that were obtained within a "tunnel zone" extending 20 feet above to 20 feet below the tunnel opening. The results are shown in Figure 4-3. Data that plot in the "unknown" portion of the figure represent soils with suspiciously high or low in-situ moisture contents, possibly indicating sample disturbance prior to testing. Such samples were most often slightly to moderately plastic and are therefore not expected to represent a clogging risk. The anticipated occurrence of significant clogging risk at tunnel depth is estimated based on high and medium

For Reference Only

See disclaimer on cover page.

clogging potential percentages in Figure 4-3 in lengths of the tunnel with Unit 1 greater than about a third of the diameter (seven feet), which from Table 4-1 constitutes 89% of the alignment length. As can be seen in Figure 4-3, 15% of the cohesive soil samples reflect a high clogging potential, and 30% reflect a medium clogging potential. The resulting overall percentage of the alignment anticipated to encounter soil with a significant clogging potential is 45% on the basis of these evaluations.

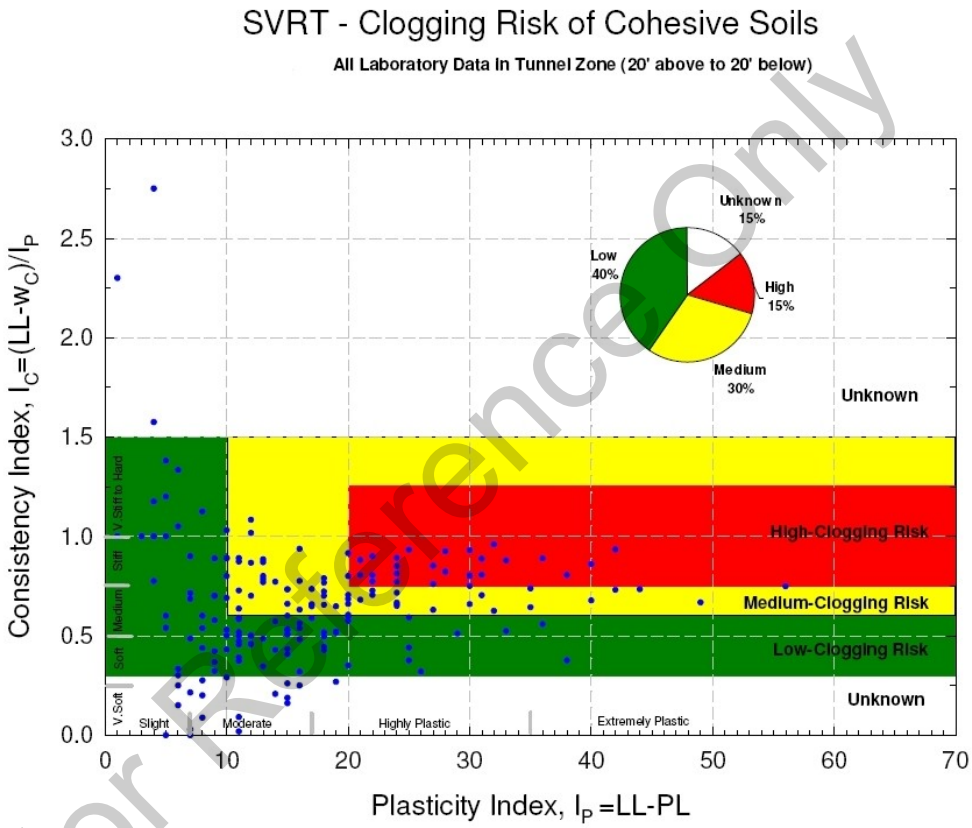


Figure 4-3 Categories of Potential TBM Clogging for SVRT Cohesive Soil at Tunnel Depth

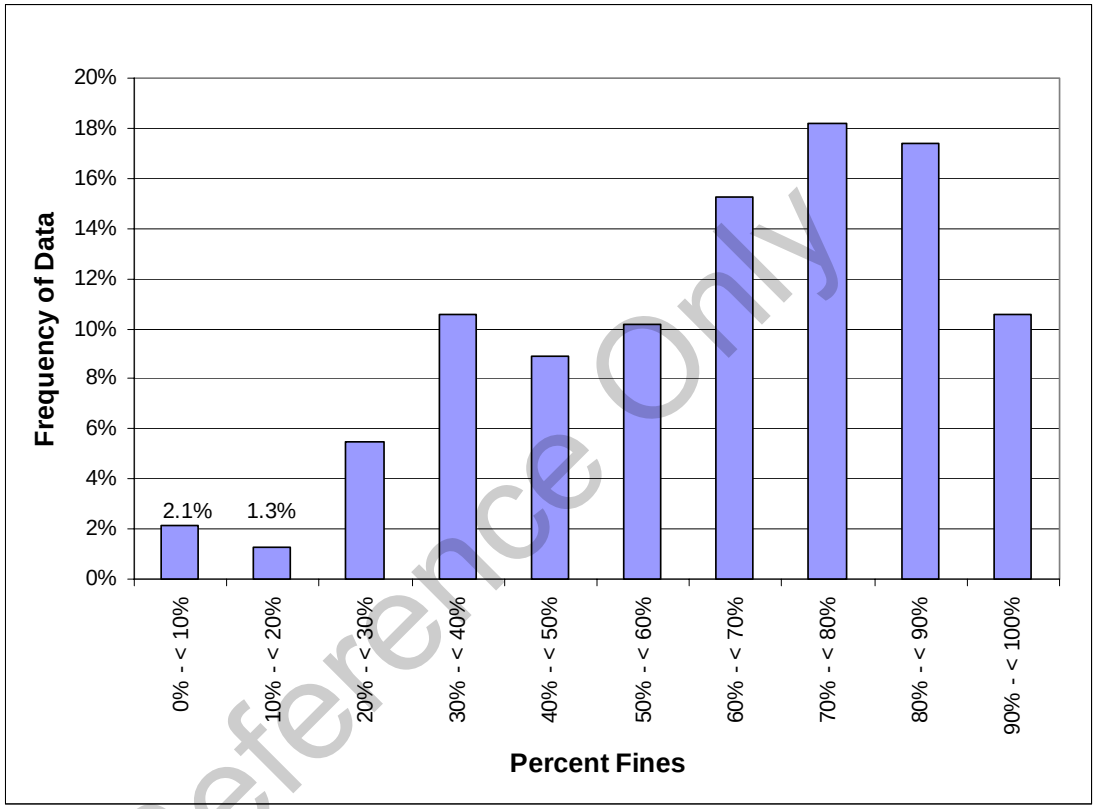
For Reference Only

4.2.6 Soil Gradation and Fines Content

The fines content has been evaluated at each borehole along the alignment with fines content values averaged over the tunnel zone described in Section 4.2.5. The fines content was averaged because the EPBM (cutterhead, cutterhead arms and screw conveyor) will at least partially if not completely mix the soil from each soil layer encountered in the tunnel face. It is the average fines content of the mixed soil in the screw conveyor that will influence the ability to control face pressure while excavating. Spoil averaging less than 20 percent fines is of

See disclaimer on cover page.

concern for EPBM tunneling. The anticipated fines contents of soils to be encountered during tunneling are summarized in Figure 4-4. The source values and their distribution along the alignment are shown on GBR Drawings C301 to C304.



Note: Percent fines are based on the 100 foot intervals as shown on the Data Parameters Tables on GBR Drawings C301 through C304

Figure 4-4 Frequency of Average Weighted Fines at Tunnel Face

Based on this assessment, about three percent of the alignment is expected to encounter spoil in the excavation chamber containing 20 percent or fewer fines. The anticipated length of alignment reduces to about two percent for a threshold of 10 percent fines. Baseline tunnel lengths are provided in Table 4-3. Two possible locations where less than 10 percent fines could be encountered in the excavation chamber are shown below. Summaries of the gradation of all soil samples from tunnel depth (+/- 20 feet) in the vicinity of these locations are shown in Figure 4-5 through Figure 4-7.

For Reference Only

See disclaimer on cover page.

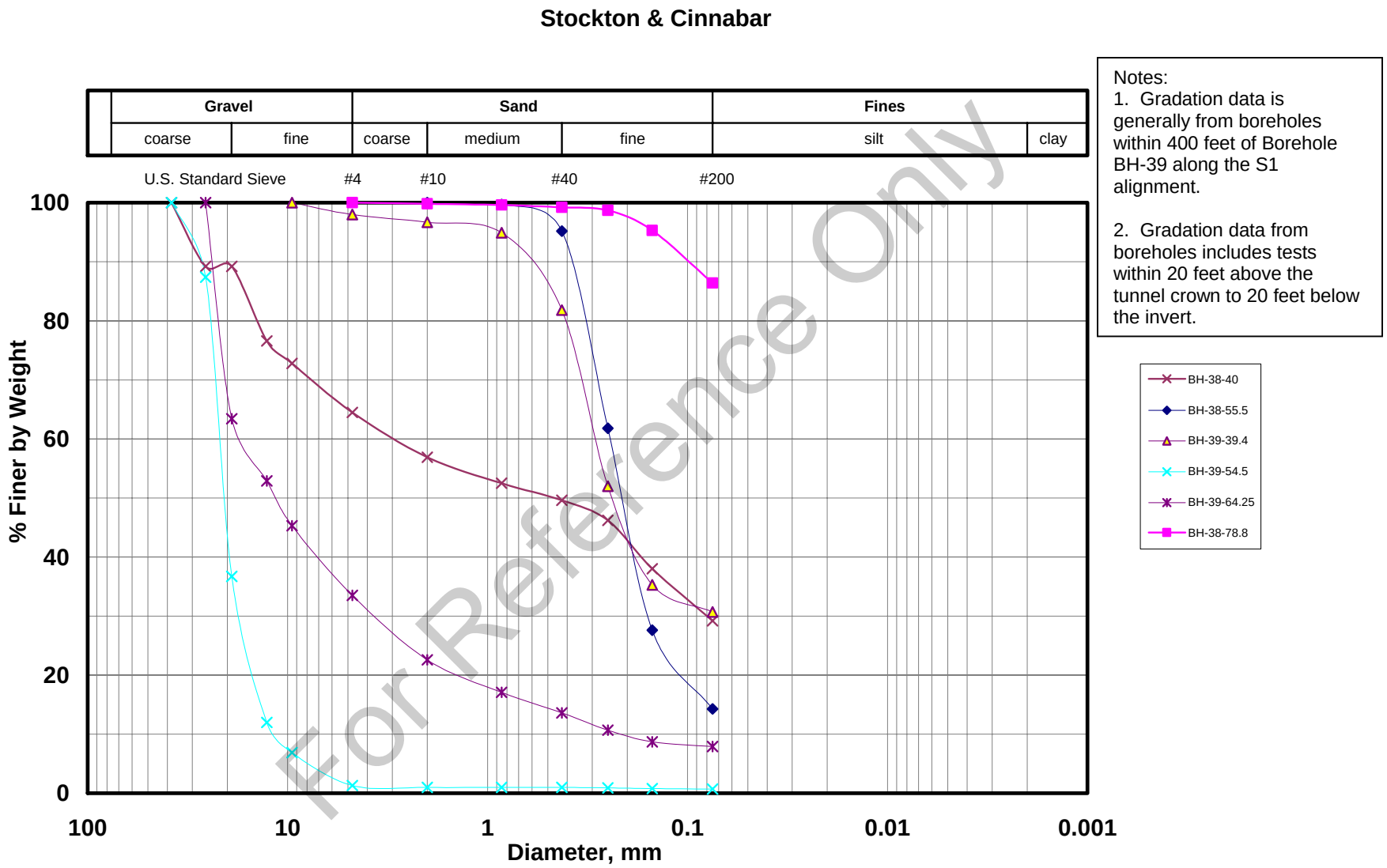
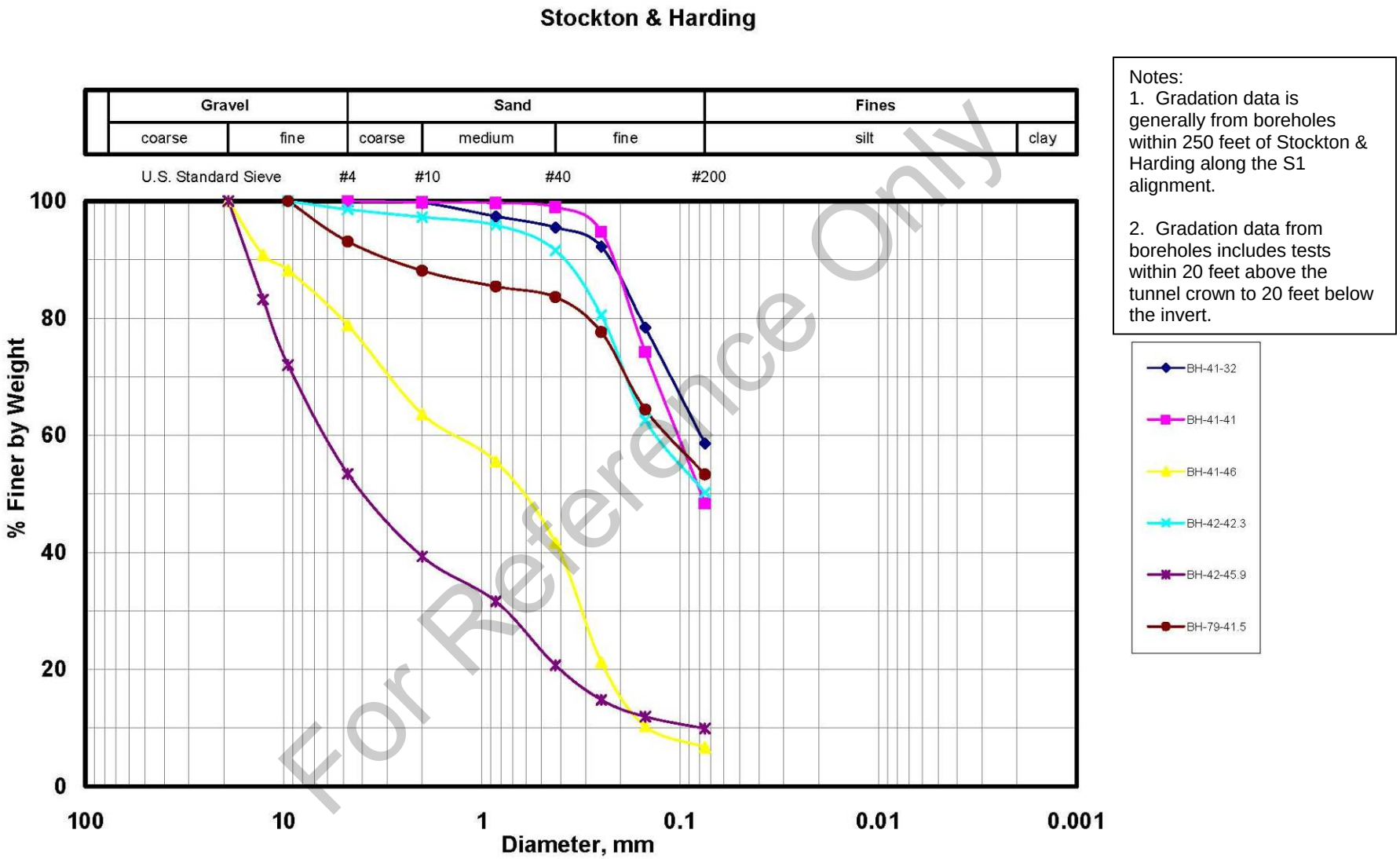


Figure 4-5 Summary Gradation Plot, Stockton and Cinnabar/Lenzen

See disclaimer on cover page.



For Reference Only

Figure 4-6 Summary Gradation Plot, Stockton and Harding

See disclaimer on cover page.

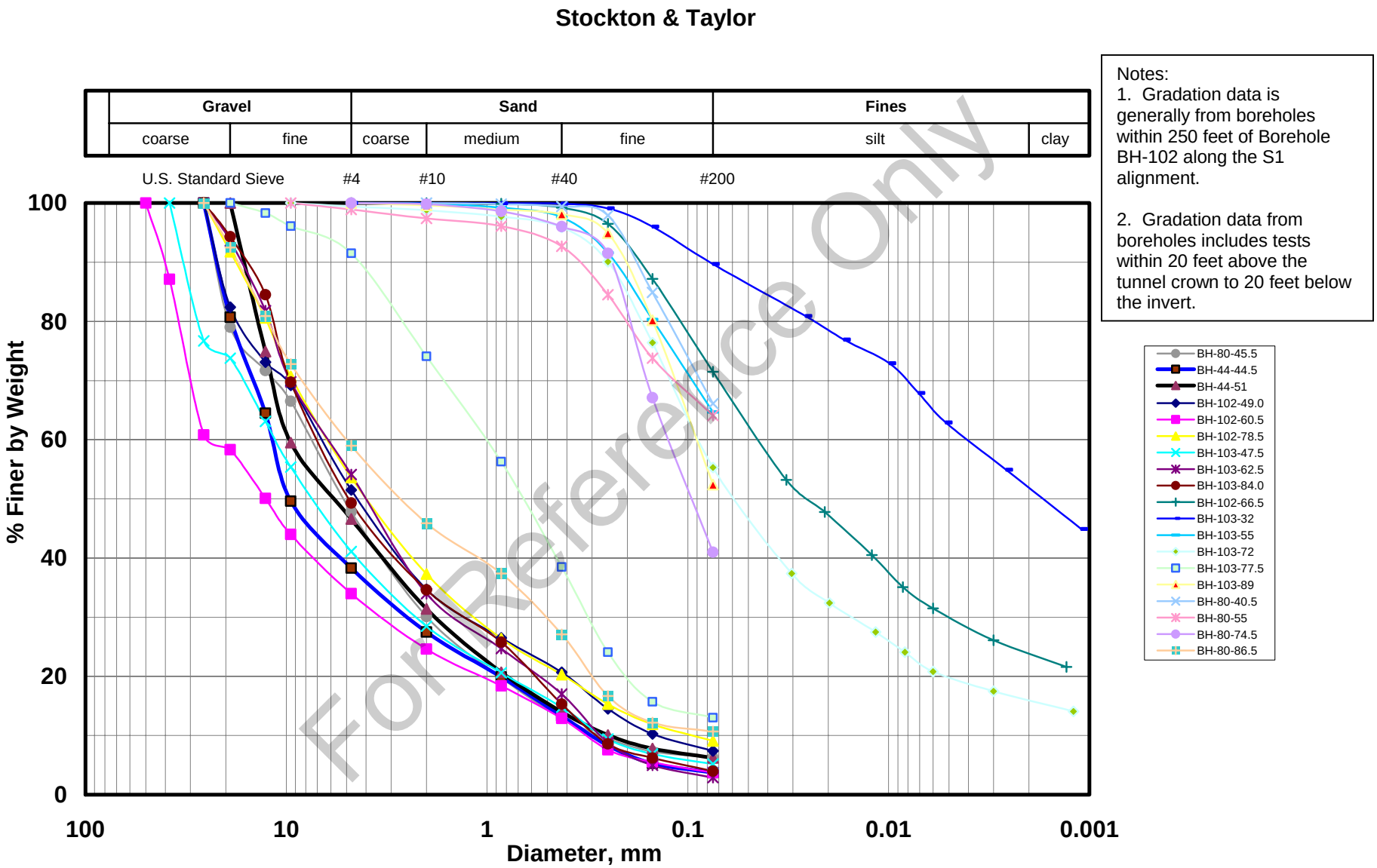


Figure 4-7 Summary Gradation Plot, Stockton and Taylor/Asbury

See disclaimer on cover page.

Table 4-3 Summary of Alignment Length Anticipated to Encounter Soil with Less than 20% Fines

Approximate Station		Baseline Length (ft)		Comments
From	To	Less than 10 % Fines	From 10% to Less Than 20 % Fines	
767+00	771+00	400		BH-39 Stockton and Cinnabar / Lenzen
779+00	782+00		300	BH-41 Stockton and Harding
798+00	799+00	100		BH-44 Stockton and Taylor / Asbury[2]

Note: [1] Lengths, not location, should be basis for design for anticipating low fines content. When low fines material is encountered, adjustments in EPBM operating parameters shall be made as required to maintain soil stability and control settlement.
[2] The sample interval is 100 feet but based on adjacent borings, low fines material is only expected intermittently in this interval
[3] Estimated lengths are taken from GBR Drawings C301 through C304

For Reference Only

4.2.7 Potential Obstructions to Tunneling

This section describes the possible types of physical obstructions to tunneling that may be encountered. Possible obstructions include the following:

- Wells
- Utilities
- Foundations and underground structures
- Wood

4.2.7.1 Wells

An abandoned water well has been identified for possible removal prior to construction on San Jose Water Company property near their building at 374 West Santa Clara Street. The casing is thought to be steel, and the well is reportedly 831 feet deep.

(Note to Reviewers: The actual position of the well in relation to the tunnels is currently unknown and will be investigated by potholing during final design. If the casing is found within five feet of a tunnel excavation, the upper 85 feet of steel casing will be removed and the hole will be backfilled with cement grout in accordance with County well abandonment procedures administered by the Santa Clara Valley Water District. It is thought that the well casing is 12 to 18 inches in diameter. The grout is expected to consist of either a neat portland cement and

See disclaimer on cover page.

water mixture or a low-strength backfill material if acceptable to the district.)

In the Hydrogeology Report (HMM/Bechtel, 2008a), the following statements are made: "In 1985, the Santa Clara Valley Sub-basin was estimated to have included 1,205 private wells, 258 municipal wells, 850 inactive wells, 3,000 destroyed wells, and possibly 12,000 additional abandoned and/or inactive wells. Water district records indicate locations of known water supply wells and monitoring wells; however, records of well locations are incomplete. Permits for well installations were not required prior to 1969; therefore, for the period of time from the installation of the first well in the mid-1800's to 1969, when formal documentation began, no master list of well locations exists."

Based on the preceding discussion, it is likely that unmarked abandoned wells and well casings will be encountered during construction. For baseline purposes, assume that five steel lined well casings, ranging from 12 to 24 inches in diameter, will be encountered that require manned interventions for removal. Additional details of interventions are presented in the Contract C301 Specifications.

4.2.7.2 Utilities

Utility relocations will be required for the stations, mid-tunnel ventilation shafts, and portal cuts. The potential for encountering unknown buried utilities is most significant near the portals, where the depth to the crown of the tunnel is less than 15 feet. Utility location services will be used to assist with field location of utilities prior to construction. Encountering unforeseen utilities, whether they are abandoned or undocumented/improperly documented lines, will be considered as discussed in the Contract C330 Specifications.

4.2.7.3 Existing Foundations

The tunnel alignment was established to avoid contact with existing foundations and underground structures provided that the tunnels are driven within the alignment tolerances. The tunnels are close to the foundations of several structures, which are shown on the Contract C330 Drawings. For baseline purposes, assume that the EPBMs will not encounter structural foundations. Settlement issues are discussed in Section 6.0.

4.2.7.4 Wood

Water well drillers have reportedly encountered redwood logs though none were encountered during the project site investigation. In total, wood of some sort was reported in 0.1% of the water wells length drilled within a depth of 40 to 70 feet. For baseline purposes, assume 0.1% of the

For Reference Only

See disclaimer on cover page.

excavated spoil volume will contain elongated wood pieces up to four inches across the narrowest dimension.

4.3 Bored Tunnels

4.3.1 Design Considerations

Soil cover above the tunnels, typically ranges from 30 to 60 feet along most of the alignment. Cover increases to about 70 feet near the Guadalupe River and decreasing to about 11 to 13 feet at the East and West Portals, respectively.

For reference and to encourage use of consistent terminology in describing EPBM components and operation, an EPBM diagram is shown on Figure 4-8.

The EPBMs will be used to install a one-pass lining system consisting of 17 foot-10 inch internal diameter, 10 inch thick pre-cast concrete segmental rings. Segment geometry is shown on the Contract C302 Drawings.

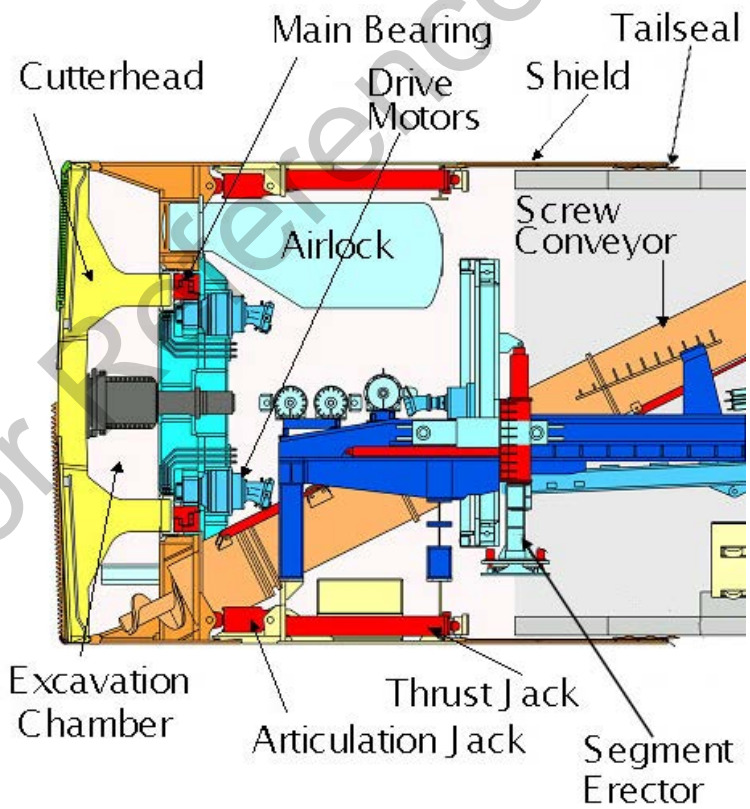


Figure 4-8 Earth Pressure Balance Machine (EPBM) Diagram

For Reference Only

See disclaimer on cover page.

4.3.2 Bored Tunnel Construction Considerations

The Contract C330 Drawings show driving the tunnels from the portals towards the Downtown Station. The EPBMs will excavate through partially backfilled mid-tunnel ventilation structures and be skidded through the previously excavated cut-and-cover excavations at Diridon/Arena Station and Alum Rock Stations. Schedule constraints dictate that tunneling must begin at the West Portal. The EPBMs will drive from the West Portal toward the Downtown Station cut and cover excavation. The EPBMs will be retrieved at the Downtown Station and re-launched from the East Portal and driven to the Downtown Crossover cut and cover excavation.

4.3.2.1 Face Pressures for Normal Operation

As described in the Contract C330 Specifications, the EPBMs are required to operate in closed mode at all times, such that the required pressurization of the excavated ground in the excavation chamber is achieved and maintained, whilst excavated material is removed by the screw conveyor during TBM advance. As defined in the Contract C330 Specifications, closed mode pressurization is maintained during excavation by synchronization of TBM rate of advance with the rate of removal of excavated material by the screw conveyor and guillotine gates. These variables are a function of cutter head rotation, TBM thrust and speed of rotation of the screw conveyor. This requires direct full-face contact between the ground in front of the cutter head and the excavated soil, soil paste or slurry that fills the excavation chamber. This necessitates the filling of open areas in the cutter head and the tool gap in front of the cutter head. Positive support pressure is pressure applied by the EPBM, within specified tolerances, to the material in the excavation chamber, the open space in the cutter head and the tool gap, at all times to balance hydrostatic and earth pressures. The excavated spoil is mixed with fluid soil conditioners injected at the cutter head, in the excavation chamber and/ or in the screw conveyors to create a plastic consistency suitable for transmitting pressure to the face and soil surrounding the EPBM.

The EPBM operator controls face pressure by controlling the thrust jack pressure, and the rate at which spoil is removed. The spoil removal rate is controlled by the rotation speed of the screw conveyor(s) and guillotine gate opening(s). A method for estimating face pressures required for ground stability for the SVRT project during boring is provided below.

Monitoring the operating pressures of EPBMs is critical to their safe and effective application. These operating pressures are monitored by sets of sensors incorporated within the EPBM operating systems, including within the excavation chamber. It is imperative that provisions be made for replacing non-functioning sensors and supporting instrumentation through the course of the tunnel drives.

For Reference Only

See disclaimer on cover page.

It is required that there be a degree of independence and multi-party checking of the procedure employed for confirming and/or revising target face pressures during tunneling. The tunneling Contractor is required to have a procedure that contains internal checks to ensure that all necessary factors are taken into account and the procedure is required to be an integral part of the Contractor's submitted project management system. A review process is required that correlates measured face pressures to the target and the observed settlement performance and calls for investigating reasons for departure from the planned operating pressure range. Details of this procedure are contained in the Contract C330 Specifications.

4.3.2.2 Face Pressure Target and Operating Range

The contractor is responsible for establishing target face pressures according to guidelines described in this GBR, and specified requirements to limit over-excavation and ground disturbance associated with tunneling. Furthermore, the Contractor is responsible for operating the EPBMs to maintain surface settlement within the threshold values indicated in the Contract C330 Specifications to control construction induced movements as described in Section 6.0.

A lower bound face pressure limit during tunneling shall be established to control surface settlement, and an upper bound limit shall be established to control heave and frac-out. EPBMs must be operated within a range defined by these upper and lower bound limits while advancing. Establishing these upper and lower bounds will involve consideration of a number of loading conditions for soil stability and evaluating which are the controlling values at a given location. Potentially limiting lower and upper bound pressures for the contractor to evaluate are described in Table 4-4.

It is expected that target face pressure values will be estimated by summing the largest (controlling) lower bound value and the face pressure control tolerance of 0.3 Bar. An operating range defined as 0.3 Bar above and below the target value will be defined, thus the operating range for a tolerance of 0.3 Bar would be a 0.6 Bar range centered on the target value. This concept is illustrated in Figure 4-9. Operating at the upper end of the allowable operating range should be checked against the lowest (controlling) upper bound value to evaluate the risk of heave and frac-out. In some instances near portals, where cover is low (approximately 11 feet at East Portal, 13 feet at West Portal), the operating range must be constrained to less than 0.6 Bar to limit the risk of heave.

For Reference Only

See disclaimer on cover page.

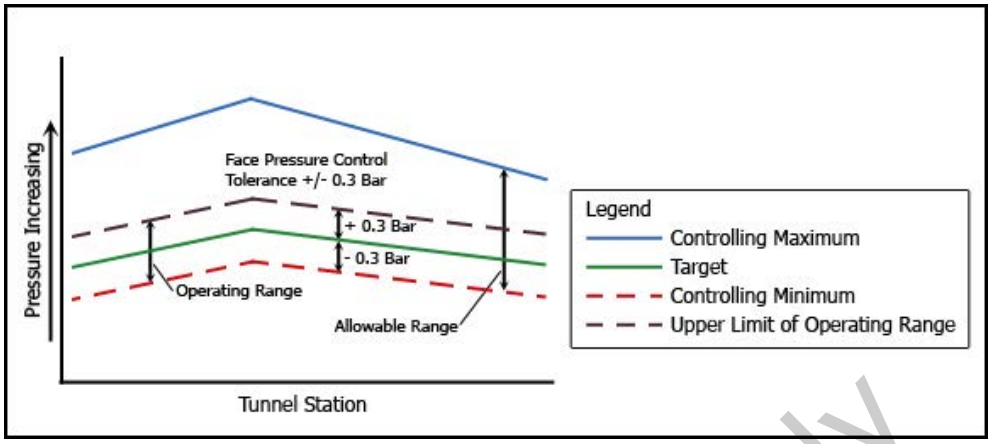


Figure 4-9 Selection of Target Face Pressure

Table 4-4 Potentially Limiting (Upper and Lower Bound) Face Pressures

Limiting Pressure	Significance
Tunnel pressure for stability of cohesive soils	Represents a possible lower bound. This pressure provides for stability in a full face of cohesive soil. Face pressure can be zero if soils have a stability ratio less than four without a supporting pressure, based on Broms and Bennermark, 1967.
Full excavation chamber pressure	Exceeding this possible lower bound value ensures the excavation chamber is full at all times. Can be controlling minimum value in low cover areas. Estimated as the product of distance from axis to excavated radius and the unit weight of conditioned spoil. Goal is to keep the excavation chamber, openings in the cutterhead and the tool gap in front of the cutterhead full of a viscous fluid spoil to ensure the EPBM is being operated in closed mode.
Active lateral earth pressure	Represents a possible lower bound. Balancing the active earth pressure prevents face instability in the form of an active soil wedge.
Hydrostatic pressure	Represents a possible lower bound. Balancing or exceeding hydrostatic pressures ensures there is no destabilizing seepage forces in the tunnel face and minimizes amount of water in the spoil.
Annulus grout pressure	Represents a possible lower bound. The annulus grout injection pressure can be slightly higher than the hydrostatic pressure to displace groundwater. Balancing the annulus grout pressure should limit the flow of annulus grout toward the face of the TBM.
At-rest lateral earth pressure	Represents a possible lower bound. This is the in-situ horizontal stress prior to tunneling. Balancing this earth pressure will theoretically limit settlement ahead of the face.
Vertical Overburden Pressure	Represents a possible upper bound. Exceeding the overburden pressure can cause heave in granular soil.
Passive Lateral Earth Pressure	Represents a possible upper bound. Applied face pressures approaching the passive lateral earth pressure will cause plastic deformation ahead of or above the TBM and can cause heave of the ground surface.

For Reference Only

See disclaimer on cover page.

Using the method described above, target face pressures at tunnel axis during the boring part of the tunneling cycle were estimated at various locations along the SVRT tunnel alignment and included in the TBM Geotechnical Basis of Design Report (HMM/Bechtel, 2008c). The Contractor is responsible for establishing target pressures using the procedures included herein and the guide values in the TBM Geotechnical Basis of Design Report. Actual pressures will vary from the guide values for target face pressure as estimating assumptions are confirmed by field measurement for items such as soil type, conditioned spoil unit weight, and free-field hydrostatic pressure. Face pressures used for construction must be compatible with EPBM design.

During stoppages (static mode) such as during the ring-build portion of the tunneling cycle, face pressures may tend to decrease to what is often called "ambient pressure". The face pressure in static mode may be allowed to drop to full-plenum pressure but no lower.

In static-mode, a system is required to ensure that face pressures remains within tolerance. This can be accomplished by the following methods:

- Increasing pressure to the upper limit of the operating range at the end of the excavation stroke
- Injecting bentonite slurry when pressure drops below tolerance.

The annulus outside the shield body may also be pressurized during stoppages to limit settlement.

A two-part screw connected end to end with an articulated joint, having independent drives for each screw segment and equipped with a guillotine gate to control the aperture of the discharge opening from screw 1 to screw 2 has been used in challenging EPBM conditions like those described in Section 4.2.6. A two-part screw with guillotine gate and adequate soil conditioning including high density slurry injection are specified to adequately control face pressure during tunneling for SVRT. Discharge from the first screw through an additional guillotine gate to the conveyor belt is also required to increase operational flexibility and reduce the potential for soil clogging in the rear screw. Operation of the two-part screw in low-fines soil is discussed in Section 4.3.2.9.

4.3.2.3 Protection against Face Pressure Variability and System Uncertainty

The pressure during rotation of the cutterhead is not uniform throughout the excavation chamber, cutterhead openings and the tool gap. Pressure gradients that develop in front of the bulkhead can create low pressure zones that lead to over-excavation in localized areas of the face and tunnel

For Reference Only

See disclaimer on cover page.

perimeter. The size of the openings in the cutterhead and proper plasticized soil consistency through use of soil conditioning are specified to control pressure gradients to within acceptable levels.

4.3.2.4 Face Pressure Maintenance during Launch and Reception

During launch and reception, jet-grout will be encountered. Because the jet-grout blocks may not be continuous, running or flowing granular soil could exist in cracks or discontinuities of the blocks. Therefore, target face pressures are established assuming a cracked jet-grout block to ensure positive control of any running or flowing ground and limited as to not damage the excavation walls. When commencing tunneling, the machine must be in closed balanced pressurized mode with a filled excavation chamber, cutterhead openings and tool gap prior to excavation.

4.3.2.5 Manned Interventions to the Excavation Chamber

Compressed air will be needed for man and material entry to the excavation chamber wherever the EPBM is below the water table and not in a full face of cohesive material. Safe havens of cohesive material and jet grouted soil at cross passages, should be sought to avoid the need for high compressed air levels during interventions. If air is used, the likelihood and consequences of loss of air pressure through granular soils, and by failure of uncured annulus grout, must be addressed. At the worst the ground may need to be stabilized in advance of compressed air interventions.

Equipment and procedures shall be designed for evacuation of spoil from the excavation chamber while maintaining soil stability in front of the cutterhead to cope with the range of anticipated conditions from self-supporting cohesive soil to flowing sands. The maximum likely pressure needed to balance hydrostatic pressure is 2.1 Bar beneath the Guadalupe River. If muck is removed from the excavation chamber during manned interventions under compressed air, the chamber must be refilled with spoil or high density slurry prior to the resumption of tunneling in closed, pressurized mode.

4.3.2.6 Tunneling Through Obstructions

The EPBM cutterheads have been designed to accommodate the mounting of picks, scrapers and disc cutters. The mounting of picks versus disc cutters is the contractor's option. It is envisioned that in considering this option, the Contractor will evaluate the time to change over versus the time savings in switching to more appropriate cutting tools for the situation. Disc cutters are intended for use in cobble rich ground, excavating support-of-excavation walls, jet-grout and other concrete.

For Reference Only

See disclaimer on cover page.

4.3.2.7 Soil Conditioning

4.3.2.7.1 Purpose of Soil Conditioning

The term soil conditioning refers to the addition of compounds that are specially designed to mix with the excavated spoil to modify the range of soil types that can be efficiently excavated by an EPBM, and to enhance EPBM performance. By correctly choosing and adapting the specially designed chemicals, their mixing recipe and injection rate to the encountered soil and ground water conditions, soil conditioning can:

- Adjust the soil consistency, plasticity and permeability
- Reduce tendency for segregation of spoil in the excavation chamber
- Reduce tendency for spoil solids to separate from pore-water
- Lower the angle of internal friction and abrasiveness of the soil
- Reduce stickiness of clays
- Reduce cutterhead torque
- Improve soil condition for conveyor transport
- Act as a lubricant

4.3.2.7.2 Soil Conditioner Dosage Rate and Mix Design

The soil conditioning mixture comprises compressed air, water, foaming agents and/or chemical and mineral additives. The basics of soil conditioner mix design are described by the European federation dedicated to specialist construction chemicals and concrete systems (EFNARC 2005) and this document can be referenced for additional information. Limited amounts of testing using soil samples from the project site have been conducted by BASF. Results of the tests are contained in the Soil Conditioning Tests at BASF Lab Technical Memorandum (P0503-D300-TM-DE-070, Rev. A), which is included as Appendix C.

Air is added to the mixture and, once it joins the mixture, expands because of lower pressure. The combination of foaming agents, additives where applicable, water, and air is termed “foam”. Table 4-5 lists terms commonly used to describe the foam mix design.

In general, spoil modified by soil conditioners is more fluid than untreated spoil and this must be recognized in the selection of spoil conveyance equipment, handling and drying areas and disposal sites.

For Reference Only

See disclaimer on cover page.

Table 4-5 Definition of Foam Mix Design Terms

Terms	Definition	Range[4]
Foaming Agent Concentration (C_F[1])	100 x Volume of foam agent / Volume of foam solution	0.5 – 5.0 %
Foam Expansion Ratio (FER[2])	Volume of foam at working pressure / Volume of foaming solution.	5 - 30
Foam Injection Ratio (FIR[3])	100 x Volume of foam at working pressure / In situ Volume of ground to be excavated (bank volume)	10 – 80 %

Notes:

1. Based on the European Federation of Specialist Construction Chemicals and Concrete Systems (EFNARC) guideline, C_F is expressed as a percentage.
2. Based on the EFNARC guideline, FER is expressed as a ratio.
3. Based on the EFNARC guideline, FIR is expressed as a percentage.
4. These values are found in practice to be relatively high, so values below the low end of the range will be considered

4.3.2.7.3 Soil Classification for Soil Conditioning

Based on discussion with soil conditioning suppliers, there is a general consensus that there are two types of problem soils for EPBM tunneling below the groundwater:

- Cohesionless soils lacking fines
- Sticky clays

In the British Tunneling Society (BTS) guidelines (2005) and from past experience a minimum of 20 percent fines in the spoil is desirable for an EPBM, with soils less than 10 percent fines requiring high density slurry to achieve the desired result. A classification system for SVRT is presented in Table 4-6 along with the baseline extent that each classification occurs along the alignment.

4.3.2.8 EPBM Operation in Normal Material

The EPBM is required to operate in closed mode at all times. Closed mode is defined as direct full-face contact between the ground in front of the cutterhead face and the excavated soil, soil paste or slurry that fills the excavation chamber. Closed mode must be maintained at all times whether boring (dynamic mode) or while stopped (static mode). Support pressure within specified tolerances must be applied to the material in the excavation chamber at all times when boring.

The TBM operator controls face pressure when boring by controlling the applied thrust, and the rate at which spoil is removed. The spoil removal rate is controlled by the screw rotation speed and guillotine gate opening.

For Reference Only

See disclaimer on cover page.

The tunneling contractor will establish face pressures, subject to acceptance by the Engineer, using a procedure similar to that described in Appendix A, based on the free-field[5] hydrostatic pressure measured in advance of tunneling.

4.3.2.9 EPBM Operation in Low Fines Material

For EPBM operation in low-fines material, where the formation of a soil plug in the screw conveyor is difficult to achieve while maintaining target pressure, the following TBM capabilities are required and can be implemented independently or together, as needed:

- Use a double screw conveyor as described below
- Vary the speeds of the screws, such that the second screw rotates at a slower speed than the first screw
- Restrict the aperture of the guillotine gate opening(s)
- Inject polymer modified foam or polymer and/or high density slurry directly in the excavation chamber to limit segregation and improve plastic behavior of the spoil

The double screw conveyor can theoretically be operated in a semi-positive displacement mode to advance through soils with very low fines content. This is done by initially filling both screws with spoil, closing the guillotine gate between the two screws and finally by discharging the spoil contained within the rear screw only, before repeating the process. Technological challenges to implementing this theoretical operating procedure is to be overcome by the EPBM supplier and operator, include closing the guillotine gate between the two screws while the screws are full of spoil, and discharging from the rear screw while there is potentially a lack of pressure to evacuate the rear screw.

4.3.2.10 Face Pressure Maintenance During Shut-Downs

In order to limit volume loss, closed mode must be maintained in the excavation chamber during stoppages of any length, unless during a planned manned intervention as described below. The annulus outside the shield body shall be pressurized during stoppages as required to maintain settlement within targets.

4.3.2.11 Measurement of Excavated Volume

Measurement of excavated quantities of spoil is critical to prevent over-excavation of the ground by the TBM. The consequences of over-

[5] Free-field means before the influence of the EPBMs. It is expected that groundwater pressures will rise in response to EPB pressure up to 100 feet out in front of the EPBMs.

For Reference Only
See disclaimer on cover page.

excavation can be very serious and include volume loss, excessive settlement and sinkholes at the ground surface. Measurement is specified to be conducted on a continuous basis, and as accurately as possible. This is something of a challenge due to the variability of the in-situ density of the ground and the effects of soil conditioning on spoil density. With the long conveyor mucking system, it will not be possible to visualize the volume of excavated spoil so information from the belt weighers, volume profilers, soil conditioning plant, bulkhead pressure cells, and annulus grout injection must be synthesized to measure and check readings.

Although the absolute comparison of theoretical to actual muck measurement cannot be carried out, one can use belt weighing on the TBM belt conveyor to measure the actual weight but not the volume, due to uncertainties about spoil density. For volume measurement using a volume profiler, the bulked solid volume can be measured but the in-place (bank) volume can only be estimated using a swell factor. Thus the trends of the measurements made, the comparisons of values ring to ring and the calculation of five ring rolling averages are very important. By using both belt weigher and volume profilers, and carrying out both volume and weight comparisons, greater control is possible.

For Reference Only

See disclaimer on cover page.

Table 4-6 Soil Type Classification System for Soil Conditioning Purposes

Classification For Soil Conditioning	Soil conditioning suppliers corresponding soil descriptions	Description of soil mixture in the excavation chamber	% Fines[2]	In Situ Moisture Content	Maximum Hydraulic Conductivity (cm / s)	% Length in each Bore [1]	Recommended Type of Foam according to EFNARC	Possible Additives	Preliminary Soil Conditioning Recommendation[3]
Sands and Gravels with 10-20% fines	Poorly graded and low fines ground (and high water pressure ground)	Full face of poorly to well-graded sands and gravels	10-20%	13%	1×10^{-1}	0.7%	Foam C – high stability and anti-segregation properties to maintain a cohesive soil as impermeable as possible	Acrylamide Polymer; CMC Polymer; Bentonite; Pulverized Limestone	C_F = 2% FER = 15 FIR = 75%
Sands and gravels with less than 10% fines	Gravelly sand, fine to coarse sand; or Poorly graded and low fines ground (and high water pressure ground)	Full face of poorly to well graded sands	<10%	(16%) [5]	1×10^{-1}	0.8%			C_F = 2% FER = 15 FIR = 75% High Density Slurry[4] at 10% by weight
Soils with significant clogging risk	Sandy clay, pure clay, soft or heavy clay	Clay layers present in tunnel face in cumulative thickness of 7 feet or more.	> 20%	37.2%	1×10^{-6}	44.6%	Foam A – high dispersing capacity (break clay bonds) and / or good coating capacity (reduce swelling effects)	Anti-clogging agents	C_F = 2% FER = 12 FIR = 70%
Non-clogging soils with sufficient fines	Silty sand, clayey sand or Silty sands to sandy clay	Mixed face may include bands of poorly to well graded sands, gravels and clays	> 20%	18.6%	1×10^{-3}	54.9%	Foam B – general purpose, with medium stability	Anti-clogging agents[5]	C_F = 2% FER = 15 FIR = 50%

Notes:
1. Percentages are based on the interpretation shown on GBR Drawings C301 to C304 within the tunnel horizon
2. Weighted average value for TBM heading as shown on GBR Drawings C301 to C304
3. TBM manufacturer responsible for designing and providing the soil conditioning equipment. The tunnel contractor is responsible for establishing soil conditioning mix
4. If pulverized limestone is used for high density slurry, it must be used in conjunction with a fluidifier to suspend the solids in water.
5. It may not be practically or advisable to limit the dosage of anti-clogging additives when clay is present in the muck so quantities of anti-clay should be estimated assuming it is used for soils with significant clogging risk and some or all of the non-clogging soils with sufficient fines.

For Reference Only

See disclaimer on cover page.

4.3.2.12 Settlement Control

Flowing sands and cohesive soils with stability ratios greater than five are materials that could be encountered throughout the alignment and which could close the annulus void around the EPBM shield rapidly before annulus grout is injected through the tail. The EPBM will be designed to limit closure of the overcut (annulus) void by minimizing the difference between cut diameter and tail shield diameter, and by having the capability to inject material under pressure to fill the void around the EPBM shield, particularly during stoppages.

The geotechnical instrumentation response value requirements established in the Contract C330 Specifications to minimize the impacts of tunneling are based on the 1% ground loss estimates (though half a percent or less is considered achievable in most ground conditions) discussed in Section 6.0 herein, but allowing for the additional effects of shaft and open cut shoring and dewatering. Volume loss, during tunneling excavation, is controlled by avoiding over-steering, by maintaining target face pressures, by keeping the annulus outside the segmental lining full of grout as the EPBM advances and by stabilizing the EPBM annulus in unstable soils, particularly when the EPBM is not advancing. The annulus between the EPBM shield and the ground will tend to close when the EPBM is not advancing, particularly when sands and gravels are present in the tunnel horizon.

For the SVRT project, the following controls are being specified:

- Use of closed mode at all times
- Use of sufficient face pressure at all times, as defined in this report. This includes the control of face pressures within specified tolerance of the target pressure while boring and reduced pressures when the TBM is approaching station headwalls.
- Measurement of excavation quantities for comparisons with theoretical as early indicator of volume loss.
- Injection of bentonite slurry around the EPBM shield.

[6] If the excavated volume equals the volume of soil displaced by the tunnel lining and overcut annulus grout, then deformation of the ground around the tunnel is kept to a minimum and surface settlement is controlled. The excavated volume that is not displaced by the lining and annulus grout is defined as volume loss.

For Reference Only
See disclaimer on cover page.

- Continuous annulus grouting through the tailseal is required whenever the EPBM is moving forward. Annulus grouting will be carried out through the tail using a two-part grout, capable of controlled set time. The grouting will be pressure controlled and pressure limited, and both pressure and volume will be measured. The grouting volume and pressure shall be sufficient to fill the excavated annulus. Additional grouting may be required in areas of over-excavation.

4.3.2.13 Tunnel Spoil

The characteristics and behavior of the tunnel spoil will be influenced by the characteristics of the soil in-situ and by the operation of the TBM and related systems like soil conditioning. The Contractor has accepted the use of a conveyor for spoil removal from the tunnel and must also accept responsibility for soil behavior during mucking, hauling and disposal. Sticky clays may adhere to the surfaces of spoil handling equipment. Improper use of soil conditioner may result in a fluid behavior of the spoil causing spills and rehandling before hauling offsite for disposal. Insufficient face pressure can lead to excessive groundwater mixed with the spoil. The Contractor is responsible for TBM operation in a way that produces spoil which is compatible with the Contractor's work plan for spoil disposal and can be handled efficiently by the Contractor's spoil handling equipment.

For estimating purposes, the Contractor should anticipate that the required use of additives for EPB tunneling will create handling and disposal issues. Upon removal from the tunnel, additive-modified spoils will be wet and should be expected to require time and effort to drain in settling basins or tanks prior to off-site transport. The Contractor is responsible for storing and modifying the spoil once removed from the tunnel so that it can be lawfully disposed of off-site. The Contractor is also responsible for the segregation of contaminated spoil and for the disposal of construction water The spoil is expected to contain residues from the additives and have little value as engineered fill and/or clay cap. Foam-conditioned spoil will need to be stored and mixed with dry spoil to create a manageable consistency for hauling and disposal. In addition to the additive residue, the water excavated along with the spoil should be expected to require treatment to correct pH and oxygen level.

The chemical properties of the by-products of spoil conditioners must be evaluated and receivers of tunnel spoil must be made aware of their concentration in the spoil.

For Reference Only

See disclaimer on cover page.

4.4 Excavation Interfaces

4.4.1 Excavation Interface Between Slurry Walls and Bored Tunnel

It is important that the ground at the transition from tunnel to structure through slurry walls and vice-versa is stable so the launch, reception, or drive through of the TBM can be performed without incident. To this end, three of five principal techniques will be employed to improve conditions in each of the transition zones prior to tunnel excavation by TBM:

Jet Grout Blocks: At all interfaces, blocks of soil, as indicated on the individual Contract Drawings for structures in the immediate interface behind the slurry walls, shall be replaced by cement with admixtures, employing jet grouting techniques from the ground surface. Admixtures shall be used to modify, or replace the cement content to limit the strength of the jet grouted materials, while achieving the specified test values for uniaxial compressive strength of recovered cores and permeability. Supplementary grouting may be required, if core sampling to determine the effectiveness of the jet grouting indicates remnant paths.

Tunnel Break-Out "Eyes": At all interfaces (except for TBM reception break-out into the Diridon Arena Station), "Tunnel eyes" will be created by the incremental systematic break-out of concrete and removal of rebar steel cast during slurry wall fabrication. Break-out of the slurry wall shall be undertaken only when the base construction slab has achieved the requisite specified strength. The "eyes" are dimensioned as noted on the Contract Drawings to accommodate TBM excavation, segmental liner placement and annular grouting.

Tunnel Seals: At the portals and stations (except for TBM reception break-out into Diridon Arena Station), mechanical "tunnel eye" seals will be located where the EPBMs break out of or into a structure, during launch and reception as indicated on the Contract C330 Drawings, to prevent water inflows, and to enable the EPBMs to commence excavation in closed mode with the required face pressure. The system is also designed to enable the EPBMs to allow for driving out of the jet grouted zone under steadily reducing levels of pressure until the bulkhead wall opening is reached with zero pressure. The cutterhead gauge cutters may be removed at the Contractor's option before passing through the seals.

Tunneling Through the Shafts: At ventilation shaft structures, the interface will not include a mechanical seal. Following placement of the invert slab, as "tunnel eyes" are incrementally excavated in the slurry walls, the shaft and "eyes" will be backfilled with layers of flowable cellular concrete to the specified elevation prior to the arrival of the EPBMs. After filling the shaft, above the concrete backfill, with water to

For Reference Only

See disclaimer on cover page.

a level equal to the external groundwater, machine boring with lining erection may take place without interruption between the shaft walls.

TBM Reception "Can": At the west slurry wall of Diridon Arena Station, due to limitations on the extent of the jet grouted block, as a consequence of the proximity of surface railroad tracks, a unique interface construction sequence shall be adopted requiring:

1. Replacement of the steel reinforcement cage within the slurry wall with fiberglass rebar in the area of the tunnel eye, as indicated on the Contract C323 Drawings.

2. The bolting of a steel reception "can" backfilled with low strength cellular concrete to the exposed internal slurry wall face following excavation of station shell and placement of the invert slab.

This will enable the TBM to mine out into the concrete "can" with the construction and grouting of the segmental liners under the required face pressure.

Thrust Frame and Cradle: A TBM thrust frame is required to transmit the thrust forces from the TBM to the invert of the excavations at each portal location and where the TBM breaks out from the station excavations. The thrust frame allows the TBM to commence excavation in closed mode immediately. Cradles are required at each location where the TBMs break in and out of structures and at stations where the TBMs skid through the stations.

Advancing TBMs through Stations and Ventilation Shafts: The sequence of breaking in and out of the cut-and-cover structures is shown on the Contract C330 Drawings.

The successful interface construction will require that each of the component techniques are built to the contract specified requirements to ensure the overall designed performance.

The Contract C330 Tunnel Contractor is responsible for the construction interface between the portals and bored tunnel as indicated in the Contract C330 Specifications and has limited responsibility for the construction interfaces at the stations and ventilation structures as indicated in the Contract C321, C322, C323, C380 and C390 Specifications.

4.5 Mined Cross Passages

A standard 11 foot diameter cross passage will be used at 27 locations. Two cross passages incorporate sumps and the remaining four cross passages are enlarged to 13 feet diameter to accommodate telephone switch panels and other equipment. The cross

For Reference Only

See disclaimer on cover page.

passages extend between running tunnels that are nominally approximately 16.5 feet apart measured from outside to outside of lining at axis level.

4.5.1 Site Conditions and Anticipated Behavior

The range of anticipated conditions at the cross passage locations has been characterized in terms of the soil types described in Table 4-7 for purposes of evaluating the required ground treatment prior to break-out and the required excavation and support design following breakout.

Table 4-7 Classification of Soils for Cross Passage Ground Treatment and Tunneling

Type	Tunneling Classification Prior to Ground Treatment	USCS Soil Types	Lowest Average Stability Ratio for Any Cohesive Soil Layer Greater than or equal to 1 foot thick	Permeation Groutable?
F1	Flowing	Sands and Gravels, non-cohesive silts and organic soils	N/A	Yes
F2			N/A	No
C1	Slow raveling	Clays and cohesive silts	Ns < 4	No
C2	Squeezing		Ns => 4	No

4.5.2 Design Considerations

The basic elements of the design are shown on the Contract C330 Drawings. The cross passages will be constructed from the running tunnels. Jet grouting at the locations shown on the Contract C330 Drawings must be performed from the surface prior to excavating the running tunnels. Pre-excavation investigation work will be performed from either or both running tunnels at the Contractor's option. Permeation grouting must be performed from both tunnels semi-concurrently, and VWP dewatering from the tunnel from which the cross-passage will be mined.

Break out from the pre-cast segmental lining will be achieved after the required investigations, by removing preformed steel framed breakout panels from the running tunnel segments at the cross passage junction location. A temporary steel support frame as shown on the Contract C330 Drawings will be installed in each running tunnel to accept the ground loading prior to removing the breakout panels for the excavation of the cross-passage. Constraints are specified on the rotation of the special break-out rings at cross passages. Flexibility in the orientation of the cross-passage excavation is incorporated into the design to accommodate stagger between the tunnel linings.

Three excavation and support basis of quantities options are indicated depending on the anticipated face stability and stand-up time of the soil in the vicinity of the cross passage excavation after the requisite ground treatment is completed.

For Reference Only

See disclaimer on cover page.

The permanent waterproofing and lining of the cross passages will consist of a PVC waterproofing membrane and cast in place reinforced concrete.

4.5.2.1 General Ground Treatment Design Considerations

Treatment method options are summarized in Table 4-8. The performance and minimum technical requirements of each treatment method are shown and specified in the construction contract. It is the contractor's responsibility to achieve the specified performance requirements by implementing the treatment options. Flexibility incorporated into the design allows the Contractor to varying the number of holes drilled, volume and pressure of grout injections, and cycles of grouting.

Vacuum wellpoint dewatering used in conjunction with permeation grouting as required to reduce permeability and pumping rates is the basis for stabilizing the F1 and F2 soils that are not jet grouted. Due to expected variability in grouted soil, the design shown on the Contract C330 Drawings does not rely on any strength gain provided by grouting alone, though some short term strength gain is anticipated.

For Reference Only

See disclaimer on cover page.

Table 4-8 Ground Treatment Options

Option	Ground Treatment Description	Performance Goal for Soil in Face and Within Eight Feet of Excavated Perimeter	Excavation Options
Probe and Sample Only	Probing and sampling conducted through the removable running tunnel lining breakout panel using specialized drilling methods	Confirm stable soils	Heading & bench
Vaccuum Well Point (VWP)	Vacuum dewatering for control of potentially flowing sand and silt layers and lenses in otherwise firm to slow raveling soils or in low head conditions	Reduce piezometric pressure to atmospheric or less	Pilot tunnel, Heading & bench
Permeation Grouting (PG) & VWP	Pressure grout injection through multiple sleeve port pipes	Reduce soil permeability	Heading & bench
Jet Grouting (JG)	Jet grouting from surface	Reduce permeability and replace the ground	Full face

Notes:

1. Refer to Figure 3-1 for description of ground condition units

The bid quantities for the number of cross passages to receive each type of ground treatment have been developed using the following guidelines:

- Jet grouting is called for at locations where high permeability granular soils are anticipated in the face and/or within eight feet of the invert of the cross passage excavation.

- The option to move directly from probing and sampling to the break-out will be taken when it is confirmed that stable cohesive soils are present in the face of excavation and in a zone extending at least eight feet radially beyond the excavation perimeter.

- Vacuum wellpoint dewatering will be used whenever silty to clayey sands and gravels, silts or grouted soils of equivalent permeability are present in the tunnel face or within eight feet radially beyond the excavated perimeter.

For Reference Only

See disclaimer on cover page.

- Permeation grouting through sleeve port grout pipes and VWP will be used for pressure injection in poorly to well-graded sands and gravels.

The aforementioned guidelines have been employed to evaluate the anticipated conditions at each cross passage shown on GBR Drawings C308-C341 in order to develop bid quantities. Table 4-9 summarizes this evaluation and Table 4-10 provides ground improvement bid quantities.

For Reference Only

See disclaimer on cover page.

Table 4-9 Cross Passage Ground Improvement and Excavation Support Basis of Quantities

SVRT Class	Ground Improvement	Excavation and Support
1	VWP	Heading & Bench
2	Jet-grout	Full Face
3	Probe only	Heading & Bench
4	Probe only	Heading & Bench
5	Probe only	Heading & Bench
6	Probe only	Heading & Bench
7	Probe only	Heading & Bench
8	Probe only	Heading & Bench
9	Probe only	Heading & Bench
10	PG & VWP	Heading & Bench
11	PG & VWP	Heading & Bench
12	PG & VWP	Heading & Bench
13	PG & VWP	Heading & Bench
14	Jet-grout	Full Face
15	PG & VWP	Heading & Bench
16	VWP	Heading & Bench
17	Jet-grout	Full Face
18	VWP	Pilot Tunnel
19	Jet-grout	Full Face
20	Jet-grout	Full Face
21	VWP	Pilot Tunnel
22	Jet-grout	Full Face
23	Jet-grout	Full Face
24	Jet-grout	Full Face
25	Jet-grout	Full Face
26	Jet-grout	Full Face
27	Jet-grout	Full Face
28	Jet-grout	Full Face
29	Jet-grout	Full Face
30	VWP	Pilot Tunnel
31	Jet-grout	Full Face
32	Jet-grout	Full Face
33	Jet-grout	Full Face

Table 4-10 Cross Passage Ground Improvement Bid Quantities

Type of Ground Improvement	Bid Quantity
Vacuum Well Point (VWP)	5
Jet-grout	16
Probe only	7
Permeation Grout & VWP	5

For Reference Only

See disclaimer on cover page.

The Contractor should not assume that the interpretation of these drawings represent a fixed design for each cross passage location. These guidelines are anticipated to be used by the Contractor and Construction Manager during tunneling to assess the results of probing and sampling and to select which in-tunnel ground treatment approach best suits each location. Additionally, the average quantities summarized in Table 4-11 shall apply as baseline averages.

Table 4-11 Permeation Grouting & Vacuum Wellpoint Ground Treatment Options

Item	Average Quantity per Cross Passage	Remarks
Permeation Grouting	**17**	From each tunnel semi-concurrently
Outer Hole Sleeve Port Grout Pipes	10	
Inner Hole Sleeve Port Grout Pipes	7	
Vacuum Wellpoints	**14**	From the tunnel which the cross-passage will be mined
Long Wellpoint	7	
Short Wellpoint	7	

4.5.2.2 Probing, Sampling, Instrumentation and Monitoring

Probing and sampling are required to confirm the anticipated conditions with respect to the effectiveness of jet grouting and for the implementation of proposed ground treatment measures prior to breakout. Probing and sampling shall be performed at all the cross passages under the full range of grout, soil and groundwater conditions. The associated equipment and drilling requirements shall be as shown on the Contract C330 Drawings and as specified.

As described in the Contract C330 Specifications, the drilling, probing, and sampling equipment and methods used for probing and sampling shall be designed and executed by a Specialty Geotechnical Drilling and Sampling Subcontractor to be compatible with the anticipated soil and groundwater conditions, borehole locations, lengths, and orientations. The associated equipment and methods for probing and sampling shall also be compatible with the Contractor's concurrent tunnel operations and available space and associated clearances. A specialist geotechnical consultant shall be retained to review the design of the testing program, and to supervise the testing, soil classification and interpretation of ground behavior."

Groundwater geotechnical instrumentation is required at all of the cross passages in the completed ground probing and sampling boreholes to monitor and confirm the effectiveness of ground treatment and/or

For Reference Only

See disclaimer on cover page.

performance criteria for lining removal and breakout. Geotechnical instrumentation generally consists of conventional plain and screened PVC pipe piezometers and vibrating wire piezometers.

Structural and ground response geotechnical instrumentation shall be installed in the TBM-bored tunnels and in the soil above the cross passage excavations at all of the cross passages. Geotechnical instrumentation in the TBM-bored tunnels and cross passages consists of optical survey target arrays and vibrating wire strain gauges on the temporary steel support frames. These arrays shall monitor the inward radial deformation or convergence response of the TBM-bored tunnel lining. Geotechnical instrumentation in the soil above the cross passage excavations shall consist of multiple-point borehole extensometers (except at cross passages constructed with jet-grouting) and ground settlement monitoring points. The extensometers shall monitor vertical ground deformation at multiple points in the soil strata directly above the cross passage excavations.

4.5.2.3 Vacuum Wellpoint Dewatering Design Requirements

Vacuum wellpoint dewatering is required to reduce hydraulic gradients and groundwater seepage forces to protect against soil piping-induced instability. Vacuum wellpoint dewatering is intended for soil pore water pressure reduction or depressurization only and not for groundwater table lowering.

Vacuum wellpoint systems are designed to provide a minimum clearance between adjacent wellpoint screens and cross passage excavation periphery to ensure an adequate vacuum seal for effective wellpoint operation and function. Details are shown on Contract C330 Drawings and the interpretive soil profiles drawings included with the GBR. Effective function is defined as a minimum of 7.36 pounds per square inch (gauge) (psig) vacuum in the wellpoint.

Both short and long wellpoints make up the complete array of possible wellpoint positions and locations shown on the Contract C330 Drawings, though only some of these wellpoints are anticipated to be required at each location depending on the soil conditions as summarized in Table 4-11. A single, full-length, design vacuum wellpoint consists of short and long wellpoints with screened lengths. Short and long wellpoints are required to enhance vacuum effectiveness of each vacuum wellpoint as excavation advances.

Vacuum wellpoint systems shall be designed with a five foot vacuum radial influence zone for groundwater capture and soil pore water pressure reduction (depressurization). To ensure effective groundwater control, vacuum wellpoint dewatering systems, limits are specified for maximum

For Reference Only

See disclaimer on cover page.

allowable circumferential wellpoint center-to-center spacing between short and long wellpoints.

The intended basis of design relies on the development of effective cohesion due to negative pore pressure derived from the applied vacuum. The effectiveness of the vacuum wellpoints must be verified by vibrating wire piezometers(s) installed in the A-holes shown on the Contract C330 Drawings. Evidence of achieving the design performance includes intercepting all seepage before it enters the excavation based on observations during cross passage excavation advance.

Vacuum wellpoint screened length slotted pipe total open area shall be designed to limit flow velocities into the wellpoints (through the slots) to less than 0.20 feet per second for a wellpoint design inflow of 15 gallons per minute (gpm) in each short and long wellpoint. The flow velocity limitation is required to reduce head losses, minimize incrustation, and protect against high flow velocities that could promote fines removal and soil disturbance. The filter pack and wellpoint screen slot width is required to develop an effective filter and protect against fines removal around the wellpoints and washing of soil particles into the groundwater influent.

Relatively cohesionless soil layers less than three (3) feet thick require a minimum of two long and two short VWP's at each peripheral location to provide effective drainage and redundancy.

The Contractor shall allow, in its bid and construction operations, for the installation and associated schedule delays, construction costs, and downstream productivity impacts of supplemental measures to enhance the effectiveness of groundwater control systems, including vacuum lances and shotcrete. These supplemental measures will be reimbursed separately according to the bid schedule and measurement and payment provisions.

Vacuum lances are spot wellpoints installed at targeted locations, in the ground around the excavation periphery and heading, to intercept excess water seepage flows into the excavations and reinforce vacuum wellpoint groundwater control and maintain excavation stability. Vacuum lances consist of rigid PVC plain and slotted pipe connected to a vacuum header pipe and vacuum pump designed by the Contractor for groundwater control. The number of vacuum lances will depend on the ground conditions and the effectiveness of the grouting and vacuum wellpoint systems. For bidding purposes, the Contractor shall expect the installation, operation, and maintenance of up to six vacuum lances each installed up to three times at various points along each of the cross passage excavations during excavation advance.

For Reference Only

See disclaimer on cover page.

Shotcrete is sprayed concrete coating on the excavation periphery and heading designed by the Contractor to enhance the vacuum seal of vacuum wellpoints for improved groundwater control at the CP excavation heading and periphery. Shotcrete is not intended to provide structural support and will require appropriate water drainage and diversion to be installed successfully as described in the Contract C330 Specifications.

4.5.2.4 Permeation Grout Design Considerations

To be provided

4.5.2.5 Jet Grout Design Considerations

The specified compressive strength of the jet-grout is indicated in the Contract C330 Specifications.

To ensure efficient excavation by drag pick equipped EPBM and mechanical hand tools such as hydraulic and pneumatic hammers and spade tools, the jet grout mix design should incorporate approved fillers and admixtures that limit and control strength gain to facilitate cross passage excavation.

Target zone coverage of jet-grouted mixed soil and cement material have been designed to provide a uniform cover thickness, as shown on the Contract C330 Drawings, around the full perimeter, depth, and length of the cross passage excavations and associated sump excavations, regardless of soil type. The design coverage is required to maintain stable excavations with sufficient redundancy for heterogeneity of the jet-grout due to the presence of cohesive soil layers and lenses.

The jet-grouted material at cross passage locations shall extend across the entire TBM heading, as shown on the Contract C330 Drawings. The full TBM-bored tunnel heading coverage is required to protect against a mixed face condition that can result in over-mining of flowing cohesionless soils and resulting ground subsidence while the TBM is penetrating through the relatively harder jet-grout.

The jet grout coverages shown are not sufficient for use as TBM intervention chamber for safe personnel access in front of the TBM cutterhead. If a TBM excavation chamber intervention is required, the Contractor shall specifically design the intervention chamber to suit its required operations based on the site-specific ground conditions.

Jet grouting is designed to replace flowing soil and associated ground behavior conditions with conditions required to maintain excavation stability.

For Reference Only

See disclaimer on cover page.

The number of jet grout holes shall be based on the required coverages, as shown on the Contract C330 Drawings, and the performance of each allowable jet grouting method as demonstrated by the jet grouting test trials in each of the representative soil materials. The design jet hole spacing shall be selected by the Contractor and its Specialty Subcontractor based on its proposed jet grouting method, anticipated soil conditions described in this GBR and the GDRs.

Supplemental groundwater control measures, such as vacuum lances may be necessary during excavation. For bidding purposes, the Contractor shall assume the installation of up to six vacuum lances per cross passage excavation.

Verification of the effectiveness of jet grouting shall be performed using the procedures described in the Contract C330 Specifications for material sampling for compressive strength testing and the permeability performance criteria.

4.5.2.6 Initial Support Design Considerations

For bidding purposes, preliminary quantity estimates for SVRT Class have been established based on generalized interpretive soil profiles of each cross-passage shown on GBR Drawings C308 - C341. Table 4-12 provides ground excavation sequence bid quantities.

Table 4-12 Cross Passage Ground Excavation Sequence Bid Quantities

Type of Excavation Sequence	Bid Quantity
Heading & Bench	14
Pilot Tunnel	3
Full Face	16

4.5.2.7 Final Lining Design Considerations

To be provided.

4.5.3 Cross Passage Construction Considerations

Construction of many of the cross passages will be carried out concurrently with the excavation of the running tunnels by TBM. This will require careful planning and scheduling.

4.5.3.1 General Construction Sequence

The following construction sequence is envisioned:

1. During TBM mining, the contractor will install special rings as part of the lining for the twin running tunnels at cross-passage

For Reference Only
See disclaimer on cover page.

locations. The cross-passage openings may vary by up to 2.5 feet from the design position because of creep of the lining resulting from variation in the gap between each ring. The cross passages will be skewed due to the relative position of the special rings in each running tunnel.

2. After the TBM and trailing gear clear the cross passage location, a geotechnical sampling and monitoring investigation will be performed, consisting of horizontal and inclined boreholes advanced from the tunnels as shown on the Contract C330 Drawings.

3. The Contractor is responsible for the design, implementation and performance of the ground improvement to meet specified criteria for break-out. Designs for jet-grouting permeation grouting and vacuum well-point dewatering are required to be prepared under the direction of and stamped by a California Professional engineer and reviewed for contract consistency by the Engineer.

4.5.3.2 Drilling Through Precast Concrete Tunnel Lining (PCTL)

Boreholes are required to be drilled through the PCTL at segment dimples to avoid the rebar in the faces of the tunnel lining and within the angular tolerance shown on the Contract C330 Drawings. PCTL boreholes shall be drilled using rotary diamond coring methods to minimize lining disturbance and avoid damage to the lining. Steel reinforcement in the PCTL on the inside face shall not be cut for any reason.

All boreholes drilled through the PCTL and in the soils outside of the PCTL must be advanced through Blow-Out Preventers (BOP) mounted into collar inserts embedded in the PCTL due to a water pressure head of up to three bars in the ground surrounding the cross passage excavations. Cohesionless soil layers shall be considered to be hydraulically connected to the overlying aquifer due to soil layer continuity. BOP's are required at all times during construction and to prevent inflows of soil and groundwater during borehole drilling from the ground outside the PCTL.

4.5.3.3 Probing, Sampling, Instrumentation and Monitoring

Probing and sampling are necessary for confirmation of anticipated conditions including effectiveness of jet grouting, or implementation of additional ground treatment measures prior to breakout. Temporary piezometers will be designed, installed, and supervised by a qualified consulting engineer, who will maintain involvement during construction.

Borehole drilling methods shall provide and maintain continuous borehole stability against borehole collapse due to soil and groundwater inflows

For Reference Only

See disclaimer on cover page.

along the entire borehole length and at the borehole heading at all times during probing, sampling, casing extraction, and installation of grout pipes, wellpoints, vibrating piezometers, and other borehole tools and instrumentation. Drilling methods shall be designed for successful undisturbed installations and completion of the work with one borehole for each probing, sampling, and installation location.

Soils shall be cut during drilling by mechanical means and methods only. Drilling fluids shall only be used for borehole heading support and cuttings transport from the borehole during advancement and borehole stabilization during temporary drill casing extraction.

As described in the Contract C330 Specifications, the drilling, probing, and sampling equipment and methods used for probing and sampling shall be designed and executed by a Specialty Geotechnical Drilling and Sampling Subcontractor to be compatible with the anticipated soil and groundwater conditions described here, borehole locations, lengths and orientations. The associated equipment and methods for probing and sampling shall also be compatible with the Contractor's concurrent tunnel operations and available space and associated clearances. A specialist geotechnical consultant shall be retained to review the design of the testing program, and to supervise the testing, soil classification and interpretation of ground behavior.

4.5.3.4 Wellpoint Installation

Due to the expected ground conditions as characterized in this GBR, vacuum wellpoint boreholes shall be drilled using temporary drill casing with lost points for borehole support and to protect against flowing (blow-back) of soils into the casing during wellpoint drilling and installation.

Wellpoint installation may be necessary in either grouted or ungrouted soil.

Vacuum wellpoints and temporary PCTL prop installation should be sequenced to protect the wellpoints and ensure the structural integrity of the temporary prop structural system.

A maximum duration of 20 days of continuous vacuum wellpoint system operation with all vacuum wellpoints at the required vacuum, as described in the Contract C330 Specifications, shall be expected and accounted for by the Contractor and its subcontractors to develop a steady-state vacuum and reduce soil pore water pressures.

The effectiveness of the vacuum wellpoint system shall be verified as described in the Contract C330 Specifications prior to opening of the PCTL or breakout into cross passage excavation.

For Reference Only

See disclaimer on cover page.

The vacuum wellpoint system and all of the associated vacuum wellpoints shall be operated at all times during cross passage construction through the end of cast-in-place concrete lining until lining has achieved design strength.

The baseline wellpoint inflow rate is 15 gpm for each short and long wellpoint.

4.5.3.5 Jet Grouting Operations

At the specified cross-passage locations, jet-grouting must be performed prior to TBM-bored tunnel excavation and PCTL installation so the precast concrete segment tunnel lining is not subjected to the extremely high injection pressures utilized in the jet-grouting process.

Maintain the stability of all jet grouting boreholes to provide an unimpeded return of jet grouting spoils (soil, water, and cement) at all times during jet grouting construction operations. Borehole stability is required to ensure positive relief of injection pressures and protect against ground displacement. Additional vent holes should be drilled to ensure venting. Based on the geotechnical investigation drilling observations and the anticipated soil and groundwater conditions described in this GBR, the use of open boreholes using drilling fluid for borehole support will not maintain a continuously stable, open borehole during drilling and jet grouting. Angled boreholes are required for jet grouting operations and will require casing for stabilization.

Jet-grouting involves the injection of fluids under high pressures. Jet grouting uses these highly pressurized fluids and suspensions to erode and replace in-situ soils with cement slurry. Jet grouting also temporarily liquefies the newly-formed mixed soil and cement material prior to its curing and hardening.

All jet grouting operations shall not result in displacement (heave, settlement, and lateral movement) of the ground sufficient to damage surface structures, underground structures, or underground utilities. The Contractor must design the jet grouting operation sequence adjacent to buildings, highway bridge and viaduct substructure foundations (including, but not to be limited to State Route 87 adjacent to cross passage CP19) to protect against the effects of temporarily softened ground adjacent to structures due to jet grouting operations.

Jet grouting shall be performed from the surface only. Much of the jet-grouting will be performed in congested urban areas and will require special provisions for maintenance and protection of vehicular and pedestrian traffic and spoil containment as described in the Contract C330 Specifications.

For Reference Only

See disclaimer on cover page.

Minimize surface disruption and impacts to pedestrian and vehicular traffic due to jet grouting operations by drilling angled boreholes from the surface confined locations to achieve the jet grout target zone coverages as shown on the Contract C330 Drawings. For the purposes of bidding, the Contractor shall expect 90% of the installed boreholes to be inclined from vertical.

All spoils (100% of the volume and mass of water, soil, cement, and grout) generated by jet grouting operations, including spoil return from the boreholes, shall be contained and intercepted at the surface to prevent infiltration via surface transport or runoff into roadways, pedestrian walkways, sewers, catch-basins, storm drains, and waterways. All jet grouting operations shall be outfitted with barriers, curtains, and tarpaulins to protect and shield the adjacent public, buildings, vehicles, and pedestrians against incidental air, water, and cement grout spray and spoil runoff. 100% of the spoil return volume shall be contained and vacuumed into a disposal vehicle immediately upon return to the surface at all times during jet grouting construction.

The spoil return volume will be up to 160% of total injected volume of water and cement.

4.5.3.6 Cross-Passage Opening Propping

The sequence of construction is shown and specified in the Contract Documents. Probing, sampling and permeation grouting work should have no conflict with the required temporary steel support frame since this work can be finished before the support frame is needed. Vacuum dewatering will be ongoing during cross passage construction and must therefore be planned to avoid conflict with the bracing.

4.5.3.7 Excavation and Support Sequence

To be provided.

4.5.3.8 Groundwater Control and Discharge from Cross Passage Work

Water encountered during the dewatering and excavation work must be treated and discharged in accordance with applicable permits. For baseline purposes, assume an average groundwater inflow per cross passage of ____ gallons per day. Values shall be averaged over the duration of work at each cross passage beginning with installation of the first vacuum well and ending once excavation and liner plate installation is complete. Residual inflows during waterproofing and concrete placement are also present.[b34]

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

5.0 Surface Excavations

5.1 General

Surface excavation is planned for construction of the underground stations, cross over, portals, and ventilation shafts. The anticipated subsurface conditions and their influence on design and construction of these excavations are described here.

5.2 Anticipated Subsurface Conditions

Interpretive soil profiles within the open excavations are shown on GBR Drawings C301 through C307 and generally correspond to the hydrogeologic Confining Layer described in Section 3.3.

For each open excavation, "construction" and "permanent" design water levels were developed for the ground water table, Confining Layer, Upper Aquifer, Major Aquitard, and the Lower Aquifer. The design water levels for construction as a function of excavation depth are shown in Figures 5-1 through 5-9 and were based on consideration of water level data and surface flooding evaluation. Further discussion of these values is provided in the Hydrogeology Report (HMM/Bechtel, 2008a).

Refer to Section 4.2.7 for details regarding possible unmarked abandoned wells and well casings that may be encountered by the surface excavations.

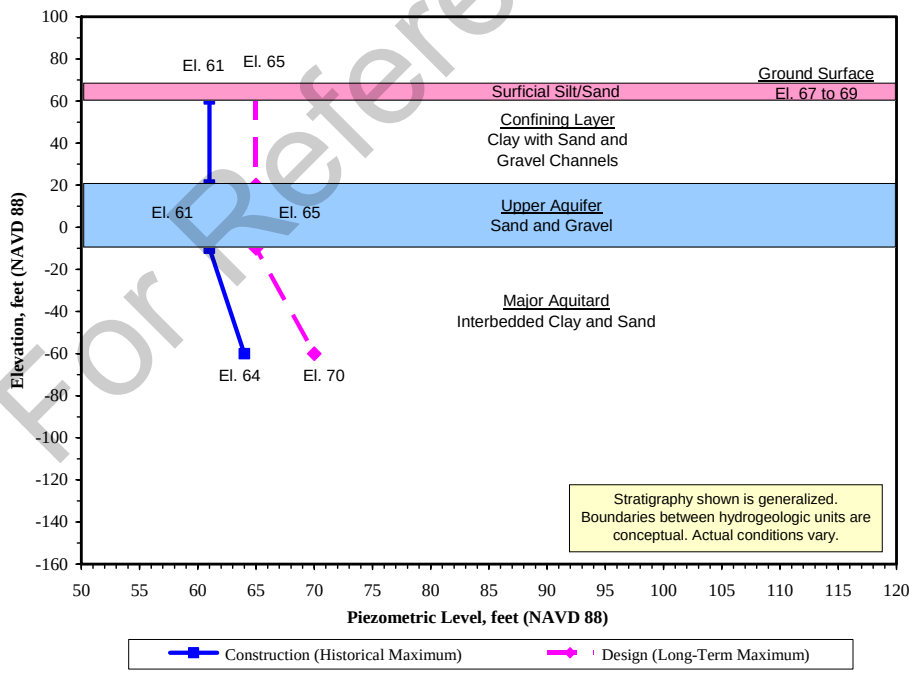


Figure 5-1 Construction and Design Water Levels for West Portal.

For Reference Only

See disclaimer on cover page.

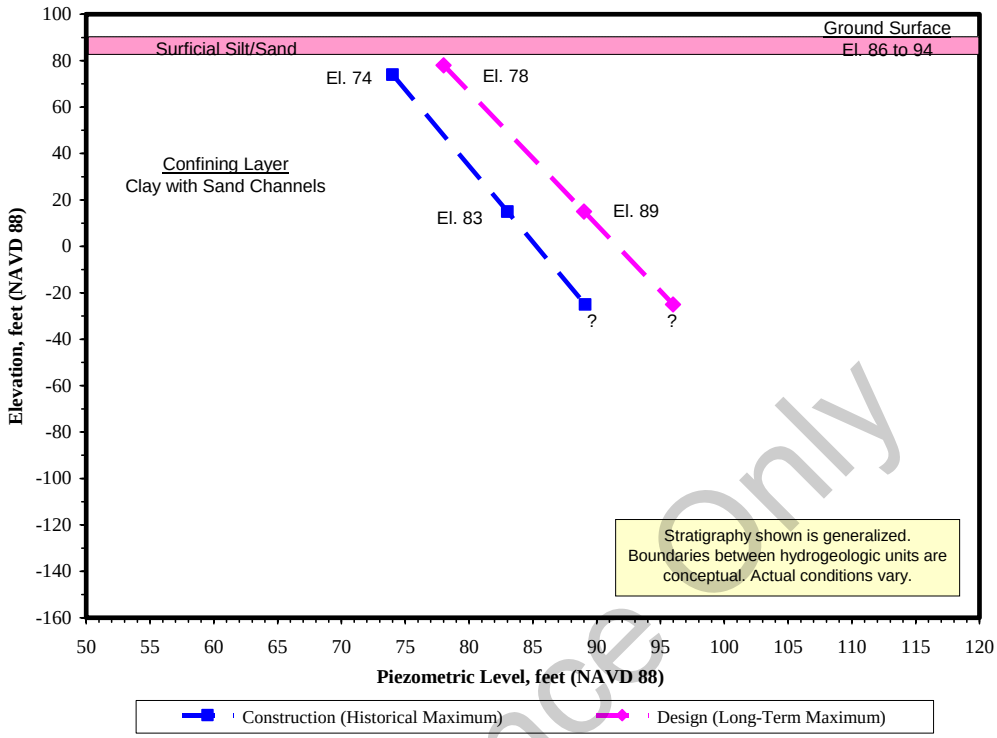


Figure 5-2 Construction Water Levels for East Portal U Wall.

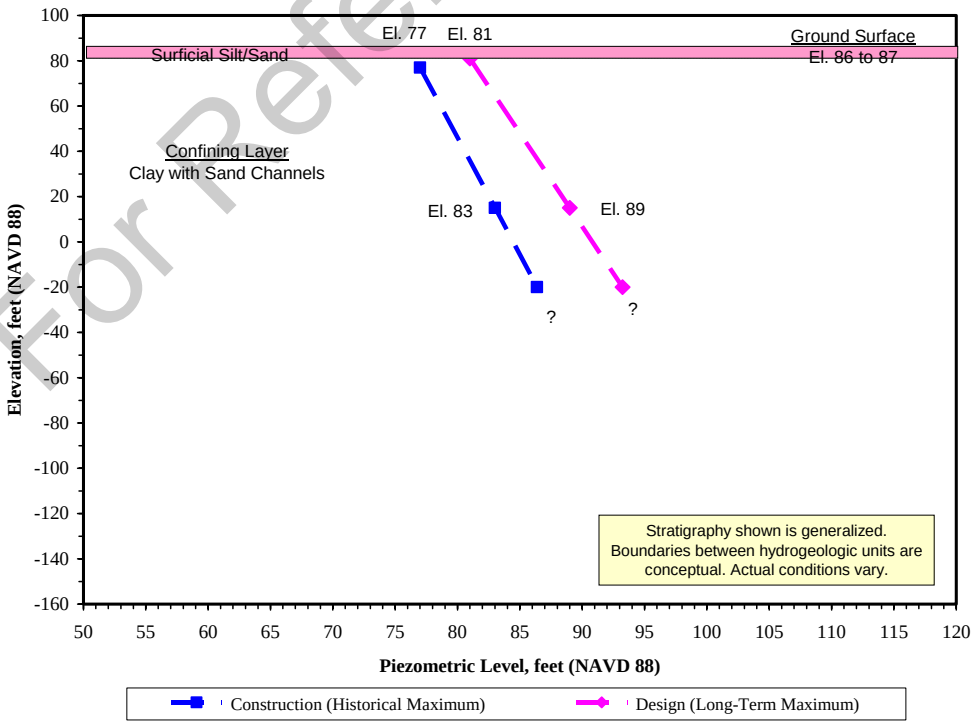


Figure 5-3 Construction Water Levels for East Portal Box Structure.

For Reference Only

See disclaimer on cover page.

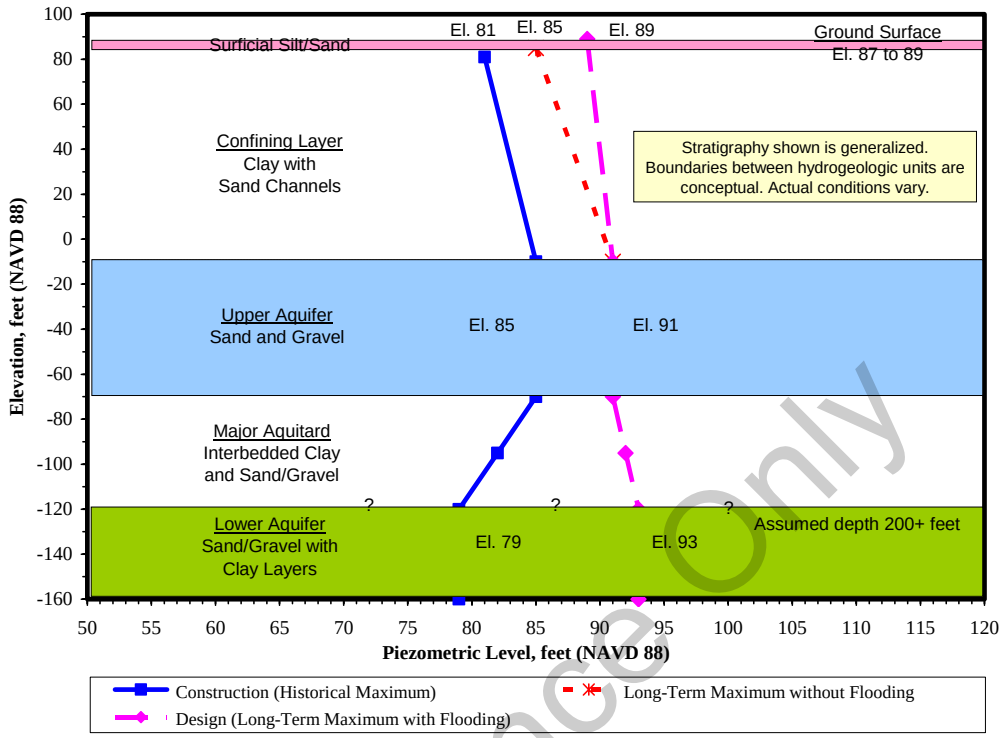


Figure 5-4 Construction Water Levels for Alum Rock Station North of STA 604+00.

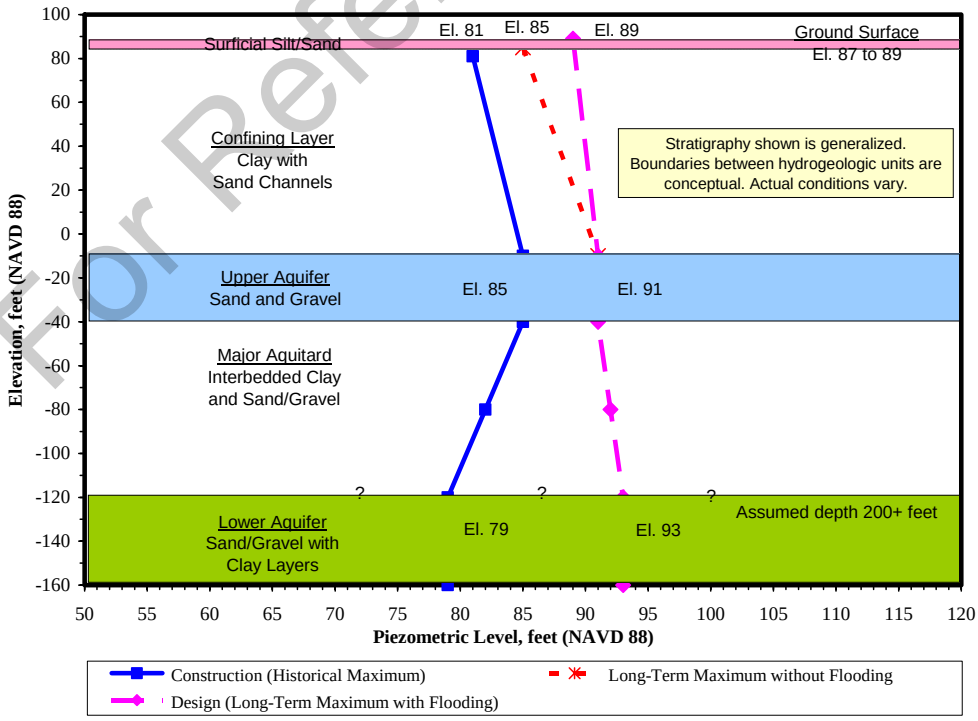


Figure 5-5 Construction Water Levels for Alum Rock Station South of STA 604+00.

For Reference Only

See disclaimer on cover page.

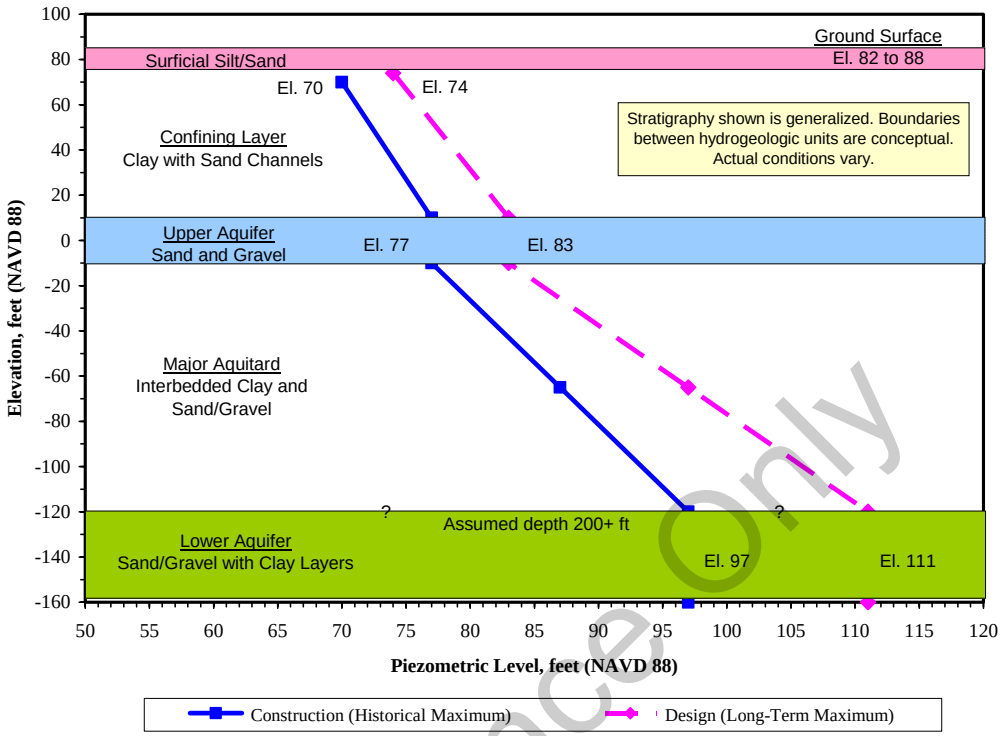


Figure 5-6 Construction Water Levels for DSJS and Crossover.

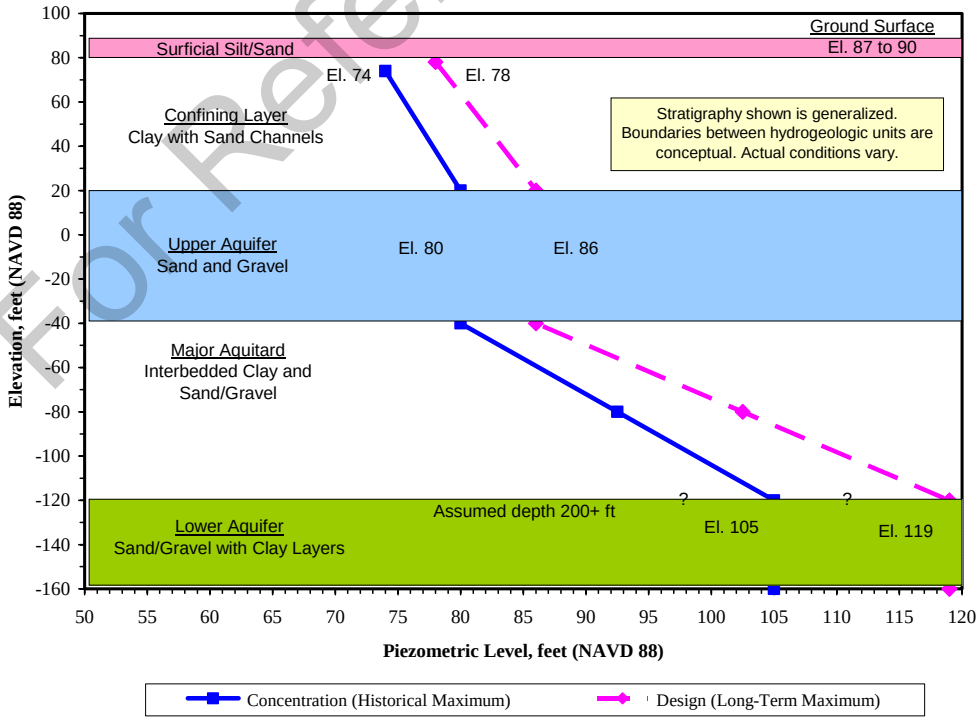


Figure 5-7 Construction Water Levels for Diridon/Arena Station.

For Reference Only

See disclaimer on cover page.

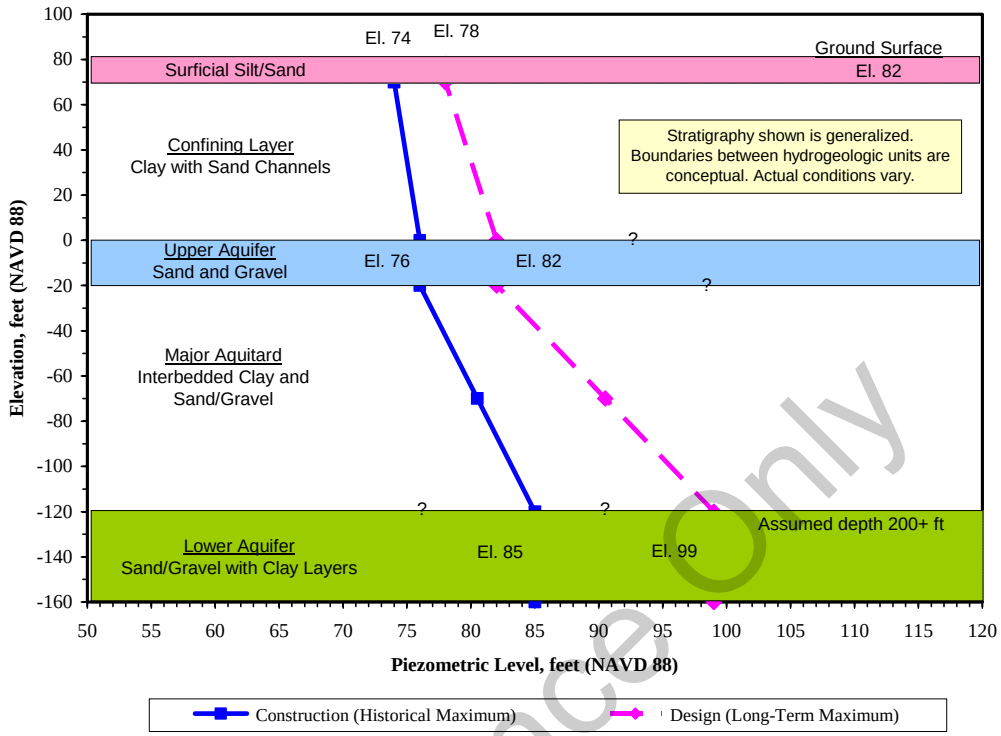


Figure 5-8 Construction Water Levels for Mid-Tunnel Ventilation Structure STS.

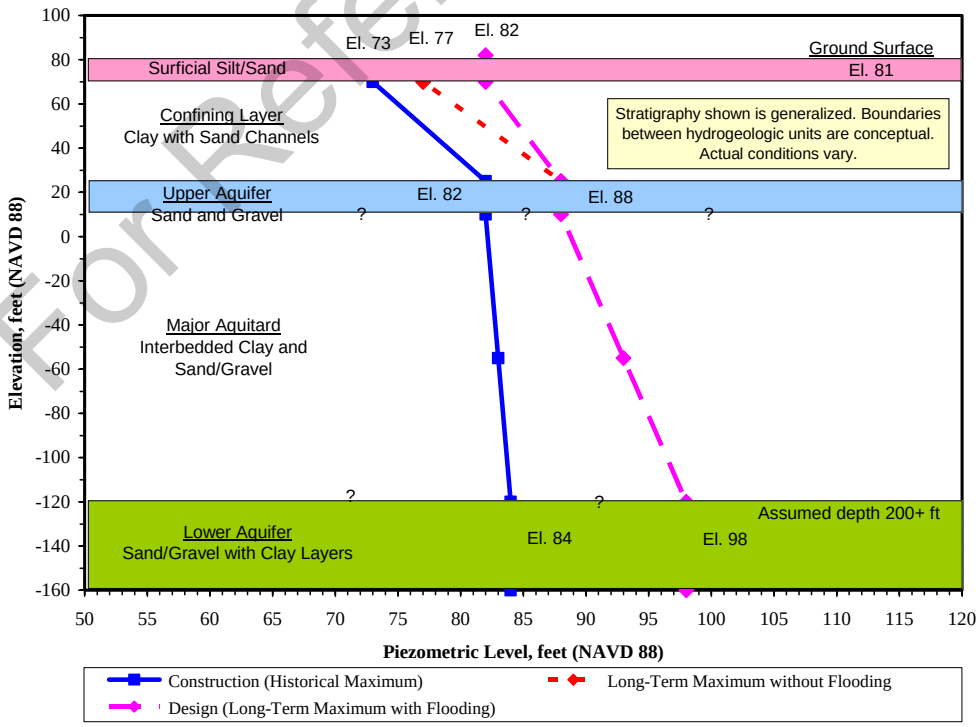


Figure 5-9 Construction Water Levels for Mid-Tunnel Ventilation Structure FSS.

For Reference Only

See disclaimer on cover page.

5.3 Basis of Design

5.3.1 Gas Classification

The drawings, geotechnical data and information on soil and groundwater contamination were provided to Cal/OSHA along with a request for an underground classification in accordance with Title 8, Tunnel Safety Orders of the California Code of Regulations. The subsurface excavations have received an underground classification of Potentially Gassy with Special Conditions from Cal/OSHA, a copy of which is included in Appendix B.

5.3.2 Groundwater Control

To prevent piezometric uplift pressures from creating uplift instabilities during construction, slurry cut-off walls were designed to minimum depths required for soils below base of excavation to counterbalance the uplift pressures. The minimum required depth is shown on the Contract Drawings. Monitoring is required during construction to confirm the ground being excavated is consistent with design assumptions. If the minimum depth shown on the Contract Drawings occurs at an elevation founded in granular material, the wall depth may need to be extended at the discretion of the Engineer so that a minimum of 10 ft of the wall is embedded into low-permeability cohesive soils. The maximum extension of wall depth directed by the Engineer will be ___ so the Contractor must have the capability to efficiently construct walls to these depths below the minimum shown on the Contract Drawings. Remedial curtain grouting below slurry walls can be implemented through the pre-installed casing shown on the Contract Drawings if inadequate groundwater cut-off occurs. For the majority of open cut excavations, the cut-off wall depths are within the Major Aquitard. The depth of the walls at each open excavation is summarized in Table 5-1.

For Reference Only

Table 5-1 Summary of Minimum Wall Depths and Baseline Maximum Pumping Rates

Location	Excavation Depth, feet	Minimum Wall Depth Range for Each Excavation, feet	Baseline Maximum Estimated Pumping Rate, GPM
East Portal	34	23 to 76	4
West Portal	37	8 to 99	32
Alum Rock Station	58	148 to 168	50
Downtown/ Crossover	62	141 to 149	60
Diridon/ Arena	64	142 to 149	66
East Mid-Tunnel Vent Shaft	71	195	10
West Mid-Tunnel Vent Shaft	86	200	16

See disclaimer on cover page.

Based on the design wall depths, minimal seepage is anticipated through the slurry walls and the cut-off layer that the walls toe into. Pumping from pressure relief wells will be required during construction to maintain the requirement that water heads in any sand or gravel layer beneath the base of excavation be kept at a minimum 10 ft below the base of excavation. For baseline purposes, assume that the maximum pumping rates required at each open cut excavation are as given in Table 5-1, and that there is a cushion of up at least __ hours following accidental pump shut-off before excavation mud slabs would become pressurized by groundwater. These quantities should not be increased if the slurry cut-off walls are constructed per specification, and the assumed cut-off layer is continuous.

5.3.3 Excavation Support

To support lateral pressures from soil, groundwater, surcharge, and seismic loadings, the slurry cut-off walls will be reinforced and will be shored by either internal bracing or tiebacks. For baseline purposes, assume that tiebacks with bond lengths in the upper 40 feet are capable of a minimum pullout capacity of 3 kips per foot and tiebacks with bond lengths below 40 feet are capable of a minimum pullout capacity of 4 kips per foot. Drill hole diameter and number of cycles of pressure grouting following installation are anticipated to be varied to achieve the required minimum pullout capacity over the bond length.

5.4 Construction Considerations

5.4.1 Slurry Wall Construction

Construction of slurry walls in mixed Unit 1 and Unit 2 materials will require special considerations. The presence of nested cobbles will create localized high permeability lenses that may cause loss of slurry and potential cave-ins if the bentonite concentration is not properly controlled and the slurry cake does not form on the wall of the slurry trench. Table 5-2 baselines the anticipated percentages of slurry wall to be excavated in cohesive and granular soils based on the interpretive soil profiles.

Additionally, groundwater and soil chemistry factors such as pH, conductivity, and contamination by salt, calcium, and organics affect the slurry by potentially creating conditions that may cause unstable bentonite slurry. The problem may relate to excessive viscosity, flocculation, slurry loss, and spalling of the excavated face.

For Reference Only

See disclaimer on cover page.

Table 5-2 Summary of Anticipated Soil Units for Slurry Wall Excavation

Structure	Slurry Wall	Anticipated Installation Conditions	
		Percent by Area	
		Cohesive	Non-cohesive
West Portal	East Side Wall	71%	29%
	West Side Wall	69%	31%
	Bulkhead Wall	50%	50%
West Mid-Tunnel Vent Shaft	East Side Wall	88%	12%
	West Side Wall	88%	12%
	North End Wall	88%	12%
	South End Wall	88%	12%
Diridon/ Arena Station	North Side Wall	79%	21%
	South Side Wall	79%	21%
	East End Wall	84%	16%
	West End Wall	73%	27%
	Walls of Entrance DA-1A	68%	32%
	Walls of Entrance DA-4A	79%	21%
Downtown San Jose Station and Crossover	North Side Wall	70%	30%
	South Side Wall	70%	30%
	East End Wall	77%	23%
	West End Wall	69%	31%
	Walls of East Appendage	79%	21%
	Walls of West Appendage	83%	17%
East Mid-Tunnel Vent Shaft	North Side Wall	85%	15%
	South Side Wall	85%	15%
	East End Wall	85%	15%
	West End Wall	85%	15%
Alum Rock Station	East Side Wall	78%	22%
	West Side Wall	78%	22%
	North End Wall	67%	33%
	South End Wall	55%	45%
	Walls of Entrance AR-1	91%	9%
East Portal	East Side Wall	100%	0%
	West Side Wall	100%	0%
	South End Wall	100%	0%

For Reference Only

See disclaimer on cover page.

5.4.2 Tieback Installation

Tiebacks will be installed in mixed soil materials at different elevations. Table 5-3 and Table 5-4 provide baselines for anticipated percentages of tiebacks to be drilled in cohesive and non-cohesive soil.

For sections of tiebacks drilled in non-cohesive soil, instability should be expected, requiring special techniques including casing and blow-out preventers. Non-cohesive soil will be encountered below the groundwater table, leading to the potential for flowing conditions. Finally, during installation of tiebacks, drilling may encounter nests of cobbles. For baseline purposes, assume that XX%, by length, of tiebacks will encounter nests of cobbles. Since it is not possible to reliably predict where unstable soils will be encountered by tie-back drilling, the special requirements applicable to non-cohesive soils must be implemented everywhere.

Table 5-3 Summary of Anticipated Soil Units for Tie-back Excavation below 40 ft

Structure		Tiers	Unfactored Design Load Range	Anticipated Installation Conditions	
			kips	*percent of holes*	
				Cohesive	Non-cohesive
Diridon	Side Walls	5	130-152	72%	28%
	West Bulkhead Wall	n/a	89-203	100%	0%
	East Bulkhead Wall	n/a	109-209	90%	10%
DTSJ	Side Walls	5	109-140	94%	6%
	West Bulkhead Wall	n/a	101-187	74%	26%
	East Bulkhead Wall	n/a	108-187	85%	15%
Alum Rock	Side Walls	5	124-173	95%	5%
	North Bulkhead Wall	n/a	101-187	98%	2%
	South Bulkhead Wall	n/a	108-187	62%	38%

For Reference Only

See disclaimer on cover page.

Table 5-4 Summary of Anticipated Soil Units for Tie-back Excavation above 40 ft

Structure		Tiers	Unfactored Design Load Range	Anticipated Installation Conditions	
			Kips	*percent of hole length*	
				Cohesive	Non-cohesive
West Portal	S1 831+20 to S1 832+20	5	82-118	95%	5%
	S1 832+20 to S1 834+50	4	77-107	61%	39%
	S1 834+50 to S1 836+00	3	95-121	87%	13%
	S1 836+00 to S1 838+50	2	112-113	100%	0%
	S1 838+50 to S1 840+00	1	103	100%	0%
	S1 841+00 to S1 844+00	1	78	100%	0%
Diridon	Side Walls	5	110-118	83%	17%
	West Bulkhead Wall	n/a	102-182	100%	0%
	East Bulkhead Wall	n/a	98-144	82%	18%
DTSJ	Side Walls	5	96-114	65%	35%
	West Bulkhead Wall	n/a	94-128	73%	27%
	East Bulkhead Wall	n/a	94-128	100%	0%
Alum Rock	Side Walls	5	110-118	98%	2%
	North Bulkhead Wall	n/a	94-128	100%	0%
	South Bulkhead Wall	n/a	94-128	100%	0%
East Portal	W Wall: S1 565+50 to S1 567+00	1	114	100%	0%
	W Wall: S1 567+00 to 568+50	2	97-105	100%	0%
	W Wall: S1 568+50 to 569+67	3	78-94	100%	0%
	E Wall: S1 564+50 to S1 565+50	1	73	100%	0%
	E Wall: S1 565+50 to S2 567+50	2	99-116	100%	0%
	E Wall: S1 567+50 to 569+67	3	81-114	100%	0%
	E and W Walls: S1 569+67 to S1 571+27	4	98-117	100%	0%

For Reference Only

See disclaimer on cover page.

5.4.3 Excavation Methods

Table 5-5 provides a summary of the types of soil to be excavated from the surface excavations.

Table 5-5 Summary of Anticipated Soil Units for Mass Excavation

Structure	Anticipated Installation Conditions	
	Unit 1	Unit 2
West Portal	XX	XX
West Mid-Tunnel Vent Shaft	XX	XX
Diridon/Arena Station	XX	XX
Downtown San Jose Station and Crossover	XX	XX
East Mid-Tunnel Vent Shaft	XX	XX
Alum Rock Station	XX	XX
East Portal	XX	XX

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

6.0 Settlement Considerations

6.1 Tunneling Induced Initial Settlement

The process of tunneling relieves the existing free-field stresses in the ground around the excavation, allowing inward movement of the ground to occur and a settlement trough to form on the ground surface. For predicting the magnitude of initial settlements, it is normally assumed that a settlement trough shaped like an inverted normal distribution curve forms over the tunnel centerline (See Figure 6-1). The volume of the settlement trough is considered equal to the loss of ground in the tunnel, usually expressed as a percentage of the excavated volume.

Settlement must be controlled during tunneling primarily to minimize deformation of the overlying utilities and near-by buildings. The VTA is also interested in minimizing property damage caused by tunnel construction.

For EPBMs as required for the SVRT project, there is considerable relevant experience of settlement over tunnels constructed in soft ground. Good tunneling practice results in volume loss in the range ½% to 1%. Thus settlements have been estimated for volume loss (V_L) of 1% and are described below. Predicted settlements were calculated using the methods and equations shown in Figure 6-1 which develop the settlement curves indicated in Figure 6-2.

Along Santa Clara Street and for businesses and residences elsewhere along the alignment, the maximum predicted settlements induced by tunneling are less than one inch. Settlements would be distributed in a trough running parallel to and centered over the twin tunnels. For typical tunnel depths, the predicted trough width is approximately 130 feet wide with the maximum settlement occurring approximately at the centerline between the two tunnels.

For Reference Only

See disclaimer on cover page.

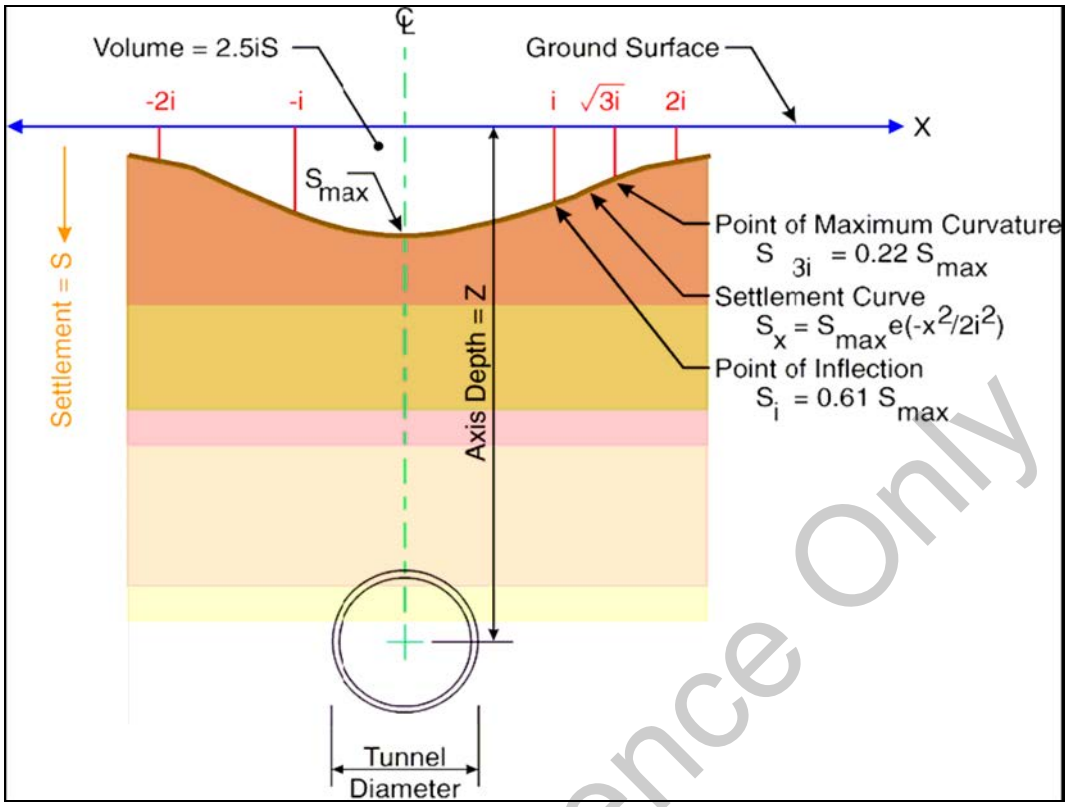


Figure 6-1 Predicting Ground Settlement above Single Bore

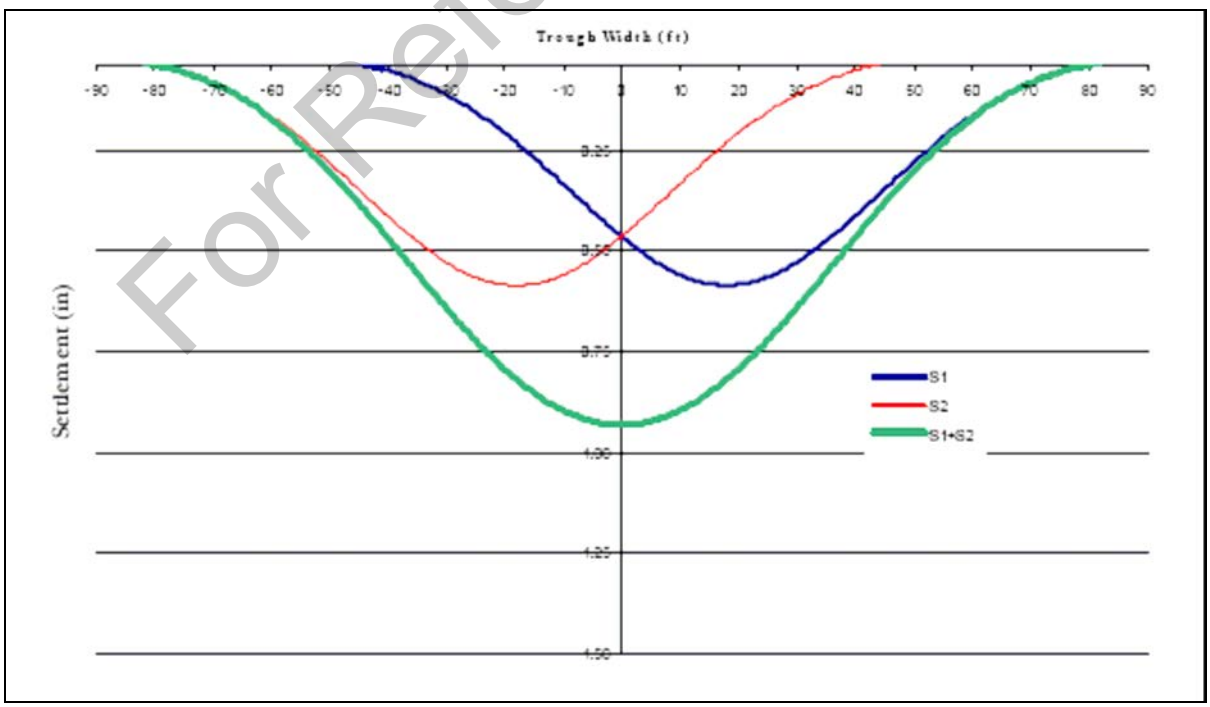


Figure 6-2 Settlement above Twin Bores by Super-position of Settlement curves

For Reference Only

See disclaimer on cover page.

6.2 Cross Passages

There are 33 cross passages on the project, sixteen of which are located in zones with principally granular material in the excavation face. Due to the high permeability and unreliability of successfully grouting and dewatering the fines and gravels in this soil, these cross passages will be hand mined through cement grout, injected by the soil displacement system of jet grouting, allowing the excavation to be completed with almost zero surface settlements. The process will however produce lateral movements and surface ground heave, which the Contractor is to control by his means and methods and maintain heave within ___ inches maximum over the jet grouted zones.

Cross Passages in zones with predominantly cohesive soils in the excavation face will be hand mined using the heading and bench system of excavation or pilot tunnels through zones of untreated ground or ground treated in the crown and/or invert by permeation grouting and/or vacuum dewatering. Hand mining the cross passages produces inward movements of the ground as the system does not maintain a constant support during the face excavation. This ground movement results in over excavation called "volume loss" which is proportional to the volume of ground settlement. With good tunneling practice these volume losses should be controlled within 3 to 4%. Using a "volume loss" of 4% on the cross passage excavation the settlement has be calculated as one inch over the center of the cross passage. Added to the maximum settlement of one inch calculated for the TBM tunneling, gives a maximum estimated settlement over the center of the cross passages of two inches.

6.3 Cut and Cover Excavations

Settlement contours for cut and cover excavations are based on settlement as a percentage of the total excavation depth. Settlement calculations for wall movements show a settlement of 1.4 inches, which is in the range of 0.2% x H, where H is the depth of the excavation. The width of the zone of influence is 2H.

In addition, settlement could occur due to the drawdown effect on the water table as a result of pumping to maintain the excavation dry. This has been estimated as 1.2 inches, giving a maximum combined estimated settlement of 2.6 inches at the outer face of the slurry walls.

6.4 Combined Effects of Tunneling and Cut and Cover Excavations

It is proposed to construct blocks of jet grouted ground at the headwalls prior to the excavation of the stations and portals and the tunneling, 50 feet long for the break-ins and 40 feet long for the break-outs of the TBMs through the headwalls. The effect of the jet grouting will be to reduce ground surface settlement adjacent to the headwalls from excavation, pumping and tunneling to almost zero. However, the jet grouting will cause ground heave which the contractor is to control by his means and methods and maintain within ?? inches maximum over the jet grouted zones.

At the west end of Diridon Station the jet grouted block is reduced to 10 feet in length from the outer face of the headwall due to the limitations adjacent to the Peninsula

For Reference Only

See disclaimer on cover page.

Corridor Joint Power Board (PCJPB) railway. Also at the ventilation structures the jet grouted blocks will be 10 feet in length from the outer face of the headwalls as the TBMs will be driving through the ventilation shafts filled with foam concrete. With these reductions of the size of the jet grout blocks from the headwalls, settlement from excavation, dewatering and tunneling will occur and be additive. This has been estimated to be in the respective order of ?? + ?? + ?? = ?? inches.

6.5 Risk of Building and Utility Damage

Settlement may cause damage to buildings and utilities; however, the occurrence of settlement does not necessarily always result in damage to buildings and utilities that are with the affected zones. This can depend on the distortions and imposed strains caused to the structure by the magnitude of slope change over length as indicated on the typical settlement curve in Figure 6-3, where the zones of maximum and minimum slopes can be demonstrated. Ground settlements are also affected by local pockets of varying ground conditions and the type of structures.

6.6 Physical Damage Categorization

A description of categories of physical damage is provided in Table 6-1 and Table 6-2. This damage can range from hairline cracks in plaster and stucco, to sticking windows and doors to structural damage where the safety of the building can be called into question. On the bored tunnel portion of the SVRT project, the maximum level of predicted damage caused by tunneling induced settlement ranges between negligible or very slight to slight.

6.7 Settlement Limits and Responsibility for Damages

The geotechnical instrumentation response value requirements established in the Contract C330 Specifications to minimize the impacts of tunneling are based on the 1% ground loss estimates (though half a percent or less is considered achievable in most ground conditions) discussed in Section 6.1 herein, and allowing for the additional effects of shaft and open cut shoring and dewatering. The Contractor is responsible for planning and conducting the work in a manner that minimizes settlement and for taking corrective action during the tunneling process if measured ground movements reach the threshold response values for instruments monitoring settlement. (50% To be determined – Subject to review prior to 95% issue) to prevent the specified limiting instrument response value.

For Reference Only

See disclaimer on cover page.

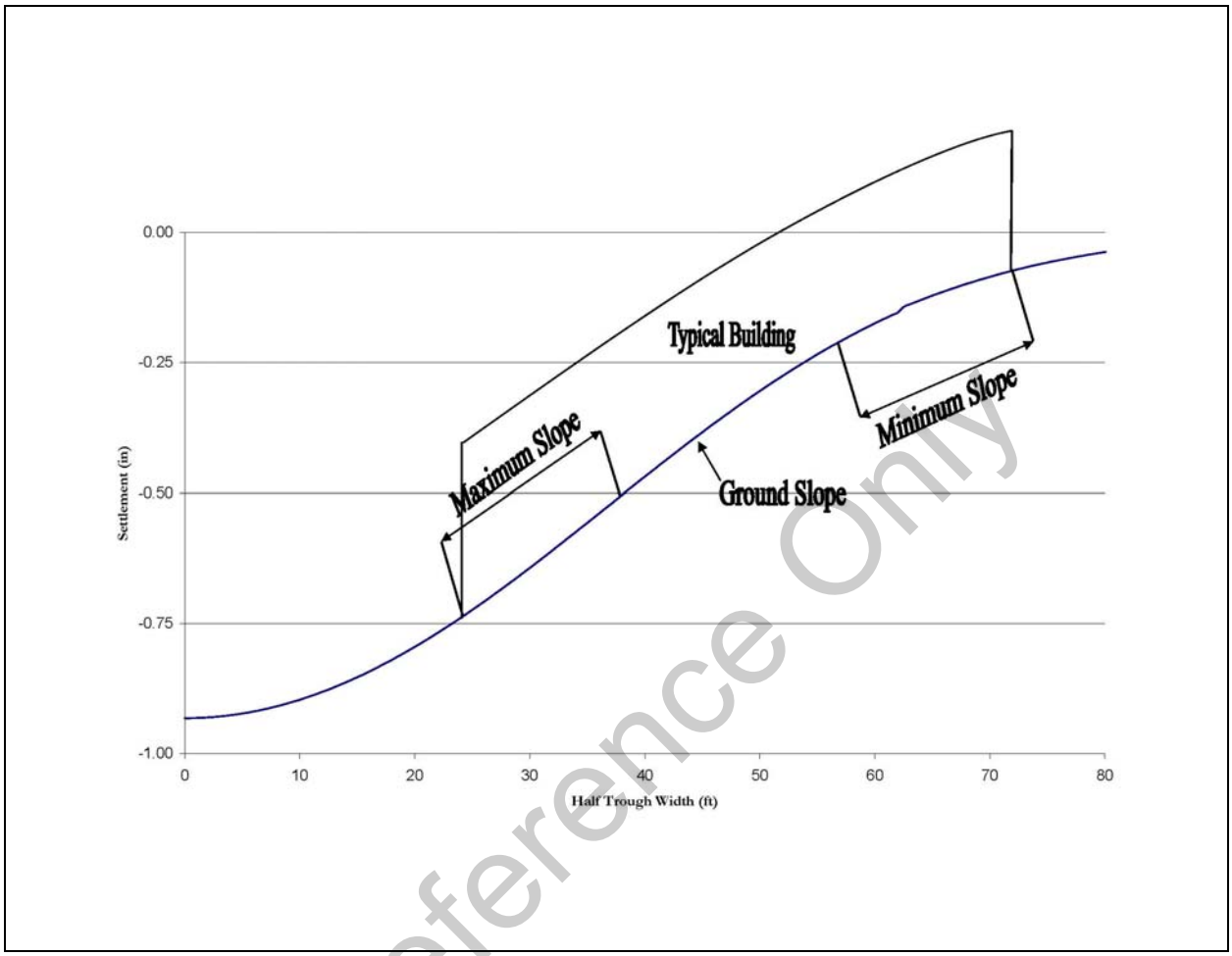


Figure 6-3 Locations of Slope Calculations

For Reference Only

See disclaimer on cover page.

Table 6-1 Damage Categories for Frame Buildings

Damage / Risk Category	Description of Typical Damage	Limiting Tensile Strain (%)	Max. Slope	Approx. Crack Width (mm)
Negligible or Very Slight (0-1)	Hairline or very fine cracks that can be easily treated during normal decoration, perhaps an isolated slight fracture in building, and cracks in external masonry visible on close inspection.	0 – 0.075	< 1/500	< 0.1 – 1.0 mm
Slight (2)	Possible superficial damage that is unlikely to have structural significance. Cracks can be easily filled and redecoration would probably be required; several slight fractures may appear showing the inside of the building; cracks that are visible externally and some repointing may be required; and doors and windows may stick. Weather-tightness concerns that may require patching of roofs, etc.	0.075 – 0.15	< 1/200	< 5.0 mm
Moderate (3)	Expected superficial damage and possible structural damage to buildings, possible damage to relatively rigid pipelines. Cracks that require some opening up and can be patched by a mason; recurrent cracks that can be masked by suitable linings; repointing of external masonry and possibly a small amount of masonry replacement; doors and windows stick requiring replacement of opening frames; utility service pipes may fracture requiring repair or replacement of connections; and weather-tightness is often impaired requiring extensive patching of roofs, etc.	0.15 – 0.3	< 1/50	5 to 15 mm, or a number of cracks > 3 mm
Severe (4)	Expected structural damage to buildings and expected damage to rigid pipelines or possible damage to other pipelines. Large cracks requiring extensive repair work involving breaking-out and replacing sections of walls (especially over doors and windows); distorted window and door frames; noticeably sloping floors; leaning or bulging walls requiring removal and replacement; some loss of bearing in beams; and disrupted utility service pipes requiring replacement of connections.	> 0.3	> 1/50	15 to 25 mm, but also depends on number of cracks
Very Severe (5)	Structural Damage often requiring major repair involving rebuilding. Beams lose bearing; walls lean and require shoring; windows are broken with distortion; and there is the danger of structural instability.	N/A	N/A	Usually > 25 mm but also depends on number of cracks

For Reference Only

See disclaimer on cover page.

Table 6-2 Damage Categories for Brick Bearing Wall Buildings

Damage / Risk Category	Description of Typical Damage	Limiting Tensile Strain (%)	Max. Slope	Approx. Crack Width (mm)
Negligible or Very Slight (0-1)	Hairline or very fine cracks that can be easily treated during normal decoration, perhaps an isolated slight fracture in building, and cracks in external masonry visible on close inspection.	0 – 0.075	< 1/2000	< 0.1 – 1.0 mm
Slight (2)	Possible superficial damage that is unlikely to have structural significance. Cracks can be easily filled and redecoration would probably be required; several slight fractures may appear showing the inside of the building; cracks that are visible externally and some repointing may be required; and doors and windows may stick. Weather-tightness concerns that may require patching of roofs, etc.	0.075 – 0.15	< 1/1000	< 5.0 mm
Moderate (3)	Expected superficial damage and possible structural damage to buildings, possible damage to relatively rigid pipelines. Cracks that require some opening up and can be patched by a mason; recurrent cracks that can be masked by suitable linings; repointing of external masonry and possibly a small amount of masonry replacement; doors and windows stick requiring replacement of opening frames; utility service pipes may fracture requiring repair or replacement of connections; and weather-tightness is often impaired requiring extensive patching of roofs, etc.	0.15 – 0.3	< 1/500	5 to 15 mm, or a number of cracks > 3 mm
Severe (4)	Expected structural damage to buildings and expected damage to rigid pipelines or possible damage to other pipelines. Large cracks requiring extensive repair work involving breaking-out and replacing sections of walls (especially over doors and windows); distorted window and door frames; noticeably sloping floors; leaning or bulging walls requiring removal and replacement; and disrupted utility service pipes requiring replacement of connections.	> 0.3	> 1/500	15 to 25 mm, but also depends on number of cracks
Very Severe (5)	Structural Damage often requiring major repair involving rebuilding. Walls lean and require shoring; windows are broken with distortion; and there is the danger of structural instability.	N/A	N/A	Usually > 25 mm but also depends on number of cracks

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

7.0 Instrumentation and Monitoring

7.1 Bored Tunnel

7.1.1 Photographic Surveys

Pre-construction and post-construction photographs are required in the Special Provisions. This information is intended to document the pre-existence of cracks and damage to existing buildings that might otherwise be considered to be caused by the work.

7.1.2 Monitoring

The Engineer will perform the monitoring. The Contractor is required to do additional monitoring as necessary to control the work.

7.1.3 Settlement Monitoring Points

Construction of monuments for ground surface, building and utility settlement monitoring is the Contractor's responsibility.

7.1.4 Ground Movement Monitoring

Location of borehole extensometers and inclinometers for measuring vertical and horizontal ground movements is the Contractor's responsibility.

7.1.5 Slurry Wall Instrumentation

Location of inclinometers in the slurry walls for measuring lateral deflections in the walls is the Contractor's responsibility.

7.1.6 Ground Water Monitoring

Construction of observation wells and piezometer sets for measuring ground water levels and hydrostatic pressures is the Contractor's responsibility. Piezometer data from the west portal and stations is also required to be taken into consideration in establishing face operating pressures for the TBMs.

7.1.7 Special Structures

Location of tiltmeters, survey reflectors, high sensitivity settlement sensors and sensor reservoirs for measuring the angles of deflection, movements and settlements of buildings, bridge abutments, piers, walls and columns as indicated on the Contract C330 Drawings is the Contractor's responsibility.

7.1.8 Monitoring Arrays

Construction of ground monitoring point and full instrument arrays at the locations indicated on the Contract C330 Drawings is the Contractor's responsibility.

For Reference Only

See disclaimer on cover page.

7.1.9 Instrumentation Zones

Construction of girder and tieback instrumentation zones as indicated on the Contract C330 Drawings and the Contract C330 Specifications is the Contractor's responsibility.

7.1.10 Tunnel Deformation Monitoring

Construction of convergence monitoring point arrays referred to in the Contract C330 Specifications are the Contractor's responsibility, and are to be established in the precast concrete tunnel lining at periodic intervals and at locations selected by the Engineer. Convergence monitoring of the running tunnel segmental lining and running tunnel cross passage opening temporary support frames will be carried out to ensure stability of the excavation and the safety of those working in the tunnels.

7.2 Structures

7.2.1 Photographic Surveys

Pre-construction and post-construction photographs are required in the Special Provisions. This information is intended to document the pre-existence of cracks and damage to existing buildings that might otherwise be considered to be caused by the work.

7.2.2 Monitoring

The Engineer will perform the monitoring. The Contractor is required to do additional monitoring as necessary to control the work.

7.2.3 Settlement Monitoring Points

Construction of monuments for ground surface and building settlement monitoring is the Contractor's responsibility.

7.2.4 Ground Movement Monitoring

Location of inclinometers for measuring horizontal ground movements is the Contractor's responsibility.

7.2.5 Slurry Wall Instrumentation

Location of inclinometers in the slurry walls for measuring lateral deflections in the walls is the Contractor's responsibility.

7.2.6 Ground Water Monitoring

Construction of observation wells and piezometer sets for measuring ground water levels and hydrostatic pressures on the slurry walls is the Contractor's responsibility.

For Reference Only

See disclaimer on cover page.

7.2.7 Special Structures

Location of tiltmeters, for measuring the angles of deflection of buildings and walls as indicated on the Contract C330 Drawings is the Contractor's responsibility.

7.2.8 Monitoring Arrays

Construction of full instrument arrays at the locations indicated on the Contract C330 Drawings is the Contractor's responsibility.

7.2.9 Instrumentation Zones

Construction of tieback, strut and beam instrumentation zones as indicated on the Contract C330 Drawings and the Contract C330 Specifications is the Contractor's responsibility.

7.3 Trigger Levels

Refer to Contract C330 Specifications for threshold and limiting values of settlements and displacements.

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

8.0 Spoil Disposal

The project will generate a spoil surplus that will be disposed by the Contractor off-site. The characteristics of TBM tunnel spoil including soil and groundwater influenced by soil conditioners and others additives induced while tunneling are discussed in Section 4.3.2.13.

Other sources of spoil will be derived by many construction activities including slurry wall construction, jet grouting and other grouting, cross passage excavation, portal station and shaft excavations.

Proper disposal of all excavated spoil and water is the Contractor's responsibility, and is subject to applicable local, State, and Federal regulations.

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

9.0 Limitations

The interpretations and assessments contained in this GBR are based upon the available information from the Geotechnical Data Reports. The geologic environment is complex, and as such, no amount of pre-construction information will convey as detailed an understanding as will exist following excavation. The anticipated conditions were developed with the standard of care commonly applied as the state of the practice in the profession. No warranty is included; either expressed or implied, that the actual geotechnical conditions encountered will conform to the conditions described herein.

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

REFERENCES

Broms, B.B. and Bennermark, H. (1967). Stability of Clay in Vertical Openings. Journal of Soil Mechanics and Foundations Division, ASCE, 193, SM1, pp.71-94.

British Tunnelling Society (BTS). Closed-Face Tunnelling Machines and Ground Stability - A Guideline for Best Practice. 2005

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

Davis, E.H., Gunn, M.J., Mair, R.J., and Senevirante, H.N. (1980). The stability of shallow tunnels and underground openings in cohesive material. *Geotechnique* 30, No. 4, 397-416.

Deere, D.U., Merritt, A.H., Coon, R.F., 1969. "Engineering Classification of In-Situ Rock". *Technical report No. AFWL-TR-67-144*, Kirkland Air Force Base, New Mexico.

Doig, Patrick.J. (2004) "Design and Construction of an LRT Tunnel in San Jose, CA". proceedings of the North American Tunneling Conference, NAT 2004.

Heuer, R. E., 1974. "Important Ground Parameters in Soft Ground Tunneling", in Subsurface Exploration for Underground Excavation and Heavy Construction, ASCE Specialty Conference.

HMM/Bechtel, 2004, "Silicon Valley Rapid Transit Project, Central Area Guideway, Surface Water Hydrology and Hydraulics for Facility Design", P0503-D300-RPT-HH-002, Rev. A, dated December 16, 2004.

HMM/Bechtel, 2008a, "Silicon Valley Rapid Transit Project, Central Area Guideway, Hydrogeology Report, P0503-D300-RPT-DE-020, Rev. 1, dated April 30, 2008.

HMM/Bechtel, 2008b, Silicon Valley Rapid Transit Project, Central Area Guideway, Grout Trial Interpretive Report", P0503-D300-RPT-DE-052., Rev. A, dated October 3, 2008.

HMM/Bechtel, 2008c, Silicon Valley Rapid Transit Project, Central Area Guideway, TBM Geotechnical Basis of Design Report", P0503-D300-RPT-DE-053, Rev. A, dated September 18, 2008.

Jacobs Associates for SFPUC. "Bay Division Pipelines Reliability Upgrade (CUW36801) – Bay Tunnel Project – Draft Geotechnical Baseline Report Rev. A." November 30, 2007.

Peck, R.B., 1969 "Deep Excavations and Tunneling in Soft Ground". Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering, State-of-the-Art Volume, Mexico City, pp225-290.

For Reference Only

See disclaimer on cover page.

Robertson, P.K. Soil Classification Using the Cone Penetration Test – Discussion by Jefferies, M. G. & Davies, M. P., (1990). Canadian Geotechnical Journal, 28 (1), 173–176.

Terzaghi, K. (1950), "Geologic Aspects of Soft Ground Tunneling", Chapter 11 in Applied Sedimentation, ed. P. Trask, John Wiley and Sons, New York..

Thewes M. and Burger W. June 2004. “Clogging risks for TBM drives in Clay”. Tunnels & Tunnelling International, pp.28-31.

VTA. See Santa Clara Valley Transportation Authority.

For Reference Only

See disclaimer on cover page.

DRAWINGS

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

D300-TR-C300
Borehole Location Plan
(See attached 28” x 40” Plot Drawing)

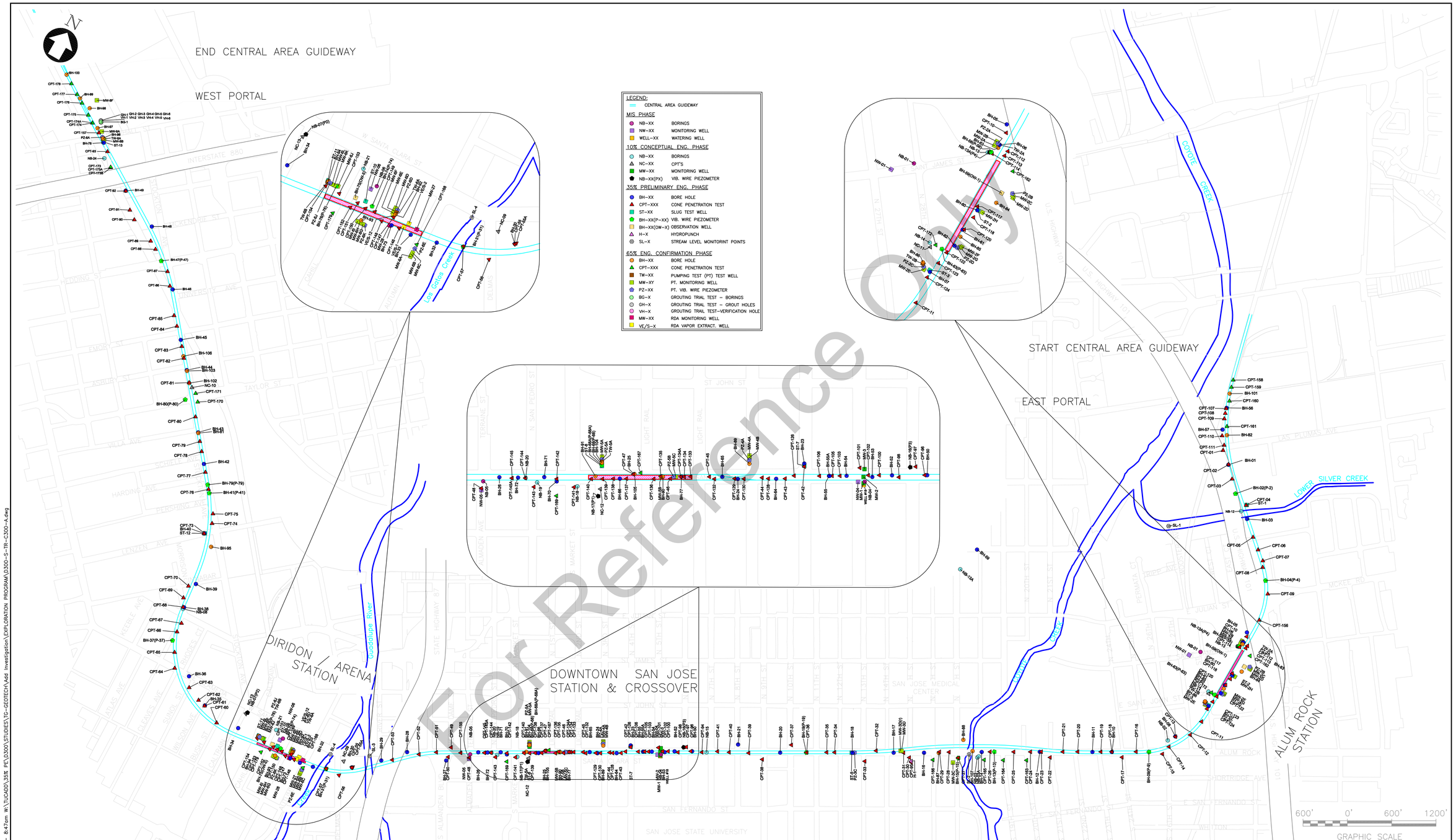
For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.



LEGEND:

CENTRAL AREA GUIDEWAY

MIS PHASE

- NB-XX BORINGS
- NW-XX MONITORING WELL
- WELL-XX WATERING WELL

10% CONCEPTUAL ENG. PHASE

- NB-XX BORINGS
- NC-XX CPT'S
- MW-XX MONITORING WELL
- NB-XX(PX) VIB. WIRE PIEZOMETER

35% PRELIMINARY ENG. PHASE

- BH-XX BORE HOLE
- CPT-XXX CONE PENETRATION TEST
- ST-XX SLUG TEST WELL
- BH-XX(P-XX) VIB. WIRE PIEZOMETER
- BH-XX(OW-X) OBSERVATION WELL
- H-X HYDROPUNCH
- SL-X STREAM LEVEL MONITORINT POINTS

65% ENG. CONFIRMATION PHASE

- BH-XX BORE HOLE
- CPT-XXX CONE PENETRATION TEST
- TW-XX PUMPING TEST (PT) TEST WELL
- MW-XY PT. MONITORING WELL
- PZ-XX PT. VIB. WIRE PIEZOMETER
- BG-X GROUTING TRIAL TEST – BORINGS
- GH-X GROUTING TRIAL TEST – GROUT HOLES
- VH-X GROUTING TRAIL TEST–VERIFICATION HOLE
- MW-XX RDA MONITORING WELL
- VE/S-X RDA VAPOR EXTRACT. WELL

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
0	20080905				IN PROGRESS

DESIGNED BY T HUNT
DRAWN BY H JOHNSON
CHECKED BY R BALL
IN CHARGE J HAWLEY
DATE 20080905

HMM / BECHTEL

A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

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

SILICON VALLEY RAPID TRANSIT PROJECT
CENTRAL AREA GUIDEWAY
GEOTECHNICAL INVESTIGATIONS
OVERALL LOCATION MAP

CADD FILENAME D300-S-TR-C300
SIZE D | SCALE 1"=600'
CONTRACT NO. D300 | REV. A
AREA CODE TR | SHEET NO. C300 | PAGE NO.

JOHNSON_H Sep 25, 2008 – 8:47am W:\TUCADD\35% PE\D300\STUDIES\TG-GEOTECH\Add Investigation\EXPLORATION PROGRAM\D300-S-TR-C300-A.dwg

See disclaimer on cover page.

D300-TR-C301 through D300-TR-C307

Interpretive Geologic Profiles

(See attached 3’ x 6’ Plot Drawings)

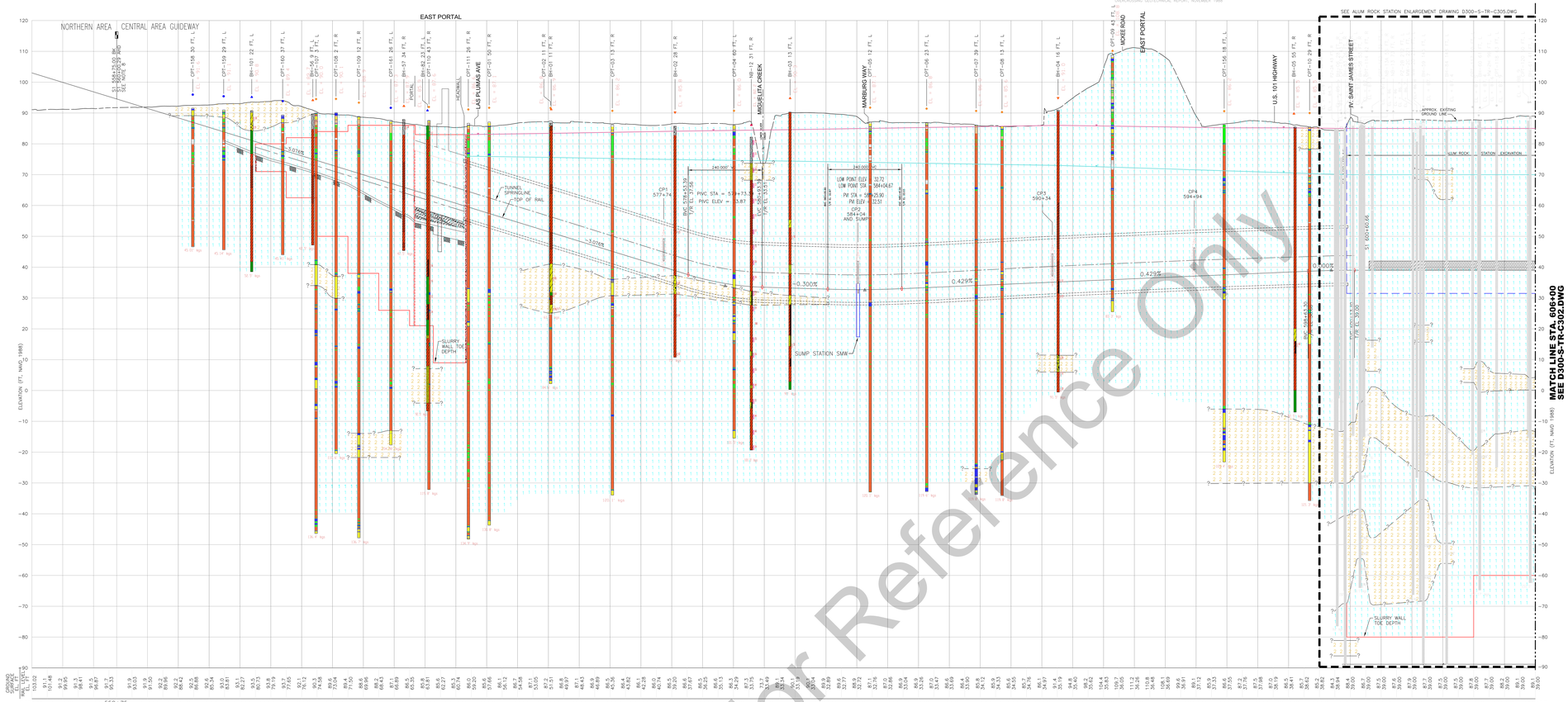
For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.



MATCH LINE STA. 606+00
SEE D300-S-TR-C302.DWG

LIMITATIONS:

THE GEOTECHNICAL INFORMATION ON THESE SUBSURFACE PROFILES IS SIMPLIFIED AND DOES NOT CONTAIN ALL THE INFORMATION PRESENTED IN THE BORING LOGS PREPARED FOR THE 10% PRELIMINARY ENGINEERING PHASE, 30% PRELIMINARY ENGINEERING PHASE AND FINAL DESIGN PHASE OF THE PROJECT. THEREFORE, THE REVIEW OF THE PROFILES MUST BE DONE IN CONJUNCTION WITH THE BORING LOGS.

THE SOIL UNIT BOUNDARIES SHOWN IN PROFILES ARE BASED ON INTERPOLATION AND EXTRAPOLATION OF GEOTECHNICAL INFORMATION FROM ADJACENT BORINGS AND CPT PROBES. THE LOCATIONS OF THESE BOUNDARIES CAN ONLY BE INFERRED APPROXIMATELY AND DO NOT REPRESENT BASELINE CONDITIONS. REFER TO DRAWINGS D300-S-TR-C308 TO D300-S-TR-C342 FOR BASELINE CONDITIONS AT CROSS PASSAGES.

STATIONING SHOWN IN ESTIMATED PARAMETERS ALONG PROFILE ARE BASED ON INTERPOLATION AND EXTRAPOLATION OF GEOTECHNICAL INFORMATION FROM ADJACENT BORINGS AND CPT PROBES AND IS THEREFORE APPROXIMATE. STATIONING AND BOUNDARIES SHOWN ARE NOT BASELINES BUT THE ESTIMATED PARAMETER DATA SHOWN IS USED TO DEVELOP AVERAGE BASELINES AS PRESENTED IN THE TEXT OF THE 65% COMBINED GBR, AUGUST 2008.

SVRT SOIL CLASSIFICATION:

UNIT	PRIMARY STRATUM DESCRIPTION	INCLUDED IN THIS UNIT (SEE NOTE 3)
1	SILTS, CLAYS AND ORGANIC SOILS. SOIL BEHAVIOR TYPE (SBT) ZONES 1 THROUGH 4, AND 9, AS CLASSIFIED BY CPTS USING ROBERTSON, 1990.	• MAN-MADE FILL CONSISTING OF ASPHALT CONCRETE AND AGGREGATE BASE ENCOUNTERED MOST OFTEN WITHIN FIVE FEET BELOW GROUND SURFACE. • THIN LAYERS OF COARSE-GRAINED SOIL, AS DESCRIBED IN NOTE 4.
2	SANDS AND GRAVELS, AND SBT ZONES 6 THROUGH 8, CLASSIFIED BY CPTS USING ROBERTSON, 1990.	• COBBLES • THIN LAYERS OF FINE-GRAINED SOIL AS DESCRIBED IN NOTE 4

DESIGN WATER LEVELS:

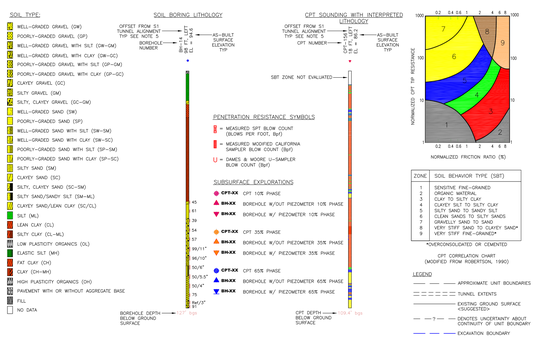
- MAXIMUM WATER LEVEL FOR CONSTRUCTION (BASED ON HISTORIC MAXIMUM)
- MINIMUM WATER LEVEL FOR CONSTRUCTION (BASED ON HISTORIC MINIMUM)

NOTES:

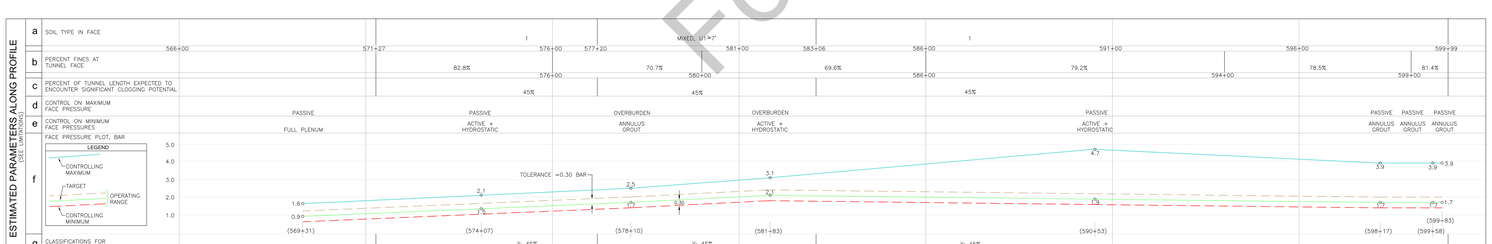
1. SOIL TYPE GRAPHICS PRESENTED ON THESE SUBSURFACE PROFILES VARY SLIGHTLY FROM THE SOIL TYPE GRAPHICS SHOWN ON THE PLAN AND PROFILES IN THE GEOTECHNICAL DATA REPORT. HOWEVER, THE UNIFIED SOIL CLASSIFICATION SYSTEM (USCS) GROUP NAMES HAVE NOT BEEN MODIFIED.
2. ALL DESIGN, CONSTRUCTION, AND MEASURED WATER LEVELS SHOWN ON THESE SUBSURFACE PROFILES REPRESENT THE REDWOOD LEVELS EXPECTED AT THE DEPTH OF THE TUNNEL. THE WATER LEVELS SHOWN ARE NOT APPLICABLE FOR THE DESIGN OF OTHER SVRT PROJECT EXCAVATIONS AND STRUCTURES.
3. THE SOIL BEHAVIOR TYPE ZONE 5 = "SAND MIXTURES – SILTY SAND AND SANDY SILT" CONSISTS OF BOTH FINE-GRAINED AND COARSE-GRAINED SOIL. THEREFORE, IT WAS INCLUDED IN EITHER SOIL CLASSIFICATION UNIT. IT IS REPRESENTED BY BLUE COLOR ON CPT STICKS.
4. LAYER THICKNESS DUE TO A HIGH VARIABILITY IN SOIL PROFILE ENCOUNTERED VERTICALLY WITH DEPTH, AND ALTERNATING OR INDIVIDUAL SOIL LAYERS BELONGING TO EITHER SOIL CLASSIFICATION UNIT 1 OR 2 THINNER THAN THE ENVIRONMENT DESCRIBED UNDER REGIONAL GEOLOGY IN THIS REPORT, FINE (S) FEET WAS SELECTED AS A THRESHOLD THICKNESS ONLY THE ALTERNATING OR INDIVIDUAL SOIL LAYERS BELONGING TO EITHER SOIL CLASSIFICATION UNIT 1 & 2 THINNER THAN FINE (S) FEET WERE CONSIDERED. THIS WAS DONE TO A SET OF CRITERIA AND JUDGEMENT AS DESCRIBED IN APPENDIX C OF THE 65% COMBINED GBR, AUGUST 2008.
5. OFFSET DISTANCE PERPENDICULAR TO ALIGNMENT S1 TUNNEL, DIRECTION LOOKING UP STATION.
6. DEEPER BORINGS TRUNCATED.
7. LENGTH OF PENETRATION RESISTANCE SYMBOLS PROPORTIONAL TO RECOVERY.
8. CHANGES IN STATIONING AT NOTED LOCATIONS OCCUR DUE TO EQUATIONS PROVIDED ON THE APRIL 2008 QUARTERLY UPDATE ALIGNMENT.

SOURCES OF INFORMATION:

1. ALIGNMENT SHOWN ON PROFILE CORRESPONDS TO APRIL 2008 QUARTERLY UPDATE ALIGNMENT SUBMITTAL.
2. CROSS PASSAGES: EMAIL FROM A. BARGROVER, FEB. 22, 2008
3. BOREHOLE AND CPT INFORMATION PROVIDED BY A. JAN VIA EMAILS APRIL 10 TO APRIL 14, 2008.
4. HYDROGEOLOGICAL INFORMATION FROM HMMB HYDROGEOLOGY REPORT P0503-0300-RPT-DE-020 MAY 2008.
5. SLURRY WALL AND DSM WALL INFORMATION FROM HMMB HYDROGEOLOGY REPORT P0503-0300-RPT-DE-020 MAY 2008.



THIS FIGURE IS STILL IN PROGRESS AND WILL BE REVISED BASED ON ADDITIONAL GEOTECHNICAL INVESTIGATION RESULTS AND REVISED MATERIAL DESCRIPTIONS



A	20080919	DY	ISSUED FOR REVIEW
REV	DATE	BY	SUB APP DESCRIPTION

DESIGNED BY D. YOUNG
DRAWN BY M. RUBRIGACION
CHECKED BY R. BALL
PL CHARGE J. HAWLEY
DATE 20080919

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

BART
VTA Silicon Valley Rapid Transit Corridor

SILICON VALLEY RAPID TRANSIT PROJECT P0503 TUNNEL SEGMENT
INTERPRETIVE GEOLOGICAL PROFILE - STUDY SECTION 1 EAST PORTAL TO ALUM ROCK STATION

CADD FILENAME D300-S-TR-C301.dwg
SIZE 22" x 36" SCALE 1"=100'H; 1"=10'V
CONTRACT NO. D300 REV. A
AREA CODE TR SHEET/BNB. NO. C301 PAGE NO. 1

For Reference Only

See disclaimer on cover page.

THIS FIGURE IS STILL IN PROGRESS AND WILL BE REVISED BASED ON ADDITIONAL GEOTECHNICAL INVESTIGATION RESULTS AND REVISED MATERIAL DESCRIPTIONS

HMM / BECHTEL

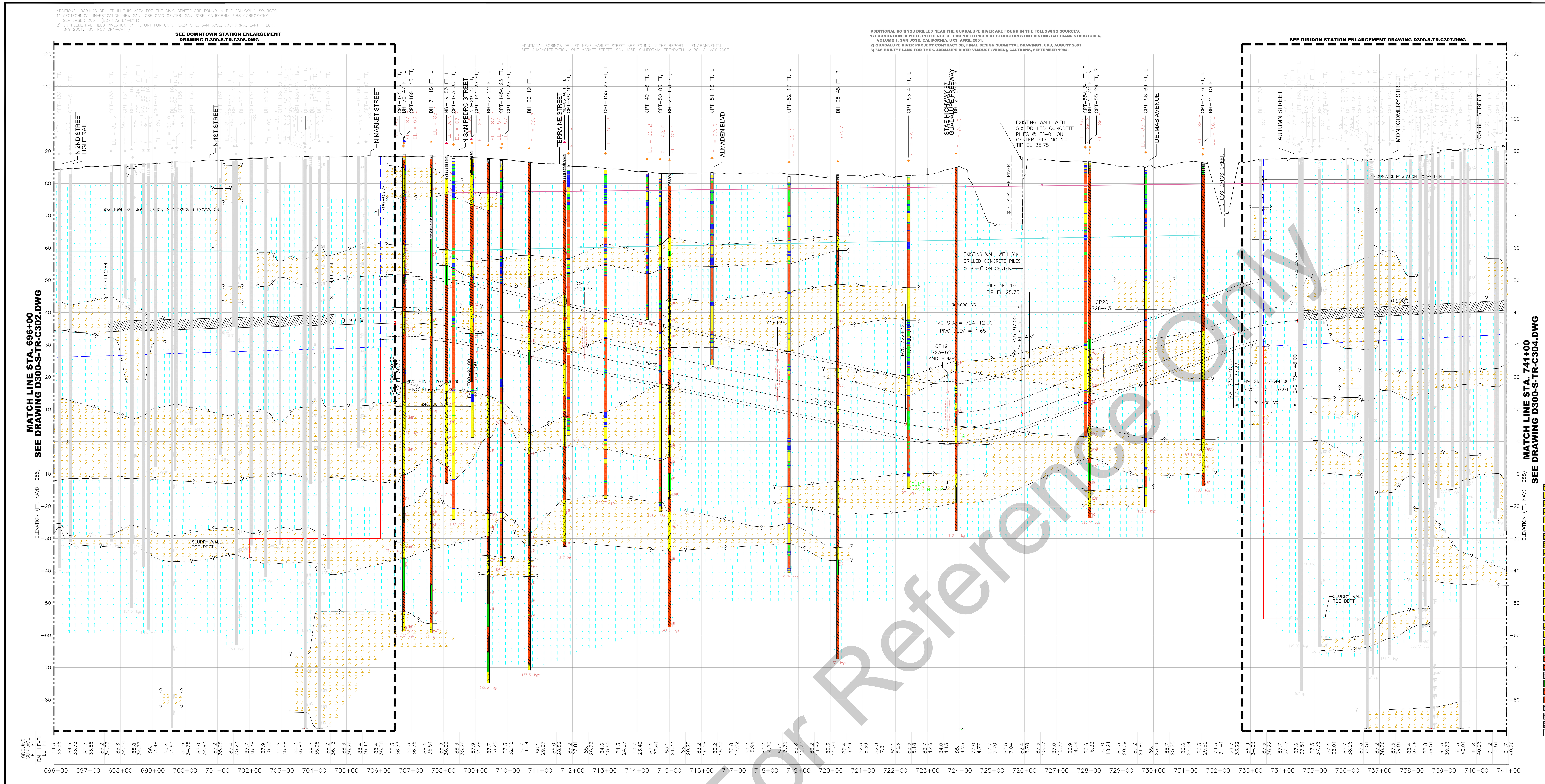
SILICON VALLEY RAPID TRANSIT PROJECT P0503 TUNNEL SEGMENT

INTERPRETIVE GEOLOGICAL PROFILE - STUDY SECTION 3 ALUM ROCK STATION TO DOWNTOWN SAN JOSE STATION AND CROSSOVER SHELL

CADD FILENAME	D300-S-TR-C302.dwg
SIZE	–
SCALE	1"=100'H; 1"=10'V
CONTRACT NO.	D300
REV	A
AREA CODE	TR
SHEET NO.	C302
PAGE NO.	2

For Reference Only

See disclaimer on cover page.



LIMITATIONS:

THE GEOTECHNICAL INFORMATION ON THESE SUBSURFACE PROFILES IS SIMPLIFIED AND DOES NOT CONTAIN ALL THE INFORMATION PRESENTED IN THE BORING LOGS PREPARED FOR THE 10% PRELIMINARY ENGINEERING PHASE, 35% ENGINEERING DESIGN PHASE AND THE 65% ENGINEERING DESIGN PHASE OF THE PROJECT. THEREFORE, THE REVIEW OF THE PROFILES MUST BE DONE IN CONJUNCTION WITH THE BORING LOGS.

THE SOIL UNIT BOUNDARIES SHOWN IN PROFILES ARE BASED ON INTERPOLATION AND EXTRAPOLATION OF GEOTECHNICAL INFORMATION FROM ADJACENT BORINGS AND CPT PROBES. THE LOCATIONS OF THESE BOUNDARIES CAN ONLY BE INFERRED APPROXIMATELY AND DO NOT REPRESENT BASELINE CONDITIONS. REFER TO DRAWINGS D300-S-TR-C308 TO D300-S-TR-C342 FOR BASELINE CONDITIONS AT CROSS PASSAGES.

STATIONING SHOWN IN ESTIMATED PARAMETERS ALONG PROFILE ARE BASED ON INTERPOLATION AND EXTRAPOLATION OF GEOTECHNICAL INFORMATION FROM ADJACENT BORINGS AND CPT PROBES AND IS THEREFORE APPROXIMATE. STATIONING AND BOUNDARIES SHOWN ARE NOT BASELINES BUT THE ESTIMATED PARAMETER DATA SHOWN IS USED TO DEVELOP AVERAGE BASELINES AS PRESENTED IN THE TEXT OF THE 65% COMBINED GBR, AUGUST 2008.

SVRT SOIL CLASSIFICATION:

UNIT	PRIMARY STRATUM DESCRIPTION	INCLUDED IN THIS UNIT (SEE NOTE 3)
1	SILTS, CLAYS AND ORGANIC SOILS. SOIL BEHAVIOR TYPE (SBT) ZONES 1 THROUGH 4, AND 9, AS CLASSIFIED BY CPTS USING ROBERTSON, 1990.	• MAN-MADE FILL CONSISTING OF ASPHALT CONCRETE AND AGGREGATE BASE ENCOUNTERED MOST OFTEN WITHIN FIVE FEET BELOW GROUND SURFACE • THIN LAYERS OF COARSE-GRAINED SOIL AS DESCRIBED IN NOTE 4
2	SANDS AND GRAVELS, AND SBT ZONES 6 THROUGH 8, CLASSIFIED BY CPTS USING ROBERTSON, 1990.	• COBBLES • THIN LAYERS OF FINE-GRAINED SOIL AS DESCRIBED IN NOTE 4

DESIGN WATER LEVELS:

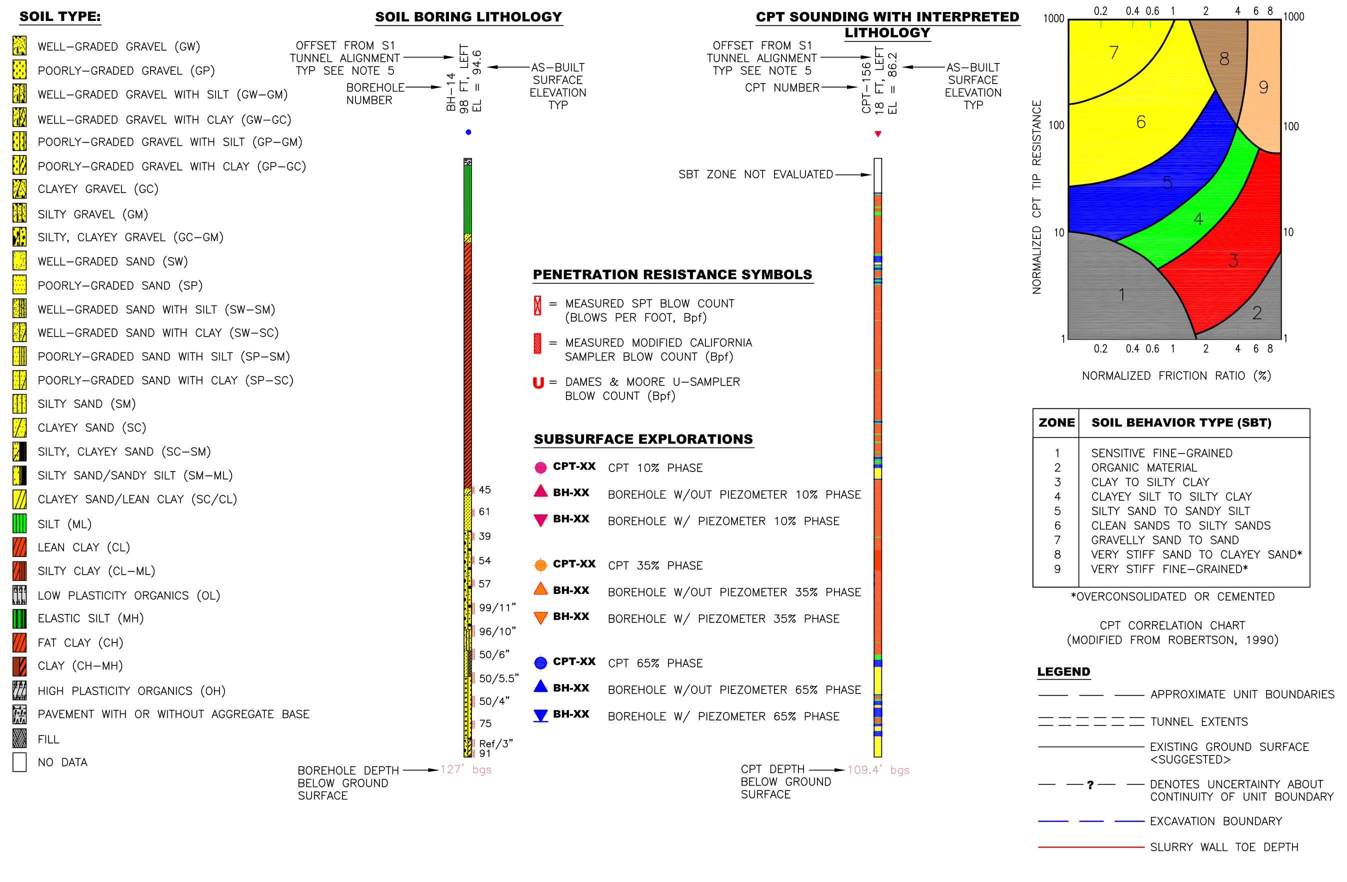
MAXIMUM WATER LEVEL FOR CONSTRUCTION (BASED ON HISTORIC MAXIMUM)
MINIMUM WATER LEVEL FOR CONSTRUCTION (BASED ON HISTORIC MINIMUM)

NOTES:

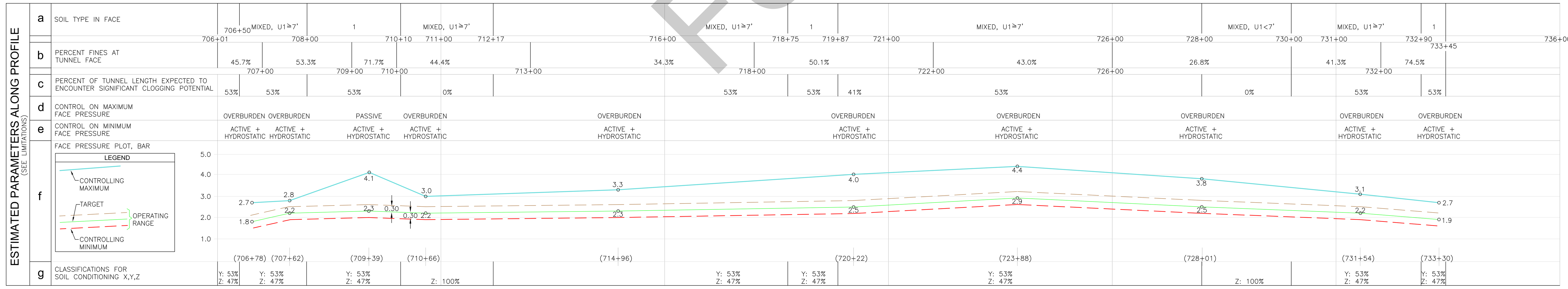
1. SOIL TYPE GRAPHICS PRESENTED ON THESE SUBSURFACE PROFILES VARY SLIGHTLY FROM THE SOIL TYPE GRAPHICS SHOWN ON THE PLAN AND PROFILES IN THE GEOTECHNICAL DATA REPORT. HOWEVER, THE UNIFIED SOIL CLASSIFICATION SYSTEM (USCS) GROUP NAMES HAVE NOT BEEN MODIFIED.
2. ALL DESIGN, CONSTRUCTION, AND MEASURED WATER LEVELS SHOWN ON THESE SUBSURFACE PROFILES REPRESENT THE PIEZOMETRIC LEVELS EXPECTED AT THE DEPTH OF THE TBM TUNNEL. THE WATER LEVELS SHOWN ARE NOT APPLICABLE FOR THE DESIGN OF OTHER SVRT PROJECT EXCAVATIONS AND STRUCTURES.
3. THE SOIL BEHAVIOR TYPE ZONE 5 – 'SAND MIXTURES – SILTY SAND AND SANDY SILT' CONSISTS OF BOTH FINE-GRAINED AND COARSE-GRAINED SOIL. THEREFORE, IT WAS INCLUDED IN EITHER SOIL CLASSIFICATION UNIT. IT IS REPRESENTED BY BLUE COLOR ON CPT STICKS.
4. LAYER THICKNESS: DUE TO A HIGH VARIABILITY IN SOIL PROFILE ENCOUNTERED VERTICALLY WITH DEPTH, AND HORIZONTALLY ALONG THE PROPOSED TUNNEL ALIGNMENT CAUSED BY THE ALLUVIAL SOIL DEPOSITIONAL ENVIRONMENT DESCRIBED UNDER REGIONAL GEOLOGY IN THIS REPORT, FIVE (5) FEET WAS SELECTED AS A 'THRESHOLD THICKNESS (th)'. THE ALTERNATING OR INDIVIDUAL SOIL LAYERS BELONGING TO EITHER SOIL CLASSIFICATION UNIT 1 & 2 THINNER THAN th WERE GROUPED ACCORDING TO A SET OF CRITERIA AND JUDGEMENT AS DESCRIBED IN APPENDIX C OF THE 65% COMBINED GBR, AUGUST 2008.
5. OFFSET DISTANCE PERPENDICULAR TO ALIGNMENT S1 TUNNEL. DIRECTION LOOKING UP STATION.
6. DEEPER BORINGS TRUNCATED.
7. LENGTH OF PENETRATION RESISTANCE SYMBOLS PROPORTIONAL TO RECOVERY.
8. CHANGES IN STATIONING AT NOTED LOCATIONS OCCUR DUE TO EQUATIONS PROVIDED ON THE APRIL 2008 QUARTERLY UPDATE ALIGNMENT.

SOURCES OF INFORMATION:

1. ALIGNMENT SHOWN ON PROFILE CORRESPONDS TO APRIL 2008 QUARTERLY UPDATE ALIGNMENT SUBMITTAL.
2. CROSS PASSAGES: EMAIL FROM A. BARNGROVER, FEB. 22, 2008
3. BOREHOLE AND CPT INFORMATION PROVIDED BY A. JAIN VIA EMAILS APRIL 10 TO APRIL 14, 2008.
4. HYDROGEOLOGICAL INFORMATION FROM HMMB HYDROGEOLOGY REPORT P0503-D300-RPT-DE-020 MAY 2008.
5. SLURRY WALL AND DSM WALL INFORMATION FROM HMMB HYDROGEOLOGY REPORT P0503-D300-RPT-DE-020 MAY 2008.



THIS FIGURE IS STILL IN PROGRESS AND WILL BE REVISED BASED ON ADDITIONAL GEOTECHNICAL INVESTIGATION RESULTS AND REVISED MATERIAL DESCRIPTIONS



DESIGNED BY	D. YOUNG
DRAWN BY	N.PURIFICACION
CHECKED BY	R. BALL
IN CHARGE	J. HAWLEY
DATE	20080919

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20080919	DY			ISSUED FOR REVIEW

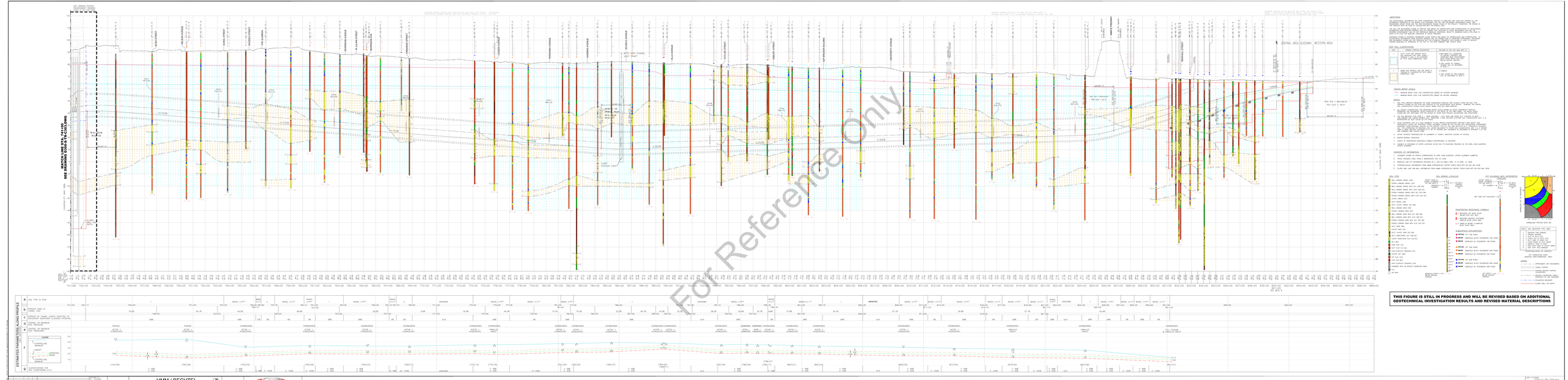
HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED ____ APPROVED ____

SILICON VALLEY RAPID TRANSIT PROJECT P0503 TUNNEL SEGMENT
INTERPRETIVE GEOLOGICAL PROFILE - STUDY SECTION 5
DOWNTOWN SAN JOSE STATION AND CROSSOVER SHELL TO DIRIDON / ARENA STATION

CADD FILENAME	D300-S-TR-C303.dwg
SIZE	–
SCALE	1"=100'H; 1"=10'V
CONTRACT NO.	D300
REV.	A
AREA CODE	TR
SHEET NO.	C303
PAGE NO.	3

See disclaimer on cover page.



For Reference Only

THIS FIGURE IS STILL IN PROGRESS AND WILL BE REVISED BASED ON ADDITIONAL GEOTECHNICAL INVESTIGATION RESULTS AND REVISED MATERIAL DESCRIPTIONS

HMM / BECHTEL

A Joint Venture of Parsons Brinckerhoff Inc. and Bechtel Infrastructure Corp.

BART

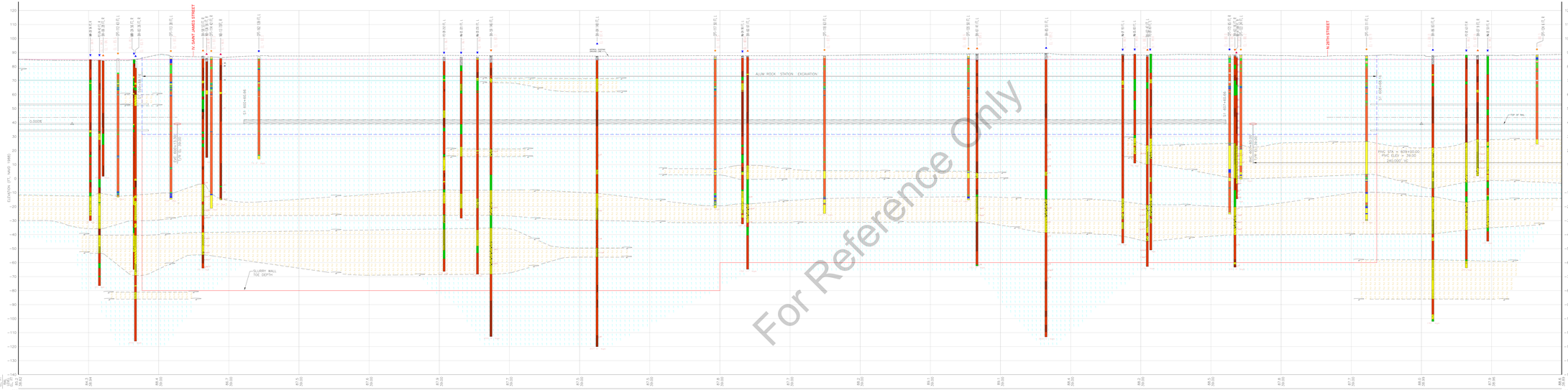
VTA

SILICON VALLEY RAPID TRANSIT PROJECT P0503 TUNNEL SEGMENT

INTERPRETIVE GEOLOGICAL PROFILE - STUDY SECTION 7 DIRIDON / ARENA STATION TO WEST PORTAL

CAD FILE	D300-9-TR-C304.dwg
DESIGN	TN/SDN, CT/KEY
CONTRACT NO.	D300
SHEET	5
TOTAL SHEETS	6
DWG NO.	TR C304
REV	4

See disclaimer on sheet page



LANDFORMS

THE HISTORICAL ALIGNMENTS ON THIS SECTIONS TYPICALLY TRAVERSED WAS LOCATED ALONG THE ALUVIAL FAN FORMED BY THE COYOTE RIVER. THE ALUVIAL FAN FORMED BY THE COYOTE RIVER IS THE PRINCIPAL LANDFORM IN THE VICINITY OF THE PROPOSED ALIGNMENT. THE ALUVIAL FAN IS COMPRISED OF ALLUVIAL SOILS, WHICH WERE DEPOSITED BY WATER FLOWING FROM THE COYOTE RIVER.

THE ALUVIAL FAN SURFACE FORMED HAS BEEN ALTERED BY HUMAN ACTIVITIES AND DEVELOPMENT OF AGRICULTURAL, RESIDENTIAL AND COMMERCIAL PROPERTIES. THE ALUVIAL FAN SURFACE HAS BEEN MODIFIED BY THE CONSTRUCTION OF NUMEROUS BUILDINGS, STREETS.

SOIL CLASSIFICATIONS

UNIT	PRIMARY STRATUM DESCRIPTION	INCLUDED IN THIS UNIT (SEE NOTE 1)
1	SILTY CLAYEY SAND (SC) - MEDIUM DENSE TO DENSE - MEDIUM STIFF TO HARD - MOISTURE CONDITION: MOIST TO VERY MOIST - COLOR: BROWN TO DARK BROWN	SOIL UNITS: FILL/OTHER, ALLUVIUM, ALLUVIAL FAN DEPOSITS, ALLUVIAL FAN DEPOSITS (OLDER), ALLUVIAL FAN DEPOSITS (OLDEST) - ALLUVIAL FAN DEPOSITS (OLDEST) IS NOTED
2	SANDY AND GRAVELLY SILT (ML/GM) - MEDIUM DENSE TO VERY DENSE - MOISTURE CONDITION: MOIST	ALLUVIUM - ALLUVIAL FAN DEPOSITS (OLDER) - ALLUVIAL FAN DEPOSITS (OLDEST) - ALLUVIAL FAN DEPOSITS (OLDEST) IS NOTED

DESIGN WATER LEVELS

1. (GW) - MAXIMUM WATER LEVEL FOR CONSTRUCTION (BASED ON HISTORIC DATA)
2. (GW) - MINIMUM WATER LEVEL FOR CONSTRUCTION (BASED ON HISTORIC DATA)

NOTES

1. SOIL TYPE DESCRIPTIONS PRESENTED ON THESE SUBSURFACE PROFILES VARY SLIGHTLY FROM THE SOIL TYPE DESCRIPTIONS PRESENTED IN THE GEOTECHNICAL DESIGN REPORT. THE SOIL TYPE DESCRIPTIONS PRESENTED IN THE GEOTECHNICAL DESIGN REPORT ARE BASED ON THE SOIL TYPE DESCRIPTIONS PRESENTED IN THE GEOTECHNICAL DATA REPORTS.
2. THE GEOTECHNICAL DESIGN REPORT DOES NOT INCLUDE THE SUBSURFACE PROFILES PRESENTED IN THE GEOTECHNICAL DATA REPORTS. THE GEOTECHNICAL DESIGN REPORT DOES INCLUDE THE SOIL TYPE DESCRIPTIONS PRESENTED IN THE GEOTECHNICAL DATA REPORTS.
3. THE FILL/OTHER SOIL TYPE DESCRIPTIONS PRESENTED ON THESE SUBSURFACE PROFILES INCLUDE ALL SOIL TYPE DESCRIPTIONS PRESENTED IN THE GEOTECHNICAL DATA REPORTS.
4. LAYER THICKNESS DUE TO A SOIL GRADUALLY OR SOIL PRESENTS FLUCTUATIONS (VERTICALLY AND/OR HORIZONTALLY) BETWEEN TWO SOIL TYPES, THE SOIL TYPE DESCRIPTION IS PRESENTED AS A TRANSITION. THE SOIL TYPE DESCRIPTIONS PRESENTED ON THESE SUBSURFACE PROFILES INCLUDE THE SOIL TYPE DESCRIPTIONS PRESENTED IN THE GEOTECHNICAL DATA REPORTS. THE GEOTECHNICAL DESIGN REPORT DOES NOT INCLUDE THE SOIL TYPE DESCRIPTIONS PRESENTED IN THE GEOTECHNICAL DATA REPORTS. THE GEOTECHNICAL DESIGN REPORT DOES INCLUDE THE SOIL TYPE DESCRIPTIONS PRESENTED IN THE GEOTECHNICAL DESIGN REPORT.
5. UPPER SURFACE REPRESENTS THE ELEVATION OF TUNNEL, VERTICAL CLEARANCE OF STATION.
6. (GW) - DENOTES GROUNDWATER.
7. (GW) - DENOTES GROUNDWATER PRESENTED (RANGE) PROPORTIONAL TO ELEVATION.
8. (GW)(R) - DENOTES GROUNDWATER (RANGE) PROPORTIONAL TO ELEVATION PRESENTED ON THE APPRX. ZONE SUBPROFILE.

SOURCES OF INFORMATION

1. ALIGNMENT SHOWN ON PROFILE CORRESPONDS TO APRIL 2008 QUARTERLY UPDATE ALIGNMENT SCHEMATIC, PROVIDED BY HMM, DATED 04/08/2008 (ALIGNMENT SCHEMATIC).
2. BOREHOLE AND CPT INFORMATION PROVIDED BY AL-TEC DRILLING APRIL 15 TO APRIL 24, 2008.
3. GEOTECHNICAL INFORMATION FROM SAMPLE GEOTECHNICAL REPORT FOR(S)-2008-001-CER-GEO AND 2008.
4. SLURRY WALL AND TOE WALL INFORMATION FROM SAMPLE GEOTECHNICAL REPORT FOR(S)-2008-001-CER-GEO AND 2008.

SOIL TYPES

- MELLOWED GRAVEL (GW)
- POORLY GRADED GRAVEL (GP)
- MELLOWED GRAVEL WITH SILT (GW-GM)
- MELLOWED GRAVEL WITH CLAY (GW-GC)
- POORLY GRADED GRAVEL WITH SILT (GP-GM)
- POORLY GRADED GRAVEL WITH CLAY (GP-GC)
- SILTY GRAVEL (GM)
- CLAYEY GRAVEL (GC)
- SILTY CLAYEY GRAVEL (GC-GM)
- WELL GRADED SAND (SW)
- POORLY GRADED SAND (SP)
- WELL GRADED SAND WITH SILT (SW-SM)
- WELL GRADED SAND WITH CLAY (SW-SC)
- POORLY GRADED SAND WITH SILT (SP-SM)
- POORLY GRADED SAND WITH CLAY (SP-SC)
- SILTY SAND (SM)
- CLAYEY SAND (SC)
- SILTY CLAYEY SAND (SC-SM)
- SILTY SANDY/SANDY SILT (ZONE MIX)
- CLAYEY SANDY/CLAYEY SILT (ZONE MIX)
- SILT (ML)
- CLAYEY SILT (CL)
- SILT CLAY (CL-ML)
- LEAN PLASTICITY ORGANICS (OL)
- SILT (MH)
- CLAY (CH)
- CLAYEY SILT (CH-MH)
- HIGH PLASTICITY ORGANICS (OH)
- SEDIMENT WITH ORGANIC ADMIXTURE (ZONE MIX)
- FILL
- NO DATA

SOIL BORING LEGEND

- LAYERS TYPE IS CLEARLY DEFINED BY SOIL SAMPLE
- LAYERS TYPE IS NOT CLEARLY DEFINED BY SOIL SAMPLE
- SOIL SAMPLE NOT AVAILABLE
- SOIL SAMPLE NOT AVAILABLE
- SOIL SAMPLE NOT AVAILABLE
- SP ZONE NOT EVALUATED

PENETRATION RESISTANCE SYMBOLS

- 5 BLOWS/FT SPT BLOW COUNT
- 5 BLOWS/FT SPT BLOW COUNT
- 5 BLOWS/FT WEIGHT OF HAMMER
- 5 BLOWS/FT WEIGHT OF HAMMER
- PENETRATION RESISTANCE (SYMBOLS) BLOW COUNT (BLC)

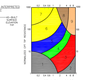
SUBSURFACE EXPLORATION

- CPT-100 CPT CONE PENETRATION
- BH-100 BOREHOLE WITH PIEZOMETER (CB PHASE)
- BH-100 BOREHOLE WITH PIEZOMETER (CB PHASE)
- BH-100 BOREHOLE WITH PIEZOMETER (CB PHASE)
- CPT-100 CPT (CB PHASE)
- BH-100 BOREHOLE WITHOUT PIEZOMETER (CB PHASE)
- BH-100 BOREHOLE WITHOUT PIEZOMETER (CB PHASE)
- CPT-100 CPT (CB PHASE)
- BH-100 BOREHOLE WITH PIEZOMETER (CB PHASE)
- BH-100 BOREHOLE WITHOUT PIEZOMETER (CB PHASE)

CPT DATA

- CPT-100 CPT (CB PHASE)
- BH-100 BOREHOLE WITH PIEZOMETER (CB PHASE)
- BH-100 BOREHOLE WITHOUT PIEZOMETER (CB PHASE)

CPT SOUNDING BOX INTERPRETED SOIL BEHAVIOR



ZONE	SOIL BEHAVIOR TYPE (SBT)
1	SENSITIVE FINE GRAINED
2	ORGANIC MATERIAL
3	CLAY TO SILTY CLAY
4	CLAYEY SILT TO SILT
5	SILTY SAND TO SANDY SILT
6	SAND TO SILTY SAND
7	SAND TO GRAVELLY SAND
8	VERY STIFF FINE GRAINED
9	SAND TO CLAYEY SAND

CONFIGURATIONS OF SOUNDINGS

- CPT SOUNDING
- BOREHOLE (CONE PENETRATION TEST)

LEGEND

- ALIGNMENT (ALIGNMENT SCHEMATIC DATED 04/08/2008)
- TUNNEL ALIGNMENT
- EXISTING GROUND SURFACE
- PROPOSED ALIGNMENT (BASED ON ALIGNMENT SCHEMATIC DATED 04/08/2008)
- SLURRY WALL TOE DEPTH

THIS FIGURE IS STILL IN PROGRESS AND WILL BE REVISED BASED ON ADDITIONAL GEOTECHNICAL INVESTIGATION RESULTS AND REVISED MATERIAL DESCRIPTIONS

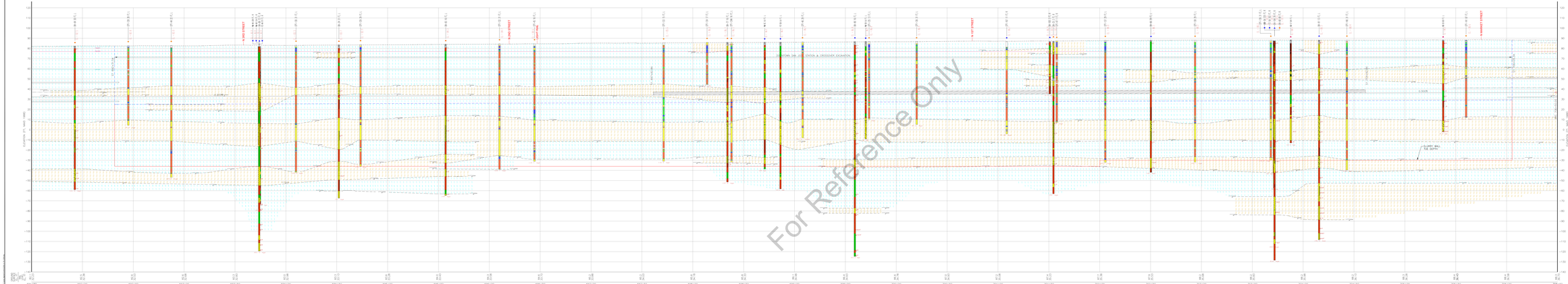
1	04/08/2008	FOR REVIEW
0	03/31/2008	FOR REVIEW
REV	DATE	DESCRIPTION

HMM / BECHTEL
A Joint Venture of Halcrow Mott MacDonald T&T, Inc. aka Bechtel Infrastructure Corp.

SILICON VALLEY RAPID TRANSIT PROJECT P0503 TUNNEL SEGMENT
INTERPRETIVE GEOLOGICAL PROFILE - STUDY SECTION 2 EAST PORTAL TO ALUM ROCK STATION

DWG. FILENAME: 2-S-TR-C.505.dwg
SCALE: H: 1:500 V: 1:100
CONTRACT NO. D3504
DWG. NO. C.505
REV. 1

See disclaimer on cover



GENERAL NOTES:

1. THIS PROFILE PRESENTS THE BEST INTERPRETATION OF SUBSURFACE CONDITIONS BASED ON THE GEOTECHNICAL INVESTIGATION DATA AVAILABLE AT THE TIME OF PREPARATION. THE ACTUAL SUBSURFACE CONDITIONS MAY VARY FROM THOSE SHOWN.
2. THE GEOLOGY PRESENTED HAS BEEN INTERPRETED FROM BORING LOGS, CPT DATA, GEOPHYSICAL DATA, AND OTHER DATA AS SHOWN. THE GEOLOGY BETWEEN BORINGS IS INTERPOLATED.
3. THE ALIGNMENT FOR BORING LOGS PRESENTED ON THIS PROFILE MAY BE OFFSET FROM THE TUNNEL ALIGNMENT. THE ALIGNMENT OF THE BORING LOGS MAY BE OFFSET FROM THE ALIGNMENT SHOWN.
4. THE GEOLOGY PRESENTED IN THE TUNNEL ALIGNMENT SHOWS THE GEOLOGY BELOW THE TUNNEL ALIGNMENT. THE GEOLOGY ABOVE THE TUNNEL ALIGNMENT IS PROJECTED FROM THE BORING LOGS. THE GEOLOGY BETWEEN THE TUNNEL ALIGNMENT AND THE BORING LOGS IS INTERPOLATED BASED ON THE BORING LOGS.
5. ALL DIMENSIONS ARE IN FEET.
6. ALL ELEVATIONS REFERENCED TO NAVD88 (NAVD 1988).
7. STATIONING REFERENCED TO SUBWAY CL TUNNEL CONSTRUCTION CENTER OF TRACKS.
8. ALL ELEVATIONS REFERENCED TO NAVD88 (NAVD 1988).
9. ALL ELEVATIONS REFERENCED TO NAVD88 (NAVD 1988).
10. ALL ELEVATIONS REFERENCED TO NAVD88 (NAVD 1988).

GEOLOGICAL INFORMATION:

1. CLOSURE WALL TOE DEPTH ESTIMATED BASED ON SAN FRANCISCO STATION CLOSURE CONCEPTS.
2. ALL DATA ON THIS PROFILE IS FOR REFERENCE ONLY.
3. GEOLOGY BELOW THE CROSSOVER CAVERNS IS BASED ON CPT DATA, CPT DATA, AND OTHER DATA.
4. GEOLOGY BELOW THE CROSSOVER CAVERNS IS BASED ON CPT DATA, CPT DATA, AND OTHER DATA.
5. FLOOR WALL DEPTH BELOW THE CROSSOVER CAVERNS IS BASED ON CPT DATA, CPT DATA, AND OTHER DATA.

THIS FIGURE IS STILL IN PROGRESS AND WILL BE REVISED BASED ON ADDITIONAL GEOTECHNICAL INVESTIGATION RESULTS AND REVISED MATERIAL DESCRIPTIONS

HMM / BECHTEL

A JOINT VENTURE OF BECHTEL INFRASTRUCTURE CORP. (50%) AND HOCHTIEF AMERICAS HOLDING (50%)

SILICON VALLEY RAPID TRANSIT PROJECT P0503 TUNNEL SEGMENT

INTERPRETIVE GEOLOGICAL PROFILE - STUDY SECTION 4 - DOWNTOWN SAN JOSE STATION AND CROSSOVER SHELL

This document is the property of HMM/Bechtel and shall not be reproduced in whole or in part without written permission.

THIS FIGURE IS STILL IN PROGRESS AND WILL BE REVISED BASED ON ADDITIONAL GEOTECHNICAL INVESTIGATION RESULTS AND REVISED MATERIAL DESCRIPTIONS

HMM / BECHTEL

A joint venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

A | ISSUED FOR REVIEW

SILICON VALLEY RAPID TRANSIT PROJECT P0503 TUNNEL SEGMENT

INTERPRETIVE GEOLOGICAL PROFILE - STUDY SECTION 6 - DIRIDON / ARENA STATION

CADD FILENAME	D300-S-TR-C307
SCALE	1"=10'H; 1"=10'V
CONTRACT NO.	D300
REV.	A
AREA CODE	TR
SHEET NO.	C307
PAGE NO.	C

See disclaimer on cover page.

D300-TR-C308 through D300-TR-C341

Cross Passage Interpretive Profiles

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

LIMITATIONS:

THE GEOTECHNICAL INFORMATION ON THESE SUBSURFACE PROFILES IS SIMPLIFIED AND DOES NOT CONTAIN ALL THE INFORMATION PRESENTED IN THE BORING LOGS PREPARED FOR 10% CONCEPTUAL ENGINEERING PHASE, THE 35% PRELIMINARY ENGINEERING PHASE AND THE 65% ENGINEERING DESIGN PHASE OF THE PROJECT. THEREFORE, THE REVIEW OF THE PROFILES MUST BE DONE IN CONJUNCTION WITH THE BORING LOGS.

THE SOIL UNIT BOUNDARIES SHOWN ARE BASED ON INTERPOLATION AND EXTRAPOLATION OF GEOTECHNICAL INFORMATION FROM ADJACENT BORINGS AND CPT PROBES. THE LOCATIONS OF THESE BOUNDARIES CAN ONLY BE INFERRED APPROXIMATELY AND DO NOT REPRESENT BASELINE CONDITIONS. REFER TO DRAWINGS D300-S-TR-C308 TO D300-S-TR-C342 FOR BASELINE CONDITIONS AT CROSS PASSAGES.

SOIL CLASSIFICATION:

UNIT	PRIMARY STRATUM DESCRIPTION	INCLUDED IN THIS UNIT (SEE NOTE 3)
1	SILTS, CLAYS AND ORGANIC SOILS. SOIL BEHAVIOR TYPE (SBT) ZONES 1 THROUGH 4, AND 9, AS CLASSIFIED BY CPTS USING ROBERTSON, 1990.	• MAN-MADE FILL CONSISTING OF ASPHALT CONCRETE AND AGGREGATE BASE ENCOUNTERED MOST OFTEN WITHIN FIVE FEET BELOW GROUND SURFACE • THIN LAYERS OF COARSE-GRAINED SOIL AS DESCRIBED IN NOTE 4
2	SANDS AND GRAVELS, AND SBT ZONES 6 THOUGH 8, CLASSIFIED BY CPTS USING ROBERTSON, 1990.	• COBBLES • THIN LAYERS OF FINE GRAINED SOIL AS DESCRIBED IN NOTE 4

DESIGN WATER LEVELS:

MAXIMUM WATER LEVEL FOR CONSTRUCTION (BASED ON HISTORIC MAXIMUM)

MAXIMUM WATER LEVEL FOR CONSTRUCTION

NOTES:

1. SOIL TYPE GRAPHICS PRESENTED ON THESE SUBSURFACE PROFILES VARY SLIGHTLY FROM THE SOIL TYPE GRAPHICS SHOWN ON THE PLAN AND PROFILES IN THE GEOTECHNICAL DATA REPORT. HOWEVER, THE UNIFIED SOIL CLASSIFICATION SYSTEM (USCS) GROUP NAMES HAVE NOT BEEN MODIFIED.
2. ALL DESIGN, CONSTRUCTION, AND MEASURED WATER LEVELS SHOWN ON THESE SUBSURFACE PROFILES REPRESENT THE PIEZOMETRIC LEVELS EXPECTED AT THE DEPTH OF THE TBM TUNNEL. THE WATER LEVELS SHOWN ARE NOT APPLICABLE FOR THE DESIGN OF OTHER SVRT PROJECT EXCAVATIONS AND STRUCTURES.
3. THE SOIL BEHAVOIR TYPE ZONE 5 – 'SAND MIXTURES – SILTY SAND AND SANDY SILT' CONSISTS OF BOTH FINE-GRAINED AND COARSE-GRAINED SOIL. THEREFORE, IT WAS INCLUDED IN EITHER SOIL CLASSIFICATION UNIT. IT IS REPRESENTED BY BLUE COLOR ON CPT STICKS.
4. LAYER THICKNESS: DUE TO A HIGH VARIABILITY IN SOIL PROFILE ENCOUNTERED VERTICALLY WITH DEPTH, AND HORIZONTALLY ALONG THE PROPOSED TUNNEL ALIGNMENT CAUSED BY THE ALLUVIAL SOIL DEPOSITIONAL ENVIRONMENT DESCRIBED UNDER REGIONAL GEOLOGY IN THIS REPORT, FIVE (5) FEET WAS SELECTED AS A 'THRESHOLD THICKNESS (th)'. THE ALTERNATING OR INDIVIDUAL SOIL LAYERS BELONGING TO EITHER SOIL CLASSIFICATION UNIT 1 & 2 THINNER THAN th WERE GROUPED ACCORDING TO A SET OF CRITERIA AND JUDGEMENT AS DESCRIBED IN AN APPENDIX OF THIS REPORT.
5. OFFSET DISTANCE PERPENDICULAR TO ALIGNMENT S1 TUNNEL. DIRECTION LOOKING UP STATION.
6. DEEPER BORINGS TRUNCATED.
7. LENGTH OF PENETRATION RESISTANCE SYMBOLS PROPORTIONAL TO RECOVERY.
8. CHANGES IN STATIONING AT NOTED LOCATIONS OCCUR DUE TO EQUATIONS PROVIDED ON THE APRIL 2008 QUARTERLY UPDATE ALIGNMENT.

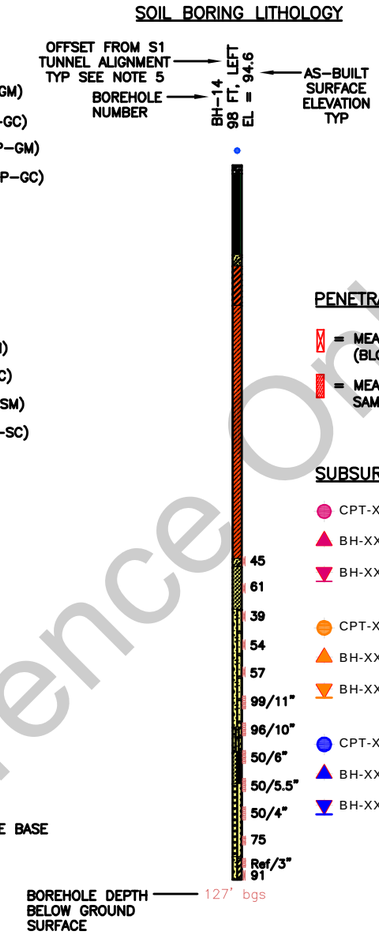
SOURCES OF INFORMATION:

1. ALIGNMENT SHOWN ON PROFILE CORRESPONDS TO APRIL 2008 QUARTERLY UPDATE ALIGNMENT SUBMITTAL.
2. CROSS PASSAGES: EMAIL FROM A. BARNGROVER, FEB. 22, 2008
3. BOREHOLE AND CPT INFORMATION PROVIDED BY A. JAIN VIA EMAILS APRIL 10 TO APRIL 14, 2008.
4. HYDROGEOLOGICAL INFORMATION FROM HMMB HYDROGEOLOGY REPORT P0503-D300-RPT-DE-020 MAY 2008.
5. SLURRY WALL AND DSM WALL INFORMATION FROM HMMB HYDROGEOLOGY REPORT P0503-D300-RPT-DE-020 MAY 2008.

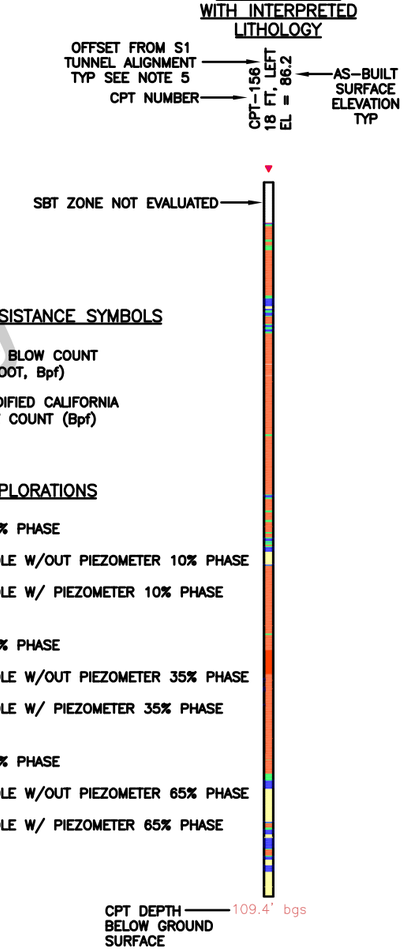
SOIL TYPE:

- WELL-GRADED GRAVEL (GW)
- POORLY-GRADED GRAVEL (GP)
- WELL-GRADED GRAVEL WITH SILT (GW-GM)
- WELL-GRADED GRAVEL WITH CLAY (GW-GC)
- POORLY-GRADED GRAVEL WITH SILT (GP-GM)
- POORLY-GRADED GRAVEL WITH CLAY (GP-GC)
- CLAYEY GRAVEL (GC)
- SILTY GRAVEL (GM)
- SILTY, CLAYEY GRAVEL (GC-GM)
- WELL-GRADED SAND (SW)
- POORLY-GRADED SAND (SP)
- WELL-GRADED SAND WITH SILT (SW-SM)
- WELL-GRADED SAND WITH CLAY (SW-SC)
- POORLY-GRADED SAND WITH SILT (SP-SM)
- POORLY-GRADED SAND WITH CLAY (SP-SC)
- SILTY SAND (SM)
- CLAYEY SAND (SC)
- SILTY, CLAYEY SAND (SC-SM)
- SILTY SAND/SANDY SILT (SM-ML)
- CLAYEY SAND/LEAN CLAY (SC/CL)
- SILT (ML)
- LEAN CLAY (CL)
- SILTY CLAY (CL-ML)
- LOW PLASTICITY ORGANICS (OL)
- ELASTIC SILT (MH)
- FAT CLAY (CH)
- CLAY (CH-MH)
- HIGH PLASTICITY ORGANICS (OH)
- PAVEMENT WITH OR WITHOUT AGGREGATE BASE
- FILL
- NO DATA

SOIL BORING LITHOLOGY



CPT SOUNDING WITH INTERPRETED LITHOLOGY



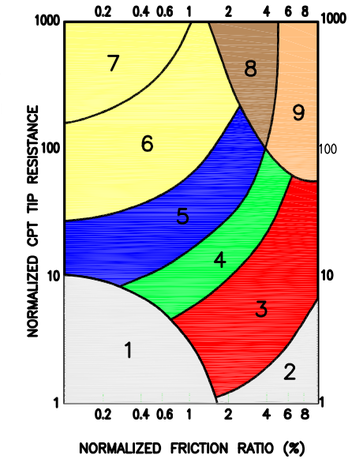
PENETRATION RESISTANCE SYMBOLS

= MEASURED SPT BLOW COUNT (BLOWS PER FOOT, Bpf)

= MEASURED MODIFIED CALIFORNIA SAMPLER BLOW COUNT (Bpf)

SUBSURFACE EXPLORATIONS

- CPT-XX CPT 10% PHASE
- BH-XX BOREHOLE W/OUT PIEZOMETER 10% PHASE
- BH-XX BOREHOLE W/ PIEZOMETER 10% PHASE
- CPT-XX CPT 35% PHASE
- BH-XX BOREHOLE W/OUT PIEZOMETER 35% PHASE
- BH-XX BOREHOLE W/ PIEZOMETER 35% PHASE
- CPT-XX CPT 65% PHASE
- BH-XX BOREHOLE W/OUT PIEZOMETER 65% PHASE
- BH-XX BOREHOLE W/ PIEZOMETER 65% PHASE



ZONE	SOIL BEHAVIOR TYPE (SBT)
1	SENSITIVE FINE-GRAINED
2	ORGANIC MATERIAL
3	CLAY TO SILTY CLAY
4	CLAYEY SILT TO SILTY CLAY
5	SILTY SAND TO SANDY SILT
6	CLEAN SANDS TO SILTY SANDS
7	GRAVELLY SAND TO SAND
8	VERY STIFF SAND TO CLAYEY SAND*
9	VERY STIFF FINE-GRAINED*

*OVERCONSOLIDATED OR CEMENTED

CPT CORRELATION CHART
(MODIFIED FROM ROBERTSON, 1990)

LEGEND

APPROXIMATE SOIL UNIT BOUNDAR

? DENOTES UNCERTAINTY ABOUT CONTINUITY OF SOIL UNIT BOUNDARY BEYOND

COLOR SCHEME FOR STABILITY RATIO PLOTS

- 4.0 TO 5.0
- 5.0 TO 6.0
- 6.0 TO 7.0
- OVER 7.0

PRELIMINARY DRAWING FOR 65% PHASE
NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D. YOUNG
DRAWN BY J. TAI
CHECKED BY A. BARNGROVER
IN CHARGE D. YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor
BART

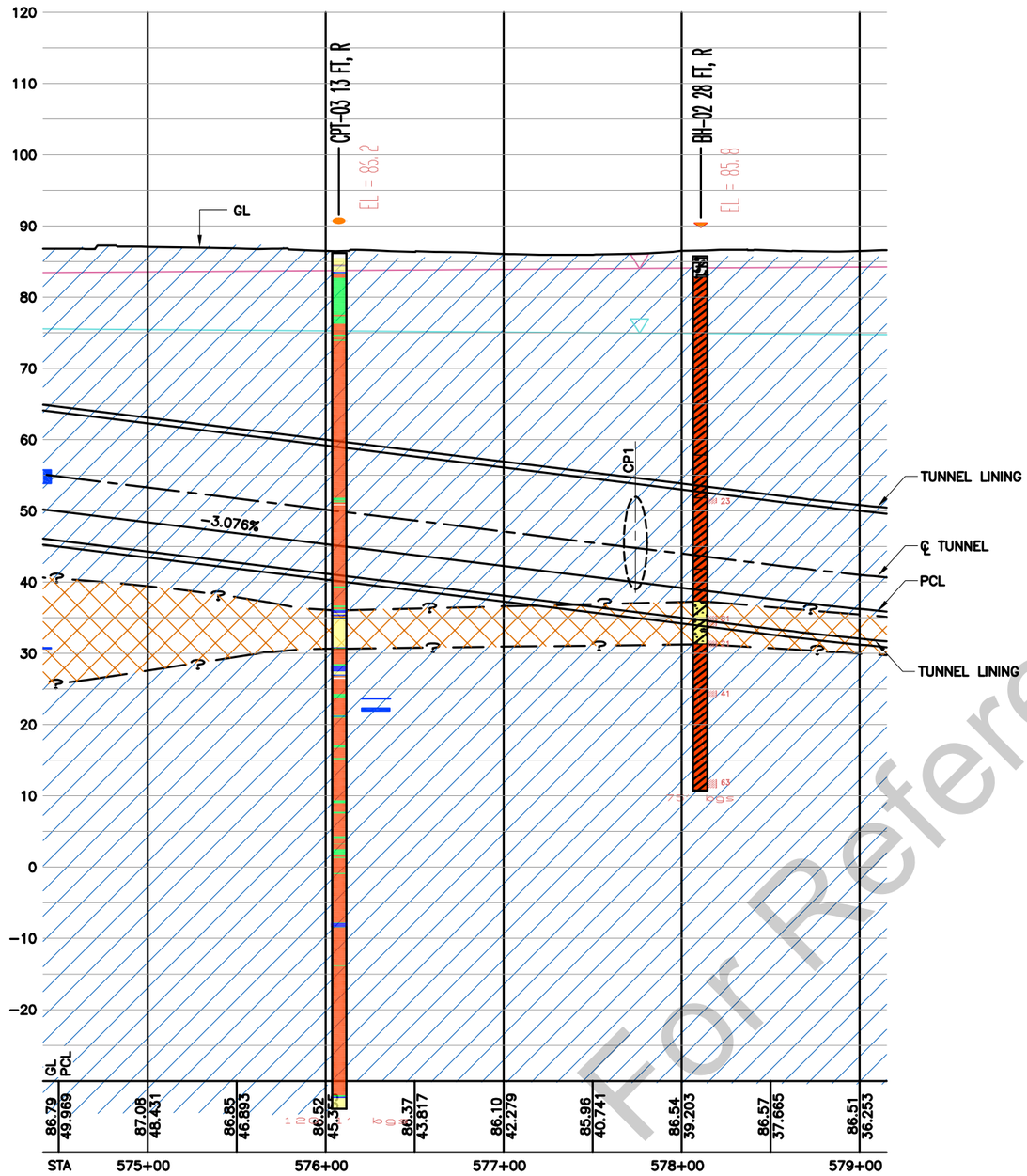
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
AT CROSS PASSAGES
LEGEND

CADD FILENAME C330-S-TR-C308.DWG
SIZE D | SCALE NOT TO SCALE
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C308 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:02pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C308.dwg

See disclaimer on cover page.



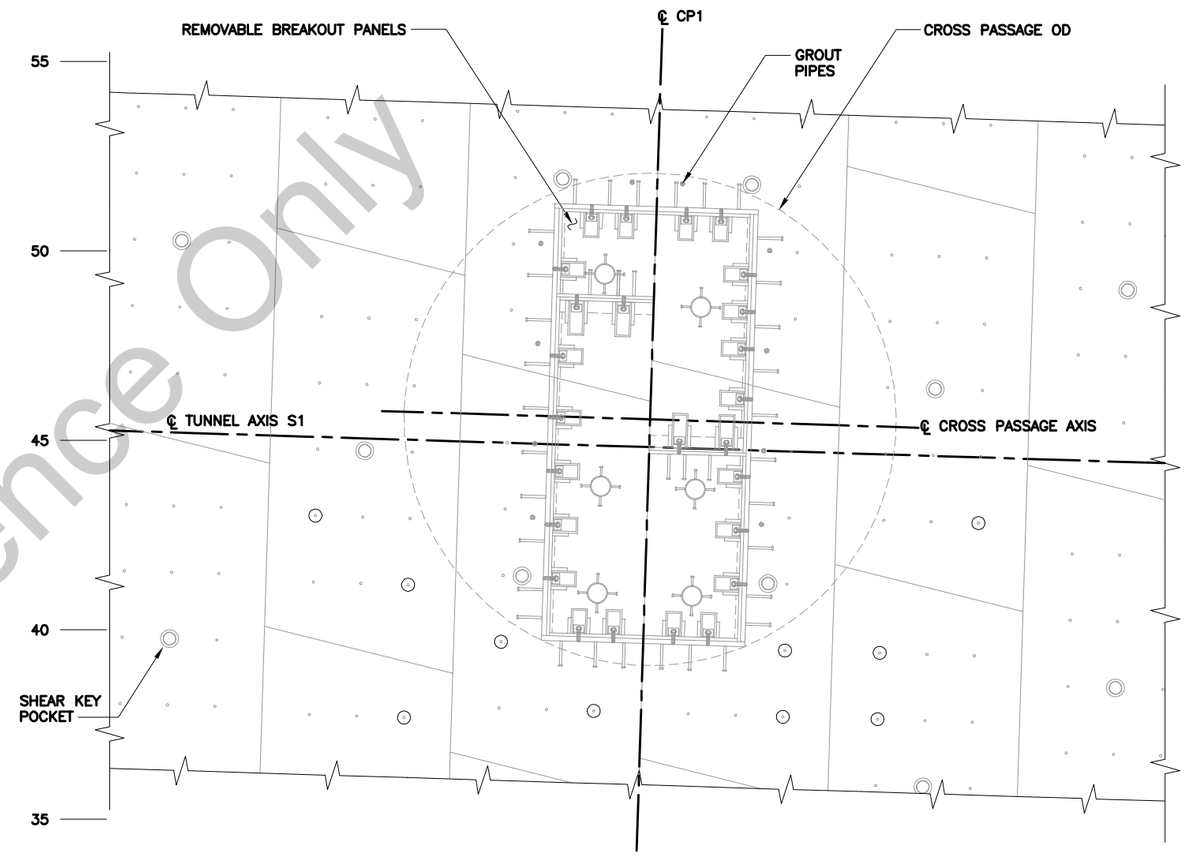
SOIL PROFILE AT CROSS PASSAGE 1 (A / TR–C309)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	VACUUM WELLPOINTS
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR–C309)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT PO503–D300–RPT–GEO–006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

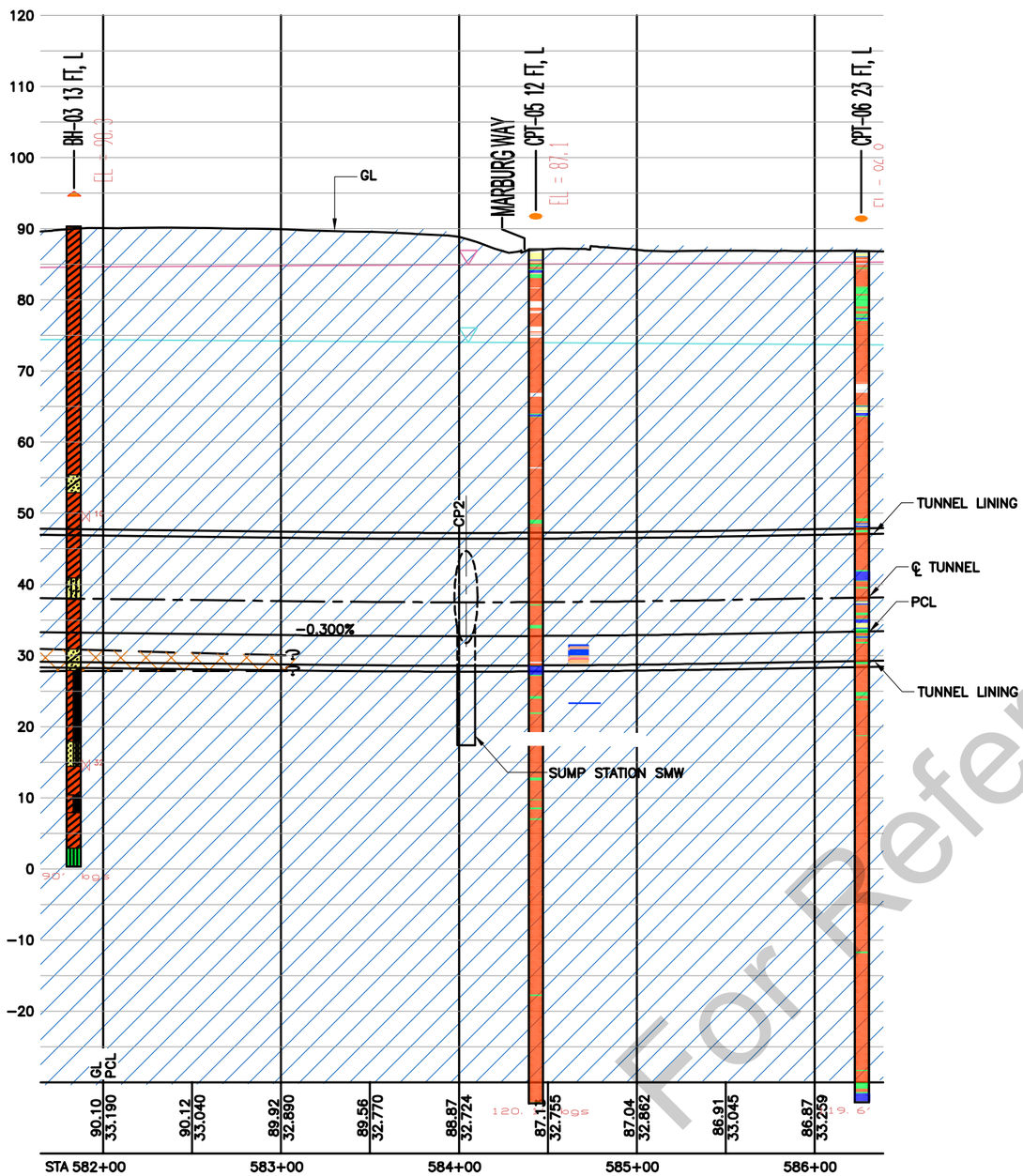
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 1

CADD FILENAME C330–S–TR–C309.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C309 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:03pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C309.dwg

See disclaimer on cover page.



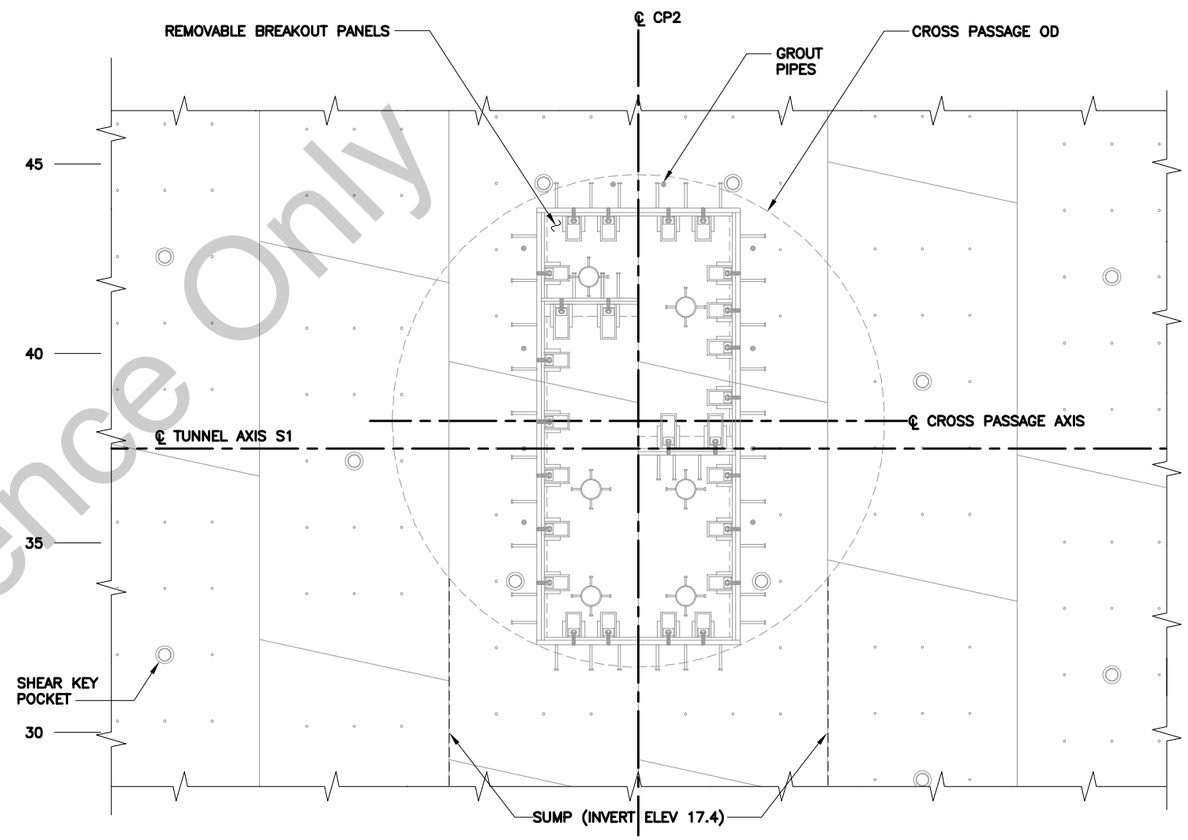
SOIL PROFILE AT CROSS PASSAGE 2 (A / TR-C310)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER
SMW	SUMP MARBURG WAY



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	13 FT

DEVELOPED ELEVATION (B / TR-C310)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

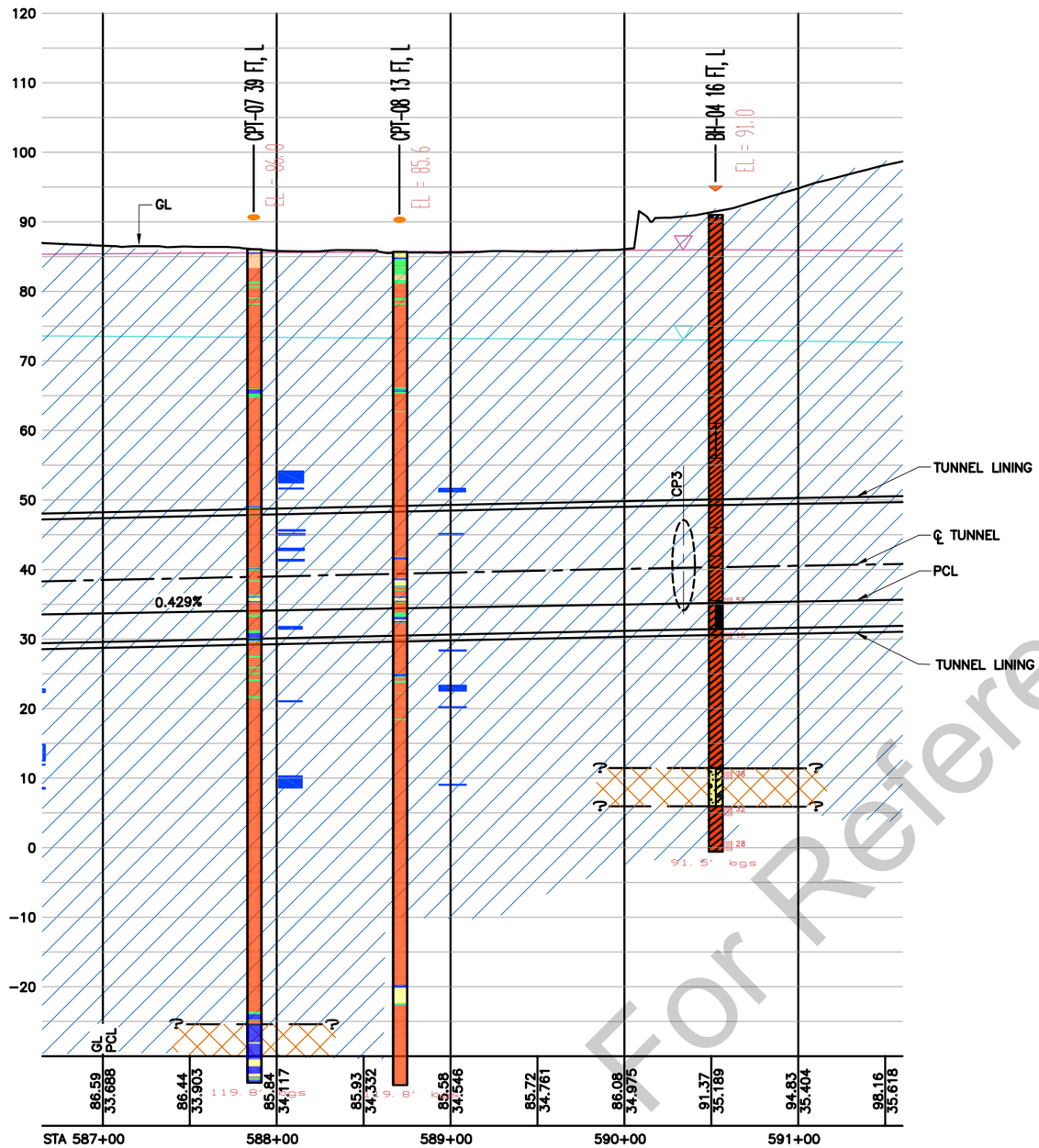
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 2

CADD FILENAME C330-S-TR-C310.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C310 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:04pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C310.dwg

See disclaimer on cover page.



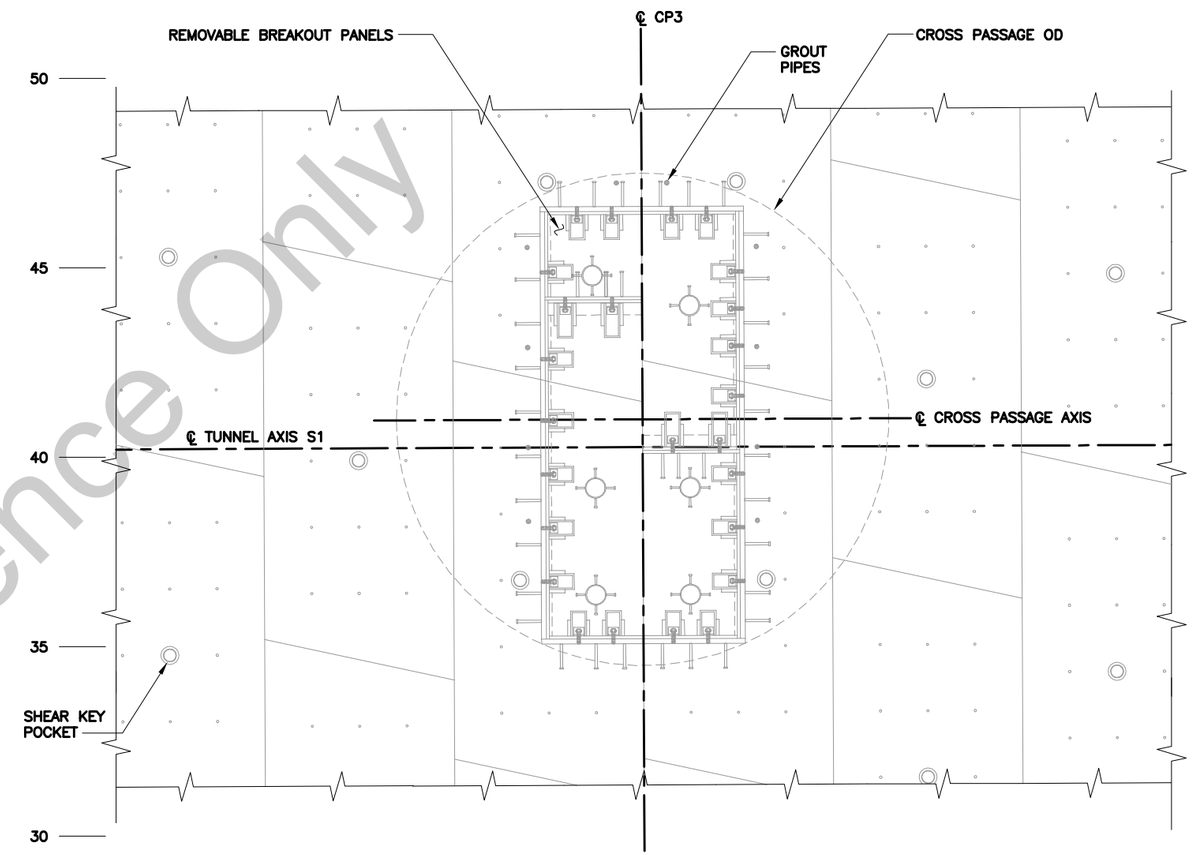
SOIL PROFILE AT CROSS PASSAGE 3 (A / TR-C311)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	NONE
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C311)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

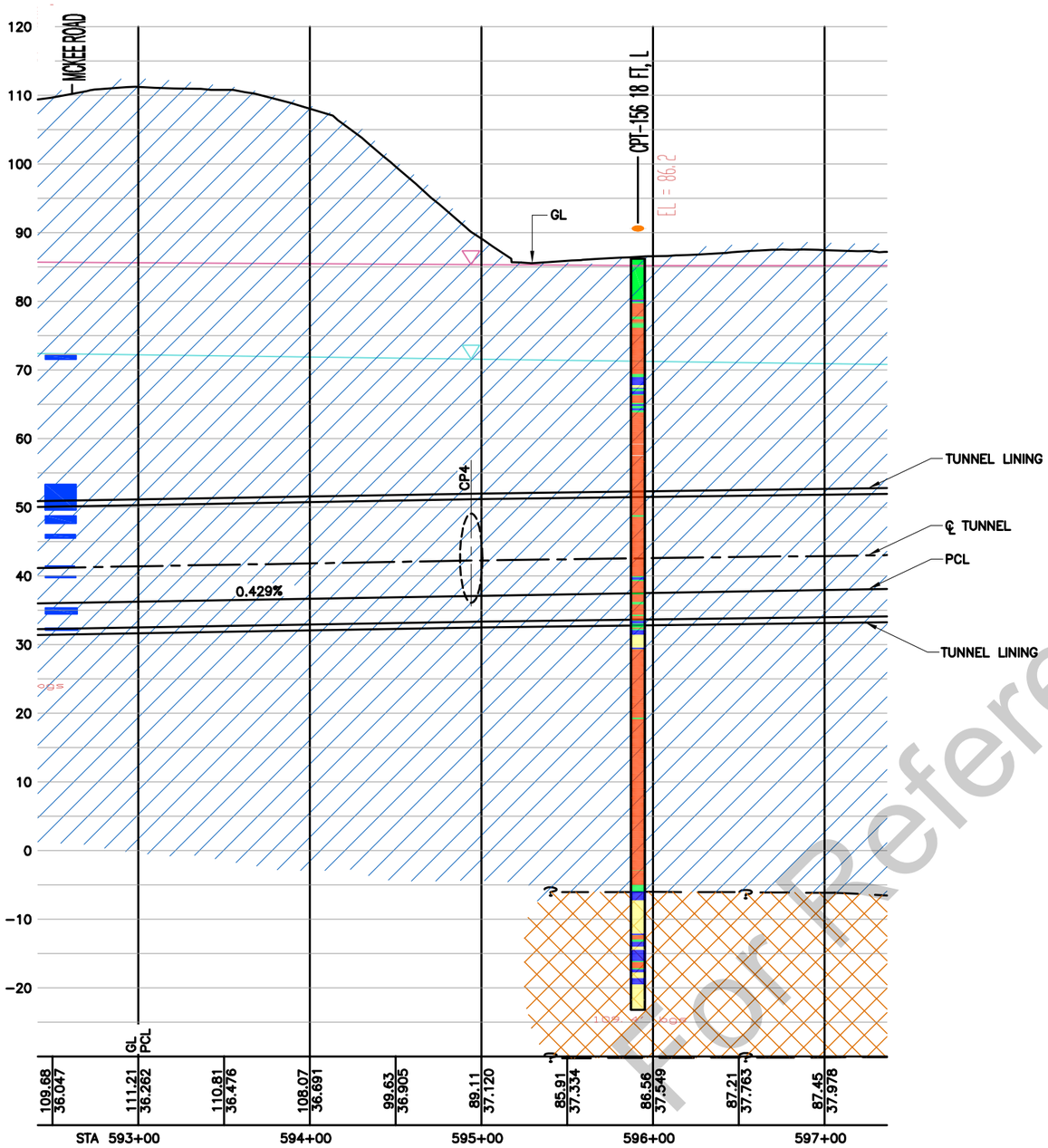
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 3

CADD FILENAME C330-S-TR-C311.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C311 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:05pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C311.dwg

See disclaimer on cover page.

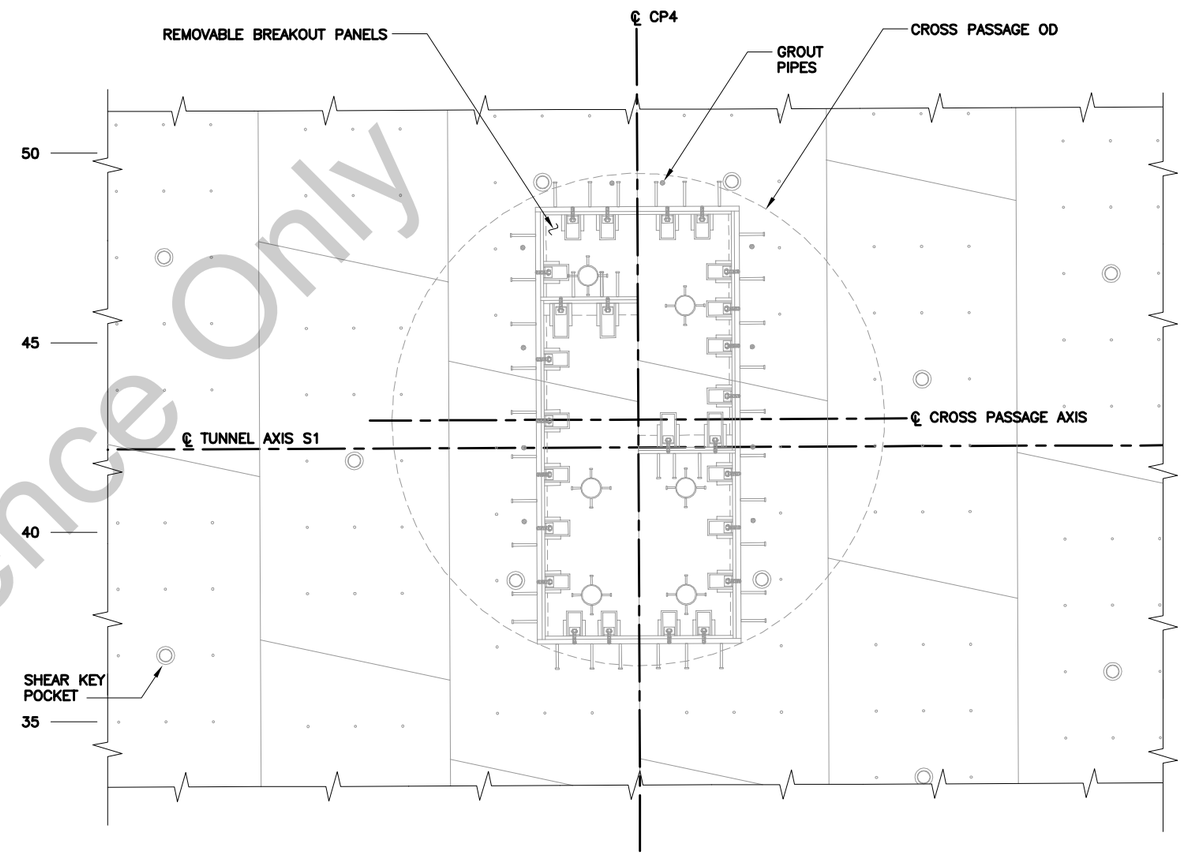


SOIL PROFILE AT CROSS PASSAGE 4 (A / TR-C312)

SCALE: 1"=40'H 1"=10'V

LEGEND:

□	PERMEATION GROUTING HOLE	STA	STATIONING ON S1 TUNNEL
○	VACUUM WELLPOINT	PCL	PROFILE CONTROL LINE
◦	CONCRETE SEGMENT LINING DIMPLE	GL	GROUND LEVEL
		OD	OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	NONE
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C312)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

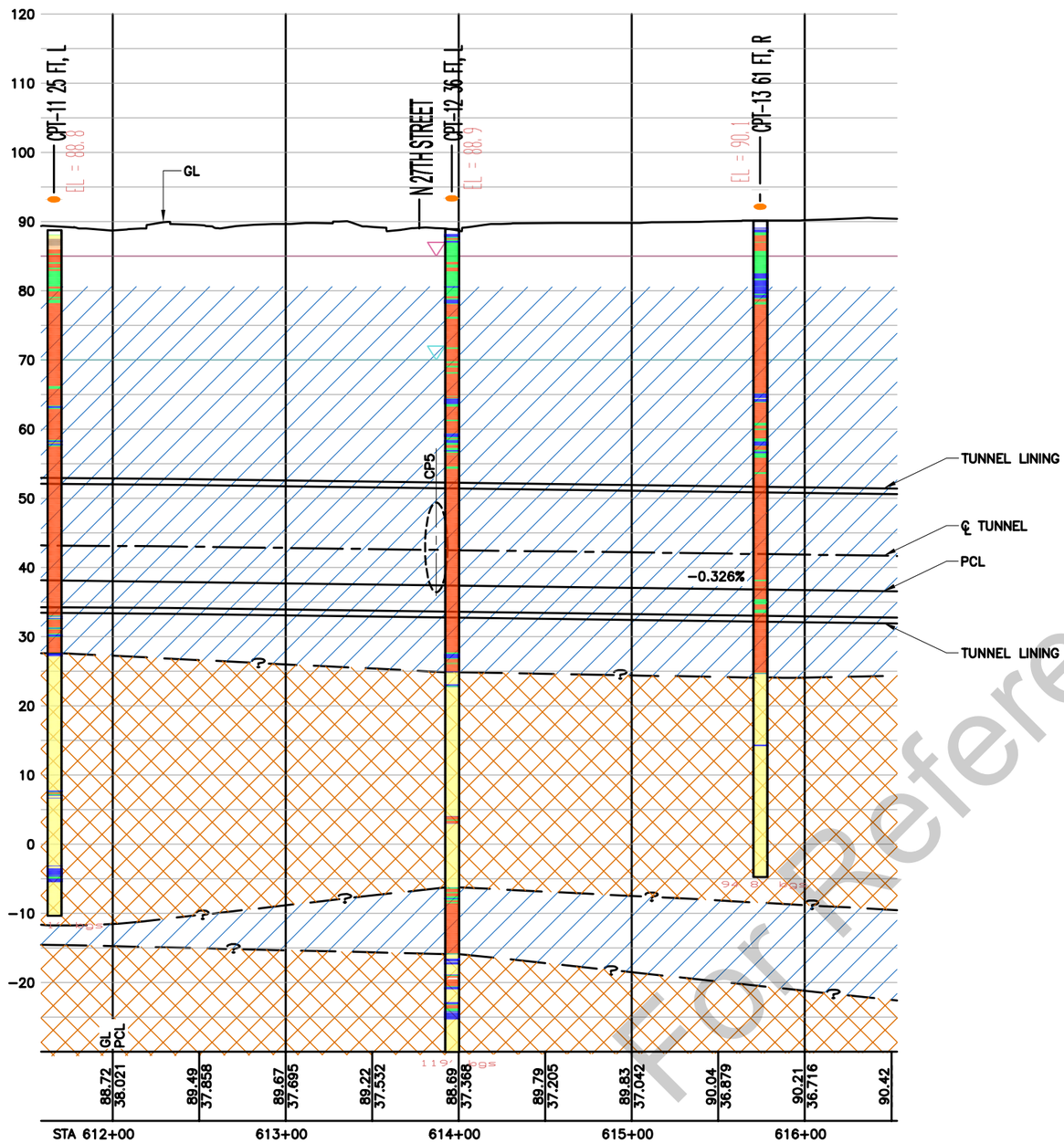
PORTALS, TUNNELS AND CRESS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 4

CADD FILENAME C330-S-TR-C312.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C312 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:06pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C312.dwg

See disclaimer on cover page.



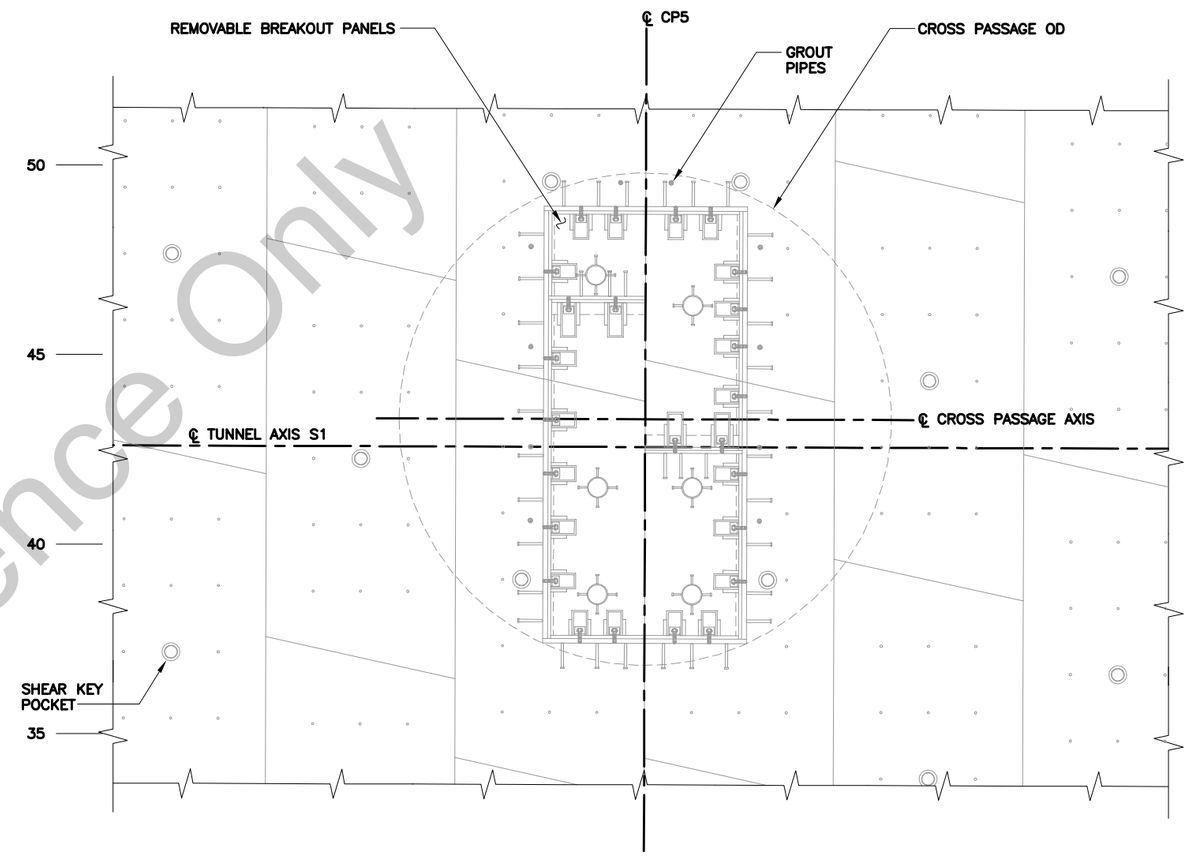
SOIL PROFILE AT CROSS PASSAGE 5 (A / TR-C313)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA STATIONING ON S1 TUNNEL
PCL PROFILE CONTROL LINE
GL GROUND LEVEL
OD OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	NONE
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C313)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

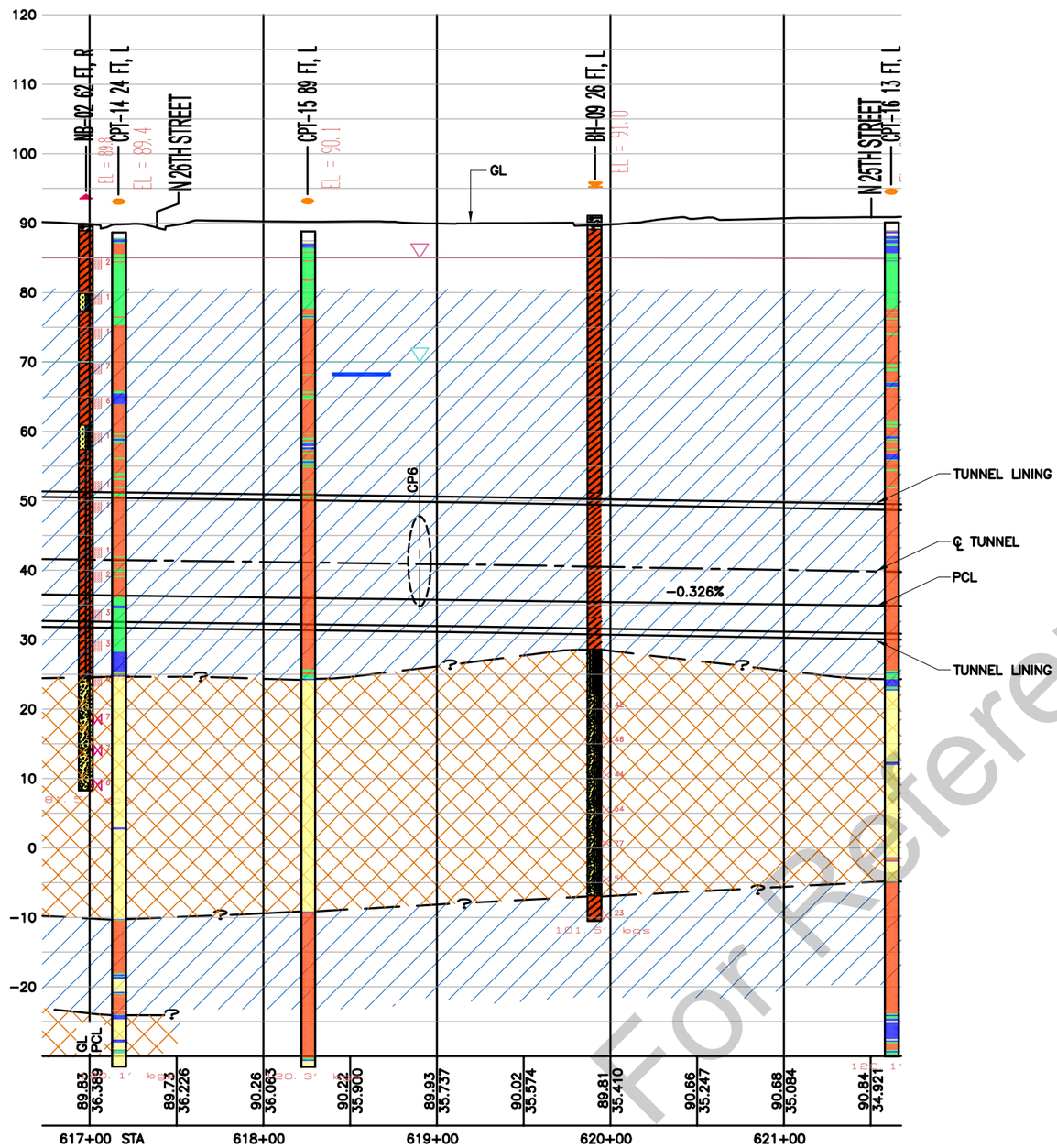
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 5

CADD FILENAME C330-S-TR-C313.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C313 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:07pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C313.dwg

See disclaimer on cover page.



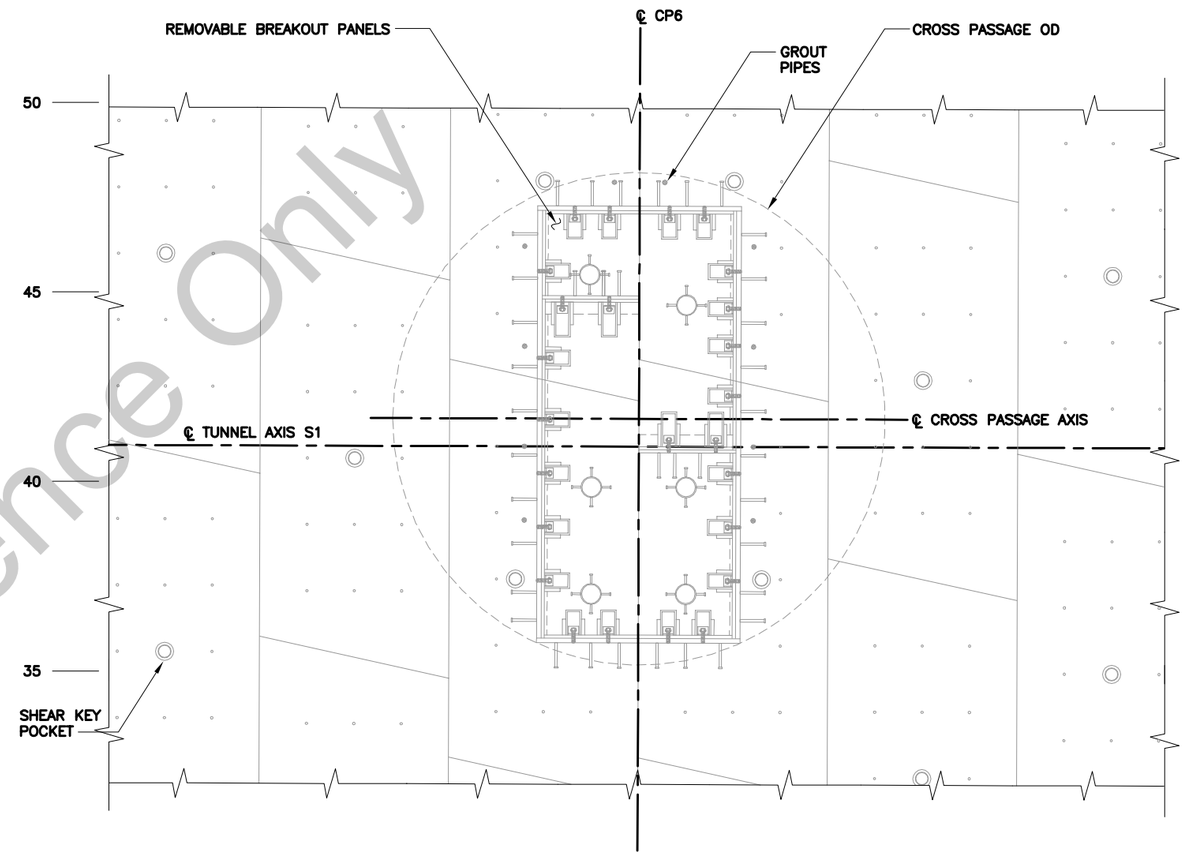
SOIL PROFILE AT CROSS PASSAGE 6 (A / TR-C314)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA STATIONING ON S1 TUNNEL
PCL PROFILE CONTROL LINE
GL GROUND LEVEL
OD OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	NONE
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C314)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT PO503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED ____ APPROVED ____

VTA Silicon Valley Rapid Transit Corridor — BART

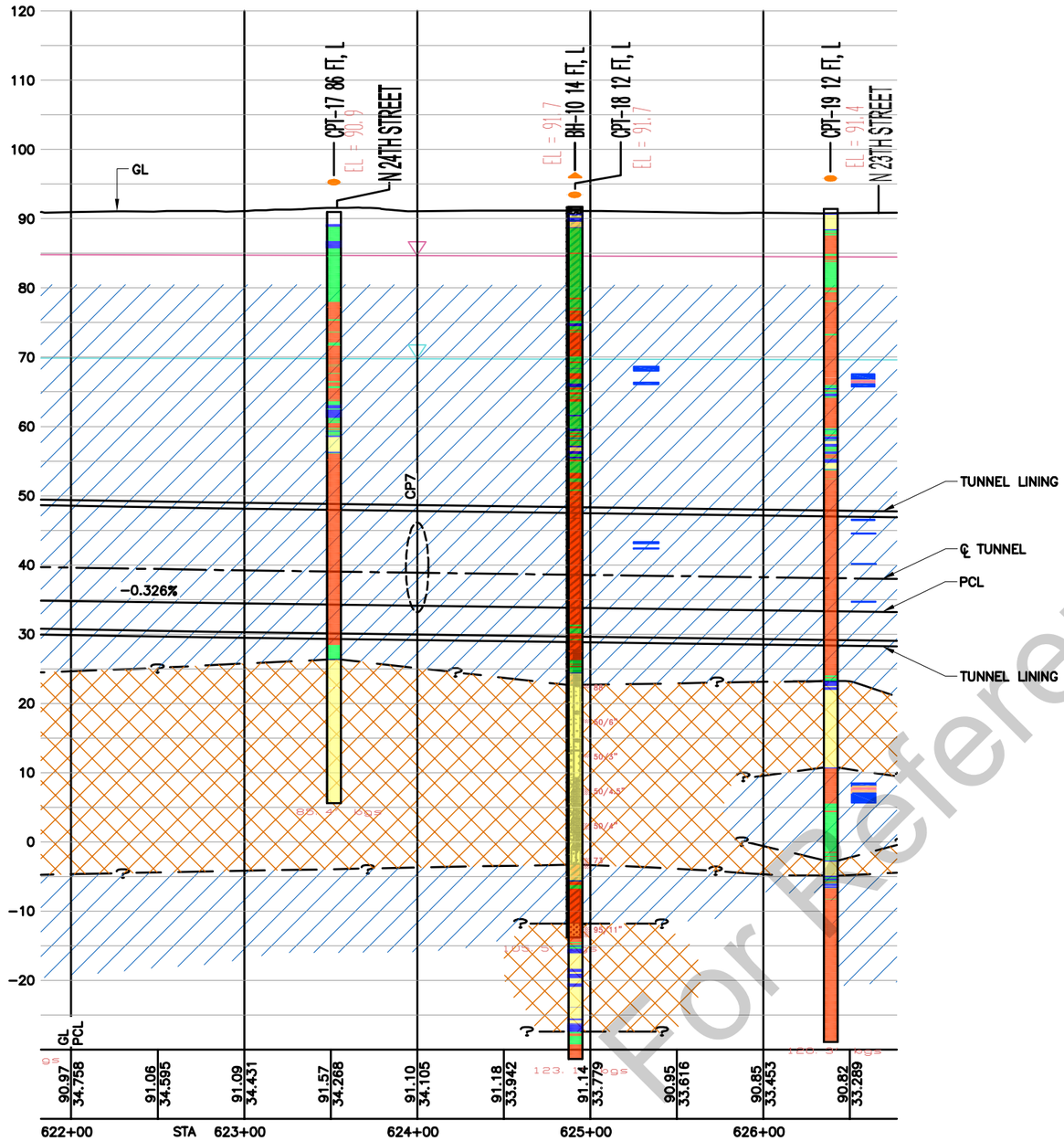
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 6

CADD FILENAME C330-S-TR-C314.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C314 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:07pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C330-S-TR-C314.dwg

See disclaimer on cover page.



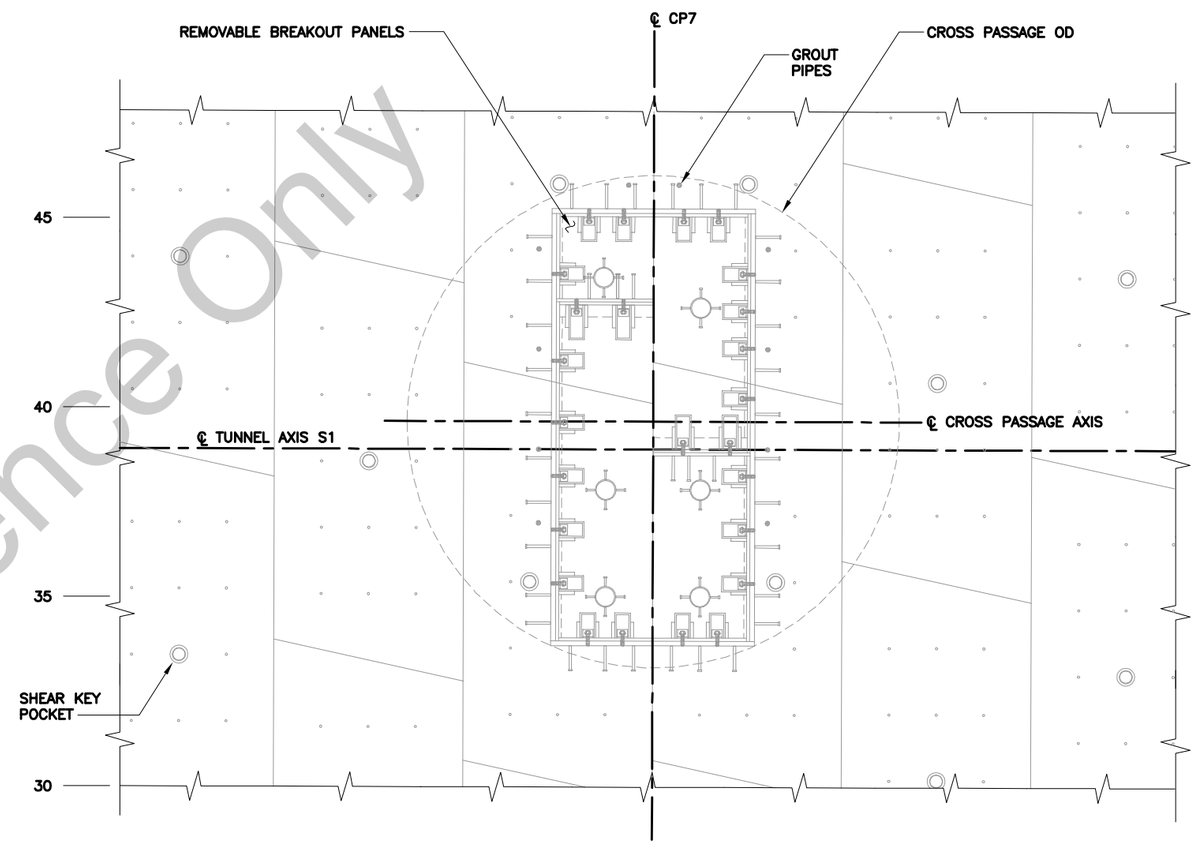
SOIL PROFILE AT CROSS PASSAGE 7 (A / TR-C315)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	NONE
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C315)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT PO503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor

PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 7

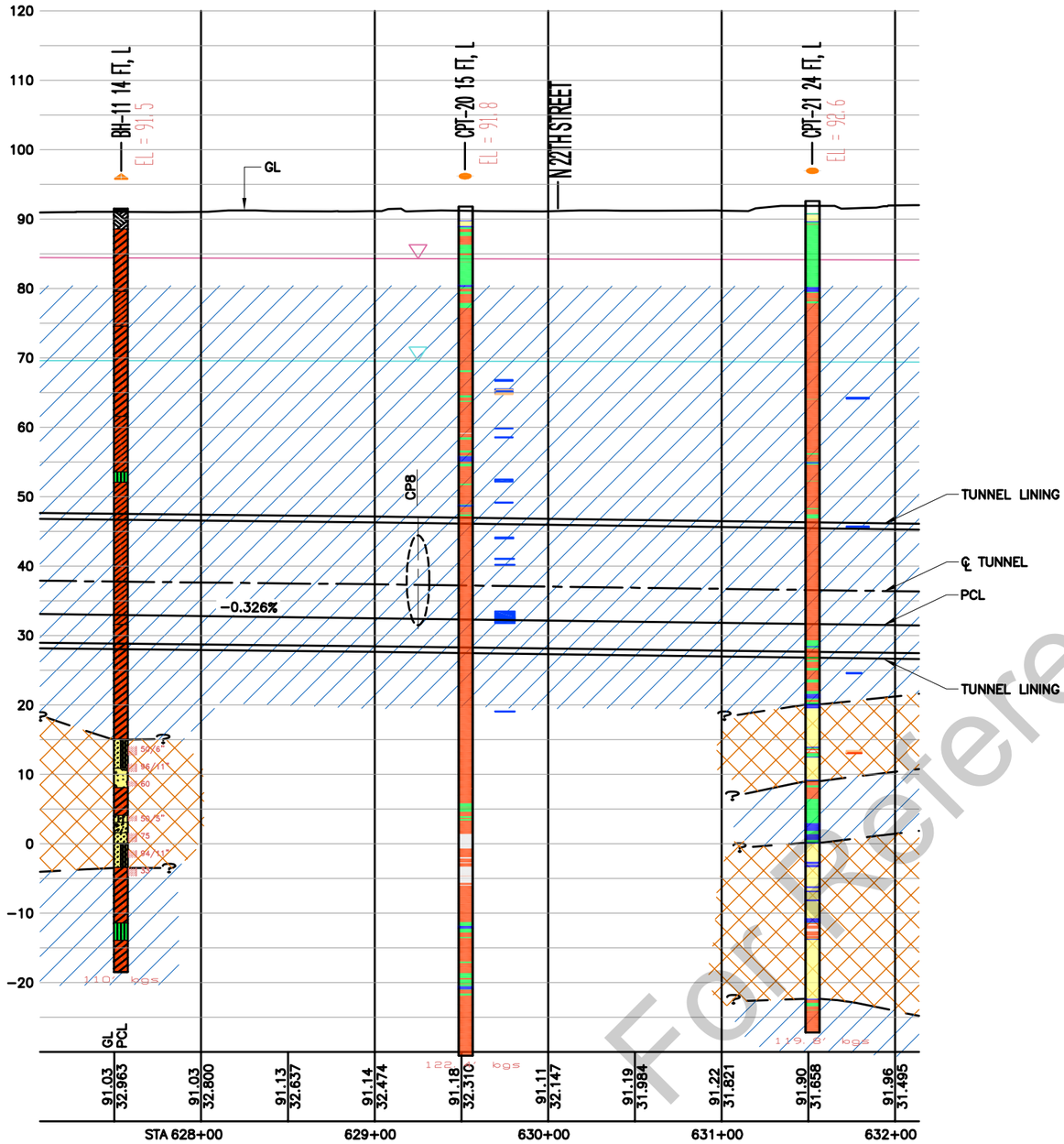
CADD FILENAME C330-S-TR-C315.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C315 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:08pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C315.dwg

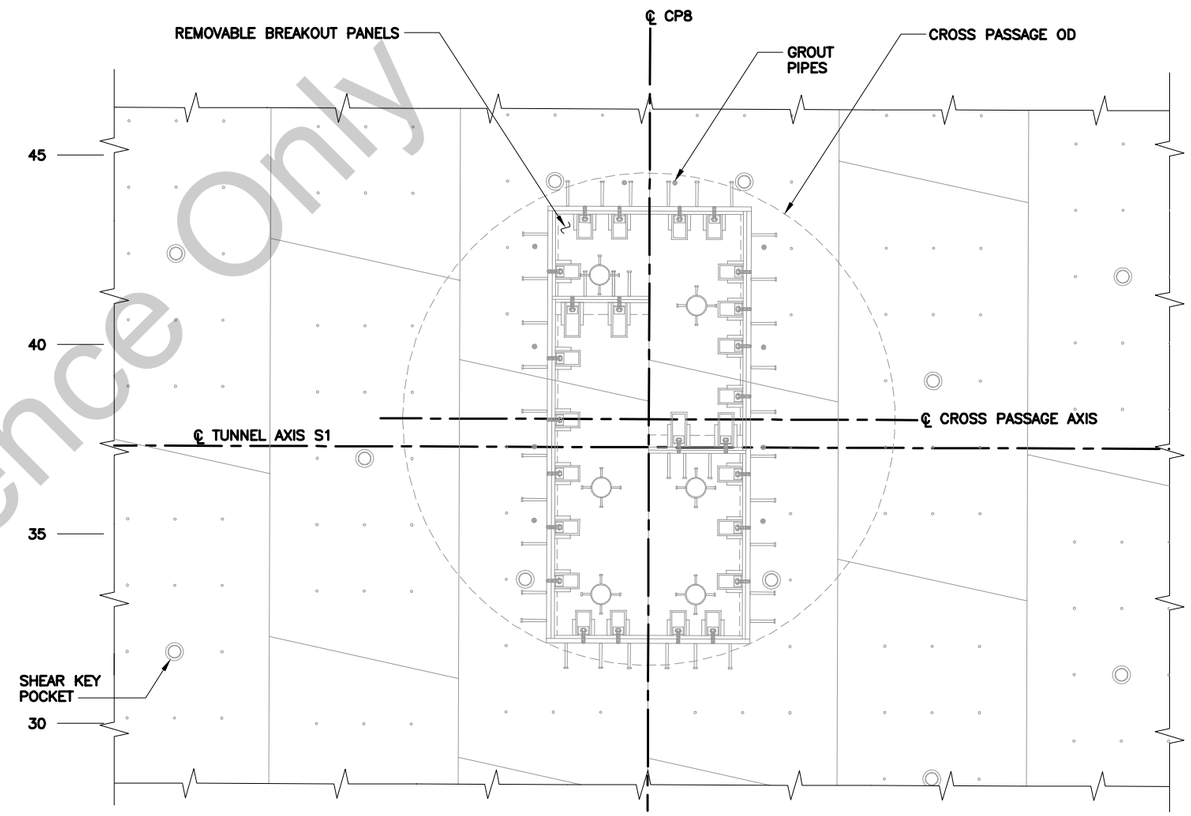
See disclaimer on cover page.

LEGEND:

□	PERMEATION GROUTING HOLE	STA	STATIONING ON S1 TUNNEL
○	VACUUM WELLPOINT	PCL	PROFILE CONTROL LINE
◦	CONCRETE SEGMENT LINING DIMPLE	GL	GROUND LEVEL
		OD	OUTSIDE DIAMETER



SOIL PROFILE AT CROSS PASSAGE 8 (A / TR-C316)
SCALE: 1"=40'H 1"=10'V



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	NONE
CP DIAMETER	13 FT

DEVELOPED ELEVATION (B / TR-C316)
SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT PO503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

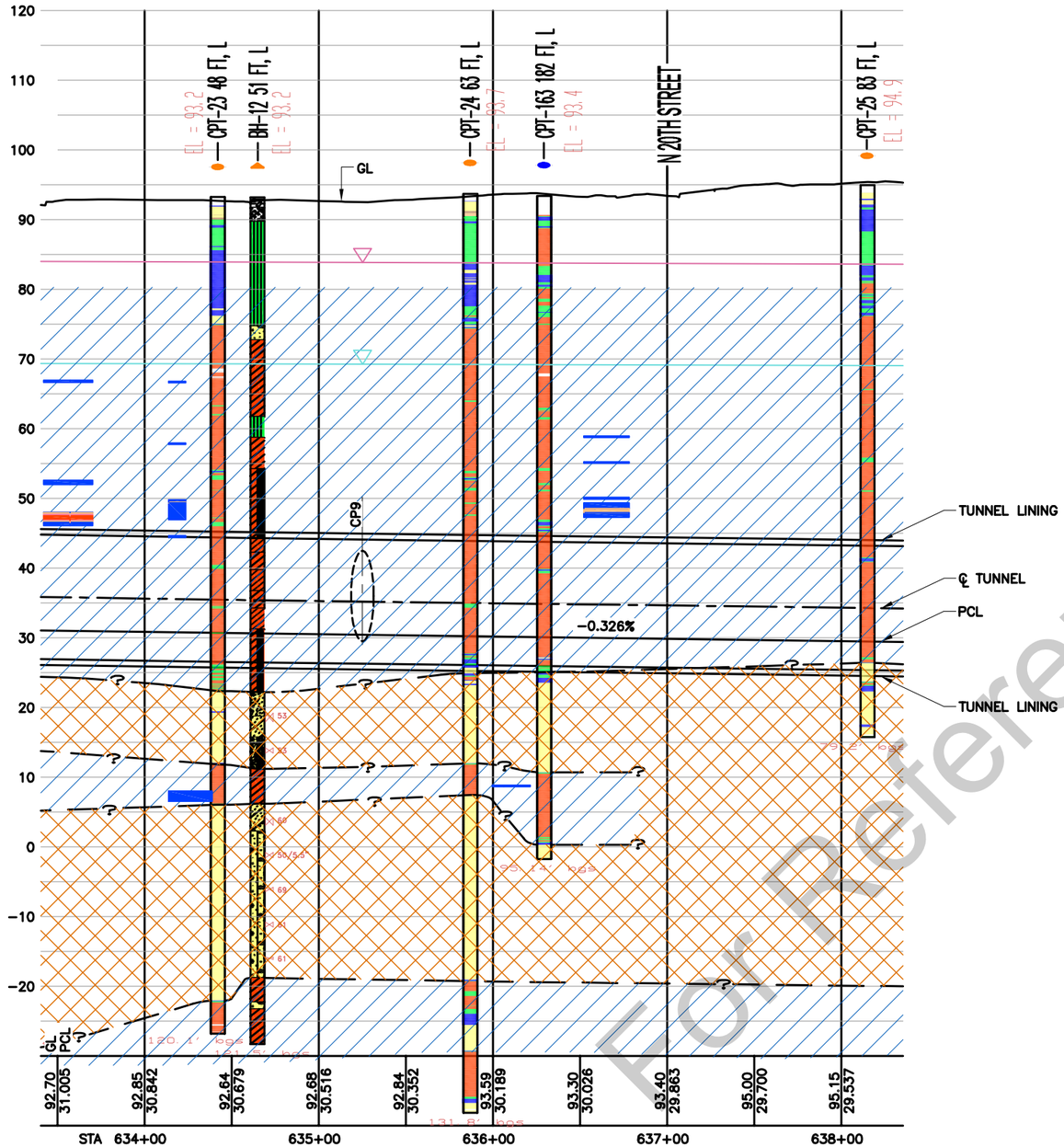
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 8

CADD FILENAME C330-S-TR-C316.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C316 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:09pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C330-S-TR-C316.dwg

See disclaimer on cover page.



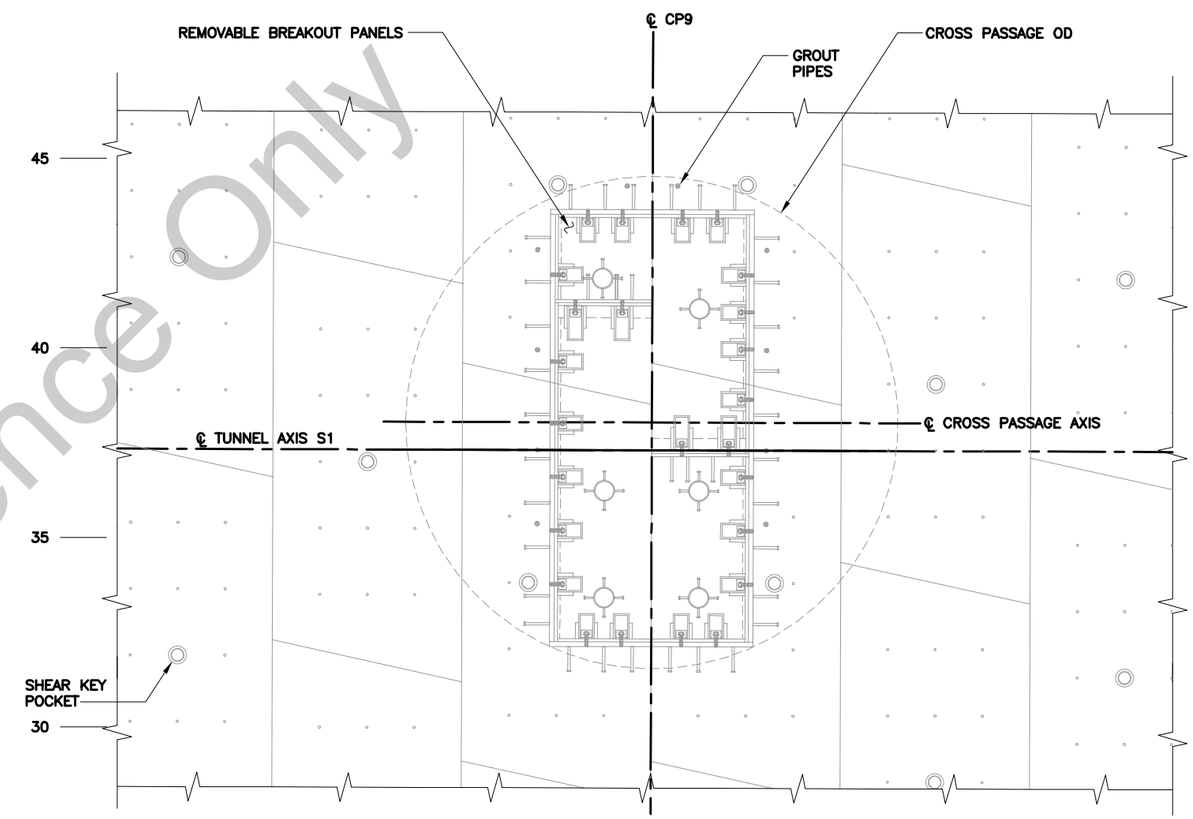
SOIL PROFILE AT CROSS PASSAGE 9 (A / TR–C317)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA STATIONING ON S1 TUNNEL
PCL PROFILE CONTROL LINE
GL GROUND LEVEL
OD OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	NONE
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR–C317)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT PO503–D300–RPT–GEO–006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

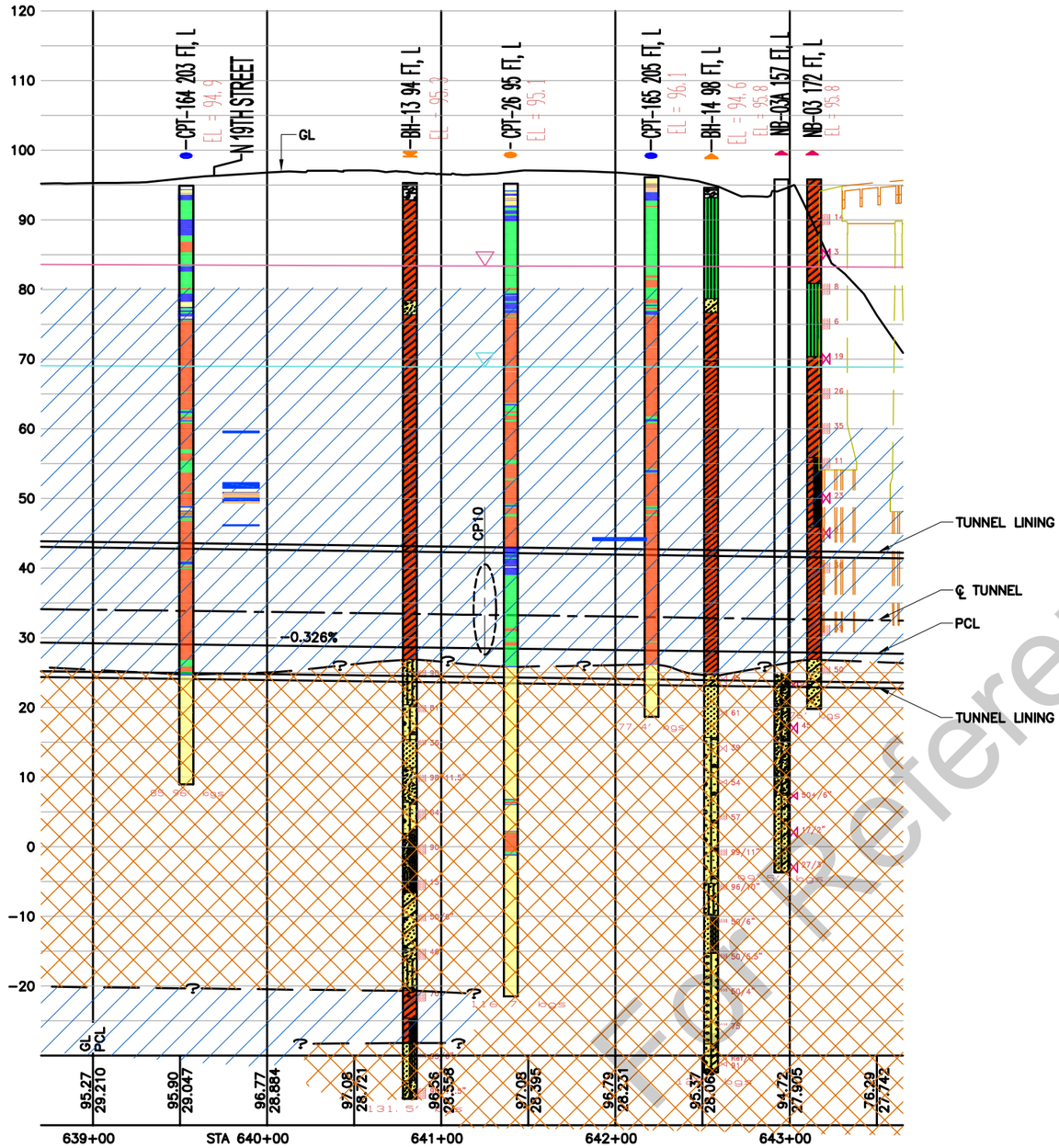
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 9

CADD FILENAME C330–S–TR–C317.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C317 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:10pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C317.dwg

See disclaimer on cover page.

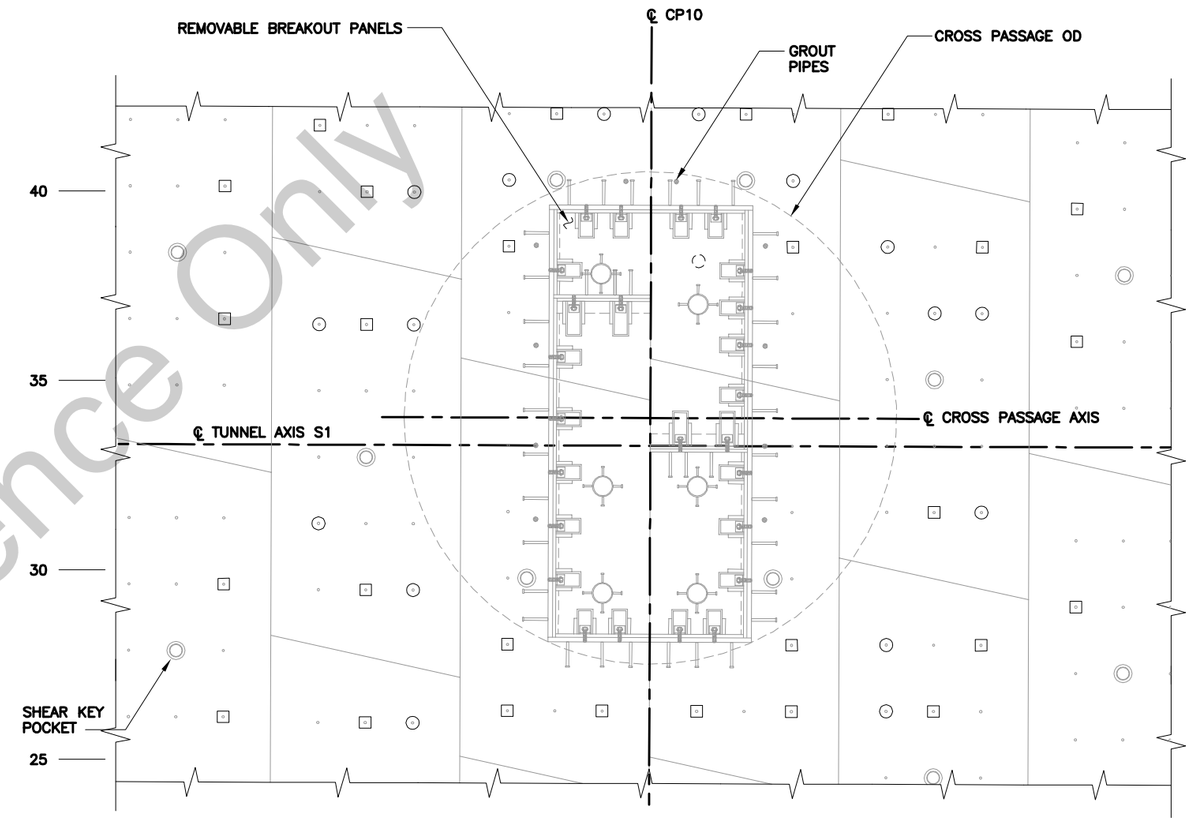


SOIL PROFILE AT CROSS PASSAGE 10 (A / TR-C318)

SCALE: 1"=40'H 1"=10'V

LEGEND:

□	PERMEATION GROUTING HOLE	STA	STATIONING ON S1 TUNNEL
○	VACUUM WELLPOINT	PCL	PROFILE CONTROL LINE
◦	CONCRETE SEGMENT LINING DIMPLE	GL	GROUND LEVEL
◐	VACUUM WELLPOINT INSTALLED AFTER PANEL REMOVED, IF REQUIRED	OD	OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	PERMEATION GROUT/VACUUM WELL POINTS
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C318)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT PO503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

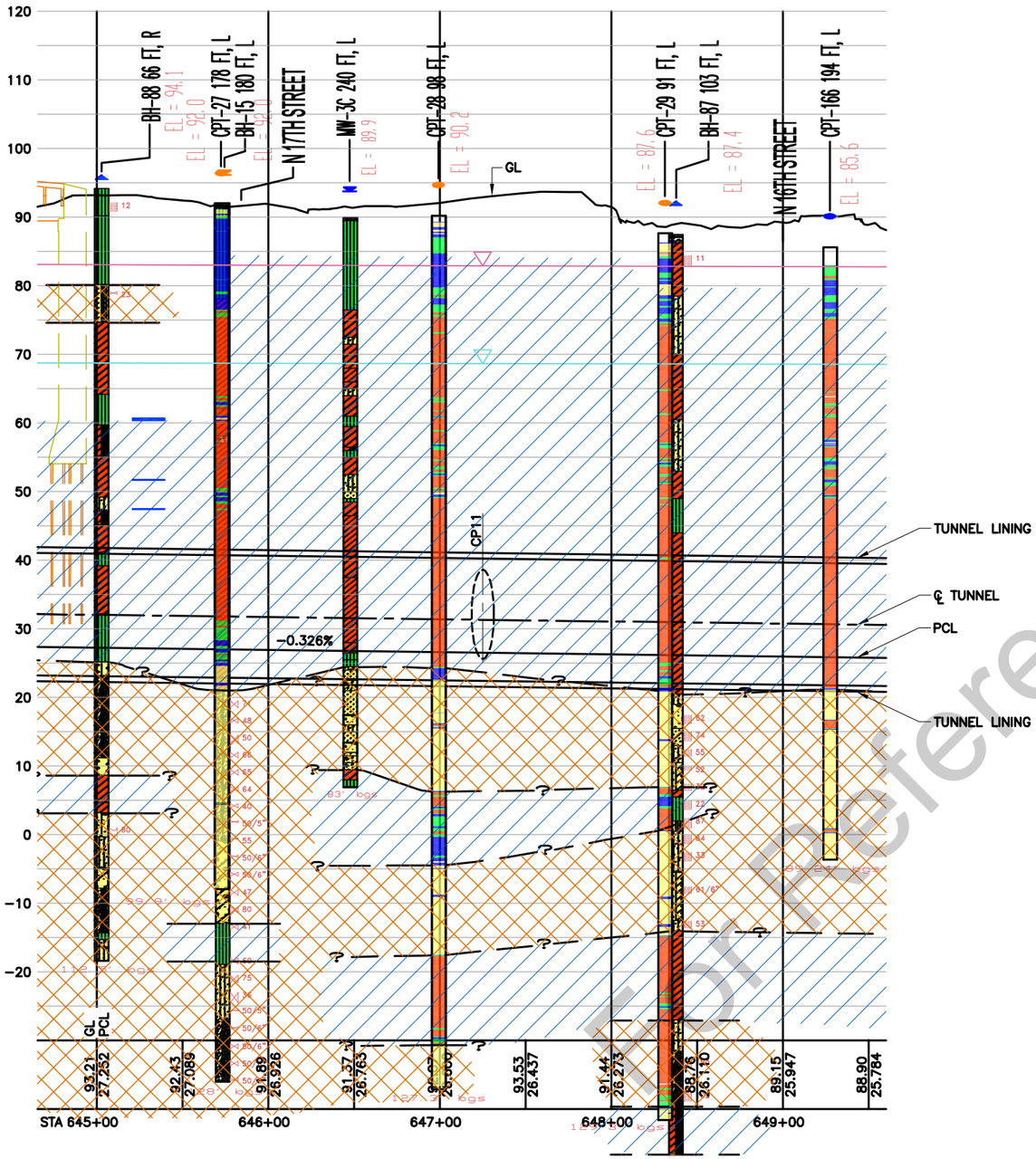
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 10

CADD FILENAME C330-S-TR-C318.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C318 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:11pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C330-S-TR-C318.dwg

See disclaimer on cover page.



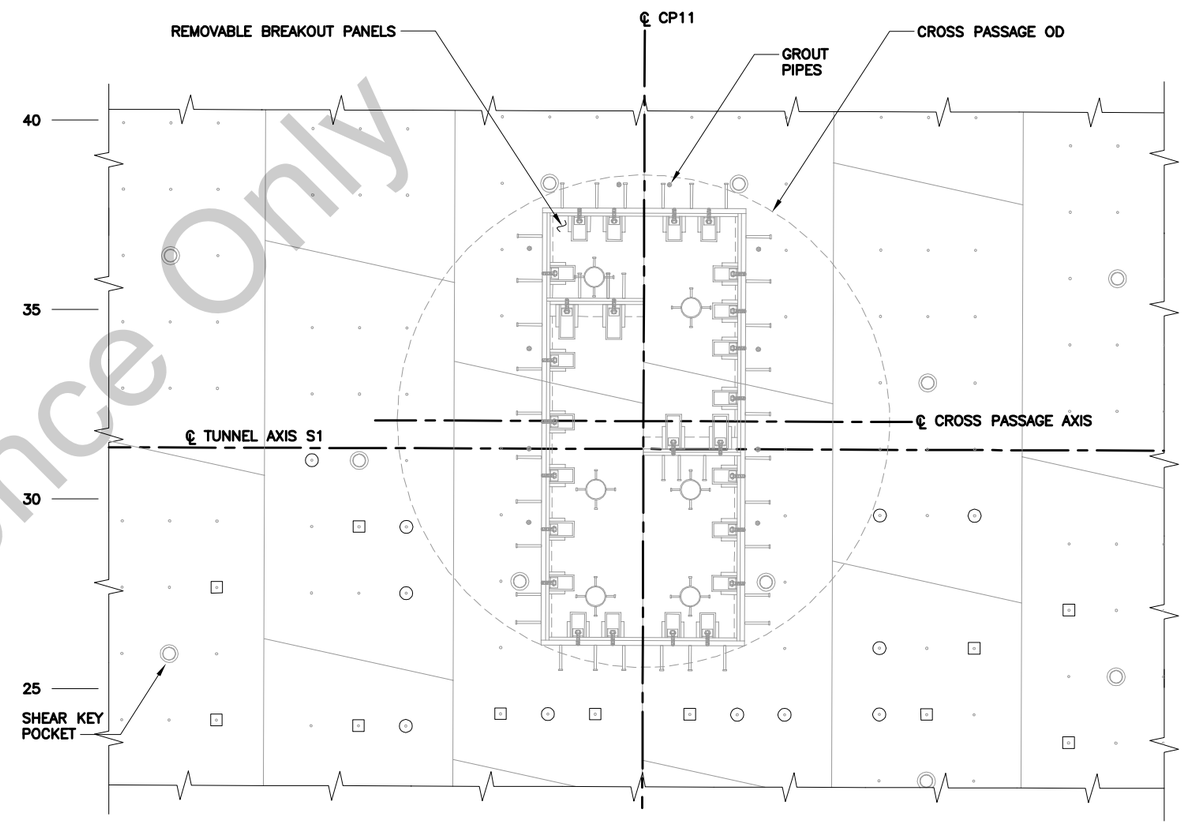
SOIL PROFILE AT CROSS PASSAGE 11 (A / TR-C319)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- · CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	PERMEATION GROUT/VACUUM WELL POINTS
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C319)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT PO503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

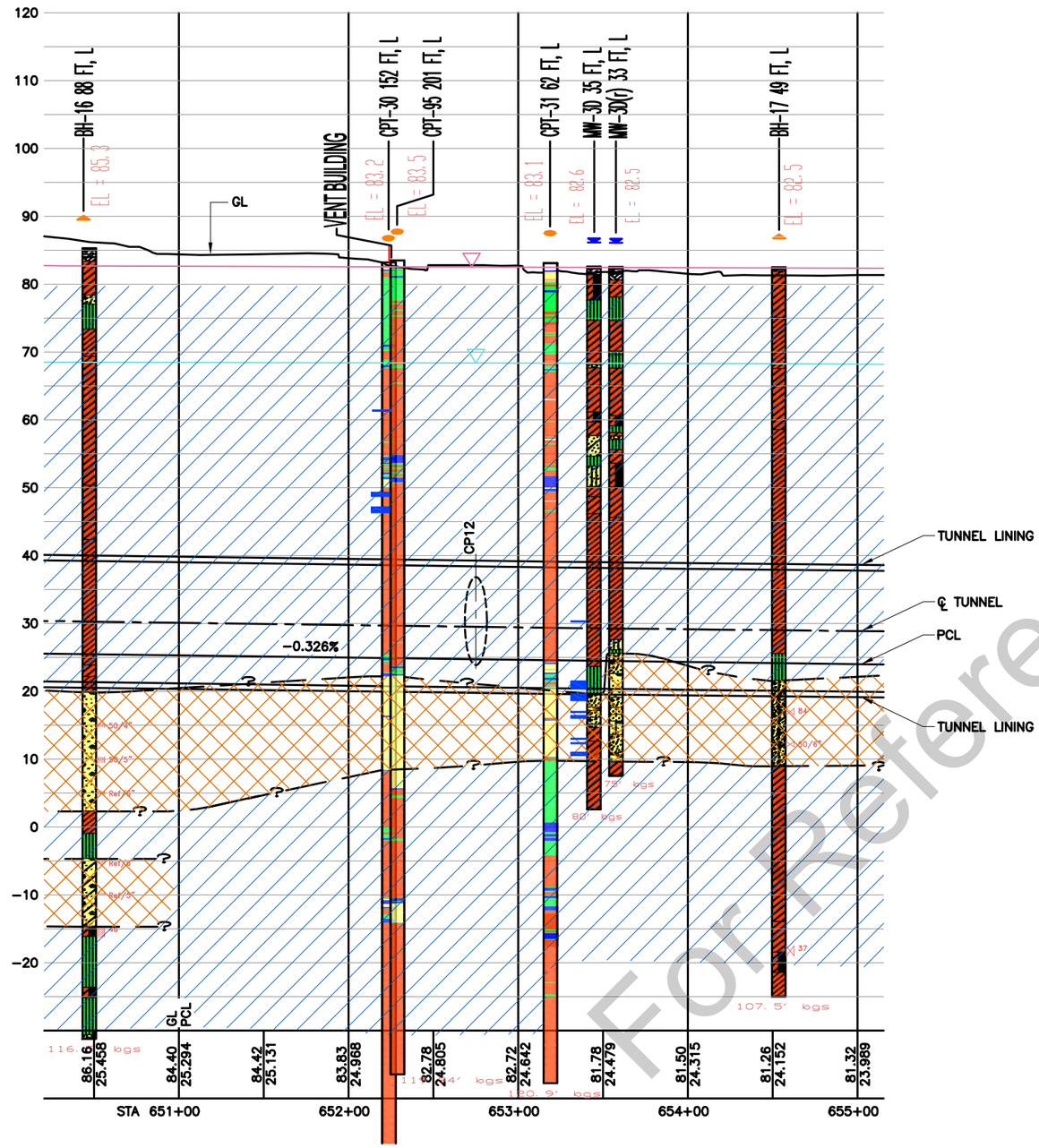
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 11

CADD FILENAME C330-S-TR-C319.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C319 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:12pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C319.dwg

See disclaimer on cover page.

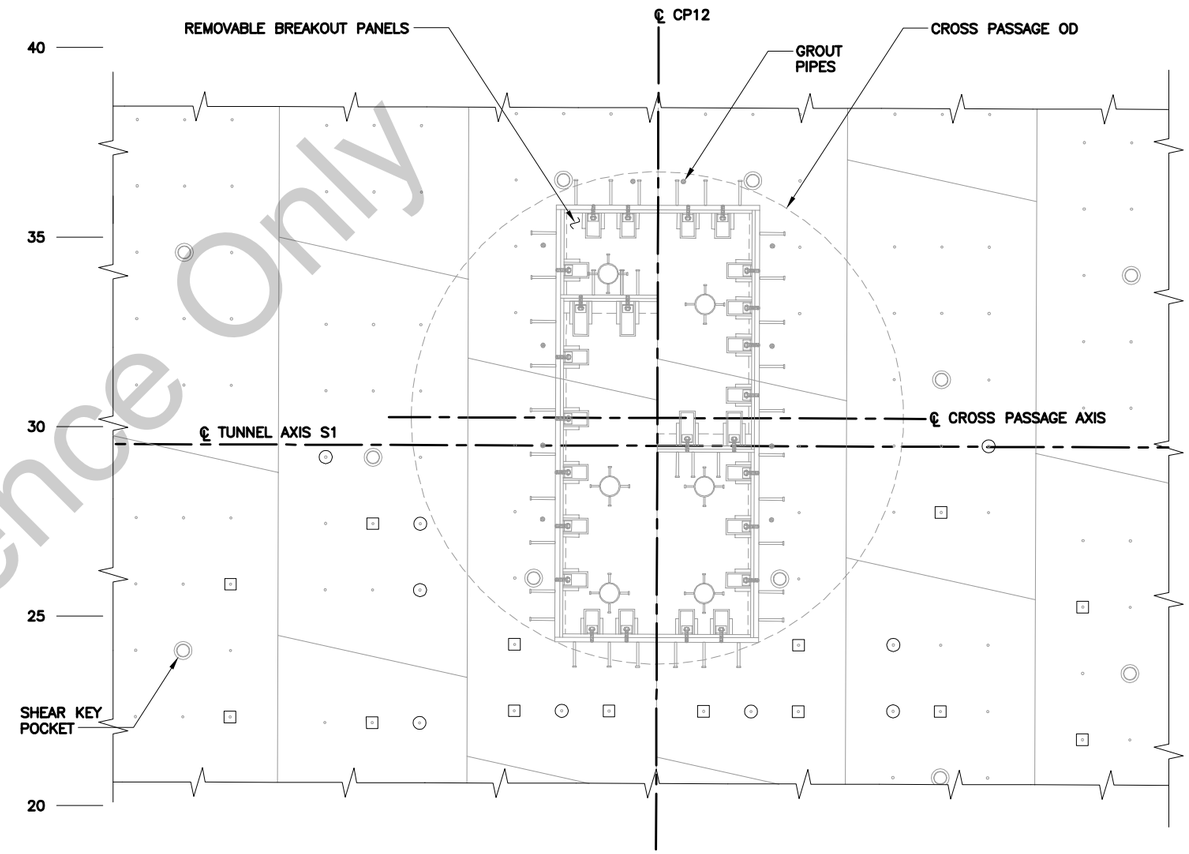


SOIL PROFILE AT CROSS PASSAGE 12 (A / TR-C320)

SCALE: 1"=40'H 1"=10'V

LEGEND:

□	PERMEATION GROUTING HOLE	STA	STATIONING ON S1 TUNNEL
○	VACUUM WELLPOINT	PCL	PROFILE CONTROL LINE
·	CONCRETE SEGMENT LINING DIMPLE	GL	GROUND LEVEL
		OD	OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	PERMEATION GROUT/VACUUM WELL POINTS
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C320)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor

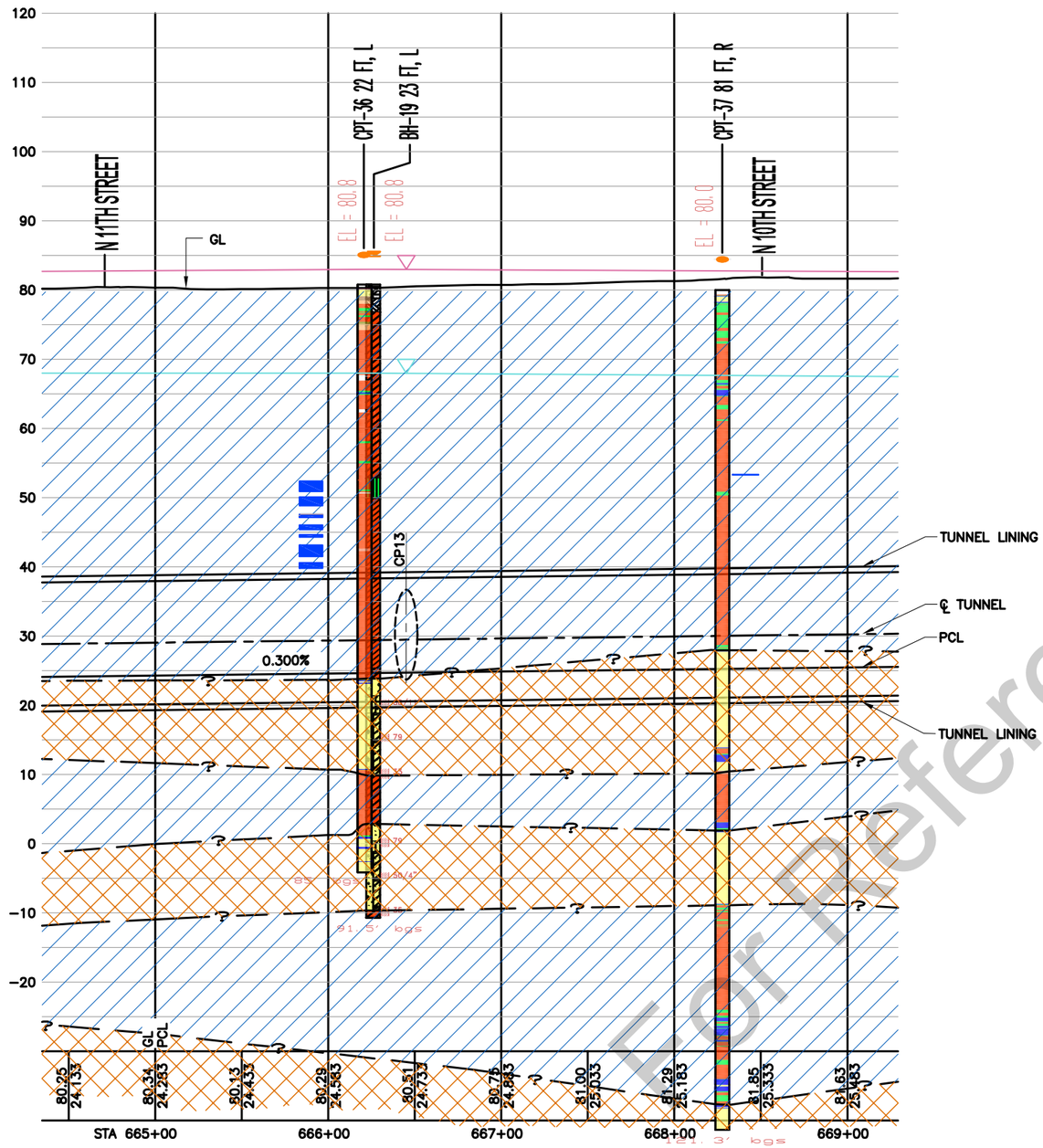
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 12

CADD FILENAME C330-S-TR-C320.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C320 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:13pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C-300-S-TR-C320.dwg

See disclaimer on cover page.



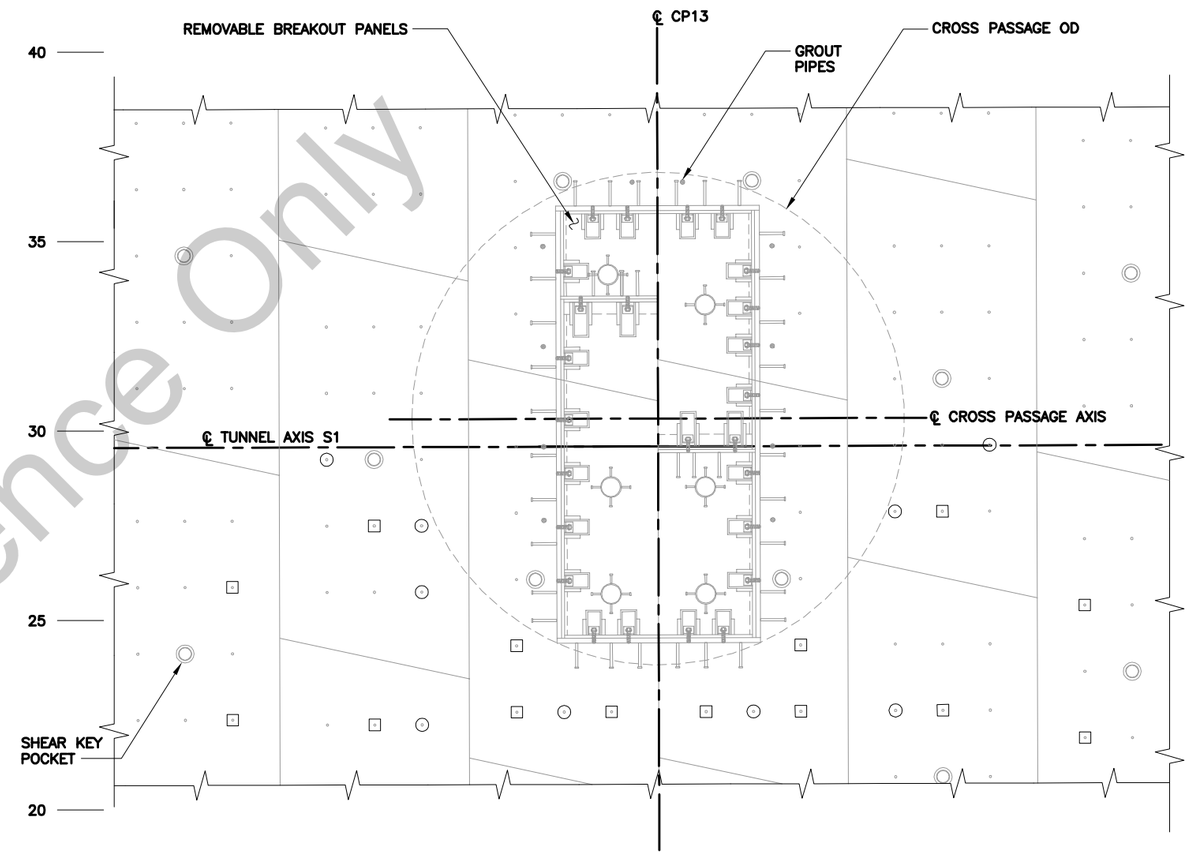
SOIL PROFILE AT CROSS PASSAGE 13 (A / TR–C321)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	PERMEATION GROUT/VACUUM WELL POINTS
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR–C321)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503–D300–RPT–GEO–006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

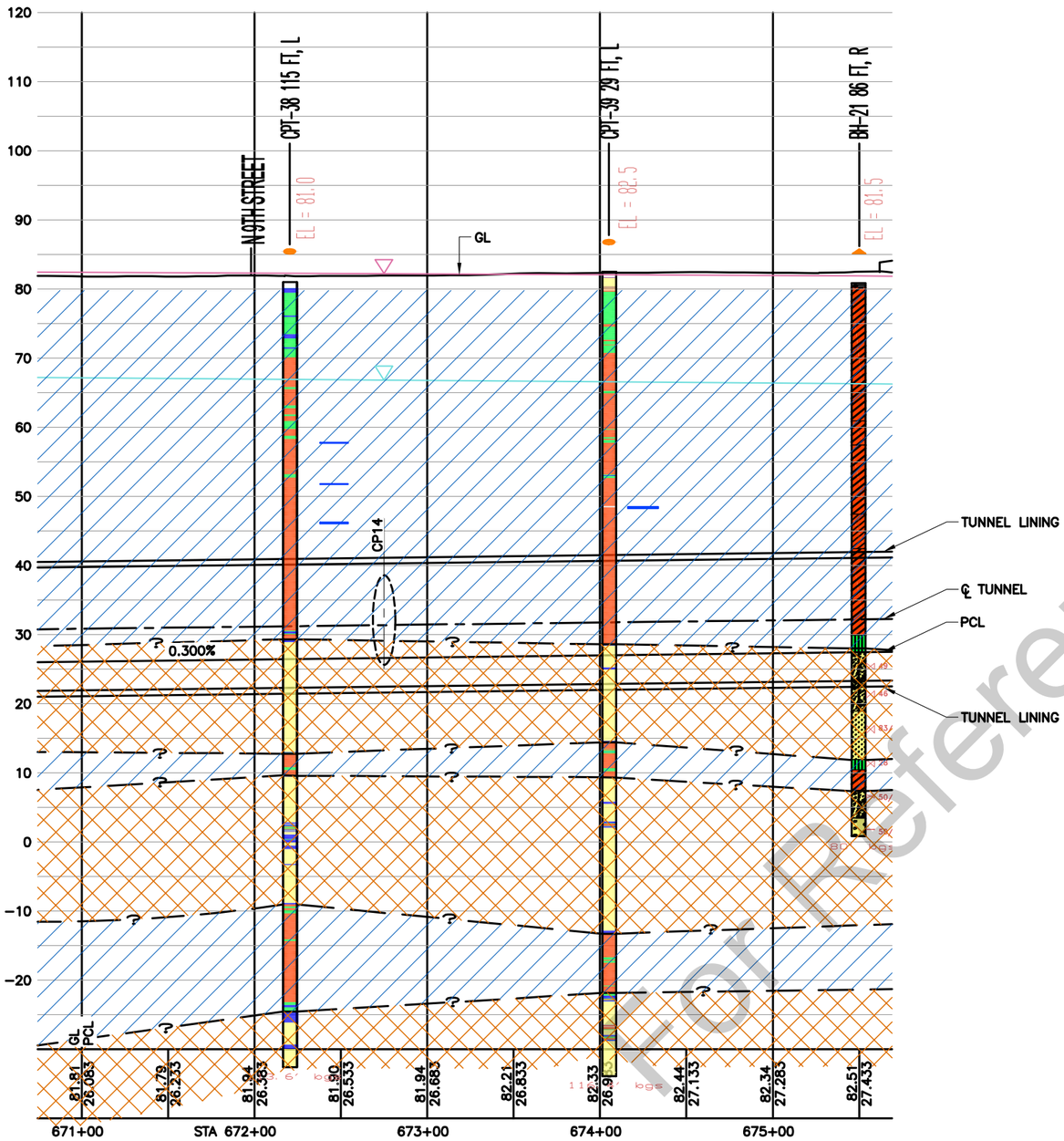
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 13

CADD FILENAME C330–S–TR–C321.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C321 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:14pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C321.dwg

See disclaimer on cover page.

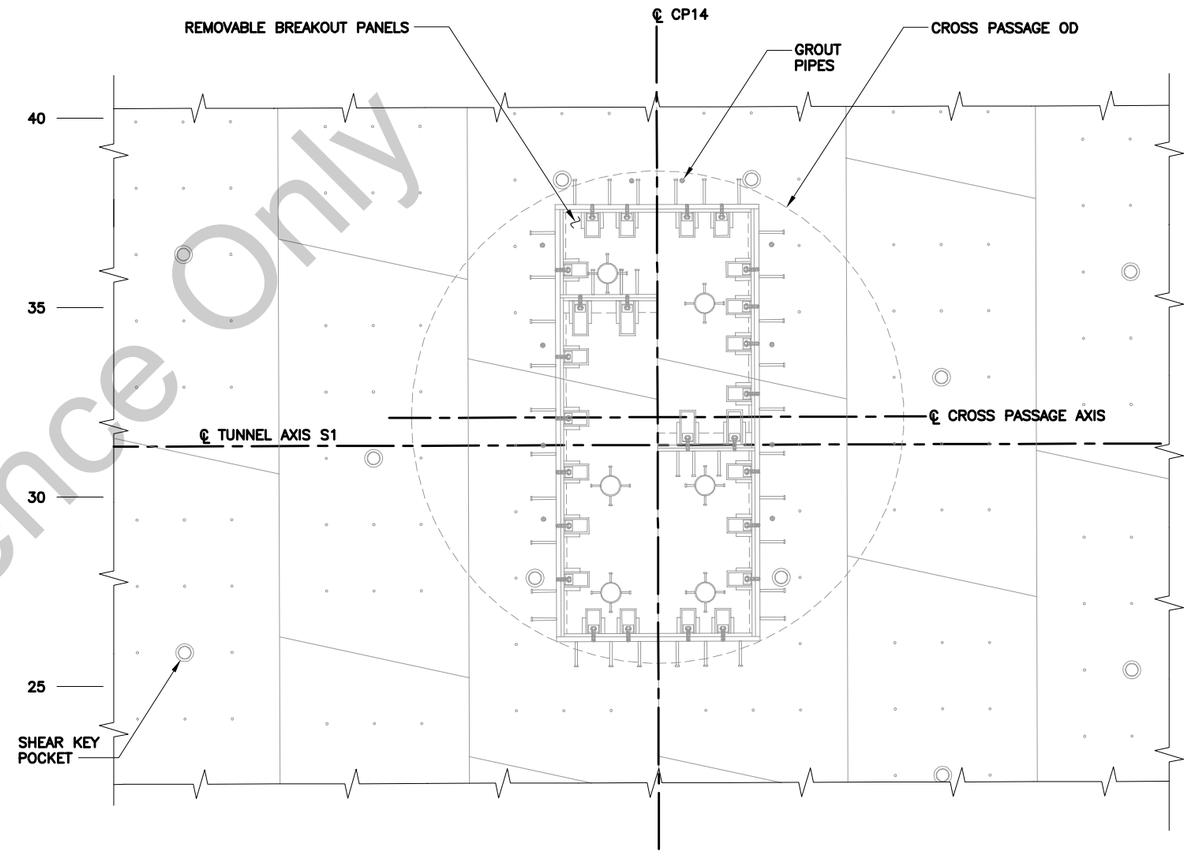


SOIL PROFILE AT CROSS PASSAGE 14 (A / TR-C322)

SCALE: 1"=40'H 1"=10'V

LEGEND:

□	PERMEATION GROUTING HOLE	STA	STATIONING ON S1 TUNNEL
○	VACUUM WELLPOINT	PCL	PROFILE CONTROL LINE
◦	CONCRETE SEGMENT LINING DIMPLE	GL	GROUND LEVEL
		OD	OURSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	13 FT

DEVELOPED ELEVATION (B / TR-C322)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

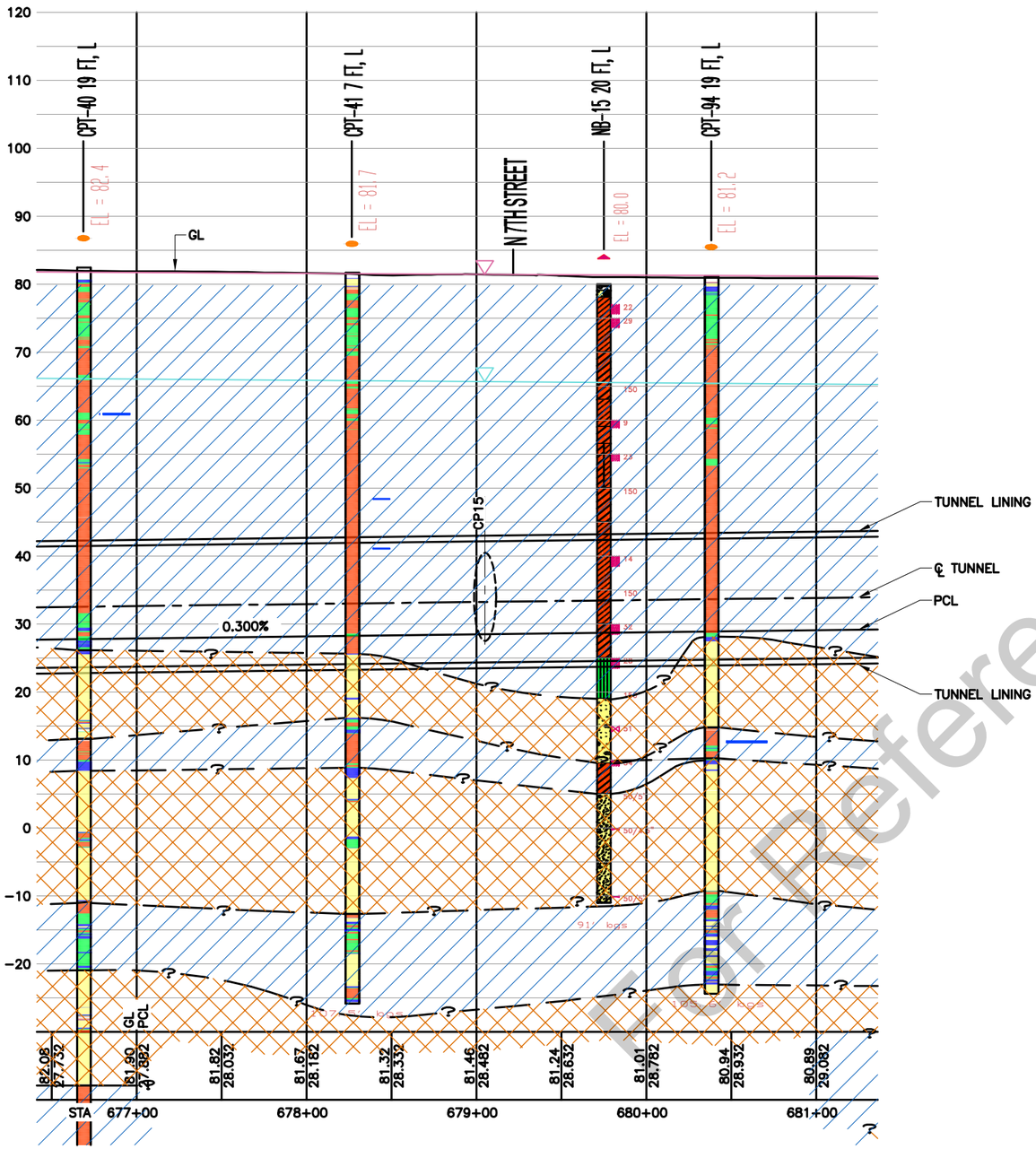
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 14

CADD FILENAME C330-S-TR-C322.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C322 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:15pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C322.dwg

See disclaimer on cover page.

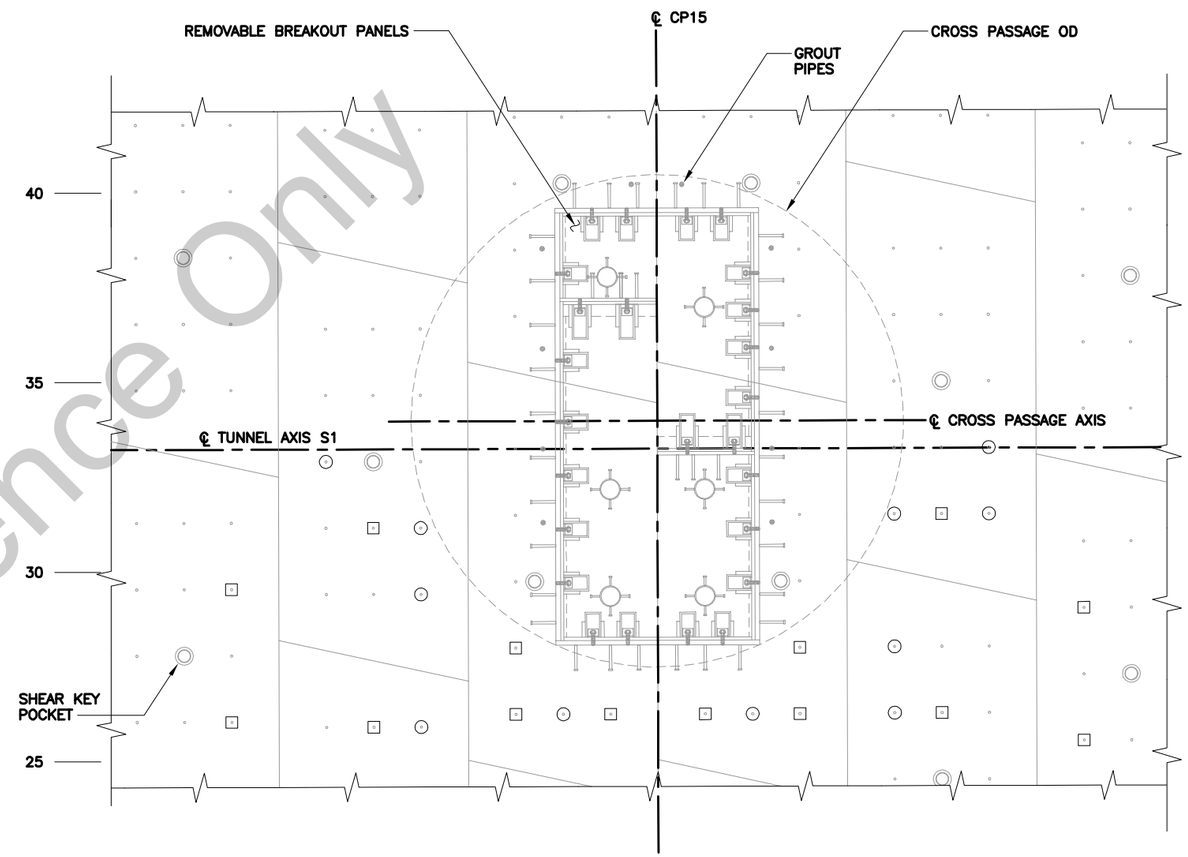


SOIL PROFILE AT CROSS PASSAGE 15 (A / TR-C323)

SCALE: 1"=40'H 1"=10'V

LEGEND:

□	PERMEATION GROUTING HOLE	STA	STATIONING ON S1 TUNNEL
○	VACUUM WELLPOINT	PCL	PROFILE CONTROL LINE
·	CONCRETE SEGMENT LINING DIMPLE	GL	GROUND LEVEL
		OD	OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	PERMEATION GROUT/VACUUM WELL POINTS
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C323)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

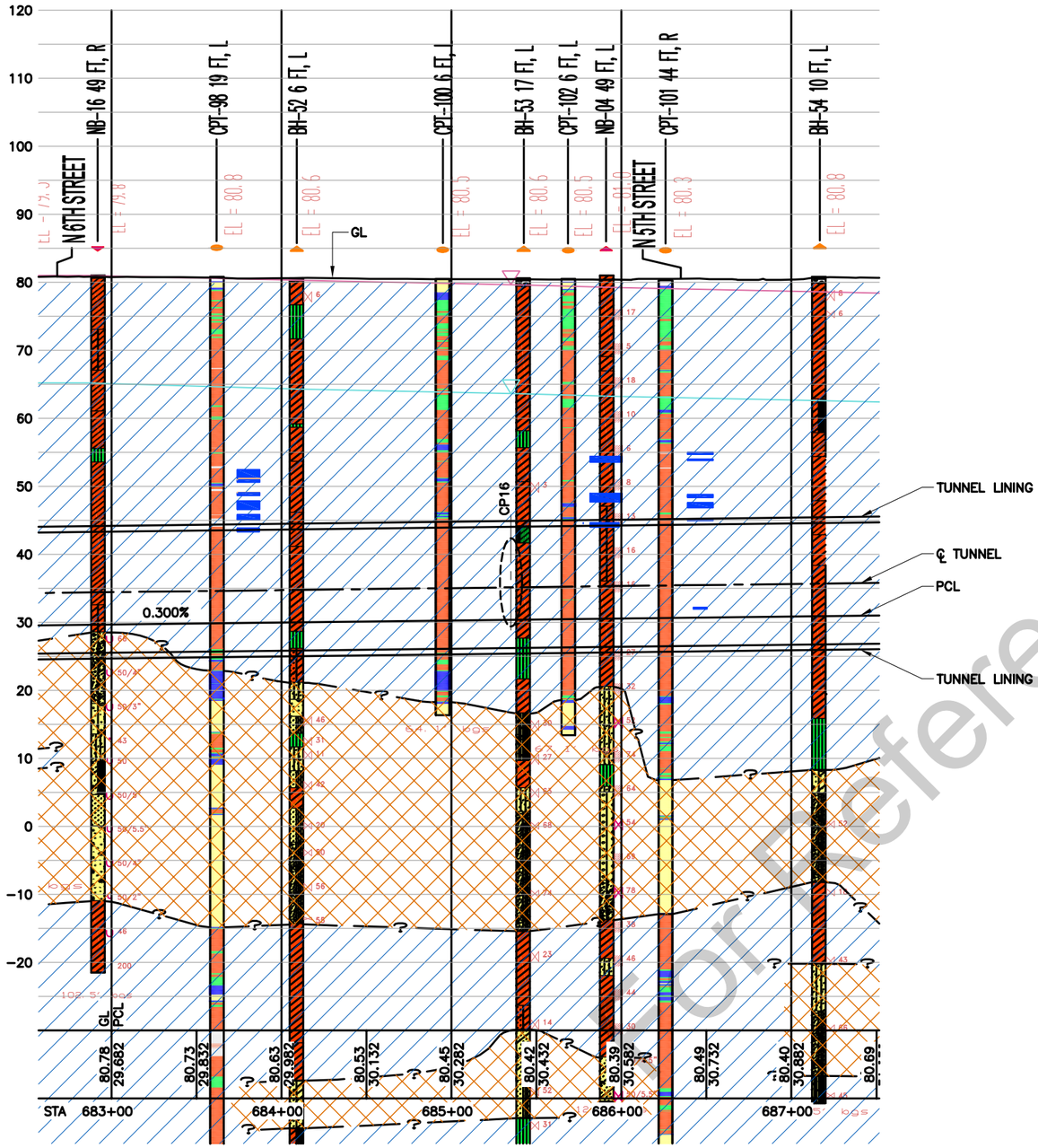
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 15

CADD FILENAME C330-S-TR-C323.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C323 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:16pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C323.dwg

See disclaimer on cover page.



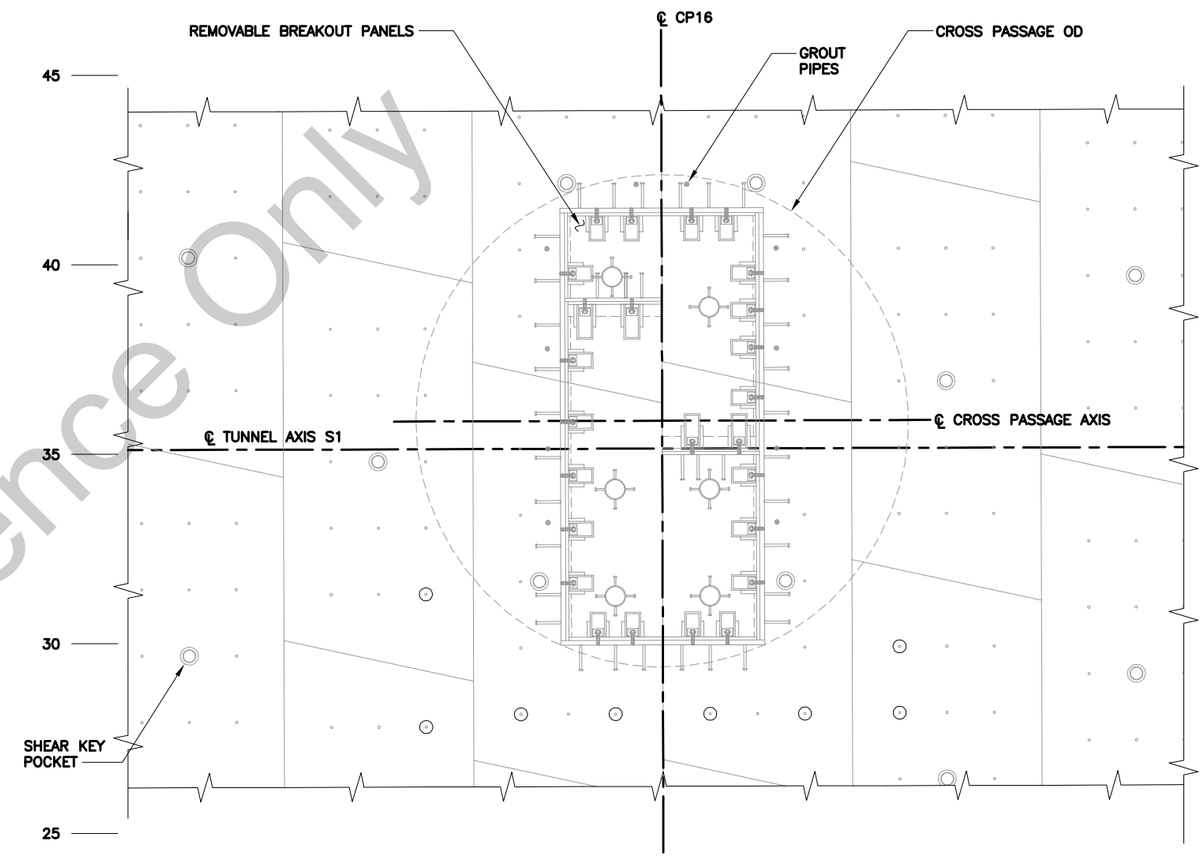
SOIL PROFILE AT CROSS PASSAGE 16 (A / TR-C324)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- · CONCRETE SEGMENT LINING DIMPLE

STA STATIONING ON S1 TUNNEL
PCL PROFILE CONTROL LINE
GL GROUND LEVEL
OD OUTSIDE DIAMETER



EXCAVATION METHOD	HEADING AND BENCH
GROUND TREATMENT METHOD	VACUUM WELL POINTS
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C324)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT PO503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 16

CADD FILENAME C330-S-TR-C324.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C324 | PAGE NO.

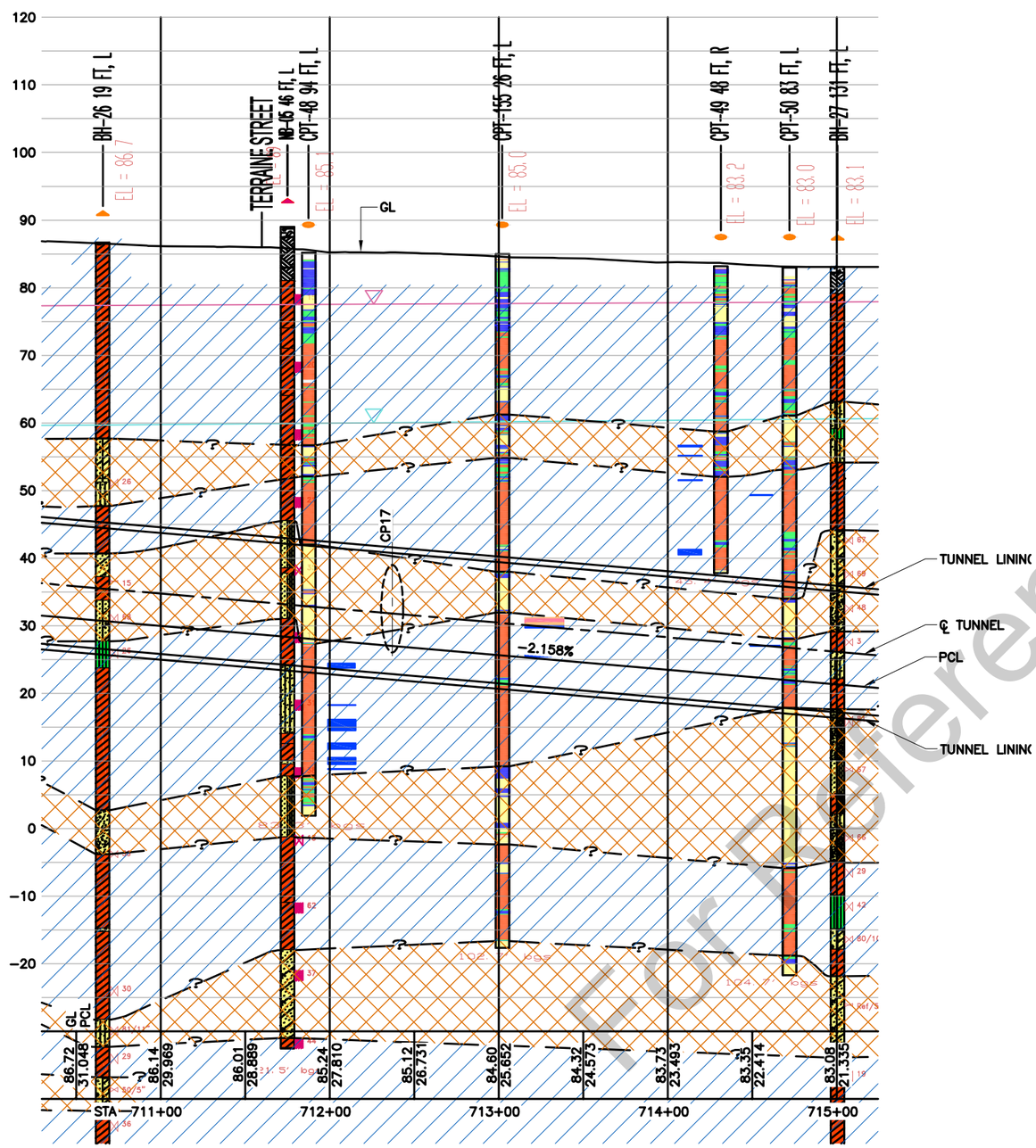
BARNGROVER_A Dec 18, 2008 - 4:17pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C324.dwg

See disclaimer on cover page.

LEGEND:

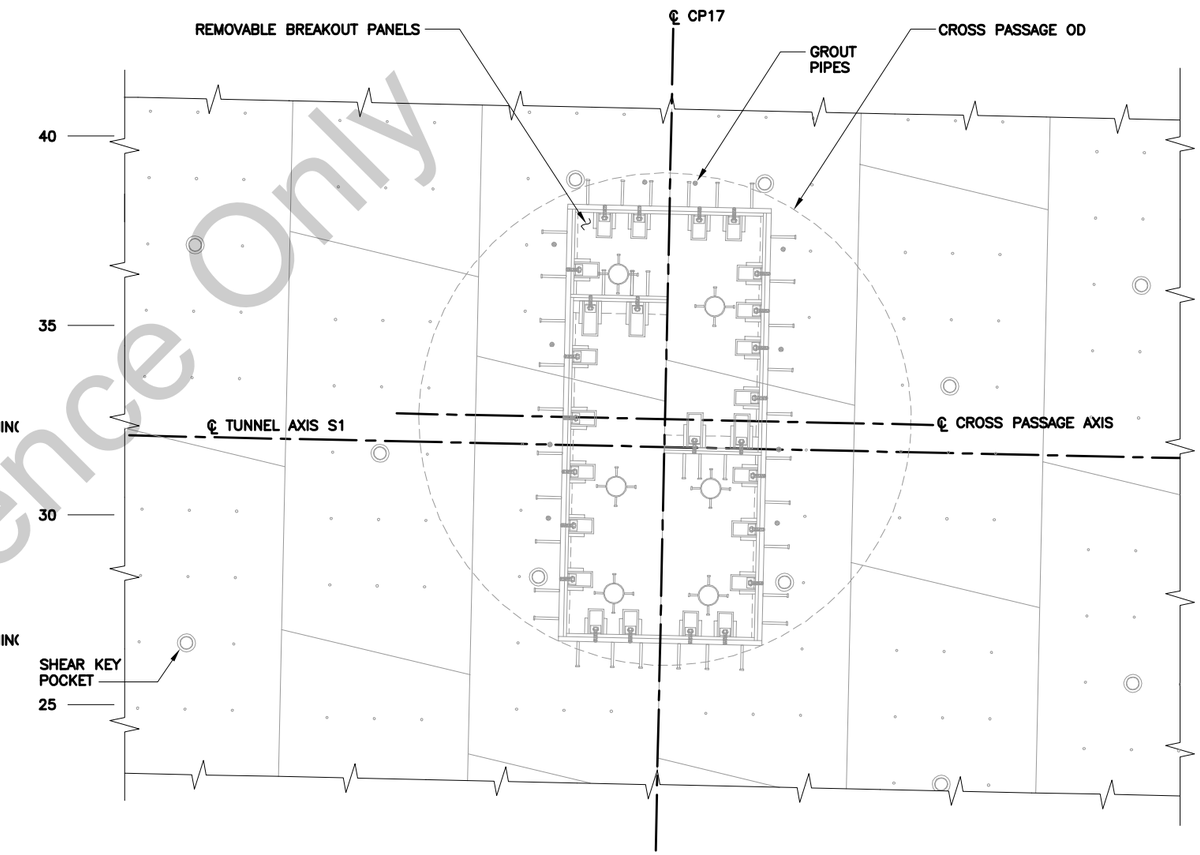
- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER



SOIL PROFILE AT CROSS PASSAGE 17 (A / TR-C325)

SCALE: 1"=40'H 1"=10'V



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C325)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

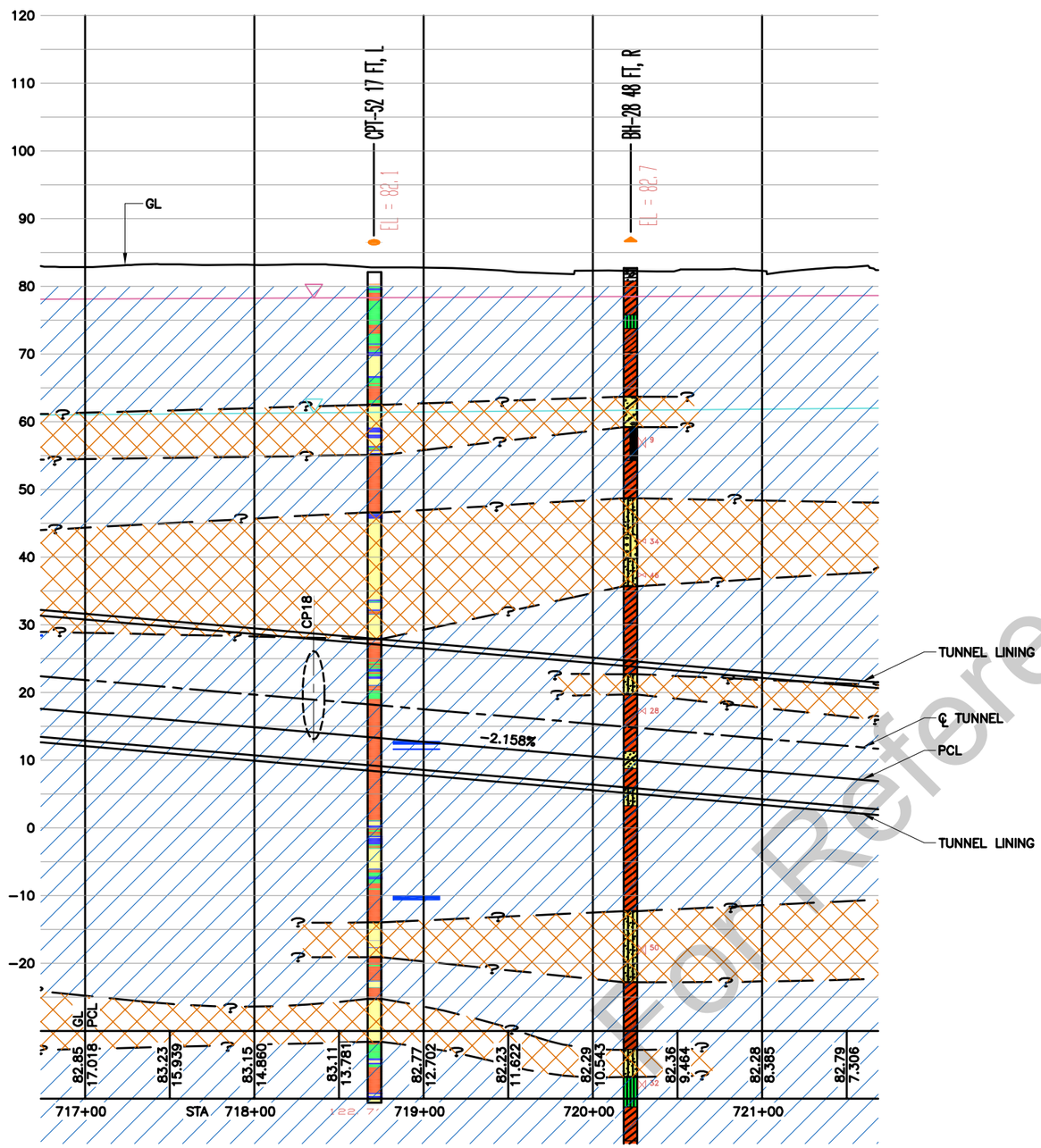
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 17

CADD FILENAME C330-S-TR-C325.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C325 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:18pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C325.dwg

See disclaimer on cover page.

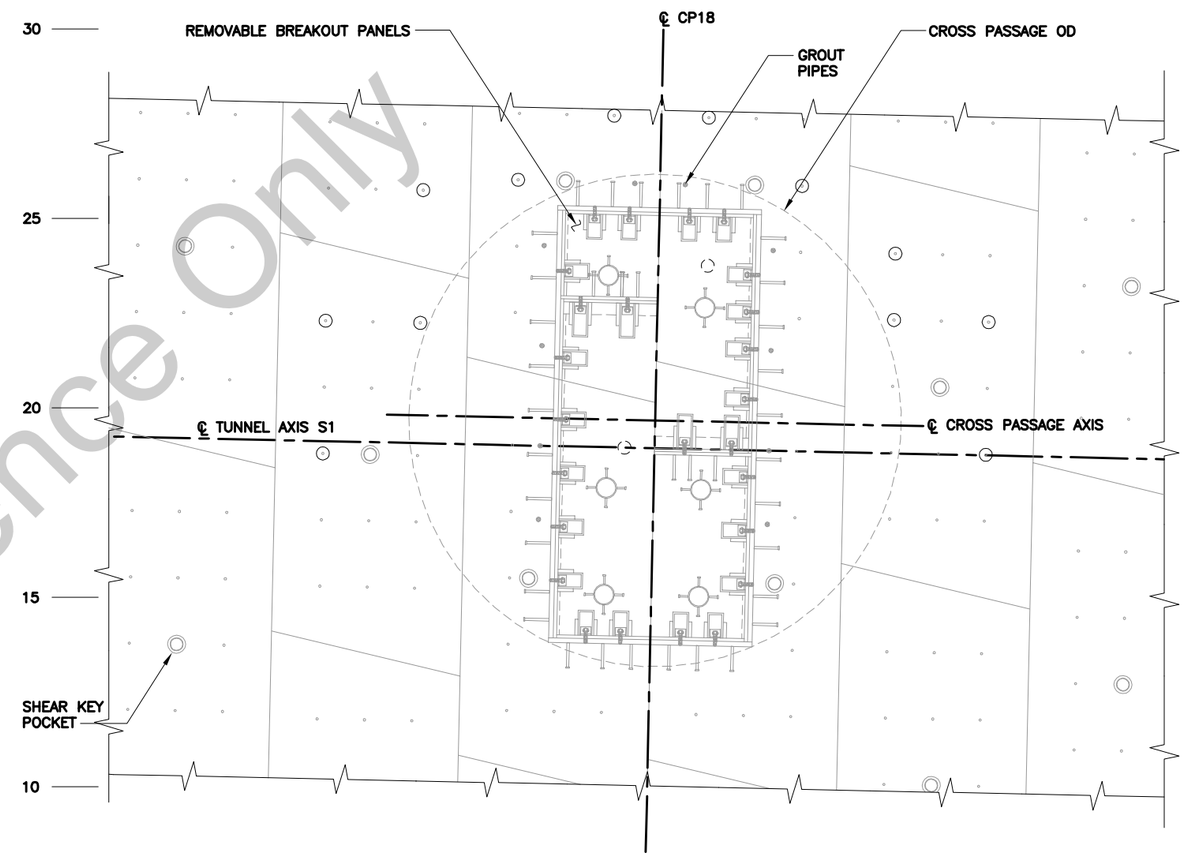


SOIL PROFILE AT CROSS PASSAGE 18 (A / TR-C326)

SCALE: 1"=40'H 1"=10'V

LEGEND:

□	PERMEATION GROUTING HOLE	STA	STATIONING ON S1 TUNNEL
○	VACUUM WELLPOINT	PCL	PROFILE CONTROL LINE
·	CONCRETE SEGMENT LINING DIMPLE	GL	GROUND LEVEL
◐	VACUUM WELLPOINT INSTALLED AFTER PANEL REMOVED, IF REQUIRED	OD	OUTSIDE DIAMETER



EXCAVATION METHOD	PILOT TUNNEL
GROUND TREATMENT METHOD	VACUUM WELL POINTS
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C326)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 18

CADD FILENAME C330-S-TR-C326.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C326 | PAGE NO.

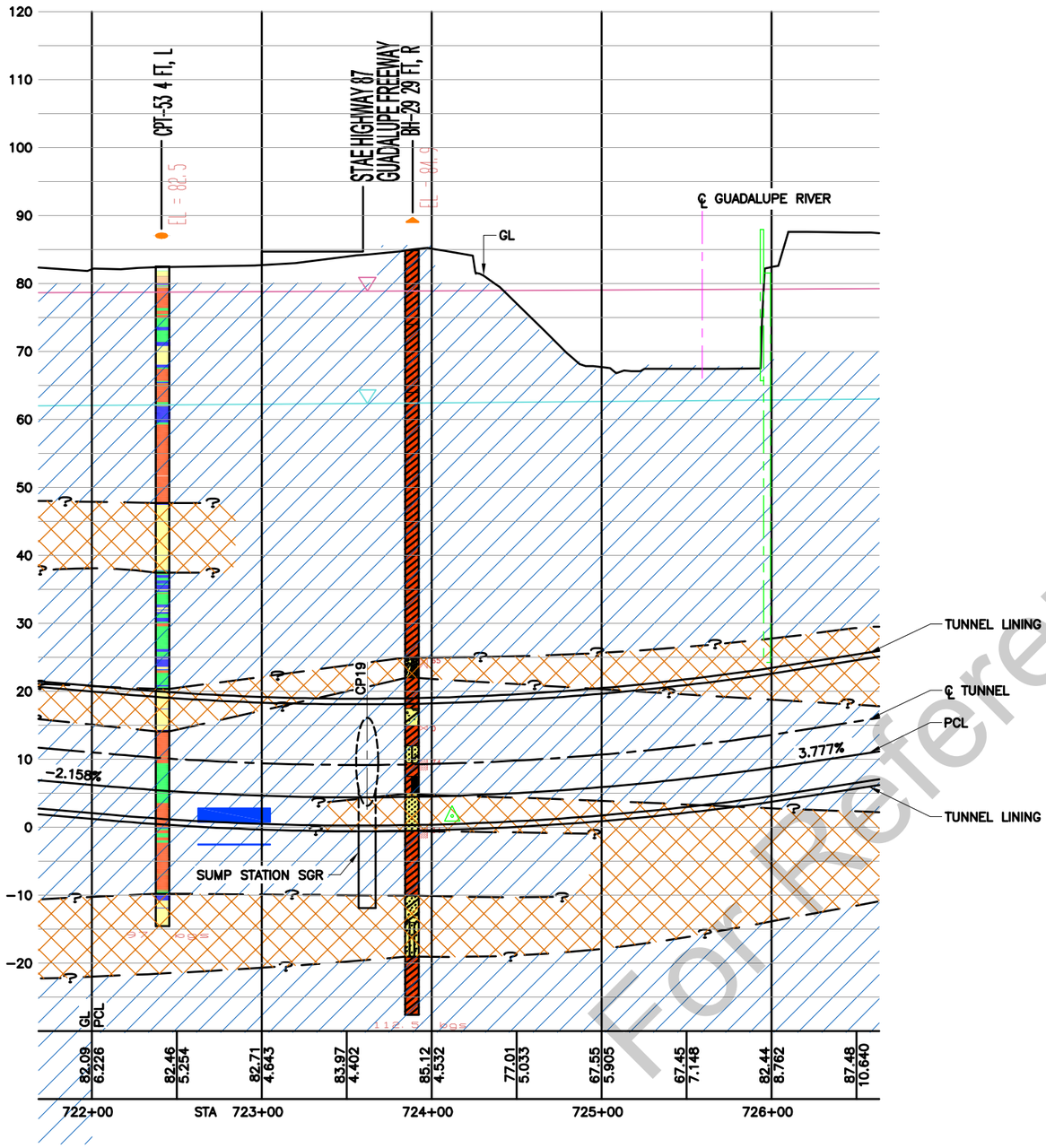
BARNGROVER_A Dec 18, 2008 - 4:19pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C326.dwg

See disclaimer on cover page.

LEGEND:

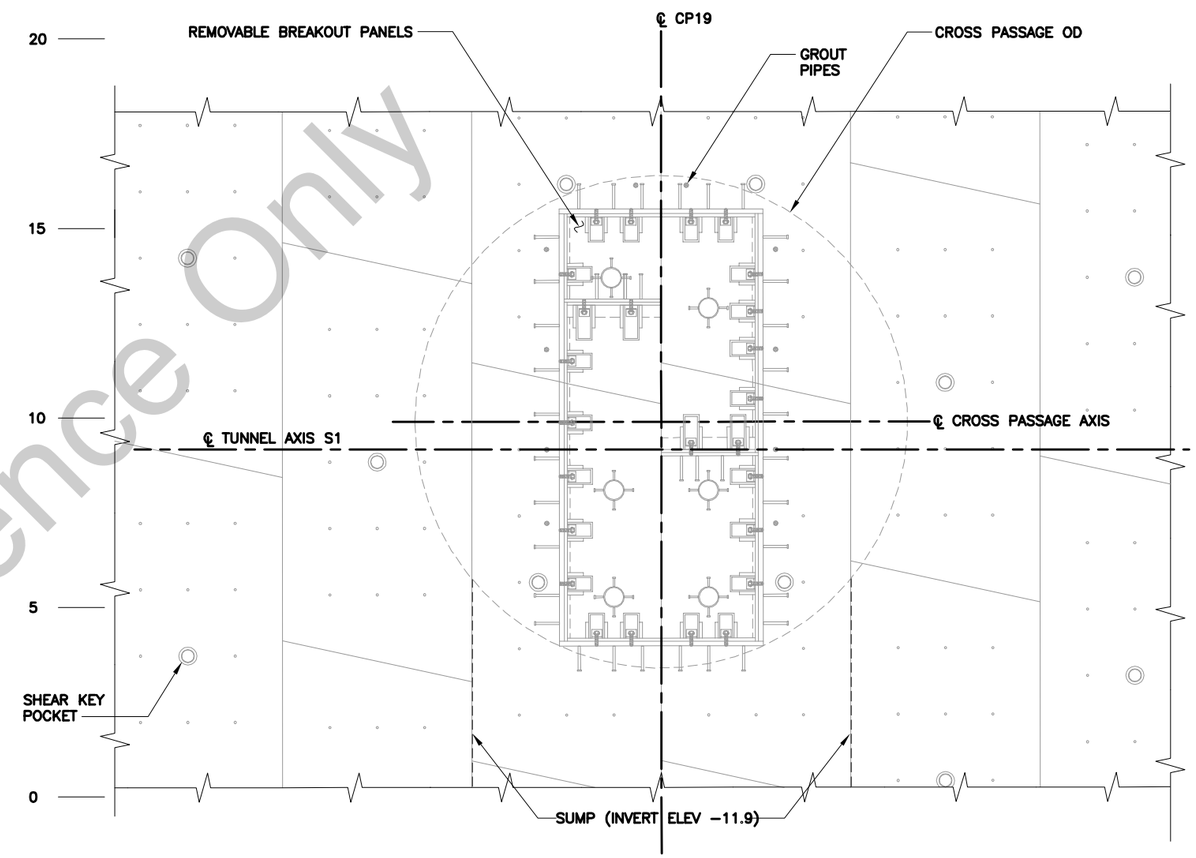
- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- · CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER
SGR	SUMP GUADALUPE RIVER



SOIL PROFILE AT CROSS PASSAGE 19 (A / TR-C327)

SCALE: 1"=40'H 1"=10'V



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	13 FT

DEVELOPED ELEVATION (B / TR-C327)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

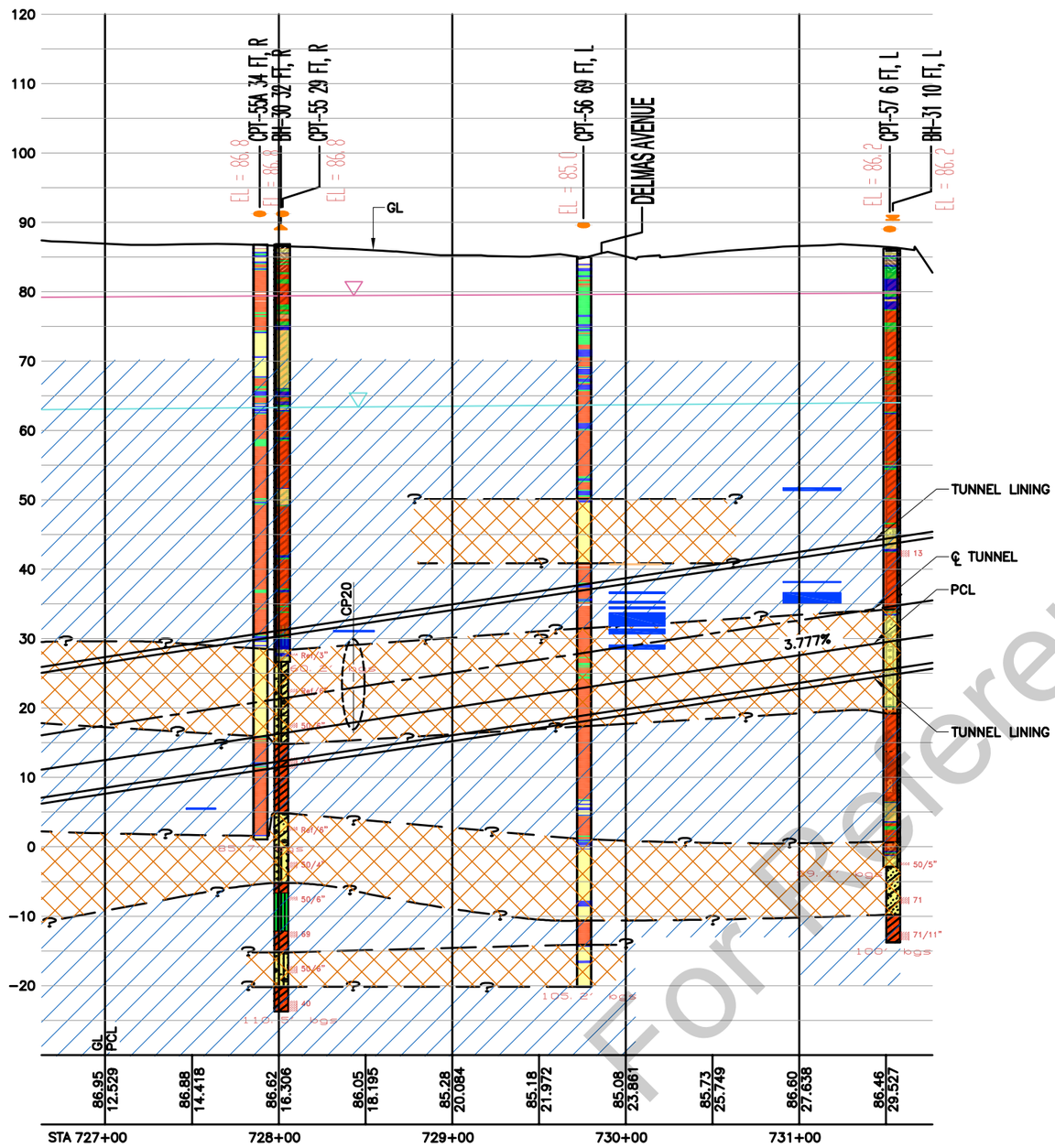
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 19

CADD FILENAME C330-S-TR-C327.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C327 | PAGE NO.

See disclaimer on cover page.

BARNGROVER_A Dec 18, 2008 - 4:20pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C327.dwg



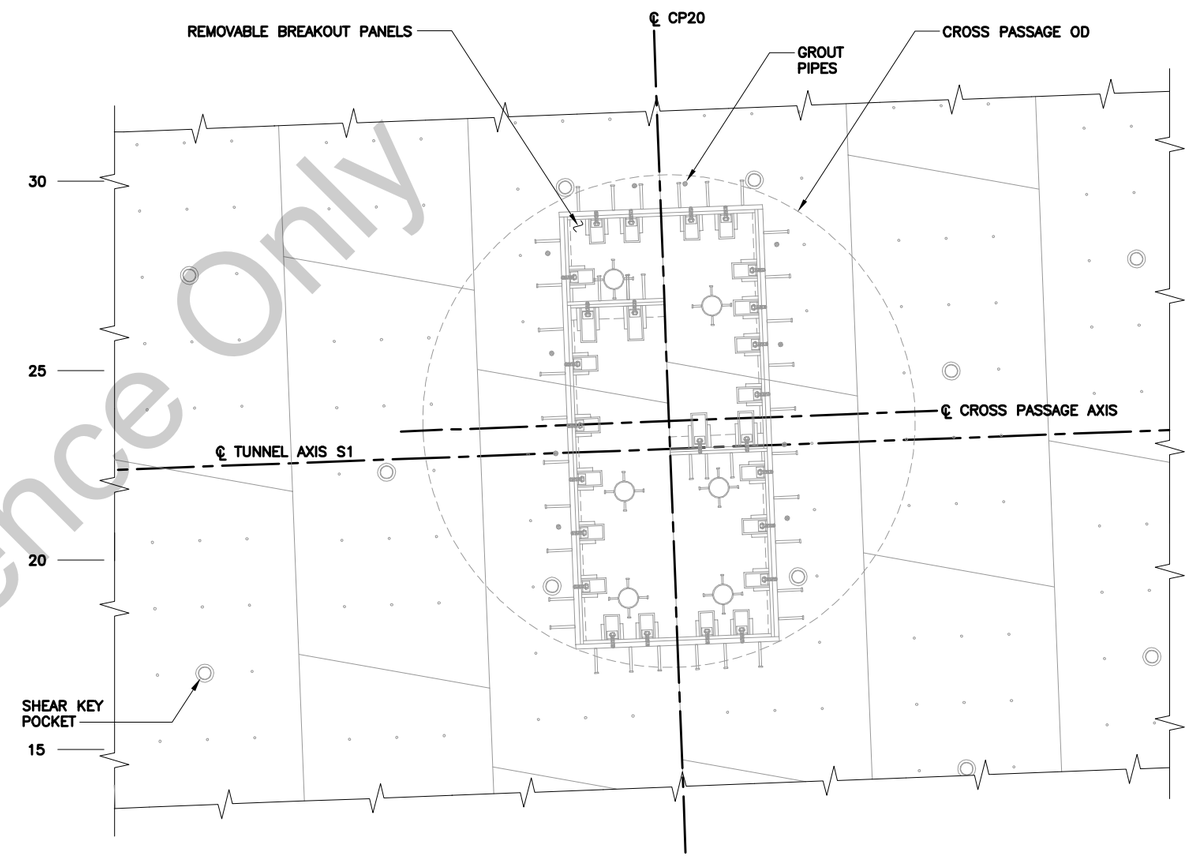
SOIL PROFILE AT CROSS PASSAGE 20 (A / TR-C328)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C328)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

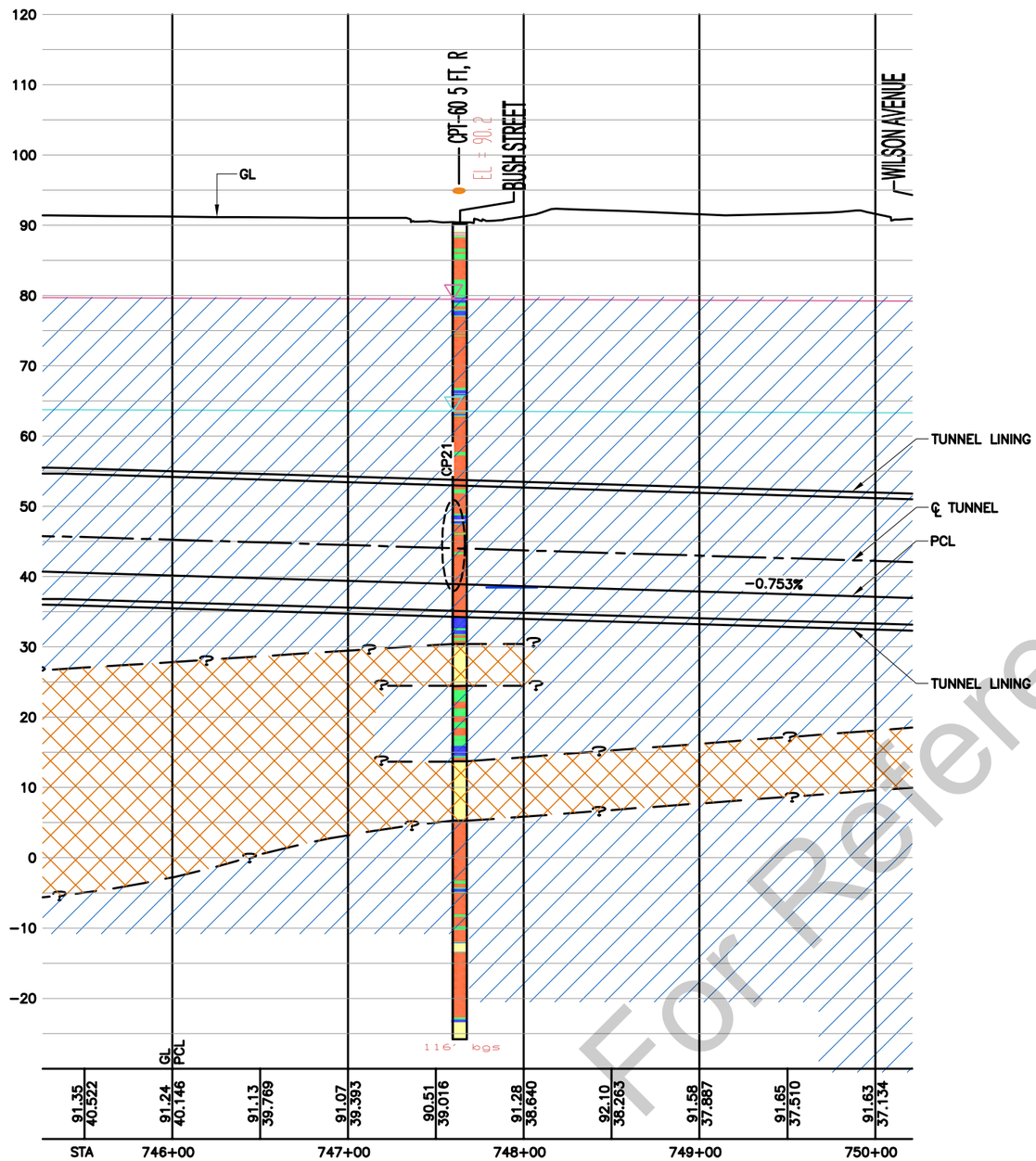
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 20

CADD FILENAME C330-S-TR-C328.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C328 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:22pm W:\TUCA00\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C328.dwg

See disclaimer on cover page.

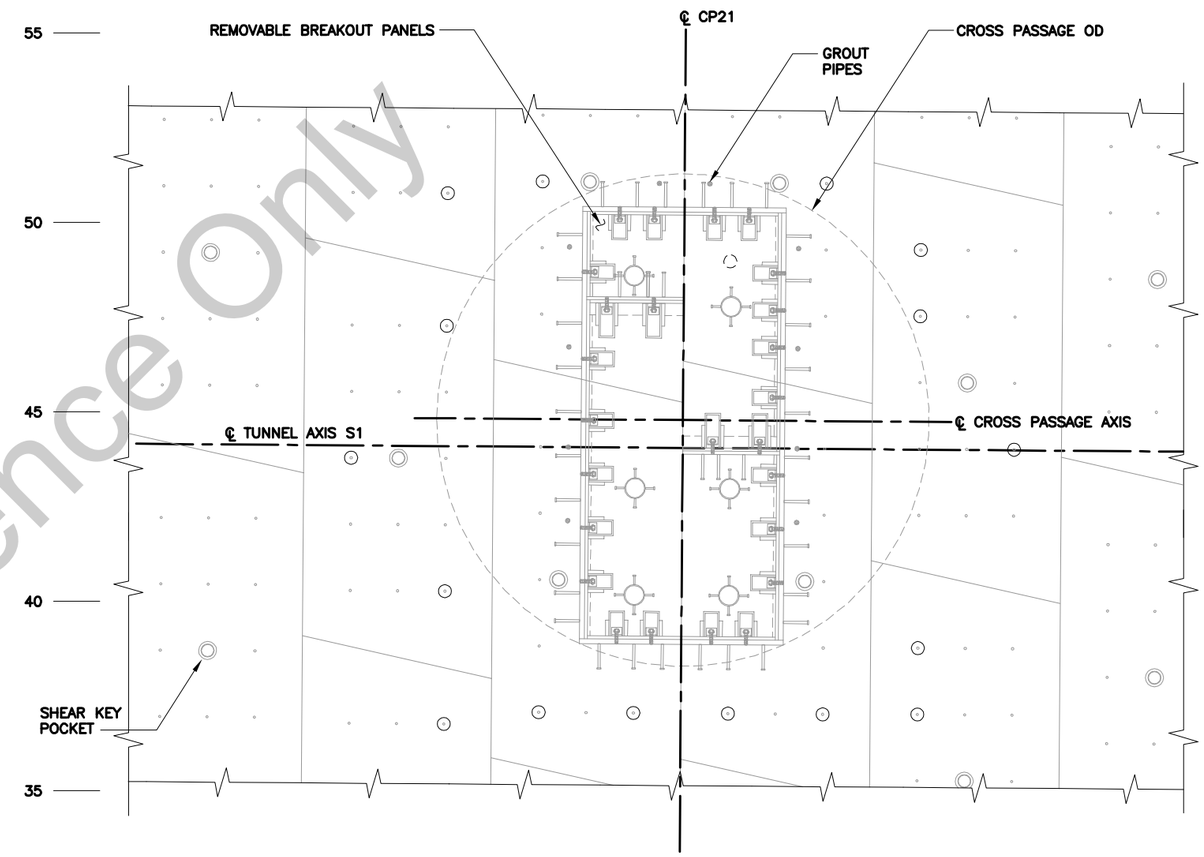


SOIL PROFILE AT CROSS PASSAGE 21 (A / TR-C329)

SCALE: 1"=40'H 1"=10'V

LEGEND:

□	PERMEATION GROUTING HOLE	STA	STATIONING ON S1 TUNNEL
○	VACUUM WELLPOINT	PCL	PROFILE CONTROL LINE
◦	CONCRETE SEGMENT LINING DIMPLE	GL	GROUND LEVEL
◌	VACUUM WELLPOINT INSTALLED AFTER PANEL REMOVED, IF REQUIRED	OD	OUTSIDE DIAMETER



EXCAVATION METHOD	PILOT TUNNEL
GROUND TREATMENT METHOD	VACUUM WELL POINTS
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C329)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

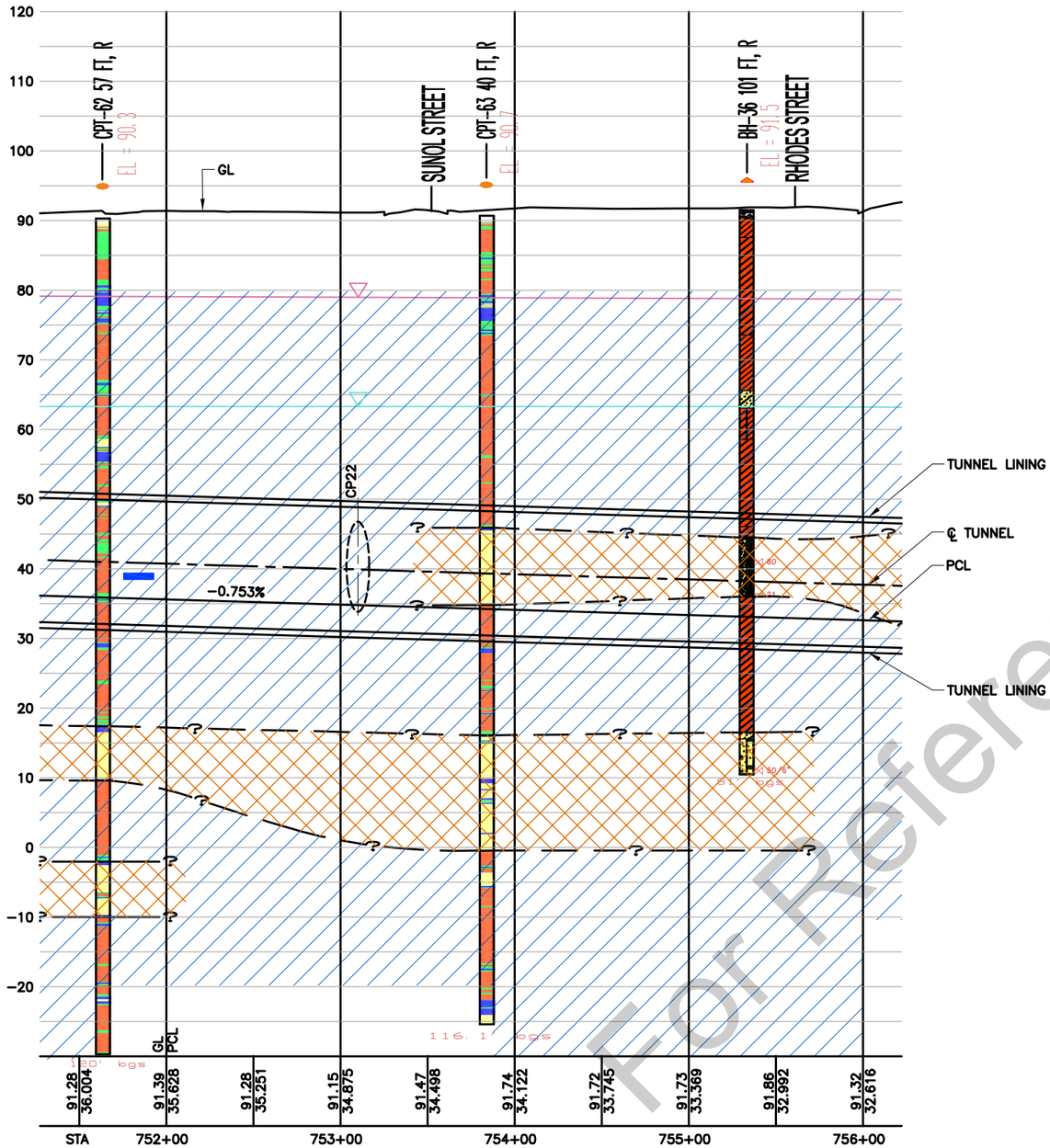
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 21

CADD FILENAME C330-S-TR-C329.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C329 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:23pm W:\TUCA0D\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C329.dwg

See disclaimer on cover page.

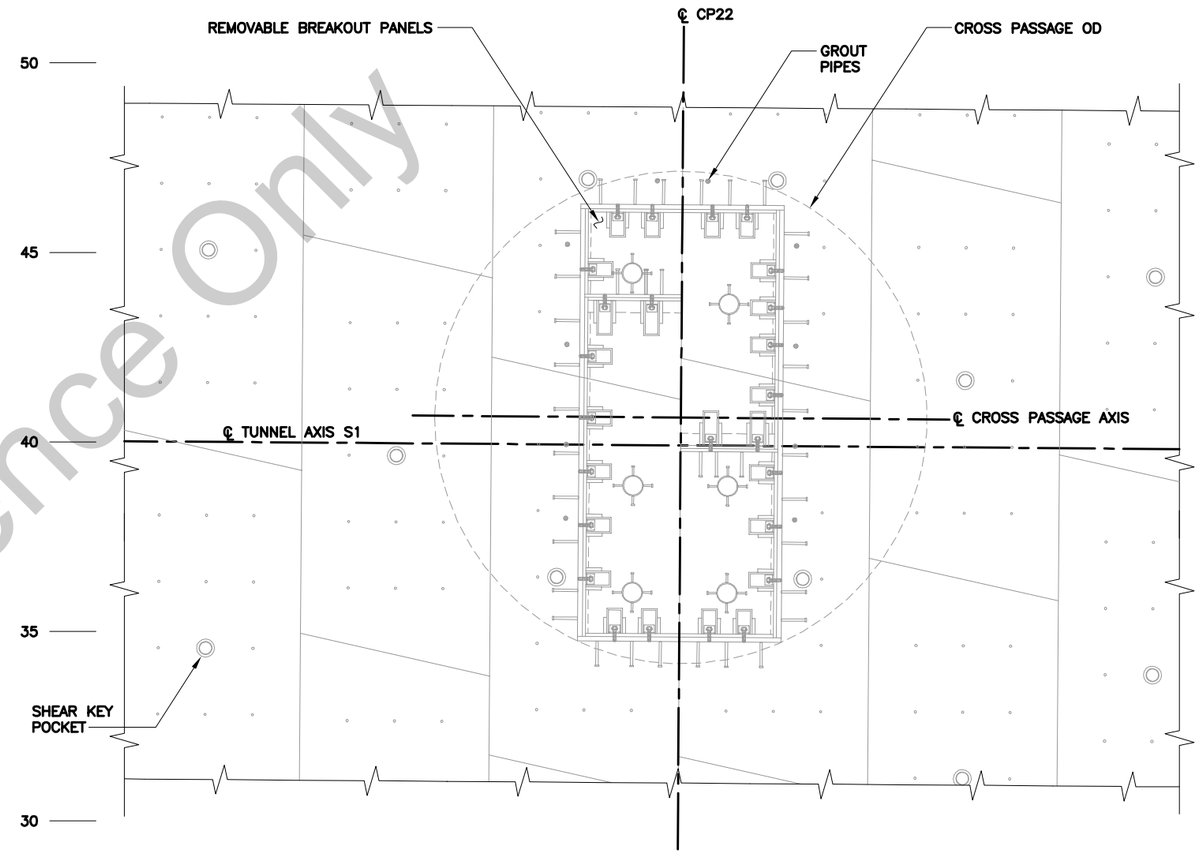


SOIL PROFILE AT CROSS PASSAGE 22 (A / TR-C330)

SCALE: 1"=40'H 1"=10'V

LEGEND:

□	PERMEATION GROUTING HOLE	STA	STATIONING ON S1 TUNNEL
○	VACUUM WELLPOINT	PCL	PROFILE CONTROL LINE
◦	CONCRETE SEGMENT LINING DIMPLE	GL	GROUND LEVEL
		OD	OUTSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C330)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

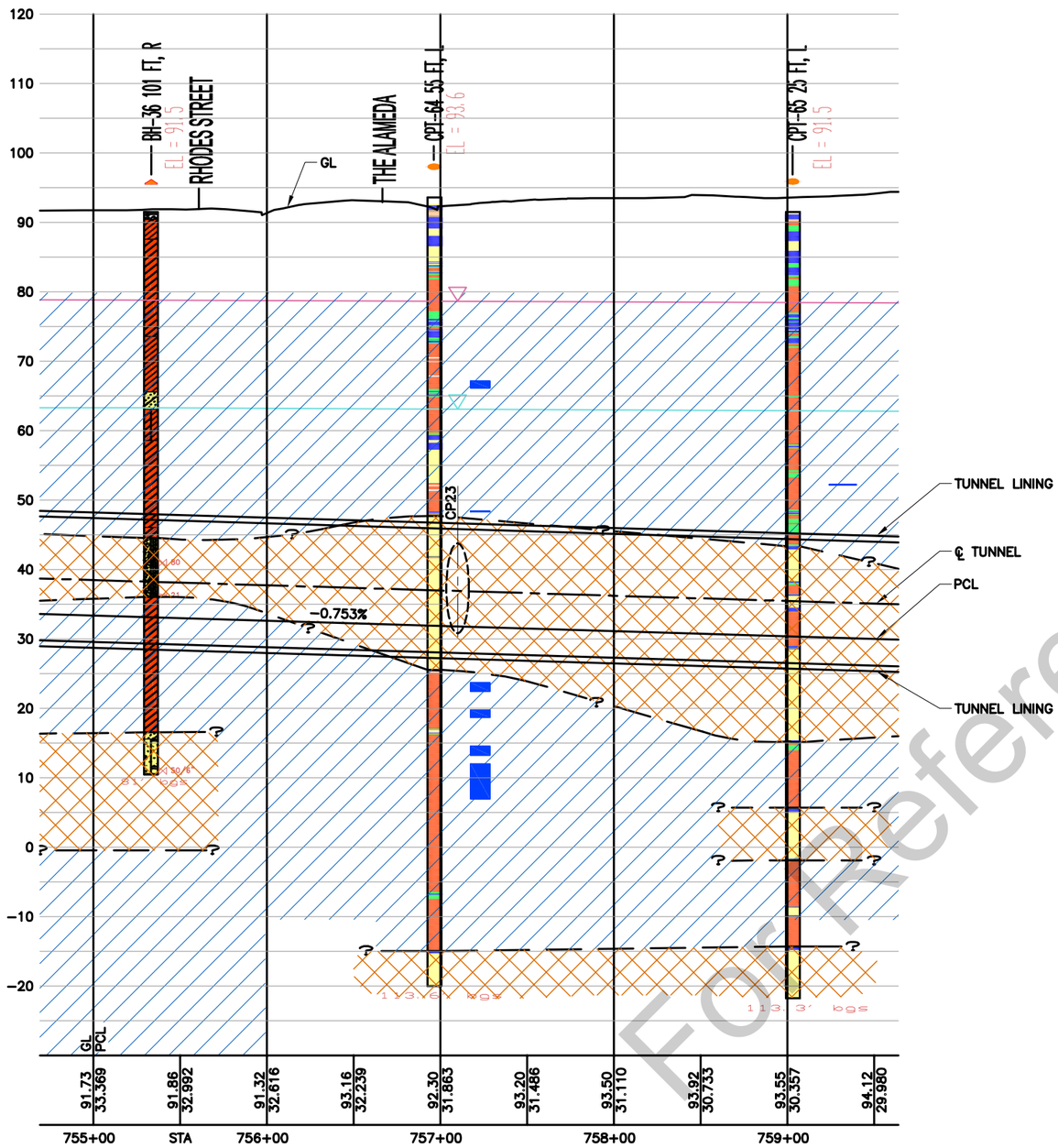
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 22

CADD FILENAME C330-S-TR-C330.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C330 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:24pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C330.dwg

See disclaimer on cover page.



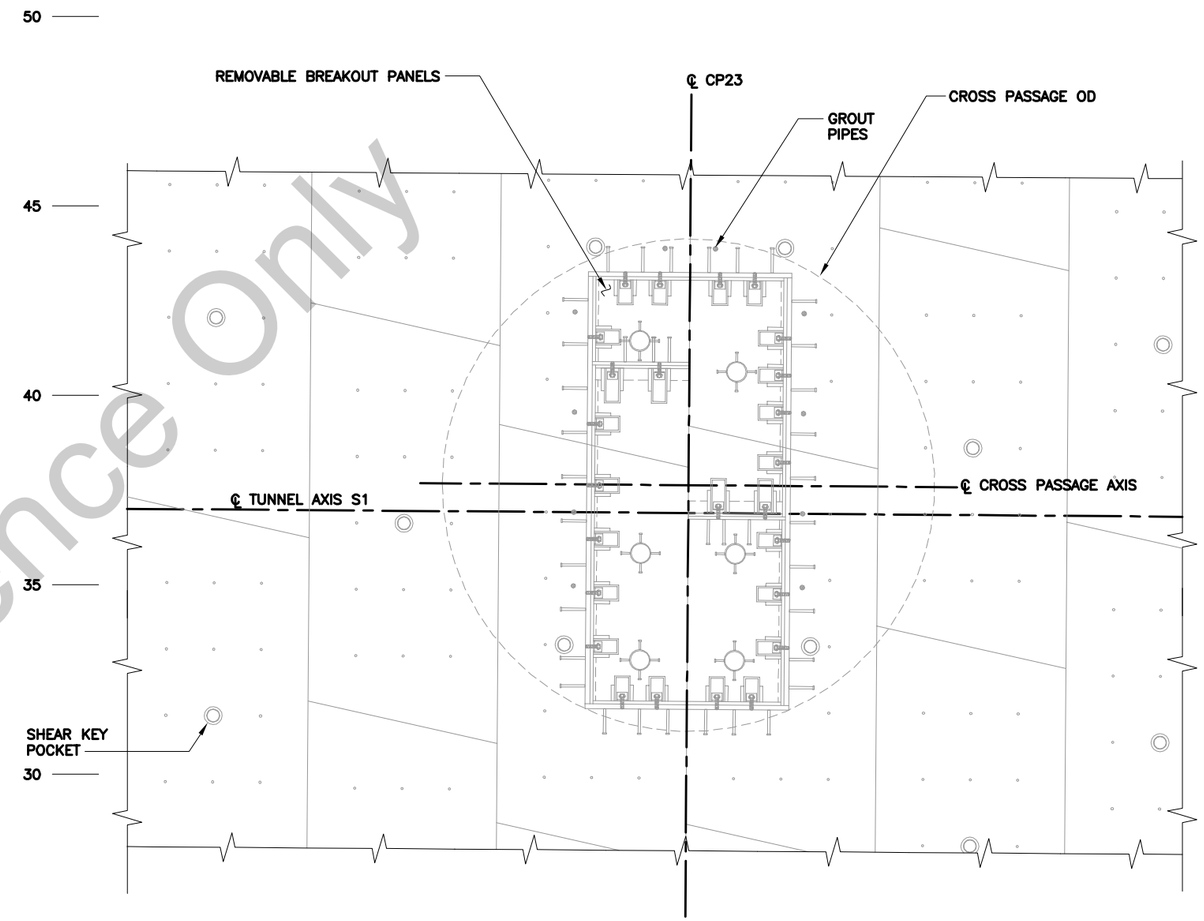
SOIL PROFILE AT CROSS PASSAGE 23 (A / TR-C331)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- · CONCRETE SEGMENT LINING DIMPLE

STA STATIONING ON S1 TUNNEL
PCL PROFILE CONTROL LINE
GL GROUND LEVEL
OD OUTSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	13 FT

DEVELOPED ELEVATION (B / TR-C331)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT PO503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

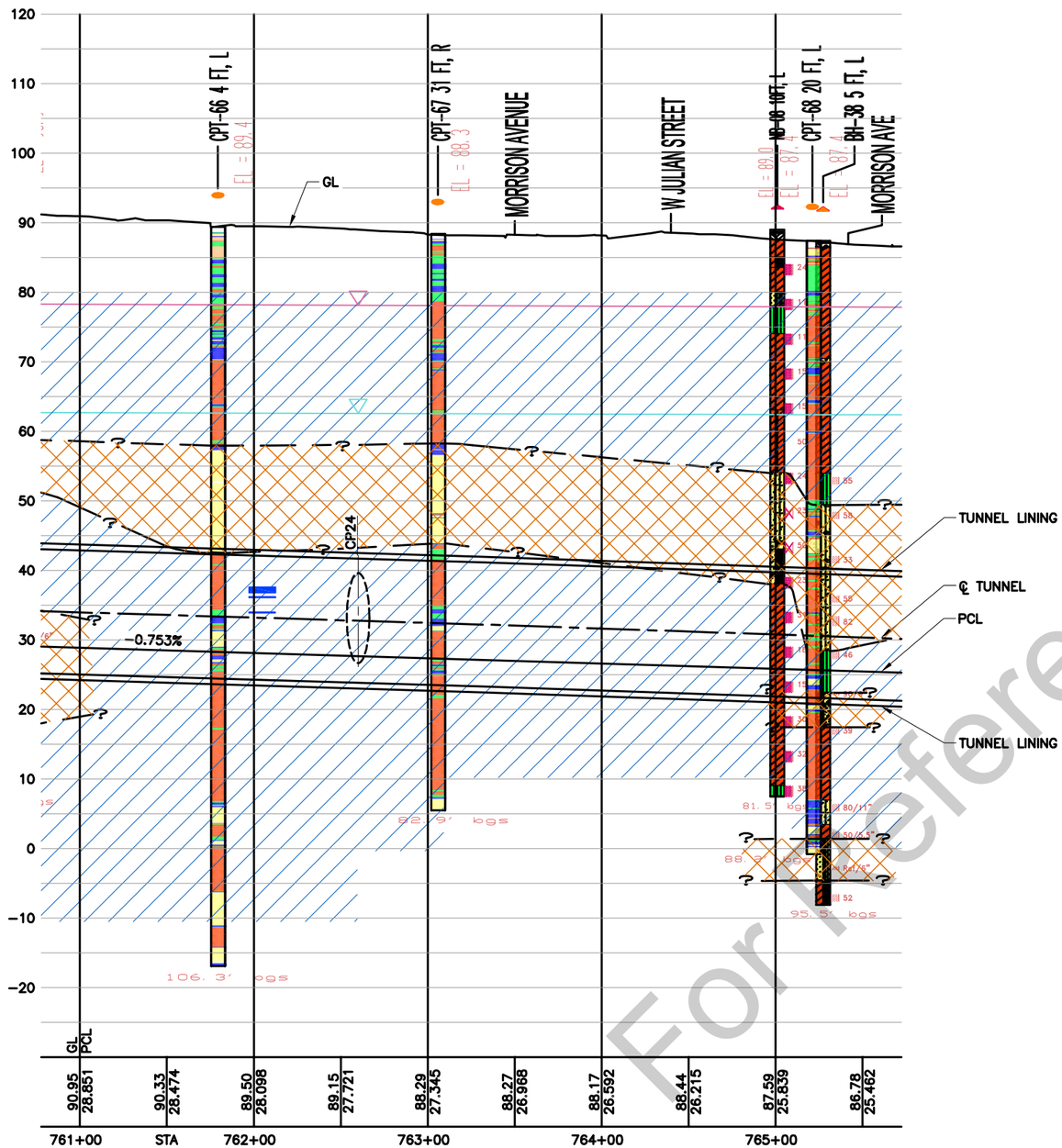
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 23

CADD FILENAME C330-S-TR-C331.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C331 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:24pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C331.dwg

See disclaimer on cover page.



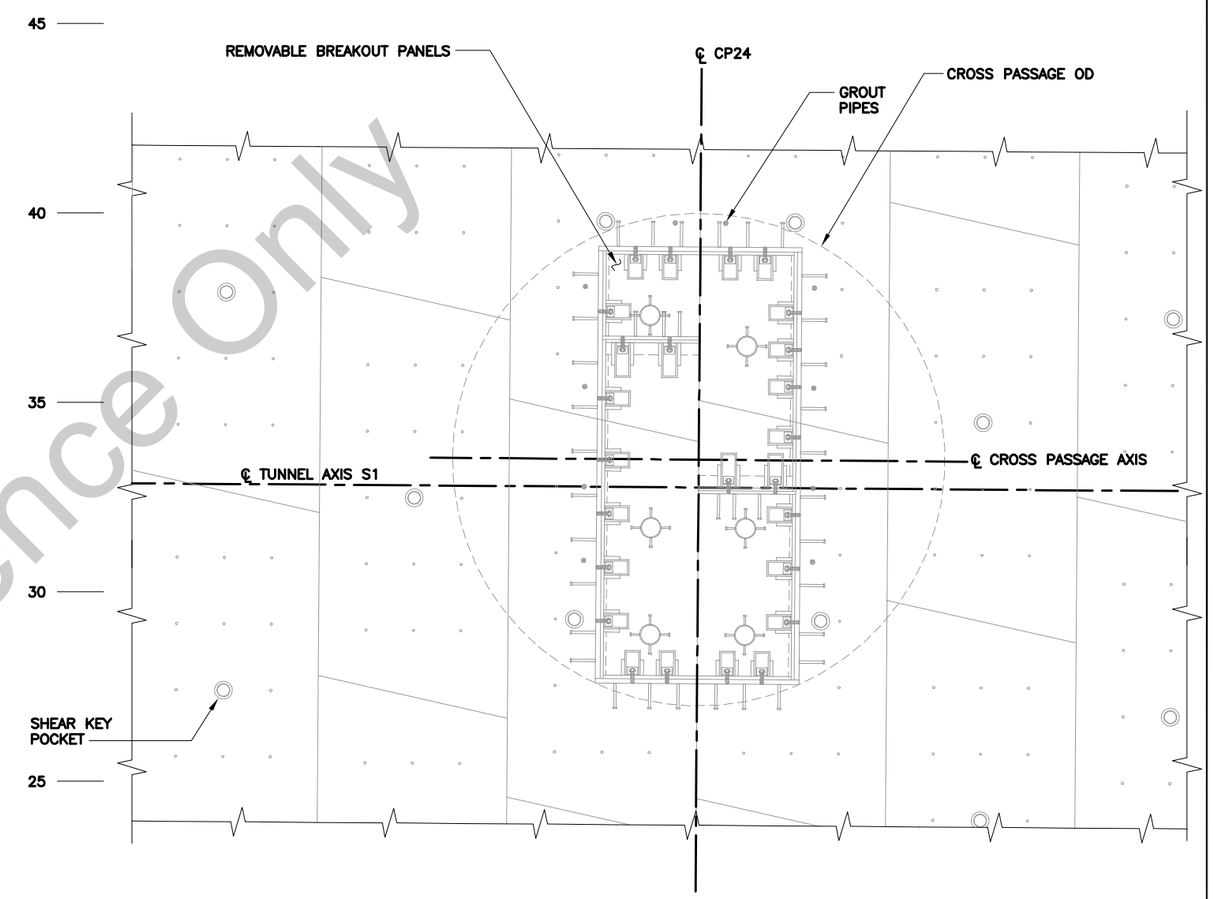
SOIL PROFILE AT CROSS PASSAGE 24 (A / TR-C332)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- · CONCRETE SEGMENT LINING DIMPLE

STA STATIONING ON S1 TUNNEL
PCL PROFILE CONTROL LINE
GL GROUND LEVEL
OD OUTSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C332)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED ____ APPROVED ____

VTA Silicon Valley Rapid Transit Corridor — BART

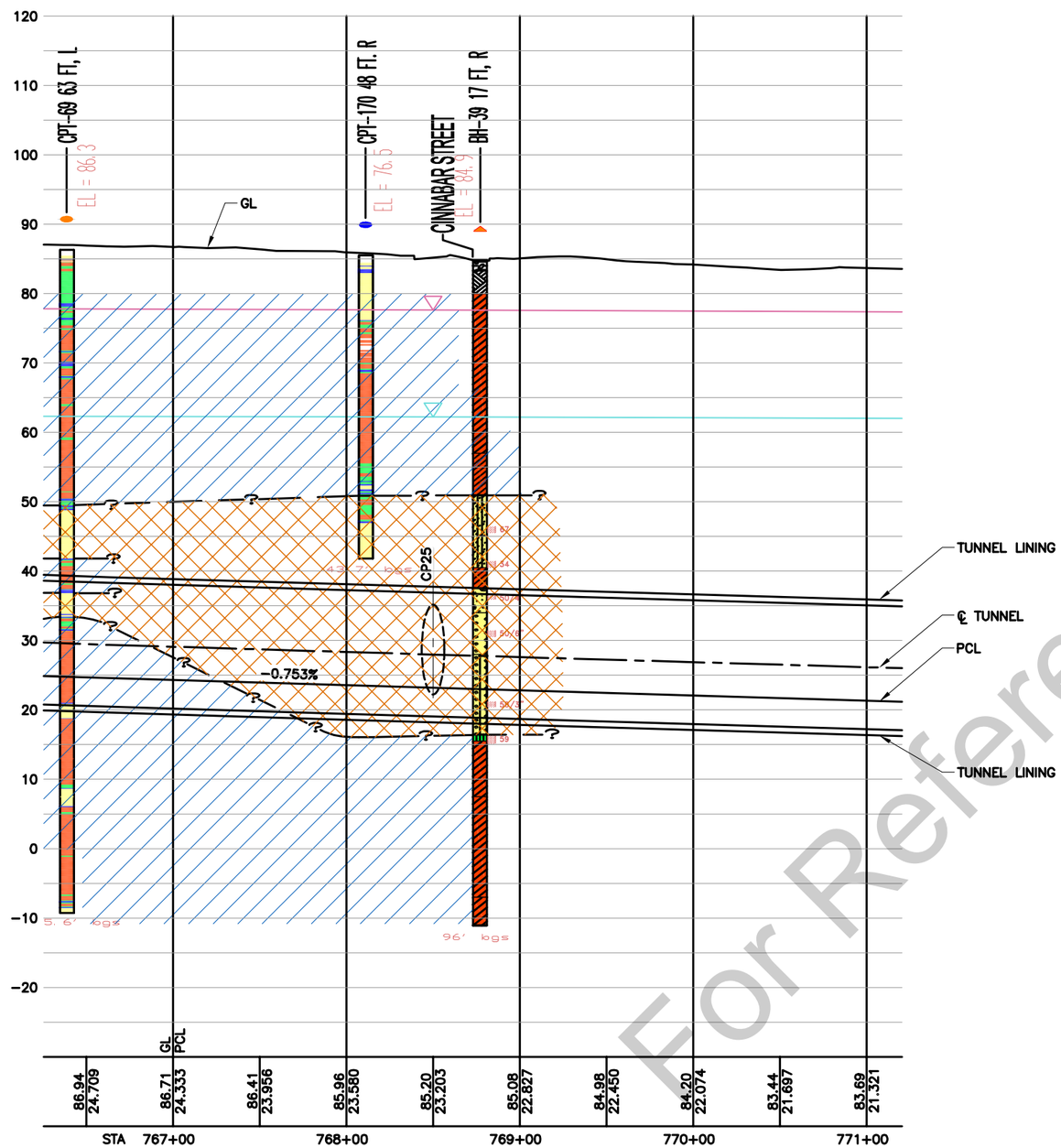
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 24

CADD FILENAME C330-S-TR-C332.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C332 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:25pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C332.dwg

See disclaimer on cover page.

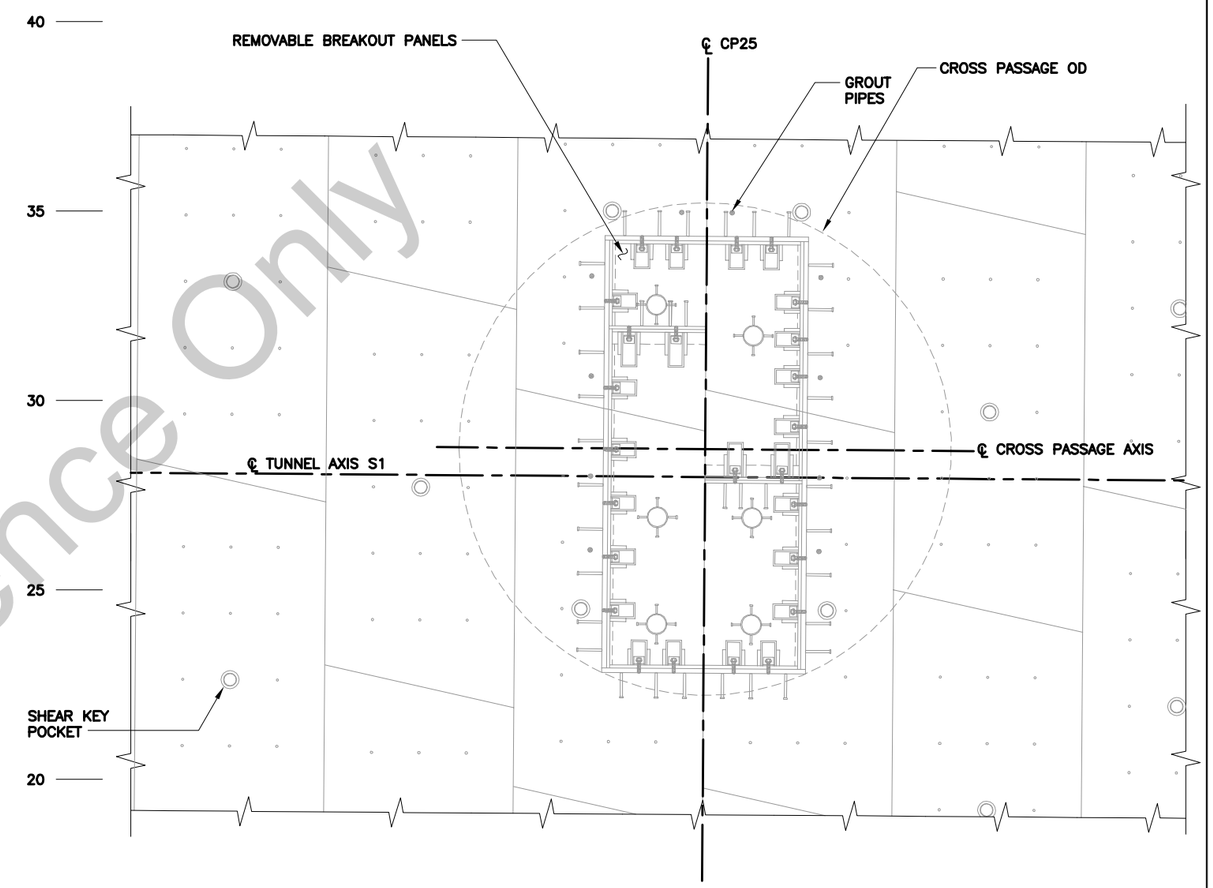


SOIL PROFILE AT CROSS PASSAGE 25 (A / TR-C333)

SCALE: 1"=40'H 1"=10'V

LEGEND:

□	PERMEATION GROUTING HOLE	STA	STATIONING ON S1 TUNNEL
○	VACUUM WELLPOINT	PCL	PROFILE CONTROL LINE
·	CONCRETE SEGMENT LINING DIMPLE	GL	GROUND LEVEL
		OD	OUTSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C333)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

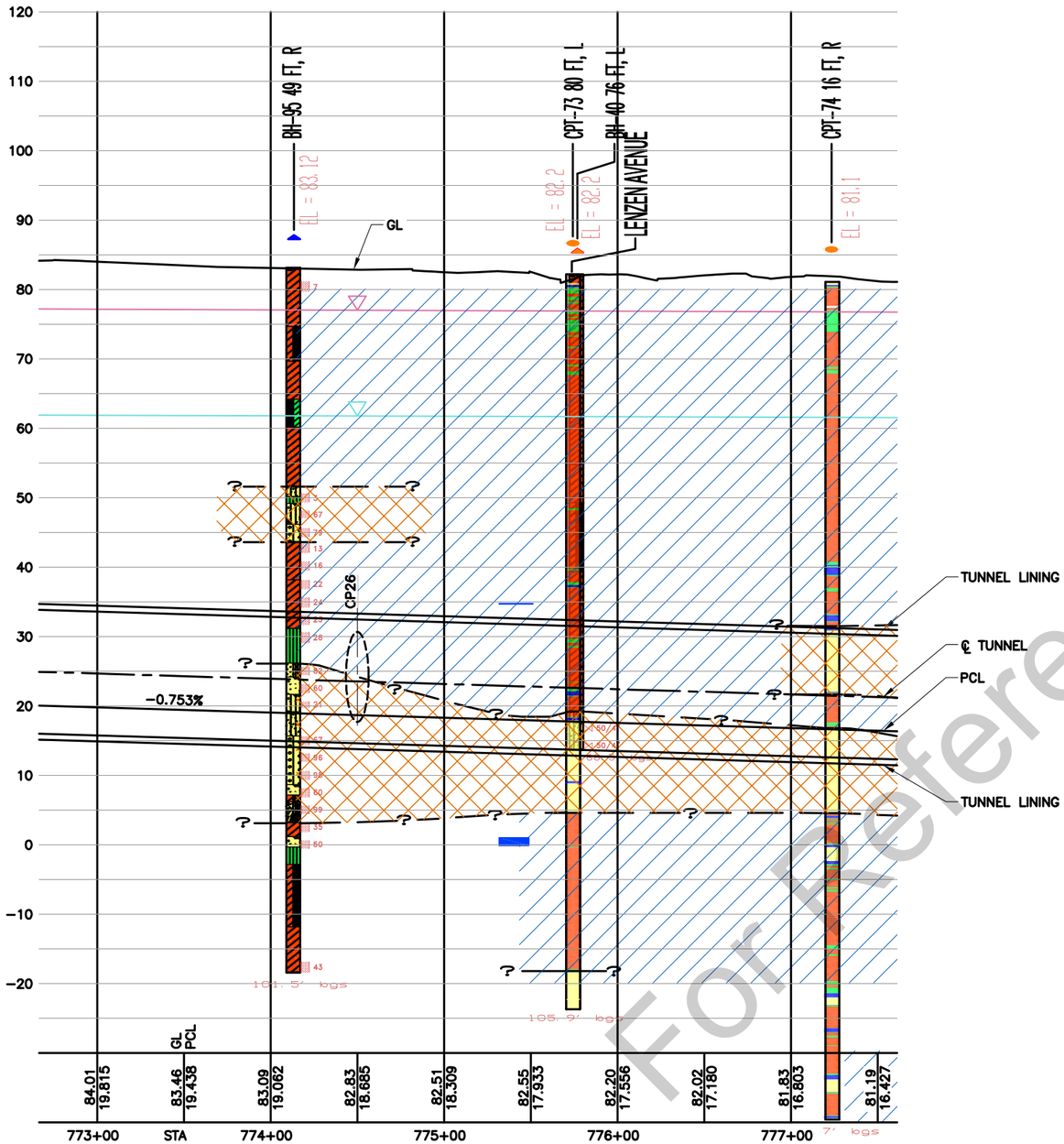
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 25

CADD FILENAME C330-S-TR-C333.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C333 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:26pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C333.dwg

See disclaimer on cover page.



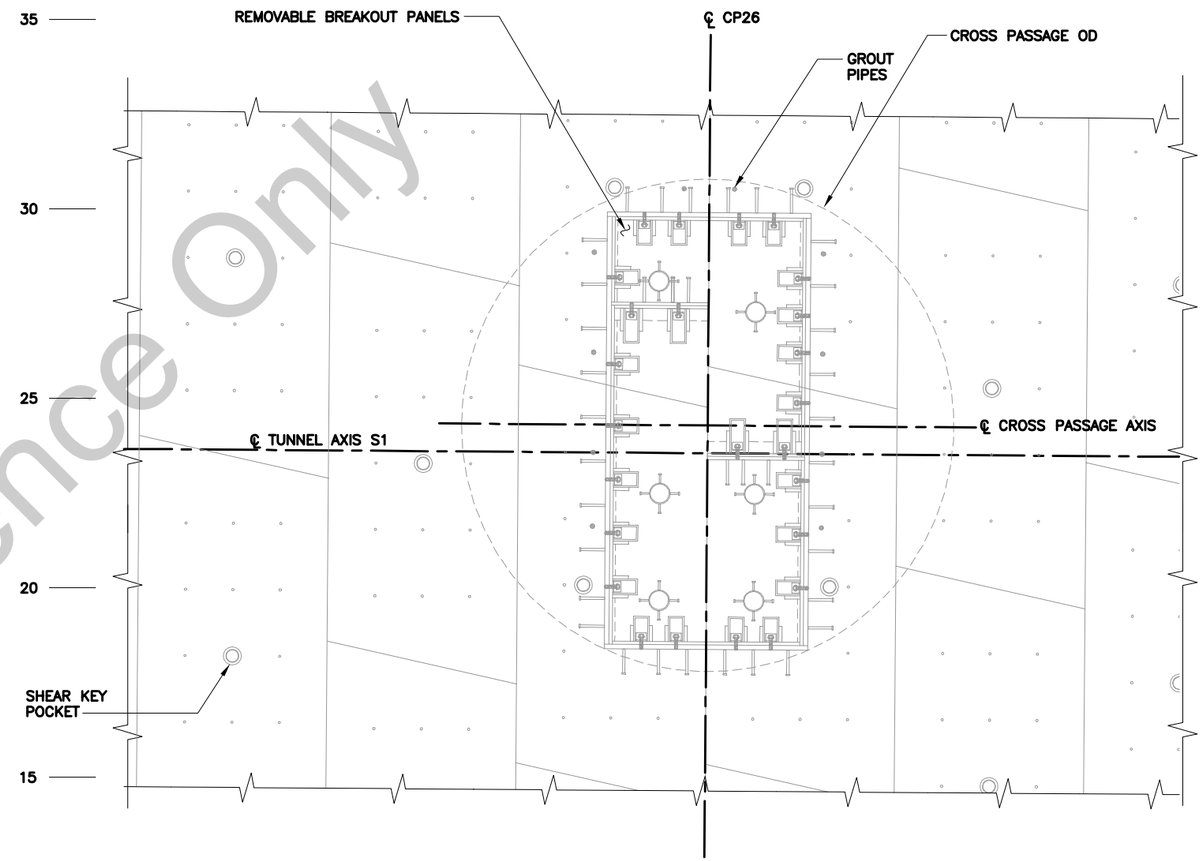
SOIL PROFILE AT CROSS PASSAGE 26 (A / TR-C334)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA STATIONING ON S1 TUNNEL
PCL PROFILE CONTROL LINE
GL GROUND LEVEL
OD OUTSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C334)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

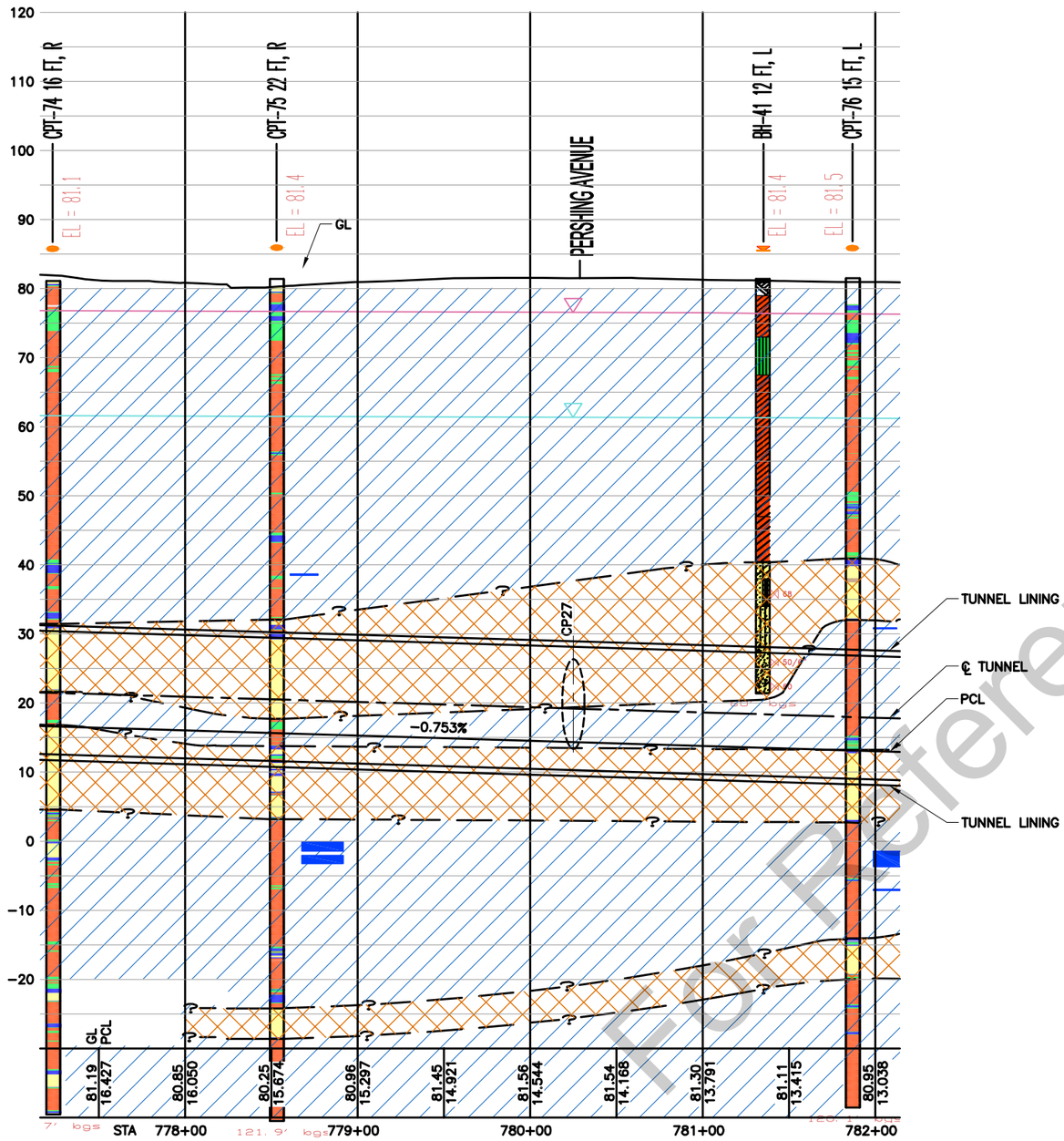
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 26

CADD FILENAME C330-S-TR-C334.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C334 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:27pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C334.dwg

See disclaimer on cover page.



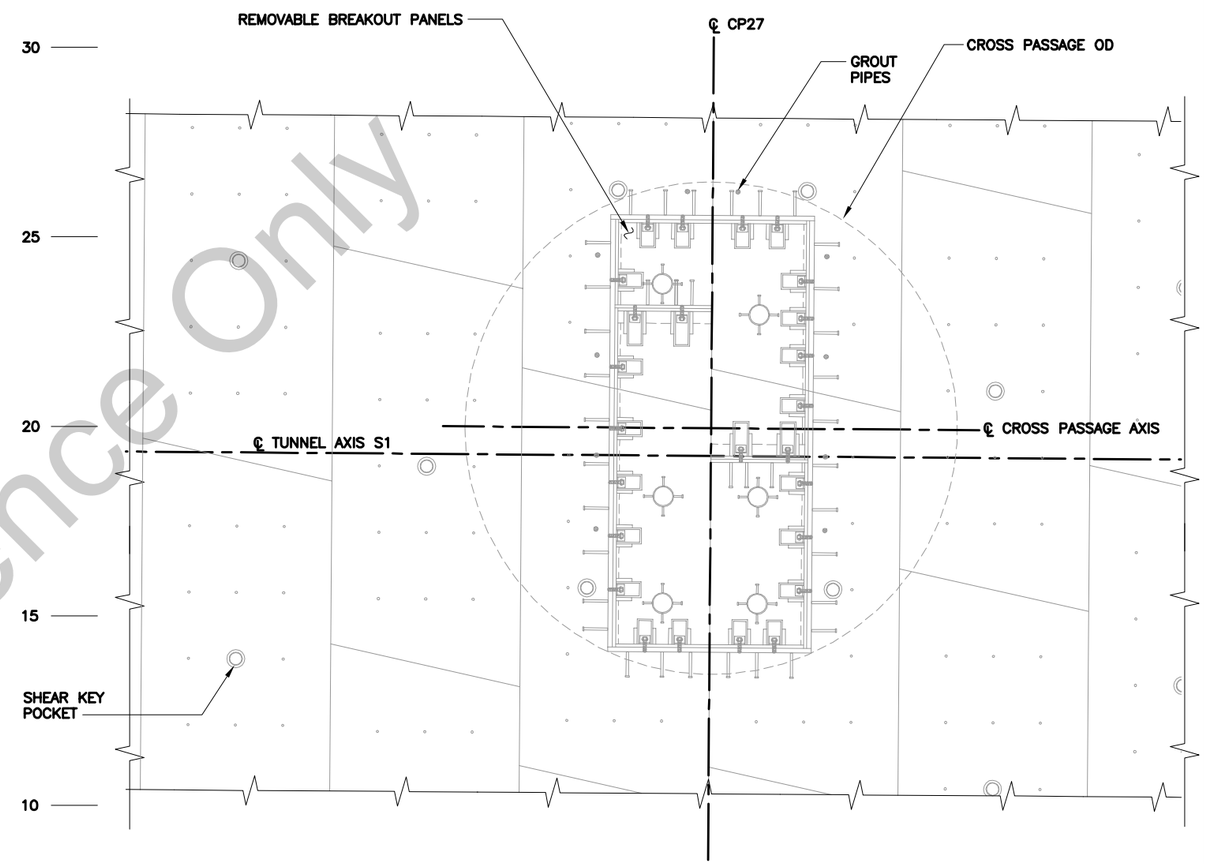
SOIL PROFILE AT CROSS PASSAGE 27 (A / TR-C335)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C335)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

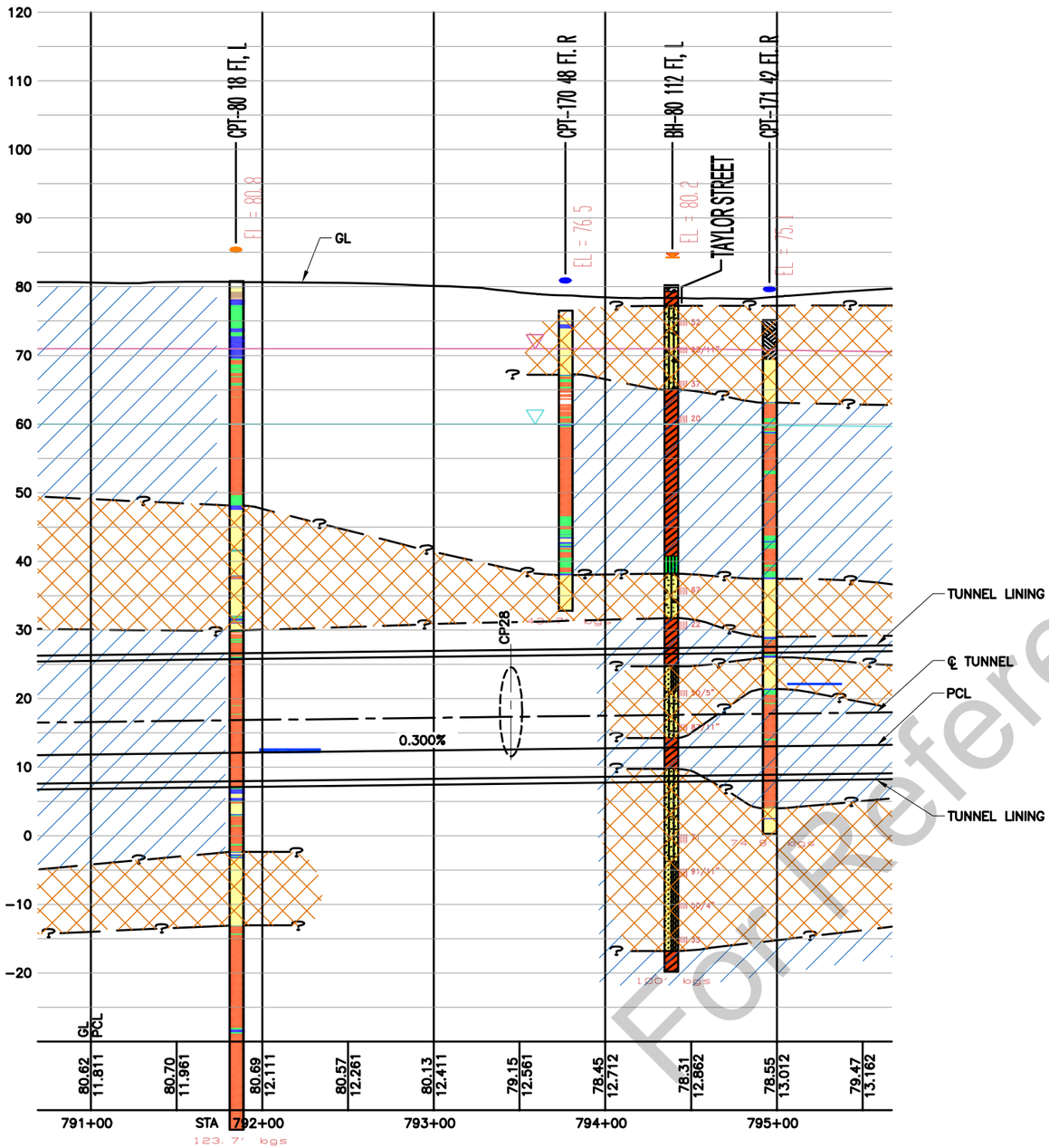
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 27

CADD FILENAME C330-S-TR-C335.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C335 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:28pm W:\TUCADD\625 EQ\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C335.dwg

See disclaimer on cover page.



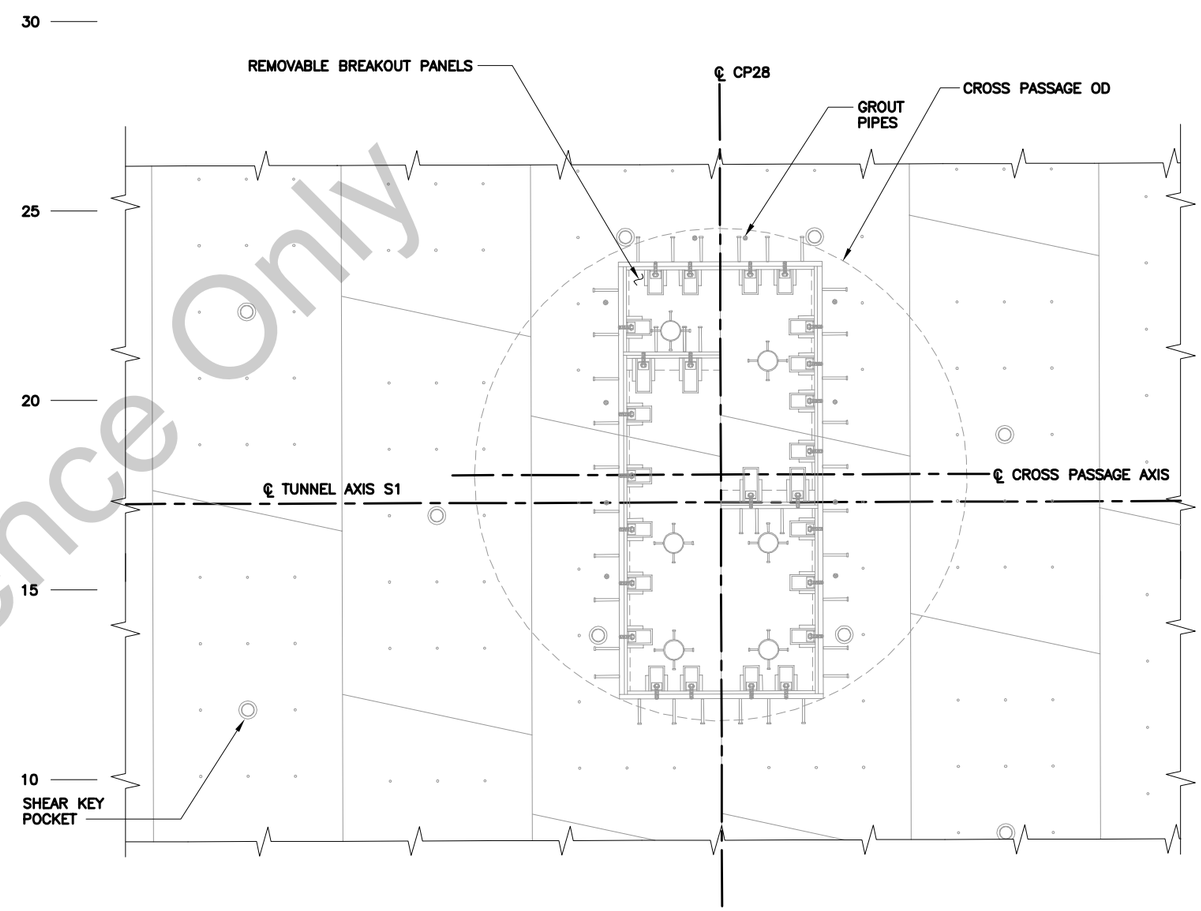
SOIL PROFILE AT CROSS PASSAGE 28 (A / TR-C336)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C336)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

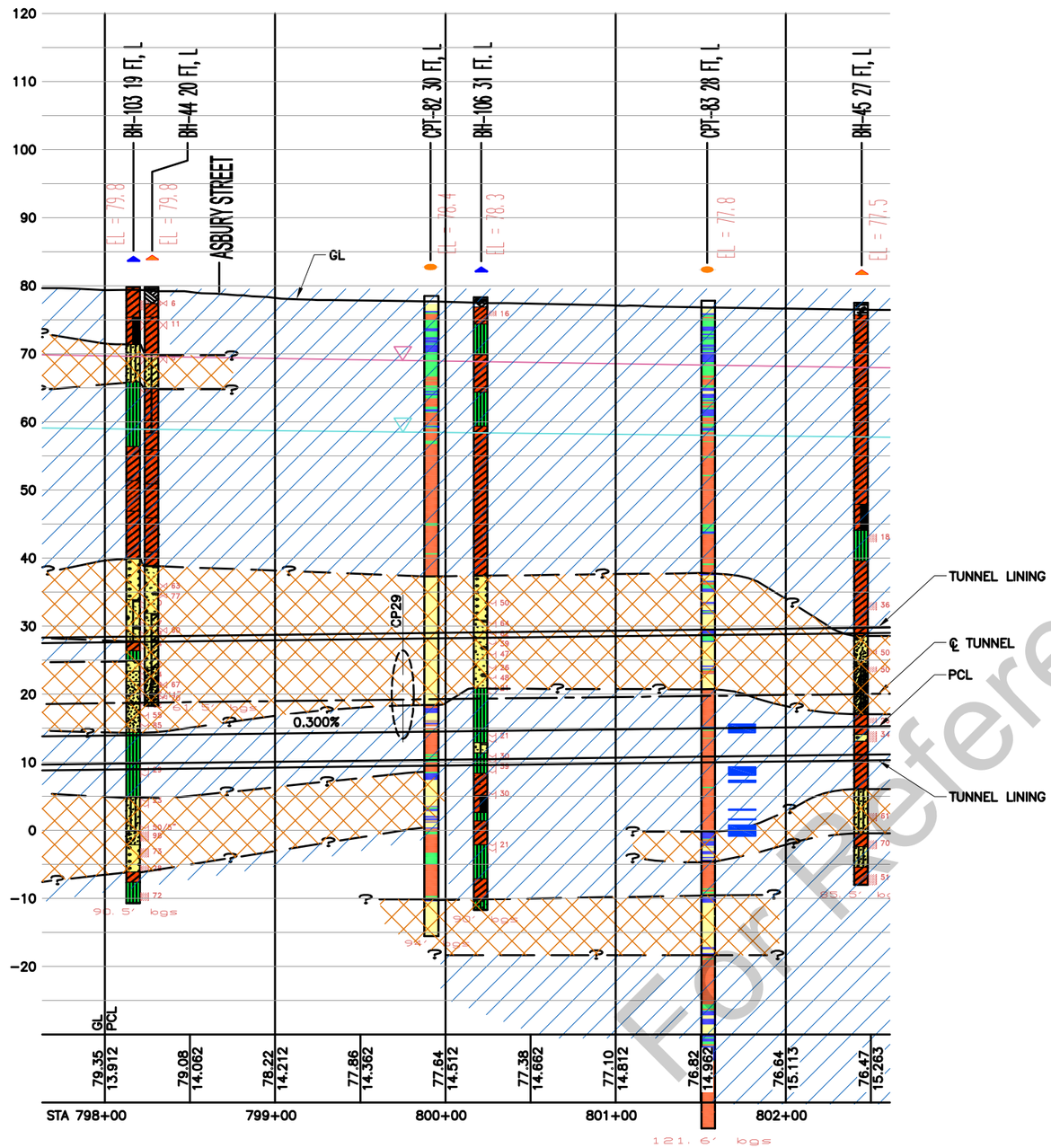
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 28

CADD FILENAME C330-S-TR-C336.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C336 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:29pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C336.dwg

See disclaimer on cover page.



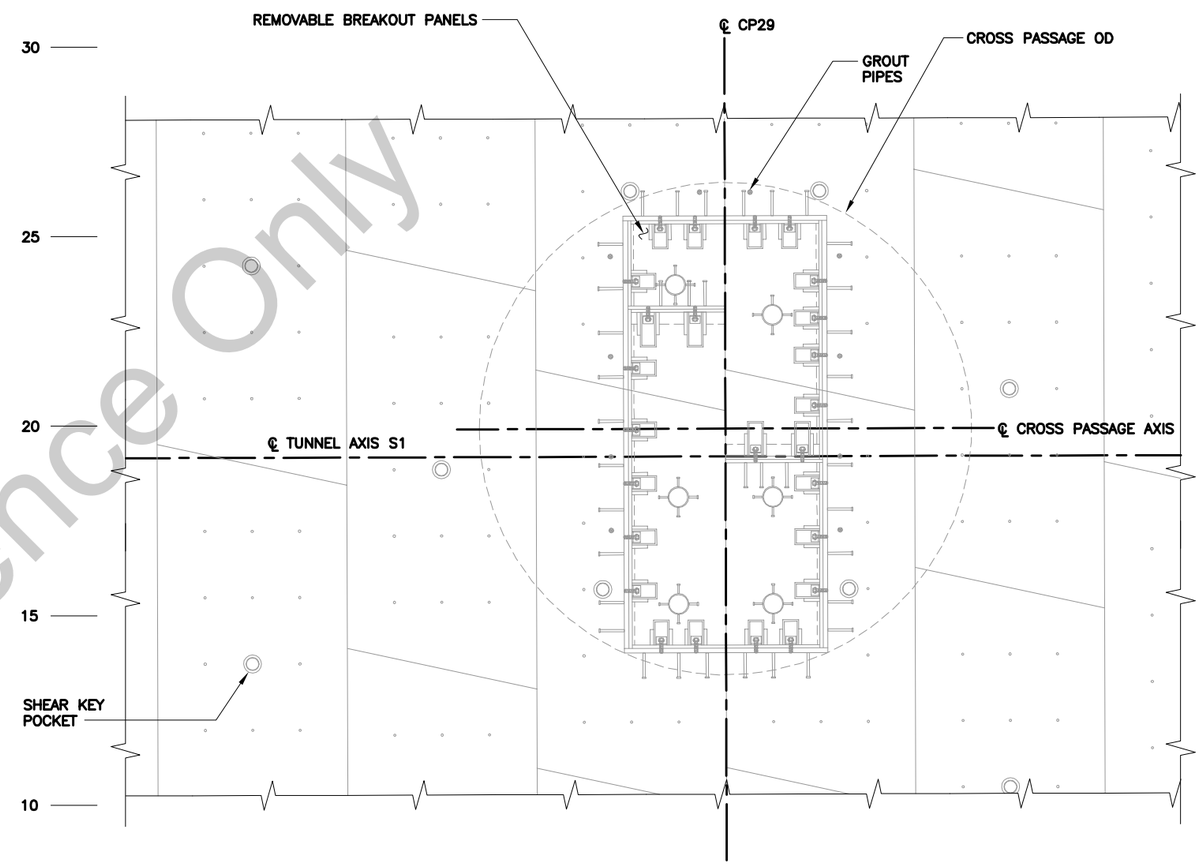
SOIL PROFILE AT CROSS PASSAGE 29 (A / TR-C337)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA STATIONING ON S1 TUNNEL
PCL PROFILE CONTROL LINE
GL GROUND LEVEL
OD OUTSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C337)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

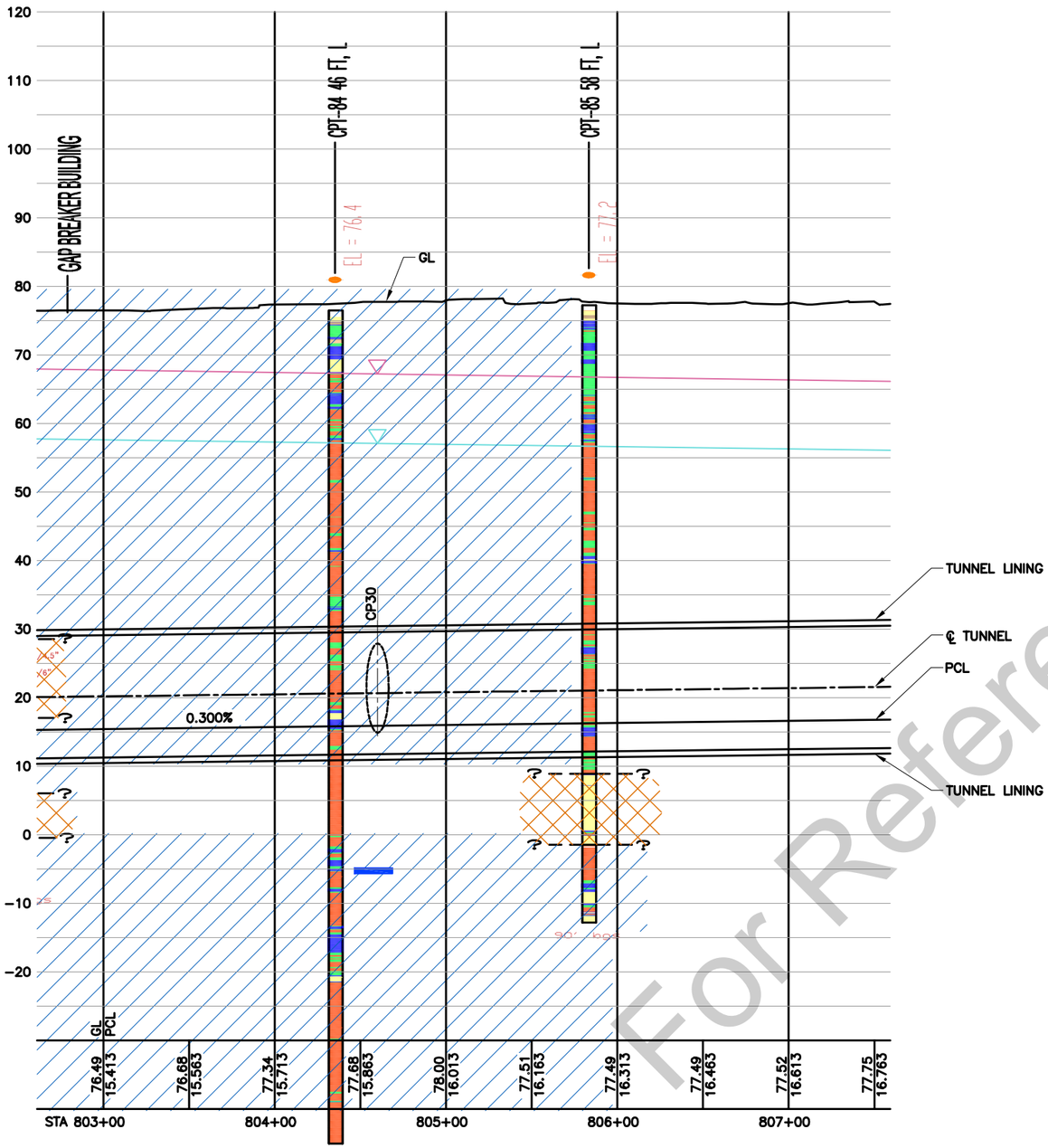
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 29

CADD FILENAME C330-S-TR-C337.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C337 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:30pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C337.dwg

See disclaimer on cover page.

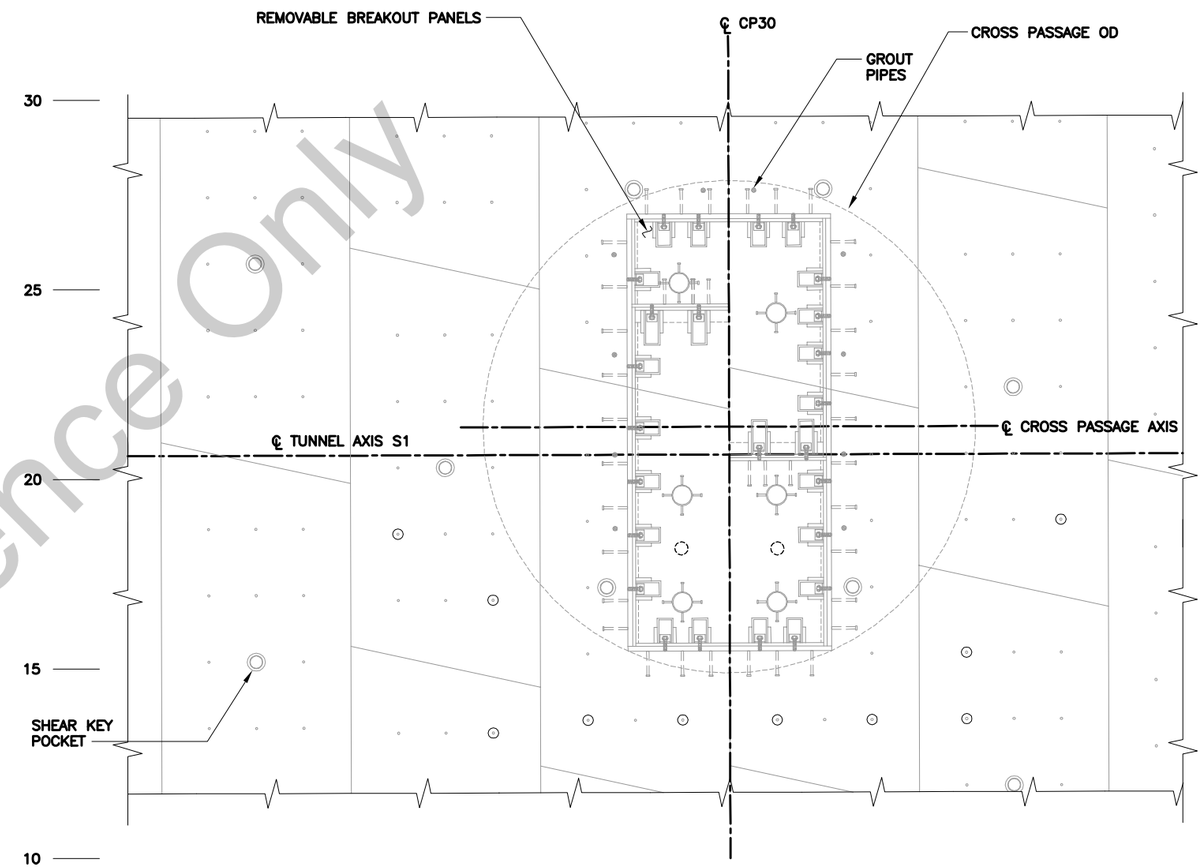


SOIL PROFILE AT CROSS PASSAGE 30 (A / TR-C338)

SCALE: 1"=40'H 1"=10'V

LEGEND:

□	PERMEATION GROUTING HOLE	STA	STATIONING ON S1 TUNNEL
○	VACUUM WELLPOINT	PCL	PROFILE CONTROL LINE
·	CONCRETE SEGMENT LINING DIMPLE	GL	GROUND LEVEL
◌	VACUUM WELLPOINT INSTALLED AFTER PANEL REMOVED, IF REQUIRED	OD	OUTSIDE DIAMETER



EXCAVATION METHOD	PILOT TUNNEL
GROUND TREATMENT METHOD	VACUUM WELL POINTS
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C338)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT PO503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

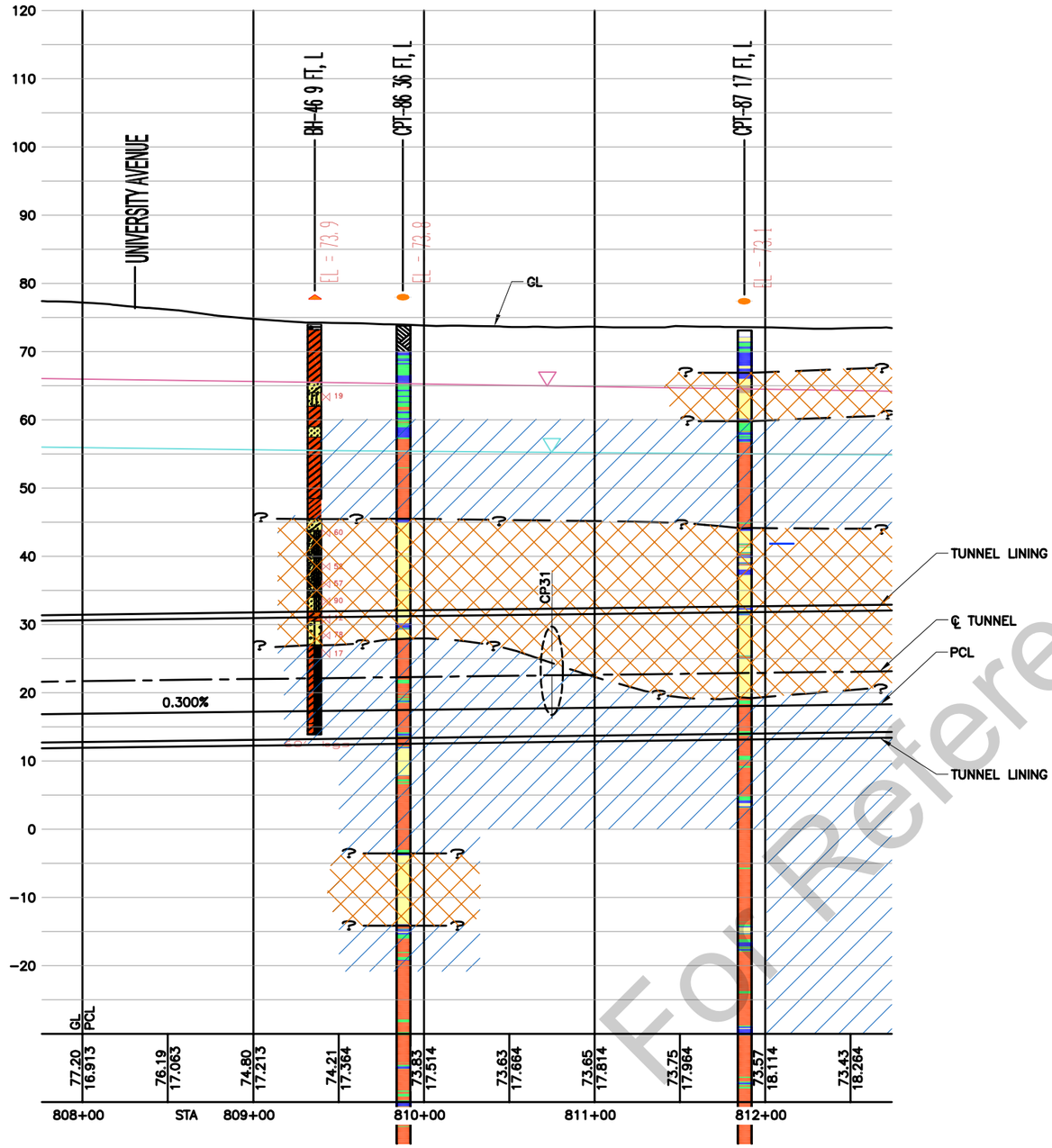
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 30

CADD FILENAME C330-S-TR-C338.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C338 | PAGE NO.

BARNGROVER_A Dec 18, 2008 – 4:31pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C338.dwg

See disclaimer on cover page.

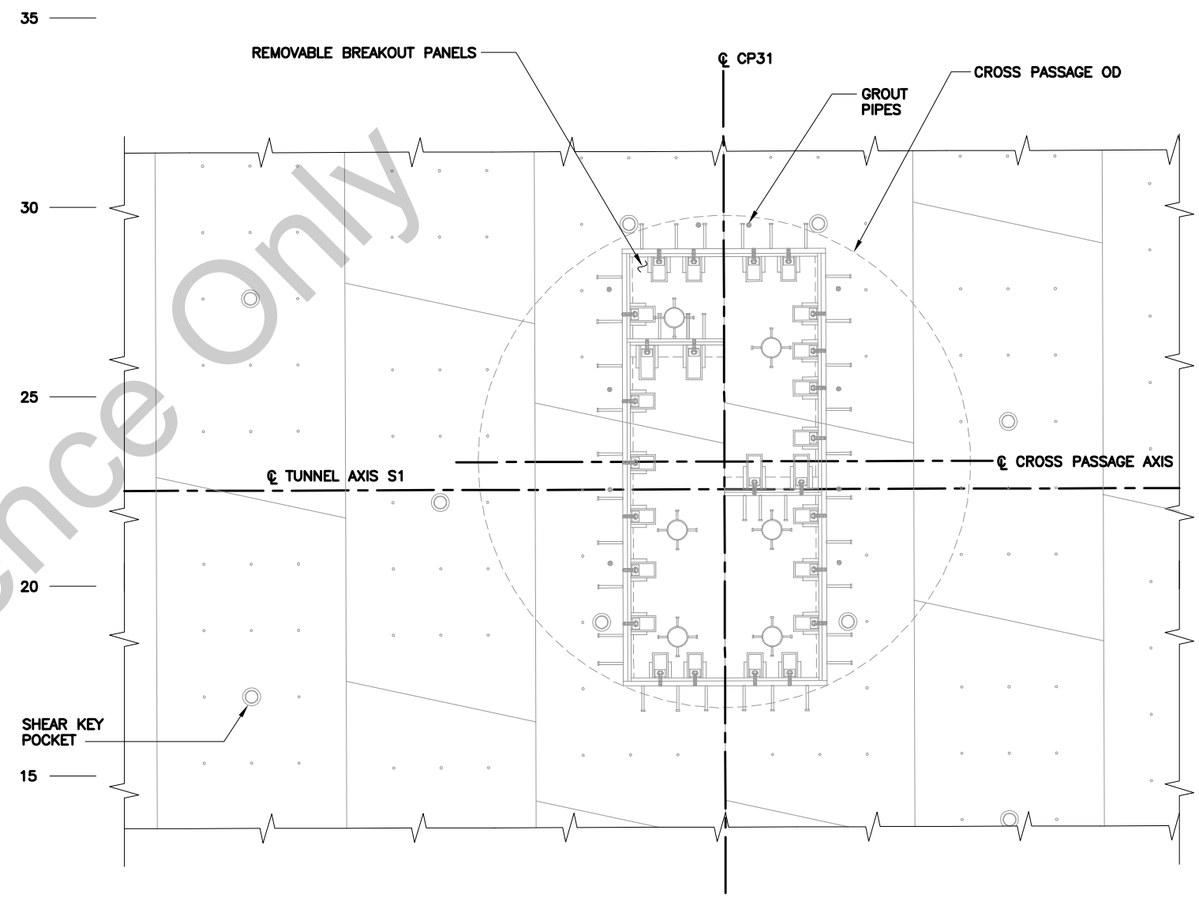


SOIL PROFILE AT CROSS PASSAGE 31 (A / TR-C339)

SCALE: 1"=40'H 1"=10'V

LEGEND:

□	PERMEATION GROUTING HOLE	STA	STATIONING ON S1 TUNNEL
○	VACUUM WELLPOINT	PCL	PROFILE CONTROL LINE
◦	CONCRETE SEGMENT LINING DIMPLE	GL	GROUND LEVEL
		OD	OUTSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	13 FT

DEVELOPED ELEVATION (B / TR-C339)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

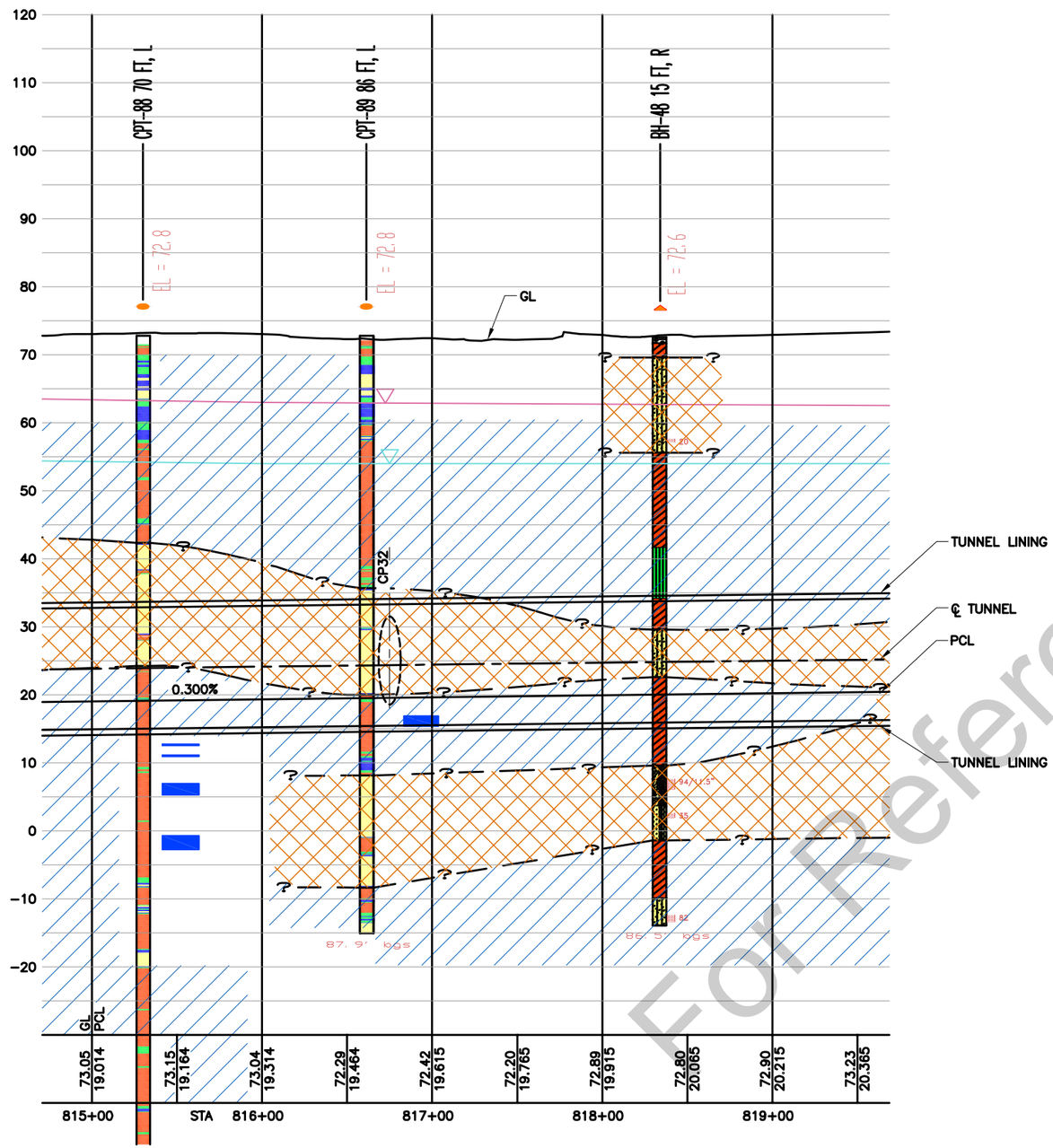
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 31

CADD FILENAME C330-S-TR-C339.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C339 | PAGE NO.

See disclaimer on cover page.

BARNGROVER_A Dec 18, 2008 - 4:32pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C300-S-TR-C339.dwg

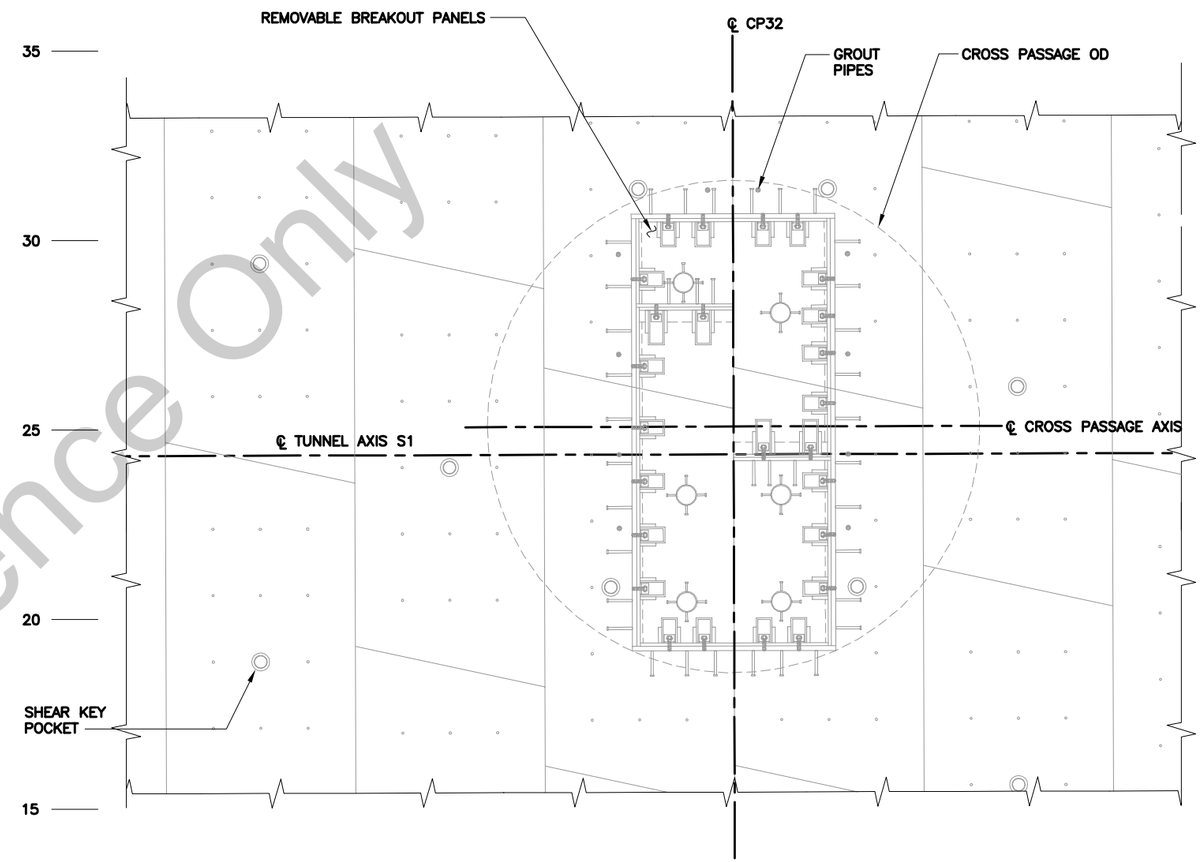


SOIL PROFILE AT CROSS PASSAGE 32 (A / TR-C340)
SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C340)
SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

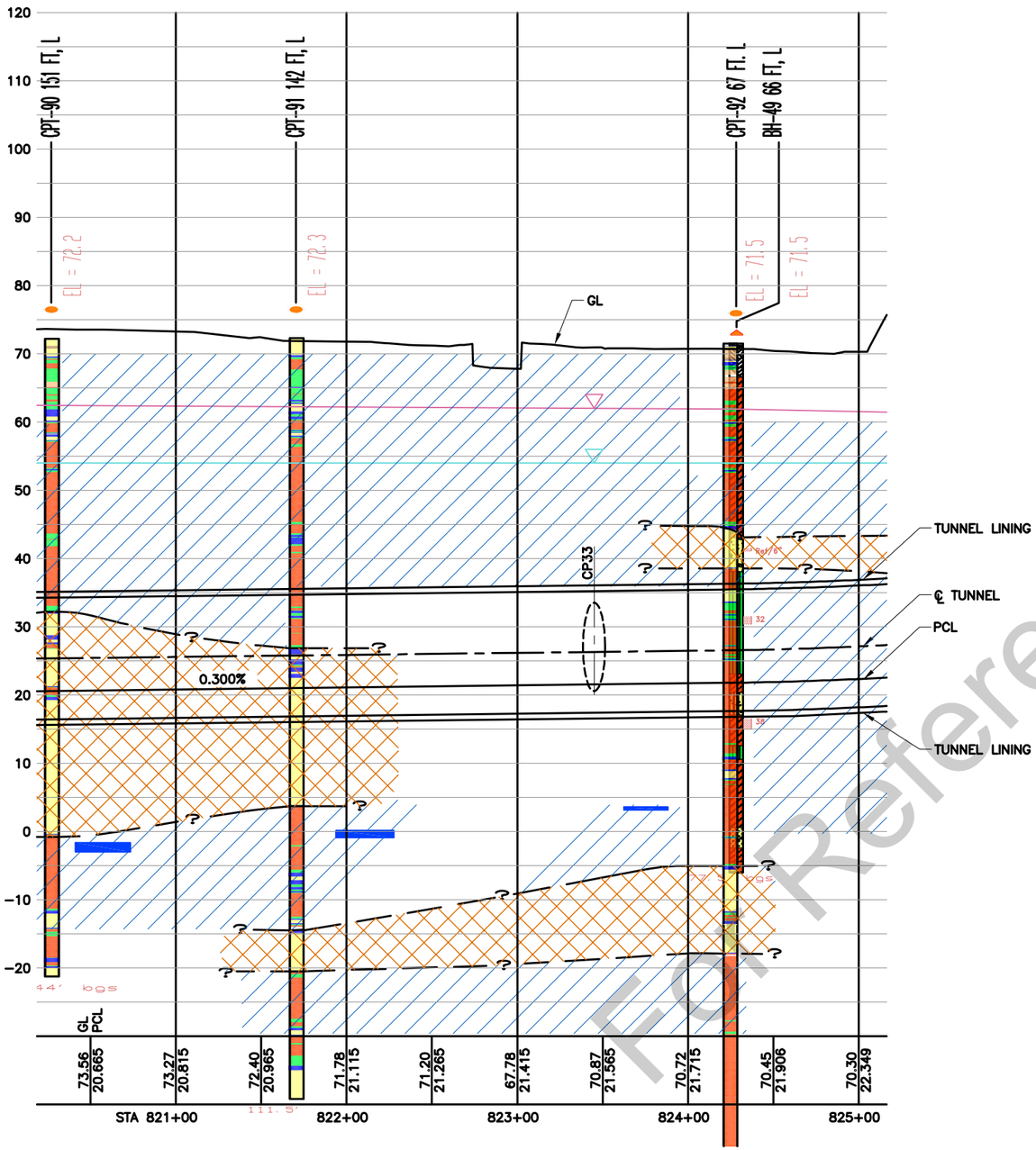
PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 32

CADD FILENAME C330-S-TR-C340.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C340 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:33pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C330-S-TR-C340.dwg

See disclaimer on cover page.



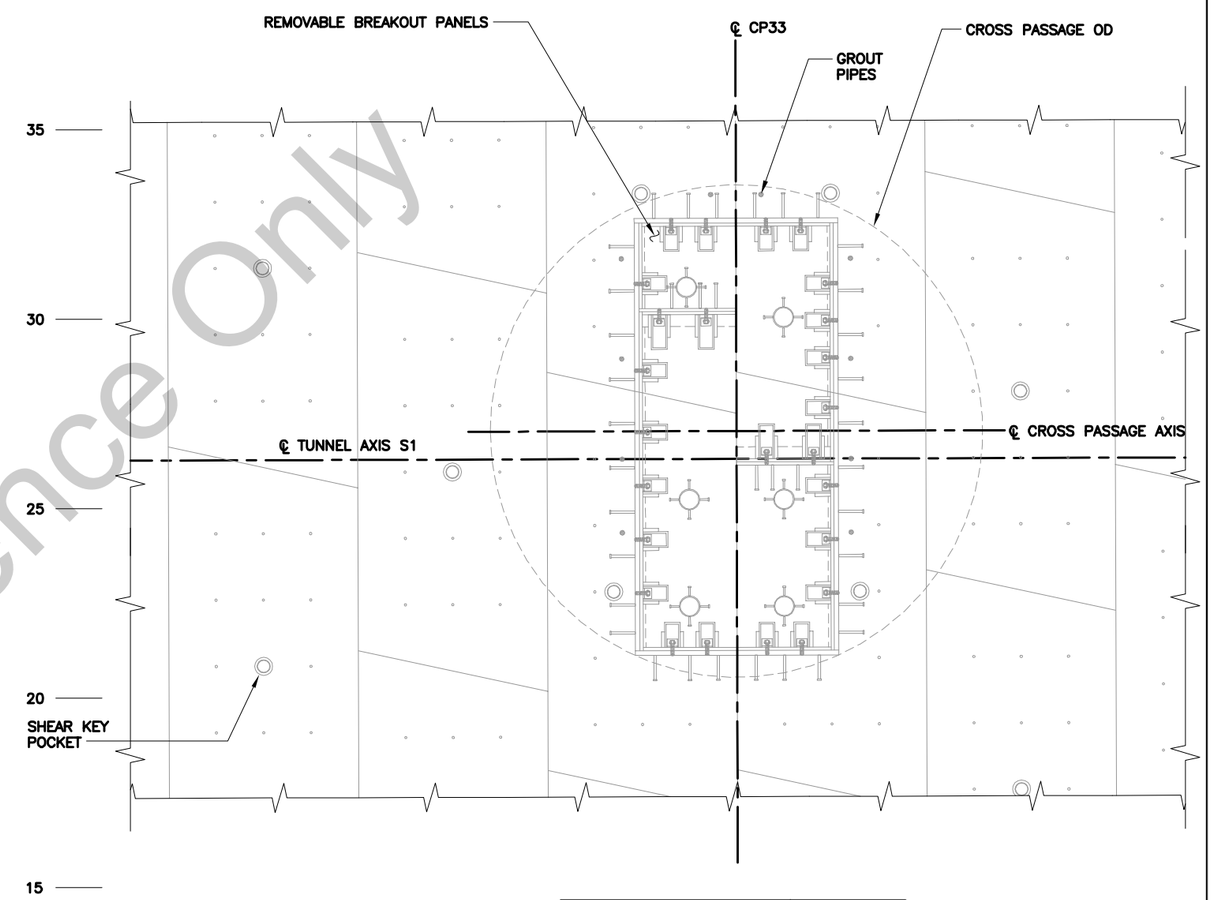
SOIL PROFILE AT CROSS PASSAGE 33 (A / TR-C341)

SCALE: 1"=40'H 1"=10'V

LEGEND:

- □ PERMEATION GROUTING HOLE
- ○ VACUUM WELLPOINT
- ◦ CONCRETE SEGMENT LINING DIMPLE

STA	STATIONING ON S1 TUNNEL
PCL	PROFILE CONTROL LINE
GL	GROUND LEVEL
OD	OUTSIDE DIAMETER



EXCAVATION METHOD	FULL FACE
GROUND TREATMENT METHOD	JET GROUTING
CP DIAMETER	11 FT

DEVELOPED ELEVATION (B / TR-C341)

SCALE: 1"=2'

PRELIMINARY DRAWING FOR 65% PHASE, NOT FOR CONSTRUCTION

For Reference Only

REV	DATE	BY	SUB	APP	DESCRIPTION
A	20081121				ISSUED FOR REPORT P0503-D300-RPT-GEO-006

DESIGNED BY D YOUNG
DRAWN BY H JOHNSON
CHECKED BY A BARNGROVER
IN CHARGE D YOUNG
DATE 20081121

HMM / BECHTEL
A Joint Venture of Hatch Mott MacDonald T&T, Inc. and Bechtel Infrastructure Corp.

SUBMITTED _____ APPROVED _____

VTA Silicon Valley Rapid Transit Corridor — BART

PORTALS, TUNNELS AND CROSS PASSAGES

INTERPRETIVE SOIL PROFILE
ESTIMATED GROUND TREATMENT
AND EXCAVATION METHOD
AT CROSS PASSAGE 33

CADD FILENAME C330-S-TR-C341.DWG
SIZE D | SCALE AS NOTED
CONTRACT NO. C330 | REV. A
AREA CODE TR | SHEET NO. C341 | PAGE NO.

BARNGROVER_A Dec 18, 2008 - 4:34pm W:\TUCADD\65% EC\STUDIES\Cross Passage Interpretive Drawings\C330-S-TR-C341.dwg

See disclaimer on cover page.

Appendix A

Guidelines for Interpretation of Boring and CPT Logs for Soil Layer Profiles

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

Silicon Valley Rapid Transit (SVRT) Project - Central Area Guideway

Interpretive Soil Profile and Soil Classification Methodology

The soil profile for the SVRT project has been classified into two (2) strata: Unit 1 and Unit 2. The classification is based on subsurface exploration by borings and cone penetrometer soundings (CPTs).

Unit 1 (or I): This unit type consists of fine-grained soils, classified as silts (M), clays (C) and organic soils (O) according to Unified Soil Classification System (USCS). It also includes man-made FILL material consisting of asphalt concrete and aggregate base encountered within ten feet below ground surface. The Soil Behavior Type (SBT) zones 1 through 4, and 9, as classified by CPTs - consisting of fine-grained and organic soil types are included in this unit. In general, fine-grained soil types are represented by red color on profile sticks.

Unit 2 (or II): This unit type consists of coarse-grained soils, classified as Sands (S) and Gravels (G) according to USCS. It also includes cobbles encountered at some locations. The SBT zones 6 through 8, classified by CPTs - consisting of coarse-grained soil are included in this unit. In general, coarse-grained soil types are represented by yellow color on the sticks.

The SBT zone 5 - 'Sand Mixtures – Silty Sand and Sandy Silt' consists of both fine-grained and coarse-grained soil. Therefore, it was included in either unit type. It is represented by green color on CPT sticks.

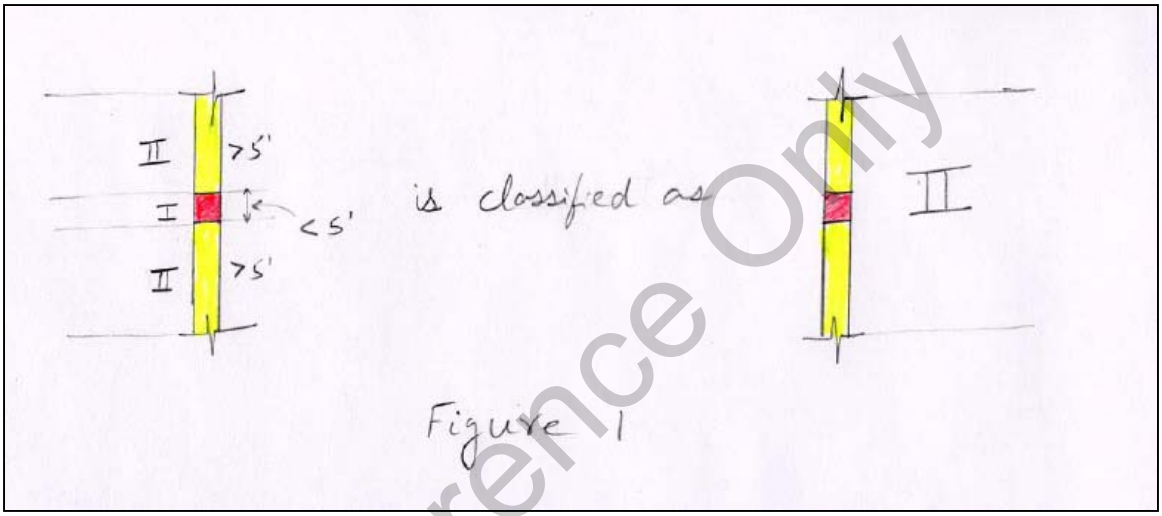
The following methodology was generally followed in soil classification and drawing the interpretive soil profile:

1. Due to a high variability in soil profile encountered vertically with depth, and horizontally along the proposed tunnel alignment caused by alluvial soil depositional environment, five (5) feet was selected as a 'threshold thickness (t_h)'.

For Reference Only

See disclaimer on cover page.

The alternating or individual soil layers belonging to either unit type I or II thinner than t_h were grouped in the other unit type (II or I, correspondingly) if they were adjacent. This is illustrated by a theoretical example in Figure 1 (See CPT-008, CPT-062 on profile drawings). The layers thicker than t_h were not grouped, and classified as individual units. Other special cases are described in the following points.



2. If within a boring or CPT, alternating layers belonging to one unit type had a combined thickness greater than t_h, and are separated by other unit type layers less than three (3) feet thick, then they were grouped together in one unit type (See CPT-126 on profile drawings). This is illustrated by the hypothetical example in Figure 2a. However, if alternating layers belonging to other unit type also had a combined thickness greater than t_h**,** then engineering judgment based on adjacent borings or CPTs was used to classify the alternating soil layer system and to draw the profile. This is illustrated by the hypothetical example in Figure 2b.

For Reference Only

See disclaimer on cover page.

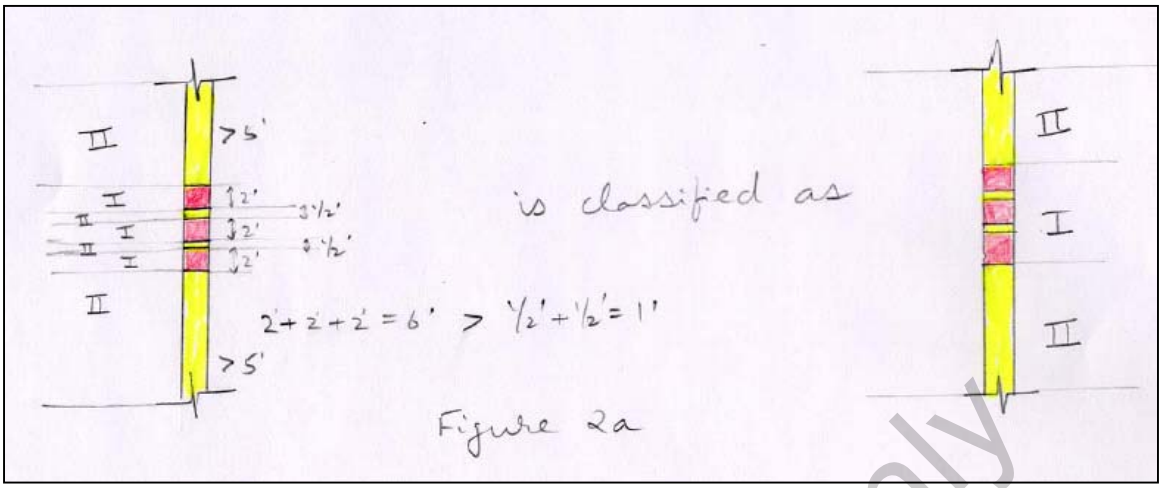


Figure 2a

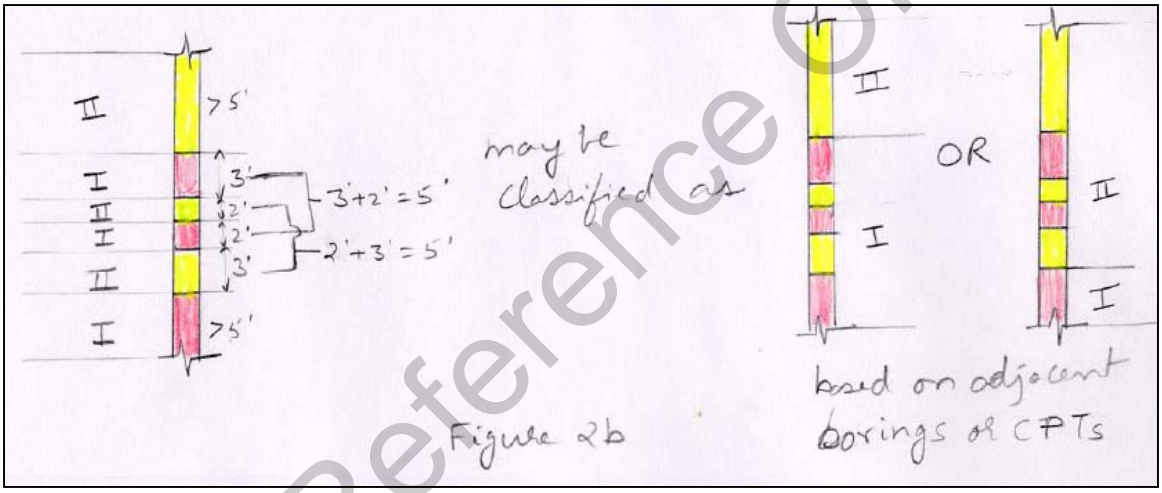
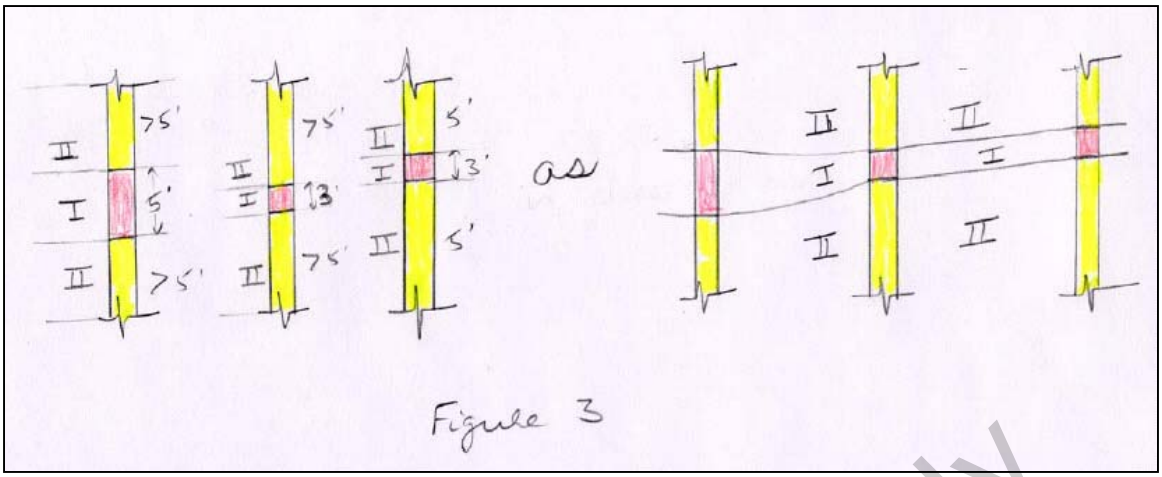


Figure 2b

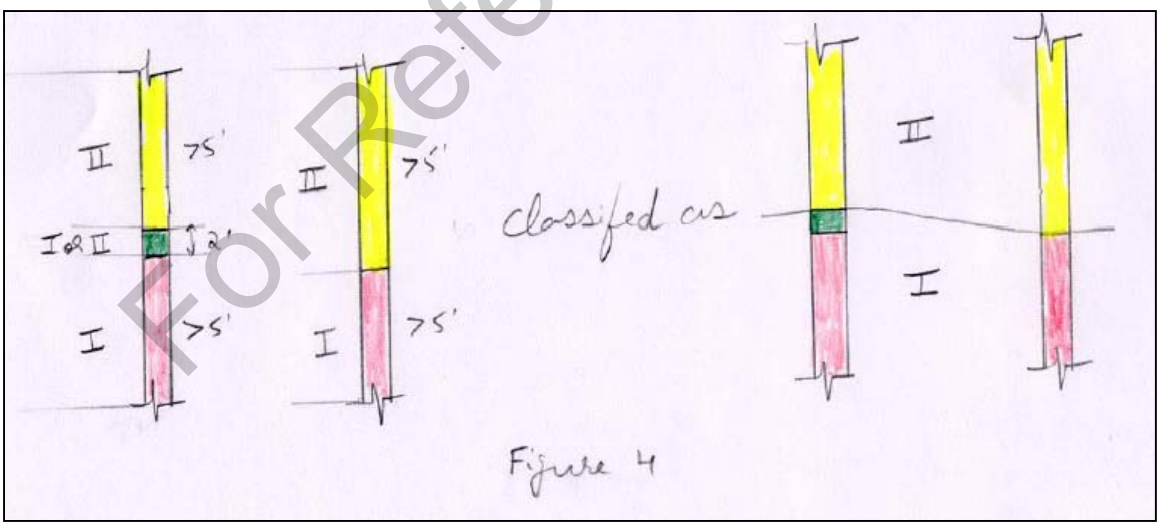
For Reference Only

3. If within a boring or CPT, an individual layer or an alternating layer system belonging to one unit type was thinner than t_h, but thicker than or equal to three (3) feet, and the adjacent borings/CPTs on either sides consisted of similar layers at similar elevations, then the boundaries of those soil layers were joined, instead of grouping the thinner layer with the layers above or below within that boring or CPT as illustrated by the hypothetical example in Figure 3 (See BH-28, CPT-053, and BH-29 on profile drawings).

See disclaimer on cover page.



4. The SBT 5 was included within unit I or II depending upon its thickness and relative location with respect to the layers above and below it, and the layers at the similar elevations in adjacent borings or CPTs. If it was present between strata I and II and adjacent CPTs or borings did not have strong evidence that it be included in II, then it was included in unit I. This is illustrated by the hypothetical example in Figure 4 (See CPT-056 on profile drawings).



For Reference Only

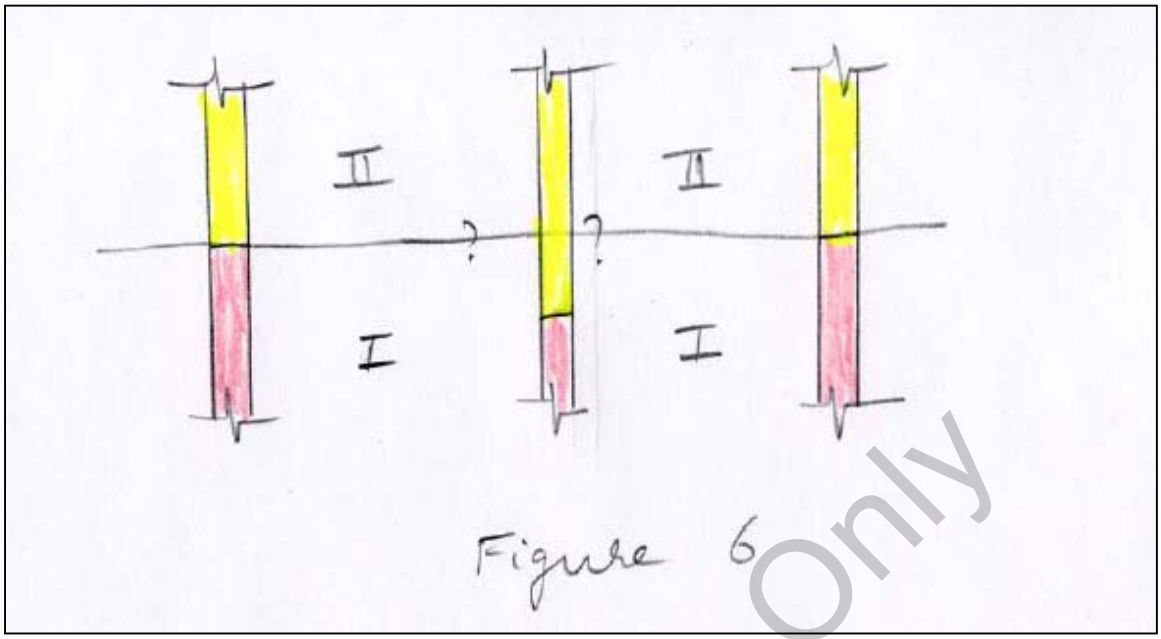
See disclaimer on cover page.

5. If within a boring or a CPT, there exist two adjacent layers thinner than t_h belonging to either unit, and each of them could be grouped in either of the strata, then a judgment based on adjacent borings or CPTs was used to group the layers, and profile accordingly drawn. This is shown by the hypothetical example in Figure 5 (See CPT-037 and CPT-052 on profile drawings).

6. If within a CPT, the boundaries of a layer at certain elevation were not confirmed by adjacent borings or CPTs on either side, then the boundaries of that layer were decided based on the adjacent borings or CPTs. Similarly, if the presence of a unit type at certain elevation in a CPT is not supported by adjacent borings or CPTs, then the layer was ignored and profile drawn based on adjacent borings and CPTs as shown by the hypothetical example in Figure 6 (See CPT-132 on profile drawings).

For Reference Only

See disclaimer on cover page.



7. If there was a conflict between a unit type at certain elevation in a CPT and a boring drilled very close to each other, then the drilled boring (confirming the unit type with a sample) was given precedence over the CPT, and the unit type at that elevation in the CPT was ignored (See CPT-068, BH-38, CPT-069, BH-39; CPT-134A or BH-77 on profile drawings).

Notes:

1. The profiles are drawn using the gINT® database software, and they are essentially projections on a selected cross-section along the S1 tunnel alignment. Therefore, the profiles are 2-D approximations of a 3-D sub-surface space.

2. The strata boundaries making up the profile are drawn as straight lines joining the layers in different CPTs and borings along the alignment. However, they may not be straight lines as existing in nature, and hence be construed only an approximation. Further, the boundary lines may vary with the availability of more information. The question marks (?) are placed at the ends of discontinuous layers, or over certain boundary lines to represent the relative uncertainty.

For Reference Only

See disclaimer on cover page.

Appendix B
Cal/OSHA Underground Classification

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

C078-085-09T

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of 3103 North First Street, San Jose, California 95134
(MAILING ADDRESS)

at SILICON VALLEY RAPID TRANSIT PROJECT – CENTRAL AREA
(LOCATION)

has been classified as *** POTENTIALLY GASSY with Special Conditions***
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.
2. Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.
3. The Division shall be notified immediately if any **Flammable Gas** or **Petroleum Vapor** exceeds 5% of the Lower Explosive Limit.
4. All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.

The 61-foot wide by 27-foot high by 160 feet long tunnel project called the East Portal Cut and Cover located near the intersection of Las Palmas Avenue and Marburg Way, San Jose, Santa Clara County.

This classification shall be conspicuously posted at the place of employment.

Date October 8, 2008

(SENIOR ENGINEER)

John R. Leahy

03 77089

For Reference Only

See disclaimer on cover page.

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

C079-085-09T

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of 3103 North First Street, San Jose, California 95134
(MAILING ADDRESS)

at SILICON VALLEY RAPID TRANSIT PROJECT – CENTRAL AREA
(LOCATION)

has been classified as *** POTENTIALLY GASSY with Special Conditions***
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.
2. Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.
3. The Division shall be notified immediately if any **Flammable Gas** or **Petroleum Vapor** exceeds 5% of the Lower Explosive Limit.
4. All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.

The 22-foot diameter by 2,823 feet long twin tunnel bores with cross-passages called the East Portal to Alum Rock bores located near the intersection of Highway 101and McKee Road, San Jose, Santa Clara County.

This classification shall be conspicuously posted at the place of employment.

Date October 8, 2008

John R. Leahy
(SENIOR ENGINEER)

John R. Leahy

03 77089

For Reference Only

See disclaimer on cover page.

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

C080-085-09T

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of 3103 North First Street, San Jose, California 95134
(MAILING ADDRESS)

at SILICON VALLEY RAPID TRANSIT PROJECT – CENTRAL AREA
(LOCATION)

has been classified as *** POTENTIALLY GASSY with Special Conditions***
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.
2. Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.
3. The Division shall be notified immediately if any **Flammable Gas** or **Petroleum Vapor** exceeds 5% of the Lower Explosive Limit.
4. All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.

The 60-foot wide by 51-foot high by 875 feet long tunnel project called the Alum Rock Station located near the intersection of Alum Rock Avenue and North 28th Street, San Jose, Santa Clara County.

This classification shall be conspicuously posted at the place of employment.

Date October 8, 2008

John R. Leahy
(SENIOR ENGINEER)

John R. Leahy

03 77089

For Reference Only

See disclaimer on cover page.

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

C081-085-09T

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of 3103 North First Street, San Jose, California 95134
(MAILING ADDRESS)

at SILICON VALLEY RAPID TRANSIT PROJECT – CENTRAL AREA
(LOCATION)

has been classified as *** POTENTIALLY GASSY with Special Conditions***
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.
2. Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.
3. The Division shall be notified immediately if any **Flammable Gas** or **Petroleum Vapor** exceeds 5% of the Lower Explosive Limit.
4. All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.

The 22-foot diameter by 8,370 feet long twin tunnel bores with cross-passages called the Alum Rock to Crossover Structure bores located near the intersection of Alum Rock Avenue and North 28th Street, San Jose, Santa Clara County.

This classification shall be conspicuously posted at the place of employment.

Date October 8, 2008

John R. Leahy
(SENIOR ENGINEER)

John R. Leahy

03 77089

For Reference Only

See disclaimer on cover page.

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

C082-085-09T

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of 3103 North First Street, San Jose, California 95134
(MAILING ADDRESS)

at SILICON VALLEY RAPID TRANSIT PROJECT – CENTRAL AREA
(LOCATION)

has been classified as *** POTENTIALLY GASSY with Special Conditions***
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.
2. Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.
3. The Division shall be notified immediately if any **Flammable Gas** or **Petroleum Vapor** exceeds 5% of the Lower Explosive Limit.
4. All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.

The 60-foot wide by 80-foot high by 31 feet long tunnel project called the East Mid-tunnel Vent Shaft located near the intersection of Santa Clara Street and North 13th Street, San Jose, Santa Clara County.

This classification shall be conspicuously posted at the place of employment.

Date October 8, 2008

(SENIOR ENGINEER)

John R. Leahy

03 77089

For Reference Only

See disclaimer on cover page.

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

C083-085-09T

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of 3103 North First Street, San Jose, California 95134
(MAILING ADDRESS)

at SILICON VALLEY RAPID TRANSIT PROJECT – CENTRAL AREA
(LOCATION)

has been classified as *** POTENTIALLY GASSY with Special Conditions***
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.
2. Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.
3. The Division shall be notified immediately if any **Flammable Gas** or **Petroleum Vapor** exceeds 5% of the Lower Explosive Limit.
4. All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.

The 60-foot wide by 51-foot high by 1,366 feet long tunnel project called the Downtown San Jose Station located near the intersection of Santa Clara Street and North 13th Street, San Jose, Santa Clara County.

This classification shall be conspicuously posted at the place of employment.

Date October 8, 2008

(SENIOR ENGINEER)

John R. Leahy

03 77089

For Reference Only

See disclaimer on cover page.

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

C084-085-09T

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of 3103 North First Street, San Jose, California 95134
(MAILING ADDRESS)

at SILICON VALLEY RAPID TRANSIT PROJECT – CENTRAL AREA
(LOCATION)

has been classified as *** POTENTIALLY GASSY with Special Conditions***
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.
2. Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.
3. The Division shall be notified immediately if any **Flammable Gas** or **Petroleum Vapor** exceeds 5% of the Lower Explosive Limit.
4. All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.

The 22-foot diameter by 2,742 feet long twin tunnel bores with cross-passages called the Diridon/Arena to Downtown San Jose Station bores located near the intersection of Santa Clara Street and South Almaden Boulevard, San Jose, Santa Clara County.

This classification shall be conspicuously posted at the place of employment.

Date October 8, 2008

(SENIOR ENGINEER)

John R. Leahy

03 77089

For Reference Only

See disclaimer on cover page.

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

C085-085-09T

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of 3103 North First Street, San Jose, California 95134
(MAILING ADDRESS)

at SILICON VALLEY RAPID TRANSIT PROJECT – CENTRAL AREA
(LOCATION)

has been classified as *** POTENTIALLY GASSY with Special Conditions***
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.
2. Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.
3. The Division shall be notified immediately if any **Flammable Gas** or **Petroleum Vapor** exceeds 5% of the Lower Explosive Limit.
4. All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.

The 60-foot wide by 51-foot high by 875 feet long tunnel project called the Diridon/Arena Station located near the intersection of Santa Clara Street and Montgomery Street, San Jose, Santa Clara County.

This classification shall be conspicuously posted at the place of employment.

Date October 8, 2008

(SENIOR ENGINEER)

John R. Leahy

03 77089

For Reference Only

See disclaimer on cover page.

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

C086-085-09T

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of 3103 North First Street, San Jose, California 95134
(MAILING ADDRESS)

at SILICON VALLEY RAPID TRANSIT PROJECT – CENTRAL AREA
(LOCATION)

has been classified as *** POTENTIALLY GASSY with Special Conditions***
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.
2. Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.
3. The Division shall be notified immediately if any **Flammable Gas** or **Petroleum Vapor** exceeds 5% of the Lower Explosive Limit.
4. All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.

The 22-foot diameter by 8,902 feet long twin tunnel bores with cross-passages called the Diridon/Arena to West Portal bores located near the intersection of Santa Clara Street and Montgomery Street, San Jose, Santa Clara County.

This classification shall be conspicuously posted at the place of employment.

Date October 8, 2008

(SENIOR ENGINEER)

John R. Leahy

03 77089

For Reference Only

See disclaimer on cover page.

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

C087-085-09T

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of 3103 North First Street, San Jose, California 95134
(MAILING ADDRESS)

at SILICON VALLEY RAPID TRANSIT PROJECT – CENTRAL AREA
(LOCATION)

has been classified as *** POTENTIALLY GASSY with Special Conditions***
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.
2. Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.
3. The Division shall be notified immediately if any **Flammable Gas** or **Petroleum Vapor** exceeds 5% of the Lower Explosive Limit.
4. All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.

The 60-foot wide by 80-foot high by 31 feet long tunnel project West Mid-tunnel Vent Shaft located near the intersection of Stockton Avenue and Schiele Avenue, San Jose, Santa Clara County.

This classification shall be conspicuously posted at the place of employment.

Date October 8, 2008

(SENIOR ENGINEER)

John R. Leahy

03 77089

For Reference Only

See disclaimer on cover page.

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

C088-085-09T

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of 3103 North First Street, San Jose, California 95134
(MAILING ADDRESS)

at SILICON VALLEY RAPID TRANSIT PROJECT – CENTRAL AREA
(LOCATION)

has been classified as *** POTENTIALLY GASSY with Special Conditions***
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.
2. Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.
3. The Division shall be notified immediately if any **Flammable Gas** or **Petroleum Vapor** exceeds 5% of the Lower Explosive Limit.
4. All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.

The 61-foot wide by 29-foot high by 200 feet long tunnel project called the West Portal Cut and Cover located near the intersection of The Alameda (Highway 82) and Newhall Street, San Jose, Santa Clara County.

This classification shall be conspicuously posted at the place of employment.

Date October 8, 2008

(SENIOR ENGINEER)

John R. Leahy

03 77089

For Reference Only

See disclaimer on cover page.

For Reference Only

See disclaimer on cover page.

Appendix C

Soil Conditioning Tests at BASF Lab
P0503-D300-TM-DE-070, Rev. A

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

TECHNICAL MEMORANDUM

P0503-D300-TM-DE-070, Rev. A

To Distribution List:

Jimmy Thompson	Alan Haldin	Alastair Biggart
Andrew Liu	Dave Young	Michael Lehnen
Ching Wu	Maria. Valdes	Doug Linares

From: **Jon Isaacson and Rory Ball**

Approved: **John Hawley, Central Area Guideway Design Manager**

Date: **December 31, 2008**

Subject: **Soil Conditioning Tests at BASF Lab**

For Reference Only

1.0 Introduction

Tunneling using an Earth Pressure Balance Machine (EPBM) typically requires the use of soil conditioning agents to adjust the behavioral properties of the soil to reduce abrasion, reduce cutterhead torque, control water, and ensure control of the spoil passing through the screw conveyor. HMMB made soil samples available for soil conditioner suppliers to prepare soil/conditioner mix designs in preparation for bidding the work. BASF is a supplier of soil conditioning agents in the U.S. who accepted the offer to receive samples. Soil collected from the SVRT alignment was sent to the BASF lab to test the effectiveness of soil conditioners on different soil types to be encountered during tunneling.

Testing by BASF was performed at no cost to the VTA. The HMMB team provided two observers to witness the testing and prepare this report. A summary of the tests is included in Table 1 and photos of the testing are located in Appendix A. Product data sheets are included in Appendix C.

2.0 Sample Collection

All of the soil samples tested were taken from geotechnical boring BG-1 (sonic borehole), drilled in March of 2008 at the west portal of the proposed SVRT tunnel alignment. Sample location and the boring log can be provided upon request and will be incorporated into reports from the next phase of subsurface investigation. In order to mimic the mixing of materials within the cutter chamber of the EPBM machines to be used, some soils samples were conglomerates of soils from different intervals along the boring. Samples representative of the three general classes of soil identified in the Soil Conditioning Section of the TBM-BOD were prepared and shipped in sealed five gallon buckets to the laboratory. Photos of the soil samples are illustrated in Figure 1.

Figure 1 – Photos of soil samples sent to BASF

Soil Type X2 was assembled from two separate layers that appeared to contain the least fines of the layers present in the boring. The aggregate Type X2 sample would be classified as sandy gravel with 4 to 6 % fines. A particle size distribution test is proposed on what remains of the pre-treated sample to confirm this.

Soil Type Y was comprised of the most plastic clay present within the boring. The vast majority (~80%) of this clay was taken from a layer of CH at a shallow depth. The remaining portion (added to increase volume) was taken from a marine clay layer (CL type material) at approximately 90 feet depth that confines the upper aquifer at the site. The samples were aggregated and mixed in the sample container. Trace amounts of fine sand were present in the CH material, and one piece of fine gravel was discovered in the mixer during testing at the lab.

Soil Type Z was comprised of samples taken from four to five layers within the boring. Equal volumes were taken from these layers and included silty sands with gravel, lean clays, and sandy silts. The aggregation of multiple soil types was conducted to achieve a general mix of various soil types. This soil type simulates typical mixed face mining along the alignment.

The total sample volume of each soil type was approximately 1/3 to 1/2 of the five gallon sample container.

For Reference Only

See disclaimer on cover page.

3.0 Soil Type X2

The first sample tested was Soil Type X2. Only one test could be performed on this material due to the limited volume. The sample was mixed without the addition of any water and returned a slump of 0 inches. In the opinion of the field staff, this did not coincide with known conditions when the sample was retrieved. To achieve a degree of saturation more commensurate with the in-situ state of the soil, assuming a porosity of 26%, 1.5 lbs of water was added to the original sample and it was re-agitated. This modified the sample moisture content to 13.15%, increasing the original degree of Saturation (S) from 63.3% to 100.6% prior to soil conditioning treatment (See Appendix B – Degree of Saturation Calculation). The re-agitated sample possessed a 7.5 inch slump; the baseline slump value for the Soil Type X2 Sample. No significant separation of water from soil solids was observed during this slump test. The order of the test and results are summarized in Table 1.

The first addition to the sample was high density limestone powder slurry at 10% of the initial sample weight before water was added, which equals 9.6% of final sample weight after water was added. This slurry was comprised of 70% limestone powder (Marblewhite 200, with a particle size in the silt range) and the remaining 30% was water. No stabilizer was added to this mix due to the short time between creating the slurry and adding it to the soil. BASF staff indicated that a higher percentage of limestone may lead to clogging of the nozzles on the TBM. Upon addition and agitation of the limestone slurry into the soil sample, a color change was immediately apparent, as well as a visual change in the consistency of the soil mix, which began to behave more uniformly as a soil slurry. After approximately three minutes of agitation, the soil was again slump tested and possessed an 8.5 inch slump. The addition of the water in the limestone slurry had affected the slump of the base soil material, but the uniformity of the mix may also have contributed to the increase in slump.

The soil was then treated by adding a 30% Foam Injection Ratio (FIR) to the mix (by volume) that consisted of water, air, and MEYCO FIX SLF 30 additive. After thorough mixing, the soil mixture was too wet to attempt a slump test, but is estimated to be 11 inches.

Next, 35ml of MEYCO FIX SLF P2 polymer was added to the soil mix to bind the water and reduce the slump. The P2 was taken from a small type II quart plastic lab bottle, and the label indicated a manufacture date of the chemical as 11-21-06. The label also stated that the chemical has a six month shelf life; this means the agent was expired. After approximately four minutes of mixing, the soil mixture was found to have an 8.5 inch slump.

An additional 15ml of P2 was added for a total of 50ml of P2 in the mix. The mix was agitated for three minutes, and yielded a 6.5 inch slump. It appeared the P2 was beginning to have an effect on the slump of the material, but the amount required seemed excessive to all involved.

An additional 15ml of P2 was again added and after 3-4 minutes of agitation, the mixture tested at 6.5 inches of slump. HMM personnel pointed out to the BASF personnel the manufacture and shelf life of the P2 product being used, and BASF personnel stopped the test while trying to locate a more recent container of P2 product.

A second container of P2 polymer was located, same type and brand. An additional 15ml of this product was then mixed with the soil sample. The manufacture date on this container was 1-25-06, making this product even older than the previously used container. In addition to the more recent P2 product, which now totaled 80ml of P2 polymer, the BASF staff added 15ml of P1 (a less potent

For Reference Only

See disclaimer on cover page.

mix) polymer was added to the mix. The sample was agitated for four minutes and yielded a 6.5 inch slump. The addition of the polymer was having a very limited effect on the sample and was likely due to the product being expired. The soil consistency at this point was very uniform, homogeneous, and slightly to moderately cohesive.

BASF decided to add another 10% by weight of the original sample of limestone slurry. In addition, 30ml of P2 polymer, and 30ml of P1 polymer were added. Agitation was performed on the soil sample for approximately four minutes after these components were added. The resulting mixture did not appear to have changed significantly in behavior or feel from the end of the previous mixing cycle. Testing of this material after mixing indicated a 7 inch slump. The addition of copious amounts of polymer with the slurry still could not counteract the additional water being introduced to the mix.

A clear polycarbonate tube (5.25 inch ID) was placed vertically and seated over a No. 8 Sieve on the floor of the lab. Next, a portion of the soil mix was poured into the tube to a height of 8.5 inches above the base of the tube, and then 15 inches of water head was surcharged above the plug of soil to test for permeability and soil seal against the container. Upon completion of all testing (2.5 hours later), no measurable leakage through the Soil Type X2 plug and No. 8 Sieve had occurred.

In the report BASF produced on soil samples tested for the North Dorchester Tunnel, the polymer is shown to work effectively on soils with low fines. It is clear from our testing that we could not produce the same effect and means something was wrong. Since the material has an advertised shelf life of six months and the age was beyond that, we assume age appears to have a considerable effect on the potency/efficacy of the polymer itself. Tracking of and adhering to age limits should be stipulated in the specifications.

4.0 Soil Type Z

The second sample to be tested was soil type Z. Only one test could be performed on this material due to the limited volume. The sample consisted of a mix of silty sands, minor gravel, and lean clays. Segregation of the pore water within the soil in the sample container occurred during transport to the laboratory, and free water was present on the soil surface. The moisture content measured was representative of the mixed sample with no free water present. The original sample possessed a slump of 1.5 inches and was considered the equivalent untreated in-situ slump value for the Soil Type Z Sample. The soil consistency after agitation was uniform and possessed good amount of cohesion. A 10% volume FIR of SLF 30 was introduced to the sample and agitated for three minutes. The slump of the material had increased to 6.25 inches.

5.0 Soil Type Y

The last soil tested was Soil Type Y, sticky clays. No dilatancy or water segregation was noted in the sampled soils prior to testing. The initial soil material was very cohesive and no slump testing was conducted during the testing. A small, measured quantity of soil was placed in a Hobart Model 11-50 mixer (similar to a domestic mixer) for agitation and testing and soil condition agents were then added.

Test Round 1

For Reference Only

See disclaimer on cover page.

Two tests were conducted on this material. For the first test, no water was added to the clay soils during the first mixing. The clay stuck to the mixing paddle stiffly, and the adhesion had fully obscured the windows through the paddle completely. A 15% FIR of SLF 30 (FER 7) was added to the sample and mixed into it thoroughly. The result was a reduction in the clogging of the mixing paddle, and a slightly visible reduction in the adhesion of the clay to the paddle. An additional 15% FIR was then added, resulting in a total of 30% FIR for the sample. After mixing was completed, the mix appeared more consistent, but was still a sticky paste and adhesion had not been significantly affected.

Next, 1% by weight of Rheosoil 211 was added to the mix and agitated. The resulting clay soil mix was still sticky, but mixer motor effort audibly reduced and the mix became more plastic in appearance. An additional 0.5% of Rheosoil 211 was added, bringing the total to 1.5%. The resulting soil mix was still sticky and the paddle remained thoroughly loaded with adhered clay soil.

Then, 5% water by weight was added, followed by an additional 2.5%, and agitated for thorough incorporation into the mix. The resulting mix exhibited less adhesion to the mixing paddle, a thinner paste, and less viscosity.

Finally, 15% FIR of SLF 30 was added, now totaling 45% FIR. The original foam matrix had been destroyed prior to this point due to the amount of previous mixing that had taken place, so the effect was more like that of only 15% FIR added. The resulting soil mix had lost its ability to bridge the gaps in the mixing paddle, becoming fluid like a thin pudding.

Test Round 2

For the second test of this material, 7.5% by weight of water was initially added to the second sample and mixed thoroughly. The resulting clay mix was still very sticky, but was slightly more plastic than the sample in the first round at this point.

Next, 1.5% of Rheosoil 211 was added (by weight) while mixing. The effect on adhesion was immediate, and was at a similar state to where the first test ended exhibiting low bridging ability of the clay across the paddle openings. Motor effort was also audibly reduced.

Then, 15% FIR of the SLF 30 foam was added to the mix, which resulted in visible fluffing (bulking). The mix was now more fluid and required even less torque from the mixer to agitate. An additional 15% FIR was then added, bringing the total to 30% by volume. The soil mix now had the appearance and viscosity of a mousse, and could be described as very fluffy (notable air entrainment) almost immediately. It required very little mixing effort. The soil mix also now had a dilatent quality when shaken, and vibrated like mousse or jello in a mold.

For Reference Only

See disclaimer on cover page.

6.0 Laboratory Equipment Used

All tests at the BASF lab were performed at atmospheric pressure with mixers that, in the opinion of the observers, likely mixed the soil more thoroughly than would take place in the TBM excavation chamber. Dosage rates determined during these tests are considered to be a general guideline and may need to be adjusted in the field under actual tunneling conditions.

The following equipment was used during testing:

1. Small Mixer for Sticky Clay Testing
    a. Manufacturer – The Hobart Company
    b. Model No. 11-50 Mixer
    c. Utilized a leaf shaped paddle mixing blade with 6 holes, axially symmetric
2. Large Mixer for Agitation of other Soils
    a. No Manufacturer Label Present
    b. Told by lab technician that it is a mixer typically used for Resins and Sand/Epoxy mixes
    c. The drum was a metallic cylinder 12-15 inches in diameter, 10 gallons in capacity
    d. It had two exterior lifting handles at 180 degree separation
    e. The base had a strip/hoop metal cage within which the cylinder was placed. The cage was angled 35-45 degrees out of vertical, and powered by an electrical motor at the base. The motor and cage were set up on a rolling dolly (see pictures). The motor may have been variable speed, but only an on-off switch was observed. It utilized 120 Volt AC power. The drum rotated at a speed of approximately 60 to 80 rpm and was agitated with a simple L-shaped fin blade affixed to a point outside the rotating drum. The lower portion of the blade, approximately 4 inches long, was parallel to the base of the bucket-like container.
3. The foam gun consisted of a spray paint pot powered by compressed air, approximately one gallon of capacity. The pot was metal with a pressure cap lid and pressure gauge. It was filled with the water and foam agent, and had a compressed air inlet line to provide driving head. The outlet line went to the foam gun, where it was fed through a separate proportioning valve before joining the air line (which had its own separate proportioning valve). The joined air and foaming compound then entered a 1 inch ID PVC line where mixing and expansion occurred prior to foam discharge. Adjustment of the proportioning valves allowed alteration of the FER.

For Reference Only

See disclaimer on cover page.

Summary Tables

Tables for Technical Memorandum on Soil Conditioning Testing

Soil Type X2							
Round	% FIR	ml	ml	min	lbs	in	Description/Result
	SLF 30	P2	P1	Time	Slurry	Slump	
1 initial	-	-	-	0		0	Permeable
2 revised	-	-	-	9		7.5	Permeable and saturated
3 slurry	-	-	-	13	3.35	8.5	Slightly cohesive, more uniform
4	30	-	-	21	3.35	-	Very liquid, possibly ~11" slump
5	30	35	-	27	3.35	8.5	No significant change, some stiffening
6	30	50	-	34	3.35	6.5	Small affect, amount may be expired P2 polymer
7	30	65	-	38	3.35	6.5	No significant change
8	30	80	15	45	3.35	6.5	Used different bottle of P2, slightly more cohesive, very uniform, homogeneous, slight to moderately cohesive
9	30	110	45	58	6.70	7.0	Polymer unable to counter affects of additional slurry

Notes:

Initial – dehydrated sample at 8.3% water content, 33.46 lbs total sample weight delivered to lab
Revised – re-hydrated sample at 13.15% water content after addition of 1.5 lbs of additional water
Slurry – high density limestone slurry comprised of 70% by weight of limestone powder and 30% water
Meyco SLF 30 Foam applied with constant FER = 7

Soil Type Y					
Round	% FIR	% by weight	%wt	Min	Description/Result
	SLF 30	Rheosoil 211	water	Time	
1a	-	-	-	0	Fully clogged mixer paddle
1b	15	-	-		Slight reduction in clogging and adhesion
1c	30	-	-	20	Better consistency, still clogging, no change in adhesion
1d	30	1.0	-		Decrease in mixer torque, more plastic
1e	30	1.5	-	26	Still sticky, still clogging
1f	30	1.5	5.0		Reduction in stiffness
1g	30	1.5	7.5	29	Less adhesion, lower viscosity, foam matrix completely destroyed
1h	45/15	1.5	7.5	33	Low adhesion, very fluid, no clogging of paddle openings on mixer
2a	-	-	7.5	3	Sticky, less viscous than last test
2b	-	1.5	7.5	5	Immediate effect on adhesion, similar to end of last test, reduction in torque
2c	15	1.5	7.5	7	Mix bulking/fluffing, mix torque reduction
2d	30	1.5	7.5	9	Further fluffing, highly reduced torque, paddle openings clear on mixer, noticeable dilatancy of soil mix, lower density

Notes:

Sample at 22.5% moisture content
Meyco SLF 30 Foam applied with constant FER = 7

Soil Type Z				
	% FIR	min	in	Description/Result
Round	SLF 30	Time	Slump	
1	-	0	1.5	Stiff, moderately cohesive mix
2	10	7	6.25	Slump increased quickly, possibly only 5% FIR could be used

Notes:

Sample at 10.22% moisture content, 41.45 lbs total sample weight
Meyco SLF 30 Foam applied with constant FER = 7

For Reference Only

See disclaimer on cover page.

For Reference Only

See disclaimer on cover page.

Attachment A - Photos

For Reference Only

See disclaimer on cover page.

For Reference Only

See disclaimer on cover page.

Soil Type X2

Round 1 (R1) – Soil as received, 0" slump

R2 – 1.5 lbs of water added, 7.5" slump

R3 – 10%* LS slurry added by wt, 8.5" slump

R4 – FIR 30% + LS slurry, no slump measured

R5 – FIR 30%+35ml P2+LS slurry, 8.5" slump

R6 – FIR 30%+50ml P2+LS slurry, 6.5" slump

R7 – FIR 30%+65ml P2+LS slurry, 6.5" slump

R8 – FIR 30%+80ml P2+15ml P1+LS slurry, 6.5" slump

R9 – FIR 30%+110ml P2+45ml P1+additional 10%* LS slurry by wt, 7" slump

***=10% by weight is based on the soil as received before the 1.5 lbs of water was added**

DRAFT
For Reference Only

See disclaimer on cover page.

Soil Type Y – Round 1

R1a – Soil as received

R1b – FIR 15%

R1c – FIR 30%

R1e – FIR 30%, 1.5% Rheosoil 211

R1g – FIR 30%, 1.5% R211, 7.5% Water

R1h – FIR 45%, 1.5% R211, 7.5% Water

Soil Type Y – Round 2

R2a – Soil as received + 7.5% Water

R2b – 7.5% Water+1.5% R211

R2c - 7.5% Water+1.5% R211+FIR 15%

R2d - 7.5% Water+1.5% R211+ FIR 30%

DRAFT – For Reference Only

See disclaimer on cover page.

Soil Type Z

R1 – Soil as received, 1.5" slump

R2 – FIR 10%, 6.25" slump

Soil Conditioning Agents

First bottle of P2 Polymer

Second bottle of P2 Polymer

Miscellaneous Laboratory Pictures

Limestone slurry being mixed

Foam gun setup

Foam gun in use

Large mixer for agitating soils

P2 polymer added by syringe

Permeability test

DRAFT
For Reference Only

See disclaimer on cover page.

For Reference Only

See disclaimer on cover page.

Attachment B – Saturated Soil Calculation

For Reference Only

See disclaimer on cover page.

For Reference Only

See disclaimer on cover page.

SILICON VALLEY RAPID TRANSIT PROJECT

P0503-D300 TUNNEL SEGMENT

HMM/Bechtel

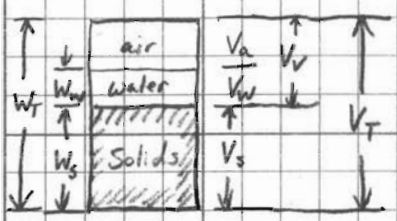
Facility/Subject:	Prepared by:	Date:	Discipline:
SVRT Soil Conditioning Tests	J. Ike Isaacson	6/23/08	Tunnel
Component:	**Checked by:**	**Date:**	
Soil Type X Lab Sample Degree of Saturation	John Schaurbusch	12/30/08	
Calculation No.: / **Rev. No.**	**Backchecked by:**	**Date:**	**Sheet No.:**
SBAC 90 / A			1

For Reference Only

Soil Sample X

$W_T = 33.46$ lbs
$\omega = 8.3\%$

sample was dried in the field and requires rehydration to simulate in-situ conditions



$W_T = 33.46 \text{ lbs} = W_W + W_S$ $\quad \omega = 0.083 = \frac{W_W}{W_S}$

$33.46 \text{ lbs} - W_W = W_S$

$$0.083 = \frac{W_W}{(33.46 \text{ lbs} - W_W)}$$

$$2.777 - 0.083 W_W = W_W$$

$$2.777 = 1.083 W_W$$

$$W_W = 2.564 \text{ lbs}$$

$W_S = 33.46 \text{ lbs} - 2.564 \text{ lbs}$

$W_S = 30.896$ lbs

Assume avg S.G. of solids = 2.7

$$V_S = \frac{30.896 \text{ lbs}}{(2.7 \times 62.4 \text{ pcf})} = 0.184 \text{ ft}^3 \text{ estimated.}$$

$$V_W = \frac{2.564 \text{ lbs}}{62.4 \text{ lbs}} = 0.041 \text{ ft}^3$$ in original sample that arrived at the lab

assume in-situ sample $n = 26\%$ $\quad n = \frac{V_V}{V_T} \quad 1 - n = \frac{V_S}{V_T}$

$$(1 - 0.26) = \frac{V_S}{V_T} \Rightarrow \frac{V_S}{(1 - 0.26)} = V_T$$

$$\frac{0.184 \text{ ft}^3}{0.74} = V_T = 0.249 \text{ ft}^3$$

degree of saturation

$$S = \frac{V_W}{V_V} = \frac{0.041 \text{ ft}^3}{0.249 \text{ ft}^3 \times 0.26} = 63.3\%$$

Original Sample $\omega = 8.3\%$ was 63.3% saturated

Added 1.5 lbs water $\quad \frac{(1.5 + 2.564)}{30.896} = \frac{W_W}{W_S} = 0.1315 = \omega \quad$ new $\omega = 13.15\%$

$$\frac{\left(\frac{1.5 + 2.564}{62.4}\right)}{(0.249 * 0.26)} = \frac{V_W}{V_V} = S = 1.006 \rightarrow S = 100.6\% \text{ prior to treatment}$$

See disclaimer on cover page.

For Reference Only

See disclaimer on cover page.

Attachment C – Product Data Sheets

For Reference Only

See disclaimer on cover page.

For Reference Only

See disclaimer on cover page.

MEYCO® FIX SLF P2

Soil-Conditioning Polymer for TBM's

Description

MEYCO FIX SLF P2 liquid polymer is used to enhance the performance of the MEYCO FIX SLF foaming products in difficult ground conditions. MEYCO FIX SLF P2 liquid polymer is used in coarse soil with low fine content or saturated soil with high water pressure.

MEYCO FIX SLF P2 liquid polymer can also be used to modify the properties of bentonite slurries for slurry TBM applications.

Applications

Recommended for use in:

- Soil conditioning in Earth Pressure Balance (EPB) shield machines
- Poorly graded and low fine ground, saturated ground and high water pressure ground in EPB shield machines
- Bentonite slurry modification in slurry shield machines

Features

- Ready-to-use liquid polymer
- Environmentally friendly

Benefits

MEYCO FIX SLF P2 liquid polymer was designed for soil conditioning with shielded TBM excavation. MEYCO FIX SLF P2 liquid polymer has excellent performance in restructuring soil and is effective in coarse, clean sands and gravels. When mixed with one of the MEYCO FIX SLF foams, MEYCO FIX SLF P2 liquid polymer provides:

- Reduced permeability and increased sealing at the face
- Even and controlled support pressure and increased face stability
- Lower inner friction and lower abrasiveness of the soil at the cutterhead through to the screw conveyor. This reduces power consumption and wear to the tools
- Increased cohesion of coarse, clean sands and gravels – smoother soil extraction
- Water soaking and swelling effect, turning wet soil into a more manageable consistency
- Improving the yield of bentonite slurries. Suitable for bentonite slurries in saline conditions

Performance Characteristics

Consumption: Typically, the consumption of MEYCO FIX SLF P2 liquid polymer is between 0.3-4% of the foam solution. In some circumstances, other dosage levels may be required. Contact your BASF Construction Chemicals sales representative for assistance.

Technical Data

Form	Viscous Liquid
Color	Light Brown
pH; 68 °F (20 °C)	8.5–10.5
Density @ 68 °F: lb/ft^3 (20 °C): (kg/m^3)	56.2–59.3 (900–950)
Viscosity; mPa•s (cps) 68 °F (20 °C)	< 500

Guidelines for Use

Application: MEYCO FIX SLF P2 liquid polymer is normally added through a separate pump into the foam solution stream before the foam generator. The mixture is then generated into the foam by the foam generator and injected into the cutterhead and working chamber. MEYCO FIX SLF P2 liquid polymer can also be added after the foam has been generated and prior to injection to the cutterhead and working chamber. MEYCO FIX SLF P2 liquid polymer can also be used alone, in which case it is injected directly to the working chamber or the bottom of the screw conveyor. The quantity of MEYCO FIX SLF P2 liquid polymer used will vary according to the soil condition.

Master Builders

For Reference Only

See disclaimer on cover page.

Storage and Handling

Storage Temperature: MEYCO FIX SLF P2 liquid polymer should be stored at temperatures between 41 °F and 95 °F (5 °C and 35 °C). Do not allow MEYCO FIX SLF P2 liquid polymer to freeze. If MEYCO FIX SLF P2 liquid polymer freezes, contact you local BASF Construction Chemicals representative prior to use.

Shelf Life: MEYCO FIX SLF P2 liquid polymer has a minimum shelf life of 6 months if stored in original tightly secured containers. Depending on storage conditions, the shelf life may be greater than stated. Please contact your BASF Construction Chemicals representative regarding suitability for use and dosage recommendations if the shelf life of MEYCO FIX SLF P2 liquid polymer has been exceeded.

Handling: MEYCO FIX SLF P2 liquid polymer contains no hazardous materials. However, it is recommended that all normal precautions be taken when handling the product such as the use of eye protection and gloves. Refer to the MSDS for MEYCO FIX SLF P2 liquid polymer for more information.

Packaging

MEYCO FIX SLF P2 liquid polymer is available in 53 gal (200 L) drums and 263 gal (1000 L) totes.

Related Documents

Material Safety Data Sheets: MEYCO FIX SLF P2 liquid polymer.

Additional Information

For additional information on MEYCO FIX SLF P2 liquid polymer, contact your BASF Construction Chemicals representative.

The Admixture Systems business of BASF Construction Chemicals is a leading provider of innovative additives for specialty concrete used in the ready mix, precast, manufactured concrete products, underground construction and paving markets throughout the NAFTA region. The Company's respected Master Builders brand products are used to improve the placing, pumping, finishing, appearance and performance characteristics of concrete.

For Reference Only

BASF Construction Chemicals, LLC
Admixture Systems
www.masterbuilders.com
United States 23700 Chagrin Boulevard, Cleveland, Ohio 44122-5544 ■ Tel: 800 628-9990 ■ Fax: 216 839-8821
Canada 1800 Clark Boulevard, Brampton, Ontario L6T 4M7 ■ Tel: 800 387-5862 ■ Fax: 905 792-0651

®Construction Research & Technology GMBH
© BASF Construction Chemicals, LLC 2007 ■ Printed in USA ■ 03/07 ■ LIT # 1025730

Master Builders

See disclaimer on cover page.

Description

Rheosoil 211 anti-clay agent is a liquid product designed to combat the problematic effects of heavy clay soils in soft ground (particularly EPB) tunneling.

Applications

Recommended for use in:

- Soft ground tunneling applications

Rheosoil® 211

Anti-Clay Agent for Use in Soft Ground Tunneling Applications

Features

- Improves and homogenizes soil consistency
- Reduces stickiness of difficult soils
- Environmentally friendly material
- Compatible with BASF's line of SLF soil conditioning foams and polymers

Benefits

- Reduces overall cutter head torque
- Effectively reduces blockages in cutter heads, working chamber, and muck transfer system
- Increases advance rate and lowers maintenance costs

Performance Characteristics

Form	Liquid
Color	Light amber, clear
Specific Gravity; @ 68°F (20°C)	1.27 – 1.30
pH @ 68°F (20°C)	7.0 – 8.0
Viscosity; cP @ 68°F (mPa·s @ 20°C)	Max. 350

Guidelines for Use

Rheosoil 211 liquid anti-clay agent can be added either to the existing foam solution or directly into the water supply for the cutter head and/or working chamber. Rheosoil 211 liquid anti-clay agent is insensitive to both temperature fluctuations and changes in water hardness.

As all TBM applications are unique, it is strongly recommended that you contact your BASF Construction Chemicals sales representative prior to using Rheosoil 211 liquid anti-clay agent in your particular application.

Dosage: The consumption rate of Rheosoil 211 liquid anti-clay agent is based on the volume of soil excavated and ranges from 0.1% to 0.5% of the excavated soil volume. Tests should be conducted to determine the optimal dosage rates for your specific needs.

For Reference Only

See disclaimer on cover page.

Storage and Handling

Storage Temperature: : Rheosoil 211 liquid anti-clay agent can be stored over a wide range of temperatures. However, it is recommended that it not be stored in temperatures below 41°F or in excess of 122°F (5°C and 50°C). Rheosoil 211 liquid agent is also not affected by freeze/thaw cycles. Should the product freeze under extreme conditions, warm to 100°F (~40°C) and mix well before using.

Shelf Life: If stored in original tightly closed containers Rheosoil 211 liquid anti-clay agent will have a shelf life of 12 months. Please consult your local BASF Construction Chemicals sales representative if you have questions regarding use of product that has been stored for longer than the recommended shelf life.

Handling: : Rheosoil 211 liquid anti-clay agent contains no hazardous substances. However, standard precautions for handling chemical products should be observed. For further information, refer to the Material Safety Data Sheet, and relevant local regulations.

The information given here is true, represents our best knowledge and is based not only on laboratory work but also on field experience. However, because of numerous factors affecting results, we offer this information without guarantee and no patent liability is assumed. For additional information or questions, please contact your local BASF Construction Chemicals sales representative.

Packaging

Rheosoil 211 liquid anti-clay agent is available in standard 55-gal (208 L) drums and 275-gal (1041 L) totes. Bulk shipments are available upon request.

Related Documents

Material Safety Data Sheets: Rheosoil 211 liquid anti-clay agent

Additional Information

For additional information on Rheosoil 211 liquid anti-clay agent, consult your BASF Construction Chemicals sales representative.

The Admixture Systems business of BASF Construction Chemicals is a leading provider of innovative additives for specialty concrete used in the ready mix, precast, manufactured concrete products, underground construction and paving markets throughout the NAFTA region. The Company's respected Master Builders brand products are used to improve the placing, pumping, finishing, appearance and performance characteristics of concrete.

For Reference Only

BASF Construction Chemicals, LLC
Admixture Systems
www.masterbuilders.com
United States 23700 Chagrin Boulevard, Cleveland, Ohio 44122-5544 ■ Tel: 800 628-9990 ■ Fax: 216 839-8821
Canada 1800 Clark Boulevard, Brampton, Ontario L6T 4M7 ■ Tel: 800 387-5862 ■ Fax: 905 792-0651

®Construction Research & Technology GMBH
© BASF Construction Chemicals, LLC 2007 ■ Printed in USA ■ 03/07 ■ LIT # 2000028

Master Builders

See disclaimer on cover page.

Description

MEYCO Fix SLF 30 liquid foaming agent is designed for use in soils containing high concentrations of clay, and as a dust suppressant inhard rock tunneling and mining applications.

MEYCO Fix SLF 30 foaming agent was developed for soil conditioning in tunneling with shielded TBM excavation.

Applications

Recommended for use in:

- Soft ground tunneling with TBMs
- Dust suppression in hard rock tunneling

MEYCO® FIX SLF 30

Soil Conditioning Foam for TBM

Features

- Liquid foaming agent

Benefits

- Improved soil behavior
- Easier mucking operation
- Environmentally friendly
- Reduced permeability and increased sealing at the face
- Even and controlled support pressure and increased face stability
- Lower inner friction and lower abrasiveness of the soil at the cutterhead through to the screw conveyor and conveyor
- Reduces power consumption; enables soil extraction and conveyance and reduces wear and tear costs
- Reduces stickiness in certain soils decreasing opportunities for blockage
- Dust suppression in hard rock tunneling and mining operations

Performance Characteristics

Technical Data

Form	Liquid
Color	Pale Yellow
pH; 68 °F (20 °C)	6.5 – 7.5
Density @ 68 °F: lb/ft^3 (20 °C): (kg/m^3)	64.6 – 65.3 (1035 – 1045)

Guidelines for Use

Dosage: The dosage range of MEYCO Fix SLF 30 liquid foaming agent is 2-6% by volume of water. MEYCO Fix SLF P2 can be added with MEYCO Fix SLF 30 foaming agent to strengthen the foam or adjust the properties of the excavated soil.

A trial evaluation should be conducted to determine the proper dosage for a given soil condition or dust suppressant application. If necessary, contact your BASF Construction Chemicals representative for assistance.

Recommendations: MEYCO Fix SLF 30 foaming agent contains no hazardous materials. However, it is recommended that all normal precautions be taken when handling the product, such as the use of eye protection and gloves. Refer to the MSDS for MEYCO Fix SLF 30 foaming agent for more information.

For Reference Only

See disclaimer on cover page.

Storage and Handling

MEYCO Fix SLF 30 foaming agent should be stored at temperatures between 41 °F and 95 °F (5 °C and 35 °C). Do not allow MEYCO Fix SLF 30 foaming agent to freeze. If MEYCO Fix SLF 30 foaming agent freezes, contact your local BASF Construction Chemicals representative prior to use.

Shelf Life: MEYCO Fix SLF 30 foaming agent has a shelf life of 12 months if stored in original tightly secured containers. Depending on storage conditions, the shelf life may be greater than stated. Please contact your BASF Construction Chemicals representative regarding suitability for use and dosage recommendations if the shelf life of MEYCO Fix SLF 30 foaming agent has been exceeded.

Packaging

MEYCO Fix SLF 30 foaming agent is available in 55 gallon (208 L) drums and 275 gallon (1041 L) totes.

Related Documents

Material Safety Data Sheet – MEYCO Fix SLF 30 foaming agent

Additional Information

For additional information on MEYCO Fix SLF 30 foaming agent, contact your BASF Construction Chemicals representative.

The Admixture Systems business of BASF Construction Chemicals is a leading provider of innovative additives for specialty concrete used in the ready mix, precast, manufactured concrete products, underground construction and paving markets throughout the NAFTA region. The Company's respected Master Builders brand products are used to improve the placing, pumping, finishing, appearance and performance characteristics of concrete.

For Reference Only

BASF Construction Chemicals, LLC
Admixture Systems
www.masterbuilders.com
United States 23700 Chagrin Boulevard, Cleveland, Ohio 44122-5544 ■ Tel: 800 628-9990 ■ Fax: 216 839-8821
Canada 1800 Clark Boulevard, Brampton, Ontario L6T 4M7 ■ Tel: 800 387-5862 ■ Fax: 905 792-0651

® Construction Research & Technology GMBH
© BASF Construction Chemicals, LLC 2007 ■ Printed in USA ■ 03/07 ■ LIT # 2000014

Master Builders

See disclaimer on cover page.

Appendix D
Description of Preliminary Ground Classification

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

Ground Classification and Assessment of the Need for Closed Face Tunneling

Ground behavior is assessed in terms of the following classification systems:

- The Tunneling (or Tunnelman's) classification system (Terzaghi, 1950, and Heuer, 1974) has been used to characterize soil conditions for tunneling purposes, and includes correlations with behavior in a tunnel heading (Table D-1). In this classification system, non-cohesive soils below the water table are "Flowing".

- The Stability Ratio method (Broms and Bennemark 1967; Peck 1969; Davis et al, 1980) provides a numerical assessment of cohesive soil stability in a tunnel heading.

- Deere's 1969 Behavioristic Classification of Various Soils is briefly discussed because it is used to assess the stability of some sands below the water table that are not addressed above.

The purpose of using these classification systems is as follows:

- Tunneling Classification System: Provides qualitative descriptions of ground behavior.

- Stability Ratio: Assess the potential for ground squeeze by comparing the free-field vertical stress to the shear strength of cohesive soil. The stability ratio can be assessed with or without face pressure.

- Behavioristic Classification: This system was used in conjunction with the Tunneling Classification System described above to assess the potential for encountering unstable soils in the tunnel heading, particularly as an aid in assessing the behavior or marginally cohesive clayey sands.

D.1.1 Tunneling Classification System

The behavior of soils is described using the Tunneling Classification System terminology from Table D-1. For successful control of surface settlement and for underground safety and stability, running and flowing ground must be either avoided or prevented.

For Reference Only

See disclaimer on cover page.

Table D-1 Tunneling Classification System (After Terzaghi, 1950 and Heuer, 1974)

Classification		Behavior	Typical Soil Type
Firm		Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.	Hard clay, cemented sand and gravel, glacial till and till-like deposits when not highly overstressed
Raveling	**Slow Raveling**	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to overstress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground).	Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays, tills and till-like soils may be slow or fast raveling depending upon degree of overstress and presence of fractures.
	Fast Raveling		
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress. Rate of squeeze depends on degree of overstress.	Ground with low frictional strength. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at excavation surface and squeezing at depth behind surface.
Running	**Cohesive Running**	Granular materials without cohesion are unstable at a slope greater than their angle of repose (-30° to 35°). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean dry granular materials. Apparent cohesion in moist sand, or weak cementation in any granular soil may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive-raveling.
	Running		
Flowing		A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter from the invert as well as the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases. Where narrow lenses occur, piping may initiate first, followed by progressive flows.	Below the water table in silt, sand or gravel without enough clay content to give significant cohesion and plasticity. May also occur in sensitive clay when such material is disturbed.
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly pre-consolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillite.

For Reference Only

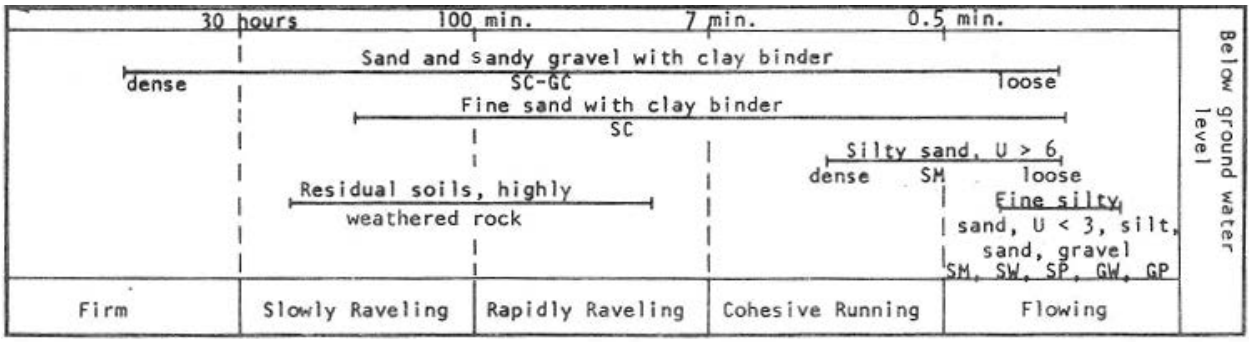
D.1.2 Deere's 1969 Behavioristic Classification of Various Soils

Deere's behavioristic classification for some sandy soils below the water table is shown in Figure D-1, with stand-up time[1] in minutes listed across the top of the chart. The chart shows that most silty sand (SM) materials are unstable below the water table. Medium dense to dense clayey sand/clayey gravel materials could be assessed in terms of stability ratio, and could have stand-up times similar to low plasticity clays, based on the stand-up times shown in Deere's chart.

Deere's classification system makes reference to the density of granular materials, which is defined by descriptive terms ranging from very loose to very dense. Descriptive terms are assigned based on standard penetration test results or equivalent obtained during the field exploration program and are not corrected for depth[2].

[1] Unit stand-up time is defined as the stand-up time of material exposed in a one foot wide strip, with stand-up time in a one foot square being 50% longer than the "unit stand-up time"
[2] For purposes of applying Deere's classification system, SPT blow counts less than 10 are considered loose and greater than 30 are considered dense

See disclaimer on cover page.



Note: 1) Air loss (in tunneling under compressed air) and water inflow is governed by the permeability, largely a function of D_{10}.
2) Behavior below ground water table under suitable air pressure is approximately the same as above ground water level.
3) Loose is here defined by N < 10 (standard penetration test), dense by N > 30.
4) Descriptive terms of materials according to the Unified Soil Classification.
5) Behavior may be somewhat better than shown above ground water level, if material is moist and fine or silty.

Figure D-1 Behavioristic Classification for Various Soils (after Deere et al, 1969)

References

Broms, B.B. and Bennermark, H. (1967). Stability of Clay in Vertical Openings. *J Soil Mech Fdns Div, ASCE*, 193, SM1, 71-94.

Deere, D.U., Merritt, A.H., Coon, R.F., 1969. “Engineering Classification of In-Situ Rock”. *Technical report No. AFWL-TR-67-144*, Kirkland Air Force Base, New Mexico.

Heuer, R. E., 1974. “Important Ground Parameters in Soft Ground Tunneling”, in Subsurface Exploration for Underground Excavation and Heavy Construction, ASCE Specialty Conference.

Terzaghi, K. (1950), "Geologic Aspects of Soft Ground Tunneling", Chapter 11 in Applied Sedimentation, ed. P. Trask, John Wiley and Sons, New York.

For Reference Only

See disclaimer on cover page.

For Reference Only

See disclaimer on cover page.

Appendix E
Description of Preliminary Face Pressure Calculations

For Reference Only

See disclaimer on cover page.

This page is left intentionally blank.

For Reference Only

See disclaimer on cover page.

Preface to Face Pressure Calculations:

E.1.1 Stability Ratio Method

The Stability Ratio is a ratio of vertical overburden free-field stress at the tunnel face to the shear strength of cohesive soils. For pressurized face tunneling the stability ratio is defined by the following equation (Davis, et al, 1980):

$$N_t = \frac{\text{[Surcharge Pressure – Face Pressure + Vertical Overburden Pressure at Springline]}}{\text{Undrained Shear Strength}}$$

According to Peck (1969), an N_t ratio of 1 is indicative of a very stable soil. Values of 2 to 3 indicate relatively stable soil that may tend to creep with time if left unsupported. Values of 4 to 5 indicate a soil that is creeping at a rate typically slow enough to allow tunneling. Finally, an N_t value of 6 or higher indicates shear failure of the soil and the onset of squeezing in unsupported soil. At this level, soil begins to extrude into openings in lining systems, or invade the tail-space between the tunnel lining and the tail shield.

Ensuring a minimum stability number of 4 by applying face pressure is one of the loading cases in the face pressure calculations. This load case is only used where CPT data indicates a stability ratio above 4. If the stability ratio of cohesive soils were the only consideration, face pressure would rarely be needed, as the stability ratios are most often less than 4, without application of any tunnel face pressure. However, fissures in the clays and the presence of water bearing sand zones at unpredictable locations within the clays reduce the influence of the stability ratio on face pressure estimates for this project. Fissuring is typically expected in approximately the top 20 feet or so of the upper cohesive unit in the soil profile, and is therefore above the bored tunnel horizon along nearly all of the alignment. As a result, the primary concern regarding face pressures is inclusions of flowing sand within zones that are otherwise cohesive.

If the extent of flowing sand zones could be determined with confidence, then zones composed of cohesive soils could be delineated with confidence. Face pressures required within the cohesive soils would be zero in most cases. Since the location of flowing sand zones within cohesive units cannot be estimated with sufficient reliability, target face pressures were developed based on the assumption that flowing sand could be encountered at any time. As a result, the potential to encounter flowing sand along the alignment generally controls the target face pressures along the tunnel rather than use of stability number estimates in cohesive materials. Also, following this logic, measurement of ambient pressures during tunneling in cohesive soil should not be considered representative of required face pressure in the event sand is encountered.

References

Davis, E.H., Gunn, M.J., Mair, R.J., and Senevirante, H.N. (1980). The stability of shallow tunnels and underground openings in cohesive material. *Geotechnique* 30, No. 4, 397-416.

For Reference Only
See disclaimer on cover page.

For Reference Only

See disclaimer on cover page.

Facility/Subject: EPBM Face Pressure Calculation		Prepared by N. Yu	Date Aug. 08, 2008	Discipline Tunnel / Civil
Component TBM GDM		Checked by R. Ball	Date Oct. 09, 2008	
Calculation No. SBAC85	Rev. No. 0			Sheet No. 1 of 12

Purpose

To estimate target operating pressures for closed-face EPBM tunnel boring during normal operations while excavating. These face pressures only apply when excavation using the TBM, and are not indicative of face pressures that may be experienced during delays or work stoppages. Soil conditions along the entire alignment are assumed to include flowing sands (the controlling soil condition for face pressure predictions), since it will not be possible to predict when flowing sands will be encountered and the TBMs need to be prepared to handle flowing sands at all times. In general, the pressure necessary to stabilize flowing sands is greater than the pressure necessary to stabilize anticipated clays.

Method

Estimating face pressures involves a four step process:

Step 1. Calculate and estimate possible upper bound and lower bound face pressures based on soil parameters, hydrogeologic conditions including minimum and maximum expected water table levels during construction, TBM parameters, and ground stability considerations.

Step 2. Establish controlling upper and lower bound values for a given location.

Step 3. Consider accuracy with which face pressures can be controlled. The face pressure control accuracy is used to estimate the tolerance in operating pressure used in the calculations.

Step 4. Based on the results of the previous steps, establish the target operating pressures as low as possible while maintaining soil stability. The operating range is the target plus and minus the tolerance. Soil stability was checked against both settlement and heave within the operating range.

Step 1 – Calculate and estimate possible upper and lower bound values

Based on the specific soil conditions, differing upper and lower bound conditions will govern. When soils are assumed to be flowing granular soils, these will present with the maximum lower bound and minimum upper bound face pressures allowed.

Face pressure boundaries based on annulus grout pressure, passive pressure, and stability ratio analysis are not analyzed because these govern in more stable, cohesive soil conditions.

For Reference Only

See disclaimer on cover page.

Estimated active pressures for cohesive soils may present higher pressures than flowing granular conditions, but the pressure, calculated from the Rankine pressure, is overly conservative. Instead, the stability of the cohesive soil face is evaluated by the stability ratio and is considerably lower than the active pressures of granular soils.

The following loading conditions are considered:

Step 1a.	Hydrostatic, including minimum and maximum water table levels anticipated during construction at tunnel depth (HMMB, 2008a), lower bound
Step 1b.	Active + Hydrostatic, including minimum and maximum water table levels anticipated during construction at tunnel depth (HMMB, 2008a), lower bound
Step 1c.	Pressure to maintain a fully charged plenum, lower bound
Step 1d.	Vertical overburden pressure, upper bound for sandy soils and fissured cohesive soils[1]

All calculations are shown on the attached spreadsheet and graph.

<u>Step 1a: Hydrostatic Pressure</u>

Hydrostatic pressure was estimated using the maximum and minimum construction water table levels at tunnel depth, shown on profile drawings in the Hydrogeology Report (HMMB, 2008a). The tunnel profile is based on the SVRT April 2008 quarterly update (Rev.G).

$p_u = \gamma_w D$, where

p_u = hydrostatic pressure for grouting
γ_w = Unit weight of water
H = Depth below water table

[1] Pressures above overburden have resulted in surface heave in EPBM tunnels, for example the LNWI Southern Sacramento River Crossing, completed in 2005 (LNWI, 2008).

For Reference Only

See disclaimer on cover page.

Step 1b: Hydrostatic and Active Earth Pressures

Active earth pressures, shown in the figure on the following page, were calculated based on the pressure formula developed as part of Rankine's Earth Pressure Theory presented by Rankine for granular materials:

$p_a = K_a \gamma_t H$, where

p_a = Active earth pressure
K_a = Active earth pressure coefficient. This parameter is calculated as described in the "Assumptions" section.
γ_t = Total unit weight of soil
H = Depth below ground surface

To estimate the active earth pressure plus hydrostatic pressure in granular soil, buoyant unit weights were used and then hydrostatic pressure was added.

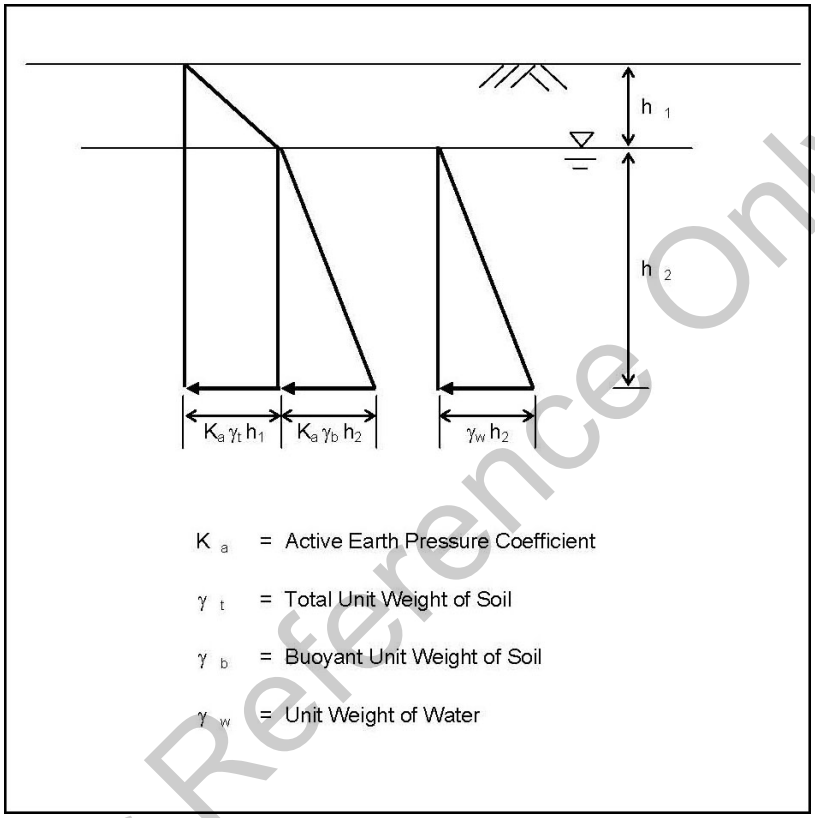
$p_{a+u} = K_a \gamma_b H + u$, where

p_{a+u} = Effective active earth pressure plus hydrostatic pressure
γ_b = Buoyant unit weight of soil
u = hydrostatic pressure
other terms as defined previously

Pressures included on the attached calculation worksheet are the pressures estimated at springline of the tunnel.

For Reference Only

See disclaimer on cover page.



Rankine Earth Pressure Diagram for Active + Hydrostatic Earth Pressure

For Reference Only

Step 1c: Pressure Needed to Maintain a Fully Charged Plenum

Calculations for pressure needed to maintain a fully charged plenum are shown at the top of the attached spreadsheet. These calculations were based on a fluid pressure caused by conditioned soil in a full plenum. A bulking factor was applied to reflect the reduction in density of excavated material in the plenum due to the application of soil conditioners. Based on construction records from the LNWI project, bulking factors as low as 1.0 are possible (LNWI, 2008).

These calculations are based on the assumption that the tunnel boring machine extrados diameter is 20 feet-8 inches.

See disclaimer on cover page.

Step 1d: Vertical Overburden Pressure

This is the upper bound pressure when granular materials are present. Exceeding this upper bound pressure with either face pressure or foam injection pressure has the potential to hydrojack (fracture) cohesionless or fissured cohesive soils with potential impact at the ground surface. Unit weights used to estimate vertical overburden pressure are provided in the "Assumptions" section.

Step 2 – Controlling Lower Bound and Upper Bound Face Pressure Values

The controlling lower bound value is the maximum lower bound value estimated at a given tunnel station. The target pressure is set as low as possible without allowing normal fluctuations in pressure within the tolerance to dip below the lower bound. The tolerance is evaluated in Step 3 below. The upper bound value and risk of heave is checked once target and tolerance are established.

In cases where the upper bound pressure is the same as the target and no tolerance is allowed, the value must be reevaluated to lower the target and decrease the tolerance. For example, at the portals where overburden is at a minimum.

Step 3 – Face Pressure Control Accuracy

Face pressure control accuracy is typically specified to the contractor. For this project, face pressure accuracy is required to be better than +/-4.35 psi, or 0.3 bar. This tolerance was added to the controlling lower bound value for the purpose of developing the recommended target face pressure. The recommended target face pressure was then used to check against the risk of heave by comparing against the controlling upper bound pressure.

Step 4 – Target Face Pressure Recommendations

Recommended target face pressures, as well as the potential upper and lower bound values, are shown on the attached calculation spreadsheet and graphs. Target face pressure values were estimated by selecting the maximum lower bound value and adding the face pressure control tolerance to this value.

For Reference Only

See disclaimer on cover page.

Assumptions

Geotechnical Parameters Used to Estimate Face Pressure

Geotechnical parameters used in the analysis are shown in the table below. Most of these parameters are from the technical memorandum entitled "Geotechnical Parameters for Design of Structures: Central Area Guideway" (HMMB, 2007b). Additional geotechnical information was obtained from the Geotechnical Data Report (HMMB, 2005d; HMMB, 2008) and the Static Geotechnical Parameters calculation (HMMB, 2007c).

While the friction angle and unit weight are parameters resulting from the analysis of geotechnical laboratory testing data, the active earth pressure coefficient K_a is a calculated parameter. The active earth pressure coefficient is calculated using the following equation developed by Rankine:

$K_a = \tan^2 (45 - \phi/2)$ where ϕ = internal friction angle of the material.

The friction angles at tunnel level for each borehole are calculated in Calculation SBAC55 – Friction Angles by Borehole (HMMB, 2008c). Friction angles for short term loadings of sands and gravels typically range from 39 to 45 degrees.

Unit weights used in these calculations are presented on the table below.

Unit Weights Used to Estimate Face Pressure

Depth Below Ground Surface	Unit Weight, γ, pcf
Ground surface to 35 feet	122 pcf
Below 35 feet	126 pcf

Source: HMMB (2007b), Section 3.0

Groundwater Level

Groundwater levels vary seasonally and with pumping and recharge in the groundwater basin. These calculations include groundwater levels provided by the Hydrogeology Report (HMMB, 2008a) labeled "Maximum Water Level for Construction (based on historic maximum)" And "Minimum Water Level for Construction". Groundwater levels should be verified as future additional piezometer readings are made. Changes in groundwater pressure

For Reference Only

See disclaimer on cover page.

estimated at tunnel depth for whatever reason, including significant artesian pressure, will influence the target pressures estimated in this calculation; however the artesian pressures encountered during the SR87 HOV project CIDH construction (HMMB, 2006a) are consistent with the water levels used in this calculation.

Tunnel Depth and Stationing

Tunnel depth and stationing was established based on the Geotechnical Plan and Profile Drawings, Rev. G released April 2008 for the quarterly update (HMMB, 2008).

TBM Parameters

TBM dimensions were obtained from Appendix C2, Tunnel Lining Geometric Design and Installation Plan of the Tunneling Methodology Report (HMMB, 2005e). This gave an external tunnel diameter of 19 feet–6 inches, and an internal diameter of 17 feet-10 inches. After inclusion of an overcut of 7-inches on radius, the excavated diameter is expected to be 20 feet – 8 inches. This diameter corresponds to the maximum overcut at the cutterhead per drawing number Z001 (HMMB, 2007a).

Calculations

Calculations are shown on the attached pages.

Results and Conclusions

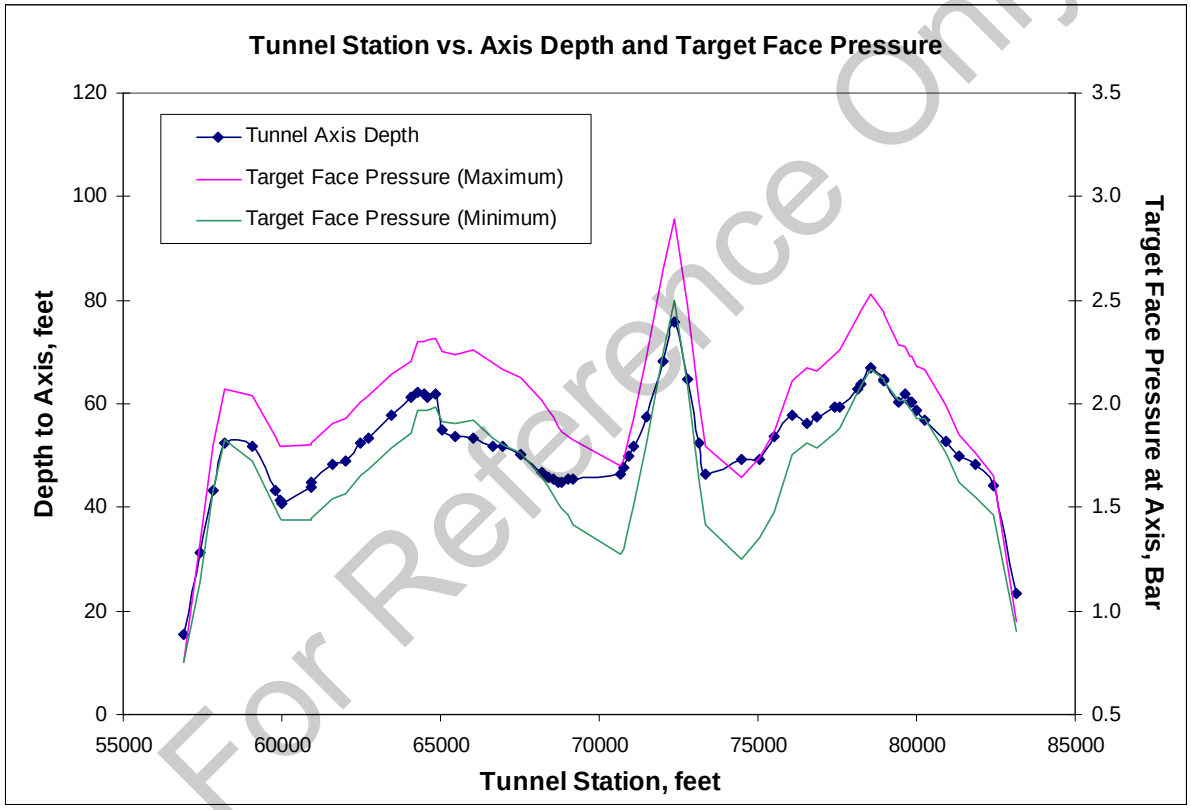
Results are summarized for each boring on the attached charts. Based on the method described above, and the calculations presented on the attached spreadsheet, minimum and maximum face pressures can be established along with target face pressures. Target pressures were generally controlled by the active earth + hydrostatic pressure; only at the portal was the target earth pressure controlled by the necessary full plenum pressure. Controlling minima are provided on the attached Table for each boring location where analyses were conducted. Target face pressures by station are shown on the chart below.

The target face pressures (primarily active earth + hydrostatic pressures) determined in this calculation are higher than those of preliminary calculation SBAC19. At many of the boreholes in the previous calculation, it is assumed to be cohesive clays in the tunnel face with minimal sands; this results in a lower target face pressure.

For Reference Only

See disclaimer on cover page.

The higher target face pressures here are recommended to avoid excessive ground settlement should sands be encountered unexpectedly. Besides at the shallower portal locations, this higher target pressures would not cause heave since overburden is sufficiently higher than the target face pressures.

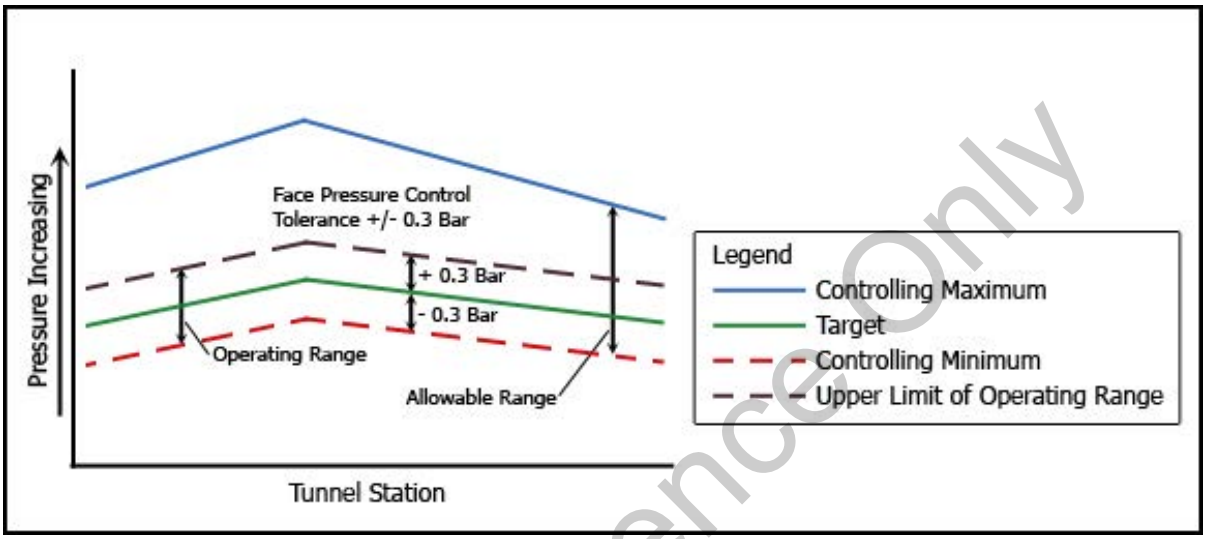


For Reference Only

Target Face Pressures by Station

It has been assumed that, through control of the TBM with soil conditioning, face pressures can be maintained within a range of plus or minus 0.3 bar (4.35 psi) of the target pressure. This tolerance was added to the controlling minimum face pressure to develop the target face pressure. This resulted in a maximum target face pressure needed to control face stability of 2.9 bar. An illustration of the relationship between the tolerance and target face pressure is presented on the following figure.

See disclaimer on cover page.



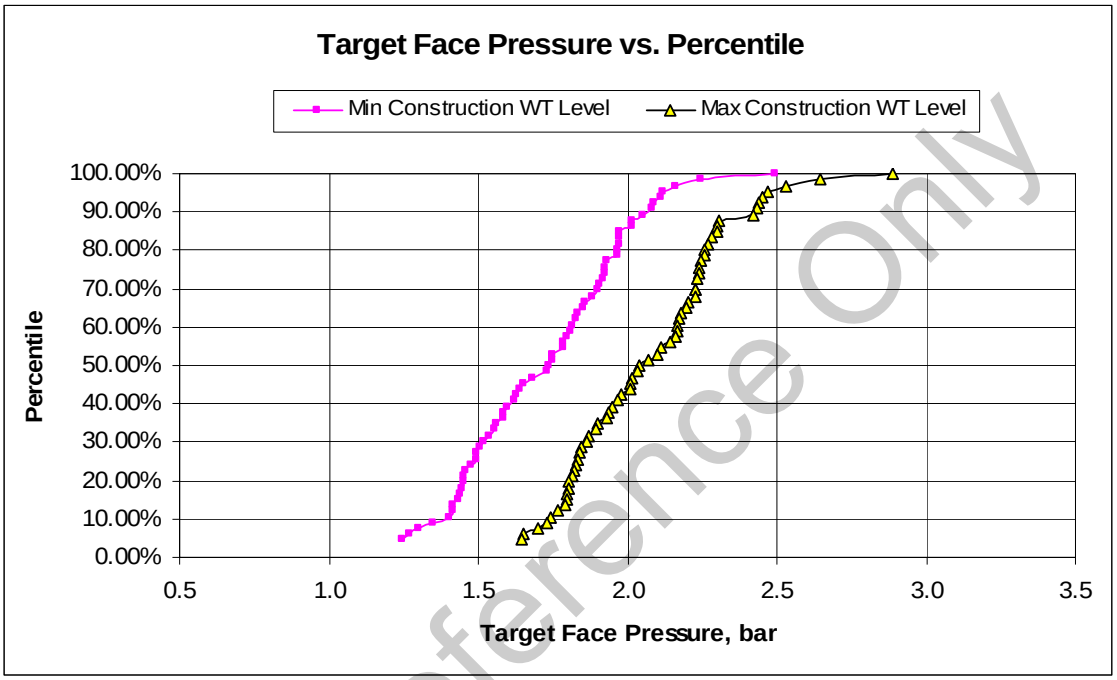
Relationship between Target Face Pressure and Tolerance

Face pressures are summarized in a percentile format for both minimum and maximum water table levels anticipated during construction, based on data points at each of the geotechnical borings evaluated in the calculations, on the figure below.

During construction, actual water table elevations will be used to establish target face pressures. Water table levels will be measured ahead of the TBM with piezometers that will be monitored throughout the excavation phase of the project. This calculation shows the expected range of anticipated target face pressures at tunnel depth resulting from the anticipated minimum and maximum water table levels during construction.

For Reference Only

See disclaimer on cover page.



Summary of Face Pressures by Percentile

From a face pressure and soil conditioning standpoint, the alignment from about Station 569+00 to 625+00 represents the most favorable ground for tunneling, and would be a foreseeable location to commence tunneling.

If the extent of flowing sand zones could be determined with confidence, then zones composed of cohesive soils could be delineated with confidence. Pressures required within the cohesive soils would be zero in most cases. Since potential flowing sand zones within cohesive units are not estimated with sufficient accuracy at this time. This is because face pressures were developed with the assumption that flowing sand could be encountered at any time. As a result, the potential to encounter flowing sand along the alignment generally controls the target face pressures.

The target pressure plus the tolerance was also checked against the total vertical overburden pressure and was found to be less than the vertical overburden pressure in all cases except at borehole BH-57 where there is a risk of heave or fracout if granular soils are encountered. It is recommended this be studied further and additional calculations performed to estimate face pressures in these areas.

For Reference Only

See disclaimer on cover page.

References

Davis, E.H., Gunn, M.J., Mair, R.J., and Senevirante, H.N. (1980). The stability of shallow tunnels and underground openings in cohesive material. *Geotechnique* 30, No. 4, 397-416.

HMM/Bechtel, 2005 (HMMB, 2005a). Tunnel Segment Geotechnical Design Memorandum for Tunnel Lining Design, Document # P0503-D300-RPT-DE-034, Rev. A. December 21, 2005.

HMM/Bechtel, 2005 (HMMB, 2005b). Technical Memorandum No. 14, "Aquifer Degassing and Artesian Conditions." August 2005.

HMM/Bechtel, 2005 (HMMB, 2005d). Geotechnical Data Report, First Draft – Internal Review Only, Document # P0503-D300-RPT-DE-021. July 1, 2005.

HMM/Bechtel, 2005 (HMMB, 2005e). Tunneling Methodology Report, Document #P0503-D300-RPT-DE-009, Rev. 0. July 5, 2005.

HMM/Bechtel, 2006 (HMMB, 2006a). Technical Memorandum, "Geotechnical Input on Coyote Creek Bridge Options," Document # 0503-D300-TM-DE-035, Rev. 0. May 24, 2006.

HMM/Bechtel, 2007 (HMMB, 2007a). TBM Technical Reference Report, Document # P0503-D300-RPT-DE-018, Rev. A, "TBM Tail Seal and Grout Annulus Study," Sheet Number Z001. June 12, 2007.

HMM/Bechtel, 2008 (HMMB, 2007b). Technical Memorandum, "Geotechnical Parameters for Design of Structures, Central Area Guideway, Tunnel Segment, SVRT Project," Document # 0503-D300-TM-DE-033, Rev. 1. July 12, 2007.

HMM/Bechtel (2007c). Calculation SOOG04 – General / Static Geotechnical Parameters. December 03, 2007.

HMM/Bechtel, 2008 (HMMB, 2008), "Silicon Valley Rapid Transit Project, Tunnel Segment Geotechnical Data Report, Phase 2, Rev 1 65% Engineering Design Investigation, P0503-D300-RPT-GEO-004, in progress.

HMM/Bechtel, 2008 (HMMB, 2008a). Hydrogeology Report. P0503-D300-RPT-DE-020, Rev. 1.

HMM/Bechtel, 2008 (HMMB, 2008c). Calculation SBAC55. Friction Angle by Borehole. P0503-D300-RPT-DE-020. May 6, 2008.

HMM/Bechtel, 2008 (HMMB, 2008e). Geotechnical Plan and Profile, Quarterly Update, Rev. G. April 2008.

For Reference Only

See disclaimer on cover page.

Lower Northwest Interceptor (LNWI), 2008. Sacramento River Crossing Tunnels Construction Report, Rev. 1.

Sowers, G. F. (1979). Introductory Soil Mechanics and Foundations: Geotechnical Engineering, 4th ed., McMillan, NY.

Attachments

1. Sample Calculation
2. Spreadsheet - Face Pressures at Maximum Water Levels During Construction
3. Spreadsheet - Face Pressures at Minimum Water Levels During Construction
4. Charts - Face Pressures by Borehole (Maximum Water Levels)
5. Charts - Face Pressures by Borehole (Minimum Water Levels)

For Reference Only

See disclaimer on cover page.

Development of Target TBM Face Pressures
Short Term Loading (Maximum Water Level)

INPUT DATA														
Machine Diameter		20.7	feet					**Lower Bound, pressure for**						
Face Pressure Control								**charged plenum at SL**						
Accuracy (Tolerance)		4.35	psi	0.30	bar			psf	bar					
Bulking Factor		1.0						1261	0.6	the minimum charged plenum pressure used in the analyses below				
Unit Weight														
from sfc to 35 ft depth		122	pcf											
below 35 ft depth		126	pcf						**Upper**		**Lower**	**Lower**	**Target**	**must be >0**
Study Section	**Borehole Number**	**Station (ft)**	**Tunnel Crown Depth (fbgs)**	**Tunnel Invert Depth (fbgs)**	**Tunnel Springline Depth (fbgs)**	**Water Surface Depth for Construction (fbgs)**	**Friction Angle**	**Unit Weight at Springline pcf**	**Vertical Overburden Pressure at Springline bar**	**Ka**	**Hydrostatic Pressure at Springline bar**	**Active + Hydrostatic at Springline bar**	**Target Face Pressure at Springline bar**	**Difference Between Operating Range and Heave Pressure [1] bar**
1	BH-57	569+31	6.0	26.7	16.3	4	41	122	1.0	0.21	0.4	0.5	0.8	-0.1
1	BH-01	574+09	21.0	41.7	31.3	4.1	41	122	1.8	0.21	0.8	1.0	1.3	0.2
1	BH-02	578+10	33.0	53.7	43.3	2.9	41	126	2.5	0.21	1.2	1.5	1.8	0.4
1	BH-03	581+83	42.0	62.7	52.3	5.4	41	126	3.1	0.21	1.4	1.8	2.1	0.7
1	BH-04	590+53	41.5	62.2	51.8	6	41	126	3.1	0.21	1.4	1.7	2.0	0.7
1	BH-05	598+17	33.0	53.7	43.3	1.3	41	126	2.5	0.21	1.3	1.5	1.8	0.4
1	BH-06	599+60	31.0	51.7	41.3	0	41	126	2.4	0.21	1.2	1.5	1.8	0.3
1	BH-83	599+83	30.5	51.2	40.8	-0.5	41	126	2.4	0.21	1.2	1.5	1.8	0.3
3	BH-86	609+08	33.5	54.2	43.8	2.5	43	126	2.6	0.19	1.2	1.5	1.8	0.5
3	BH-07	609+40	34.5	55.2	44.8	3.5	43	126	2.6	0.19	1.2	1.5	1.8	0.5
3	BH-08	615+76	38.0	58.7	48.3	5	43	126	2.9	0.19	1.3	1.6	1.9	0.6
3	BH-09	619+91	40.0	60.7	50.3	6	43	126	3.0	0.19	1.3	1.6	1.9	0.7
3	BH-10	624+90	42.0	62.7	52.3	6.4	43	126	3.1	0.19	1.4	1.7	2.0	0.8
3	BH-11	627+53	43.0	63.7	53.3	6.6	43	126	3.2	0.19	1.4	1.7	2.0	0.8
3	BH-12	634+65	47.5	68.2	57.8	9.1	43	126	3.4	0.19	1.5	1.8	2.1	1.0
3	BH-13	640+82	51.0	71.7	61.3	11.6	43	126	3.6	0.19	1.5	1.9	2.2	1.1
3	BH-14	642+55	52.0	72.7	62.3	11.7	40	126	3.7	0.22	1.5	2.0	2.3	1.1
3	BH-88	645+03	51.5	72.2	61.8	10.9	40	126	3.7	0.22	1.5	2.0	2.3	1.1
3	BH-15	645+73	51.0	71.7	61.3	10	40	126	3.6	0.22	1.5	2.0	2.3	1.0
3	BH-87	648+42	45.5	66.2	55.8	4.2	40	126	3.3	0.22	1.5	1.9	2.2	0.8
3	BH-16	650+43	44.5	65.2	54.8	2.3	40	126	3.2	0.22	1.6	1.9	2.2	0.7
3	BH-17	654+53	43.5	64.2	53.8	0.6	41	126	3.2	0.21	1.6	1.9	2.2	0.6
3	BH-18	660+09	43.0	63.7	53.3	-1.1	41	126	3.2	0.21	1.6	2.0	2.3	0.6
3	BH-19	666+26	41.5	62.2	51.8	-2	43	126	3.1	0.19	1.6	1.9	2.2	0.6
3	BH-20	669+79	41.5	62.2	51.8	-0.6	43	126	3.1	0.19	1.6	1.9	2.2	0.6
3	BH-21	675+49	39.0	59.7	49.3	-0.4	40	126	2.9	0.22	1.5	1.8	2.1	0.5
3	BH-50	681+71	36.5	57.2	46.8	-0.1	40	126	2.8	0.22	1.4	1.7	2.0	0.4
3	BH-52	684+09	35.5	56.2	45.8	0.2	40	126	2.7	0.22	1.4	1.7	2.0	0.4
3	BH-53	685+42	35.0	55.7	45.3	0.9	40	126	2.7	0.22	1.3	1.6	1.9	0.4
3	BH-54	687+16	34.5	55.2	44.8	1.9	40	126	2.6	0.22	1.3	1.6	1.9	0.4
3	BH-55	688+35	34.5	55.2	44.8	3	40	126	2.6	0.22	1.3	1.6	1.9	0.5
3	BH-23	690+02	33.9	54.6	44.2	3.9	40	126	2.6	0.22	1.2	1.5	1.8	0.5
3	BH-64	691+93	35.0	55.7	45.3	5.5	40	126	2.7	0.22	1.2	1.5	1.8	0.5
5	BH-70	706+78	36.0	56.7	46.3	11	42	126	2.7	0.20	1.1	1.4	1.7	0.7
5	BH-71	707+62	37.5	58.2	47.8	12	42	126	2.8	0.20	1.1	1.4	1.7	0.8
5	BH-72	709+39	39.5	60.2	49.8	10.8	42	126	2.9	0.20	1.2	1.5	1.8	0.8
5	BH-26	710+66	41.5	62.2	51.8	9.6	42	126	3.1	0.20	1.3	1.6	1.9	0.8
5	BH-27	715+00	47.0	67.7	57.3	5.6	42	126	3.4	0.20	1.5	1.9	2.2	0.9
5	BH-28	720+22	58.0	78.7	68.3	4.5	42	126	4.1	0.20	1.9	2.3	2.6	1.1
5	BH-29	723+88	65.5	86.2	75.8	5.6	42	126	4.5	0.20	2.1	2.6	2.9	1.3
5	BH-30	728+01	54.5	75.2	64.8	6.6	42	126	3.8	0.20	1.7	2.2	2.5	1.1
5	BH-31	731+54	42.0	62.7	52.3	7.2	42	126	3.1	0.20	1.3	1.7	2.0	0.8
5	BH-32	733+30	36.0	56.7	46.3	7	42	126	2.7	0.20	1.2	1.5	1.8	0.6
7	BH-34	744+65	39.0	59.7	49.3	15.7	45	126	2.9	0.17	1.0	1.3	1.6	1.0
7	BH-35	750+48	39.0	59.7	49.3	11.8	45	126	2.9	0.17	1.1	1.4	1.7	0.9
7	BH-36	755+33	43.5	64.2	53.8	13.2	45	126	3.2	0.17	1.2	1.6	1.9	1.0
7	BH-37	760+59	46.4	67.1	56.7	12.6	39	126	3.4	0.23	1.3	1.8	2.1	1.0
7	BH-38	765+23	46.0	66.7	56.3	9.1	39	126	3.3	0.23	1.4	1.9	2.2	0.9
7	BH-39	768+77	47.0	67.7	57.3	7.4	43	126	3.4	0.19	1.5	1.9	2.2	0.9
7	BH-95	774+14	49.0	69.7	59.3	5.9	45	126	3.5	0.17	1.6	1.9	2.2	1.0
7	BH-40	775+76	49.0	69.7	59.3	5.1	45	126	3.5	0.17	1.6	2.0	2.3	1.0
7	BH-41	781+35	52.5	73.2	62.8	4.6	44	126	3.7	0.18	1.7	2.1	2.4	1.0
7	BH-79	782+50	53.5	74.2	63.8	4.7	44	126	3.8	0.18	1.8	2.2	2.5	1.0
7	BH-42	785+37	56.5	77.2	66.8	5.9	44	126	4.0	0.18	1.8	2.2	2.5	1.1
7	BH-81	789+62	54.5	75.2	64.8	7.1	43	126	3.8	0.19	1.7	2.1	2.4	1.1
7	BH-43	789+71	54.0	74.7	64.3	6.7	43	126	3.8	0.19	1.7	2.1	2.4	1.1
7	BH-80	794+39	50.0	70.7	60.3	7	43	126	3.6	0.19	1.6	2.0	2.3	1.0
7	BH-102	796+49	51.5	72.2	61.8	9.8	43	126	3.7	0.19	1.6	2.0	2.3	1.1
7	BH-103	798+17	50.0	70.7	60.3	9.4	43	126	3.6	0.19	1.5	1.9	2.2	1.0
7	BH-44	798+27	50.0	70.7	60.3	9.4	43	126	3.6	0.19	1.5	1.9	2.2	1.0
7	BH-106	800+21	48.5	69.2	58.8	9.2	43	126	3.5	0.19	1.5	1.9	2.2	1.0
7	BH-45	802+44	46.5	67.2	56.8	9	40	126	3.4	0.22	1.4	1.9	2.2	0.9
7	BH-46	809+36	41.4	62.1	51.7	8.4	41	126	3.1	0.21	1.3	1.7	2.0	0.8
7	BH-47	813+52	39.5	60.2	49.8	9.1	44	126	2.9	0.18	1.2	1.5	1.8	0.8
7	BH-48	818+34	38.0	58.7	48.3	10.3	44	126	2.9	0.18	1.1	1.5	1.8	0.8
7	BH-49	824+28	34.0	54.7	44.3	9.1	44	126	2.6	0.18	1.1	1.4	1.7	0.7
7	BH-78	831+41	13.0	33.7	23.3	7	44	122	1.4	0.18	0.5	0.6	0.9	0.1

Notes:

1 This column is used to check to see that there is an adequate factor of safety against heave. Heave pressure conservatively assumed to be vertical overburden total pressure.

For Reference Only

See disclaimer on cover page.

Development of Target TBM Face Pressures
Short Term Loading (Minimum Water Level)

INPUT DATA														
Machine Diameter		20.7	feet					**Lower Bound, pressure for**						
Face Pressure Control								**charged plenum at SL**						
Accuracy (Tolerance)		4.35	psi	0.30	bar			psf	bar					
Bulking Factor		1.0						1261	0.6	the minimum charged plenum pressure used in the analyses below				
Unit Weight														
from sfc to 35 ft depth		122	pcf											
below 35 ft depth		126	pcf						**Upper**		**Lower**	**Lower**	**Target**	**must be >0**
Study Section	**Borehole Number**	**Station (ft)**	**Tunnel Crown Depth (fbgs)**	**Tunnel Invert Depth (fbgs)**	**Tunnel Springline Depth (fbgs)**	**Water Surface Depth for Construction (fbgs)**	**Friction Angle**	**Unit Weight at Springline pcf**	**Vertical Overburden Pressure at Springline bar**	**Ka**	**Hydrostatic Pressure at Springline bar**	**Active + Hydrostatic at Springline bar**	**Target Face Pressure at Springline bar**	**Difference Between Operating Range and Heave Pressure [1] bar**
1	BH-57	569+31	6.0	26.7	16.3	11	41	122	1.0	0.21	0.2	0.3	0.8	-0.1
1	BH-01	574+09	21.0	41.7	31.3	11.9	41	122	1.8	0.21	0.6	0.8	1.1	0.4
1	BH-02	578+10	33.0	53.7	43.3	12.1	41	126	2.5	0.21	0.9	1.3	1.6	0.7
1	BH-03	581+83	42.0	62.7	52.3	15.6	41	126	3.1	0.21	1.1	1.5	1.8	1.0
1	BH-04	590+53	41.5	62.2	51.8	19	41	126	3.1	0.21	1.0	1.4	1.7	1.0
1	BH-05	598+17	33.0	53.7	43.3	15.9	41	126	2.5	0.21	0.8	1.2	1.5	0.8
1	BH-06	599+60	31.0	51.7	41.3	14.9	41	126	2.4	0.21	0.8	1.1	1.4	0.7
1	BH-83	599+83	30.5	51.2	40.8	14.5	41	126	2.4	0.21	0.8	1.1	1.4	0.7
3	BH-86	609+08	33.5	54.2	43.8	17.5	43	126	2.6	0.19	0.8	1.1	1.4	0.8
3	BH-07	609+40	34.5	55.2	44.8	18.5	43	126	2.6	0.19	0.8	1.2	1.5	0.9
3	BH-08	615+76	38.0	58.7	48.3	20	43	126	2.9	0.19	0.8	1.2	1.5	1.0
3	BH-09	619+91	40.0	60.7	50.3	21	43	126	3.0	0.19	0.9	1.3	1.6	1.1
3	BH-10	624+90	42.0	62.7	52.3	21.3	43	126	3.1	0.19	0.9	1.3	1.6	1.1
3	BH-11	627+53	43.0	63.7	53.3	21.4	43	126	3.2	0.19	1.0	1.4	1.7	1.2
3	BH-12	634+65	47.5	68.2	57.8	23.7	43	126	3.4	0.19	1.0	1.5	1.8	1.3
3	BH-13	640+82	51.0	71.7	61.3	26.1	43	126	3.6	0.19	1.1	1.6	1.9	1.5
3	BH-14	642+55	52.0	72.7	62.3	26.1	40	126	3.7	0.22	1.1	1.7	2.0	1.4
3	BH-88	645+03	51.5	72.2	61.8	25.3	40	126	3.7	0.22	1.1	1.7	2.0	1.4
3	BH-15	645+73	51.0	71.7	61.3	24.3	40	126	3.6	0.22	1.1	1.7	2.0	1.4
3	BH-87	648+42	45.5	66.2	55.8	18.4	40	126	3.3	0.22	1.1	1.6	1.9	1.1
3	BH-16	650+43	44.5	65.2	54.8	16.5	40	126	3.2	0.22	1.1	1.6	1.9	1.0
3	BH-17	654+53	43.5	64.2	53.8	14.7	41	126	3.2	0.21	1.2	1.6	1.9	1.0
3	BH-18	660+09	43.0	63.7	53.3	13	41	126	3.2	0.21	1.2	1.6	1.9	0.9
3	BH-19	666+26	41.5	62.2	51.8	13	43	126	3.1	0.19	1.2	1.5	1.8	0.9
3	BH-20	669+79	41.5	62.2	51.8	14.6	43	126	3.1	0.19	1.1	1.5	1.8	1.0
3	BH-21	675+49	39.0	59.7	49.3	15.2	40	126	2.9	0.22	1.0	1.4	1.7	0.9
3	BH-50	681+71	36.5	57.2	46.8	15.8	40	126	2.8	0.22	0.9	1.3	1.6	0.8
3	BH-52	684+09	35.5	56.2	45.8	16.2	40	126	2.7	0.22	0.9	1.3	1.6	0.8
3	BH-53	685+42	35.0	55.7	45.3	16.9	40	126	2.7	0.22	0.9	1.3	1.6	0.8
3	BH-54	687+16	34.5	55.2	44.8	17.9	40	126	2.6	0.22	0.8	1.2	1.5	0.8
3	BH-55	688+35	34.5	55.2	44.8	19	40	126	2.6	0.22	0.8	1.2	1.5	0.8
3	BH-23	690+02	33.9	54.6	44.2	19.9	40	126	2.6	0.22	0.7	1.1	1.4	0.9
3	BH-64	691+93	35.0	55.7	45.3	23.2	40	126	2.7	0.22	0.7	1.1	1.4	1.0
5	BH-70	706+78	36.0	56.7	46.3	29	42	126	2.7	0.20	0.5	1.0	1.3	1.2
5	BH-71	707+62	37.5	58.2	47.8	30	42	126	2.8	0.20	0.5	1.0	1.3	1.2
5	BH-72	709+39	39.5	60.2	49.8	28.6	42	126	2.9	0.20	0.6	1.1	1.4	1.2
5	BH-26	710+66	41.5	62.2	51.8	27.3	42	126	3.1	0.20	0.7	1.2	1.5	1.3
5	BH-27	715+00	47.0	67.7	57.3	22.9	42	126	3.4	0.20	1.0	1.5	1.8	1.3
5	BH-28	720+22	58.0	78.7	68.3	21.3	42	126	4.1	0.20	1.4	1.9	2.2	1.5
5	BH-29	723+88	65.5	86.2	75.8	22.1	42	126	4.5	0.20	1.6	2.2	2.5	1.7
5	BH-30	728+01	54.5	75.2	64.8	22.7	42	126	3.8	0.20	1.3	1.8	2.1	1.5
5	BH-31	731+54	42.0	62.7	52.3	23	42	126	3.1	0.20	0.9	1.3	1.6	1.2
5	BH-32	733+30	36.0	56.7	46.3	23	42	126	2.7	0.20	0.7	1.1	1.4	1.0
7	BH-34	744+65	39.0	59.7	49.3	31.7	45	126	2.9	0.17	0.5	0.9	1.2	1.4
7	BH-35	750+48	39.0	59.7	49.3	27.6	45	126	2.9	0.17	0.6	1.0	1.3	1.3
7	BH-36	755+33	43.5	64.2	53.8	28.9	45	126	3.2	0.17	0.7	1.2	1.5	1.4
7	BH-37	760+59	46.4	67.1	56.7	28.2	39	126	3.4	0.23	0.9	1.4	1.7	1.3
7	BH-38	765+23	46.0	66.7	56.3	24.6	39	126	3.3	0.23	0.9	1.5	1.8	1.2
7	BH-39	768+77	47.0	67.7	57.3	22.8	43	126	3.4	0.19	1.0	1.5	1.8	1.3
7	BH-95	774+14	49.0	69.7	59.3	21.2	45	126	3.5	0.17	1.1	1.6	1.9	1.4
7	BH-40	775+76	49.0	69.7	59.3	20.3	45	126	3.5	0.17	1.2	1.6	1.9	1.3
7	BH-41	781+35	52.5	73.2	62.8	19.7	44	126	3.7	0.18	1.3	1.8	2.1	1.4
7	BH-79	782+50	53.5	74.2	63.8	19.8	44	126	3.8	0.18	1.3	1.8	2.1	1.4
7	BH-42	785+37	56.5	77.2	66.8	21	44	126	4.0	0.18	1.4	1.9	2.2	1.5
7	BH-81	789+62	54.5	75.2	64.8	20.4	43	126	3.8	0.19	1.3	1.8	2.1	1.4
7	BH-43	789+71	54.0	74.7	64.3	19.9	43	126	3.8	0.19	1.3	1.8	2.1	1.4
7	BH-80	794+39	50.0	70.7	60.3	18	43	126	3.6	0.19	1.3	1.7	2.0	1.3
7	BH-102	796+49	51.5	72.2	61.8	20.6	43	126	3.7	0.19	1.2	1.7	2.0	1.4
7	BH-103	798+17	50.0	70.7	60.3	20	43	126	3.6	0.19	1.2	1.7	2.0	1.3
7	BH-44	798+27	50.0	70.7	60.3	20.1	43	126	3.6	0.19	1.2	1.7	2.0	1.3
7	BH-106	800+21	48.5	69.2	58.8	19.6	43	126	3.5	0.19	1.2	1.6	1.9	1.3
7	BH-45	802+44	46.5	67.2	56.8	19.2	40	126	3.4	0.22	1.1	1.6	1.9	1.1
7	BH-46	809+36	41.4	62.1	51.7	18.1	41	126	3.1	0.21	1.0	1.5	1.8	1.0
7	BH-47	813+52	39.5	60.2	49.8	18.3	44	126	2.9	0.18	0.9	1.3	1.6	1.0
7	BH-48	818+34	38.0	58.7	48.3	19	44	126	2.9	0.18	0.9	1.3	1.6	1.0
7	BH-49	824+28	34.0	54.7	44.3	17	44	126	2.6	0.18	0.8	1.2	1.5	0.8
7	BH-78	831+41	13.0	33.7	23.3	14	44	122	1.4	0.18	0.3	0.5	0.9	0.2

Notes:

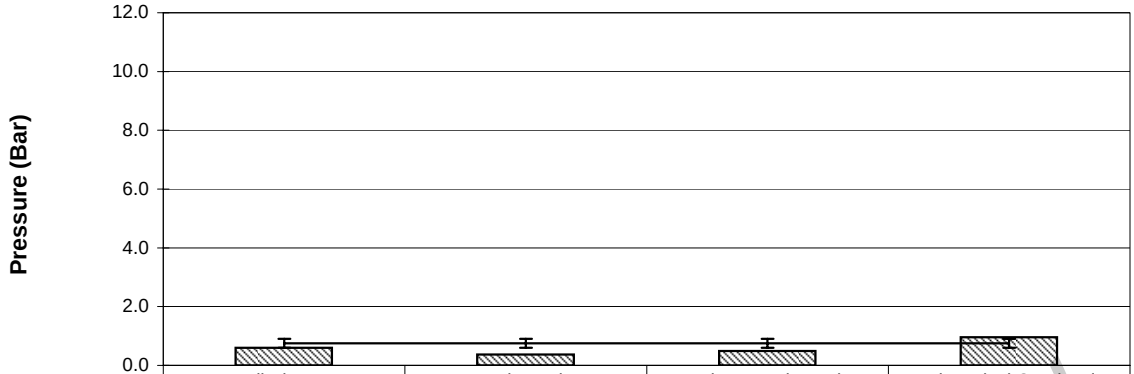
1 This column is used to check to see that there is an adequate factor of safety against heave. Heave pressure conservatively assumed to be vertical overburden total pressure.

For Reference Only

See disclaimer on cover page.

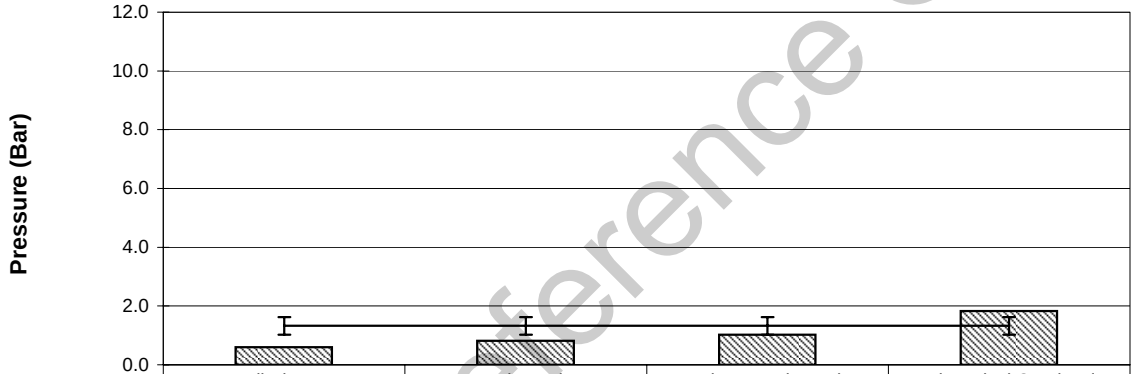
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-57 (Sta. 569+31)



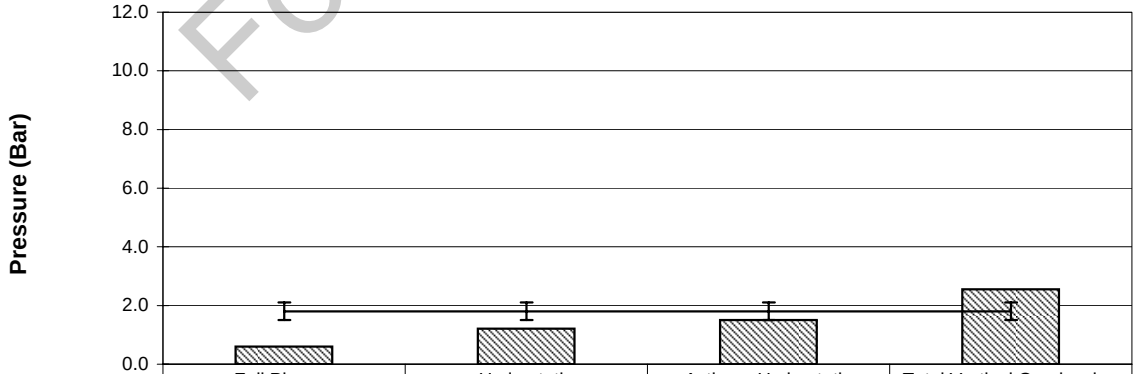
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.4	0.5	1.0
Target Pressure, bar	0.8	0.8	0.8	0.8

BH-01 (Sta. 574+09)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.8	1.0	1.8
Target Pressure, bar	1.3	1.3	1.3	1.3

BH-02 (Sta. 578+10)



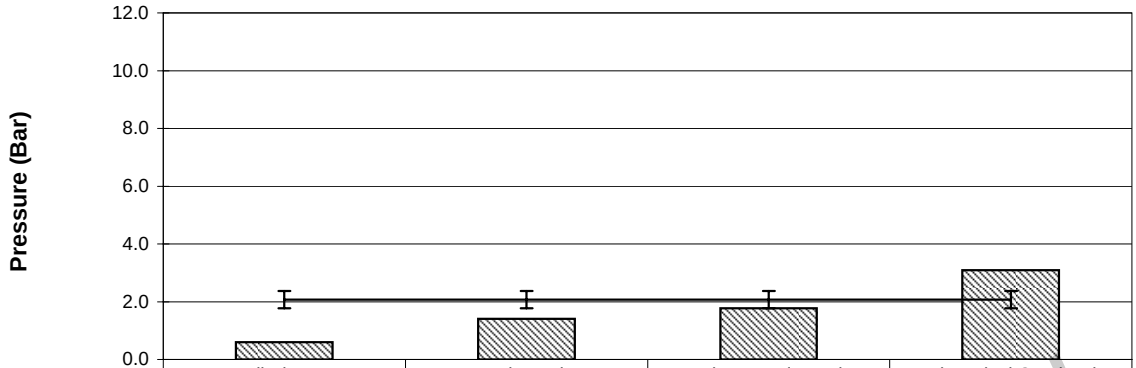
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.5	2.5
Target Pressure, bar	1.8	1.8	1.8	1.8

For Reference Only

See disclaimer on cover page.

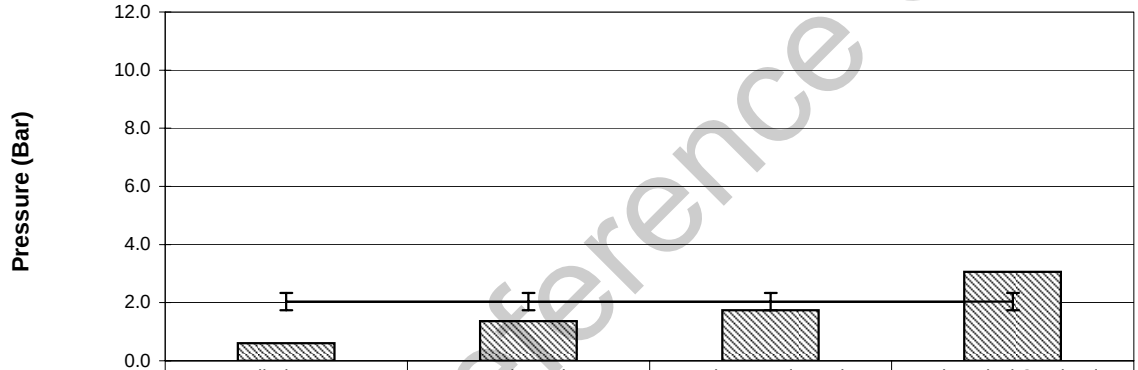
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-03 (Sta. 581+83)



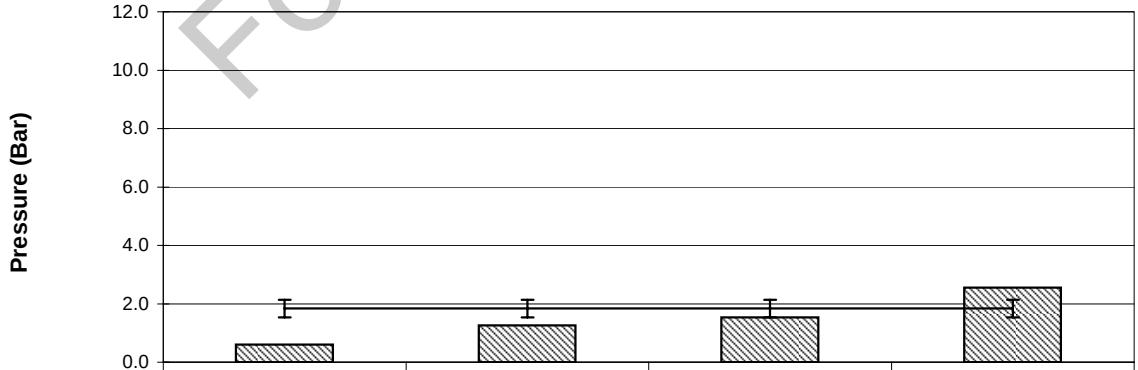
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.4	1.8	3.1
Target Pressure, bar	2.1	2.1	2.1	2.1

BH-04 (Sta. 590+53)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.4	1.7	3.1
Target Pressure, bar	2.0	2.0	2.0	2.0

BH-05 (Sta. 598+17)



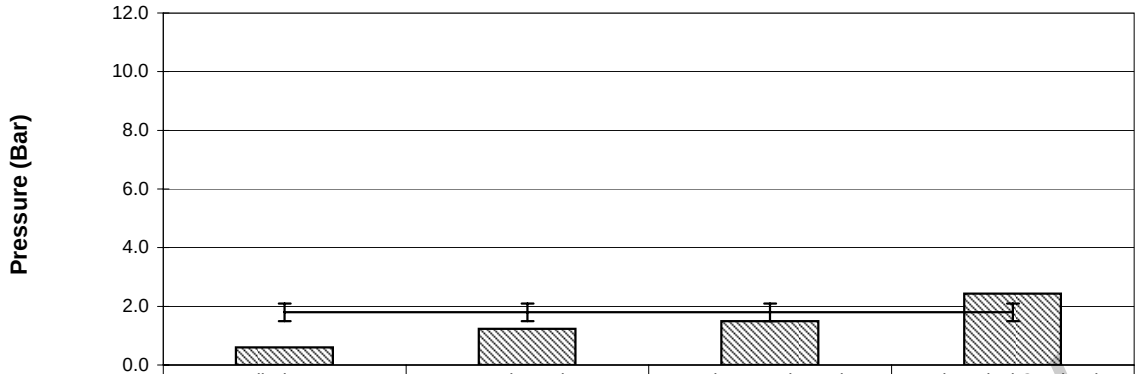
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.5	2.5
Target Pressure, bar	1.8	1.8	1.8	1.8

For Reference Only

See disclaimer on cover page.

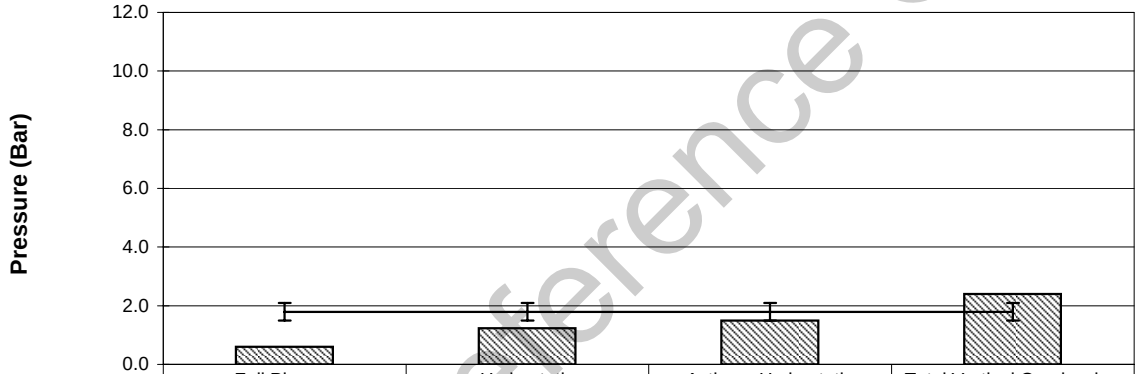
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-06 (Sta. 599+60)



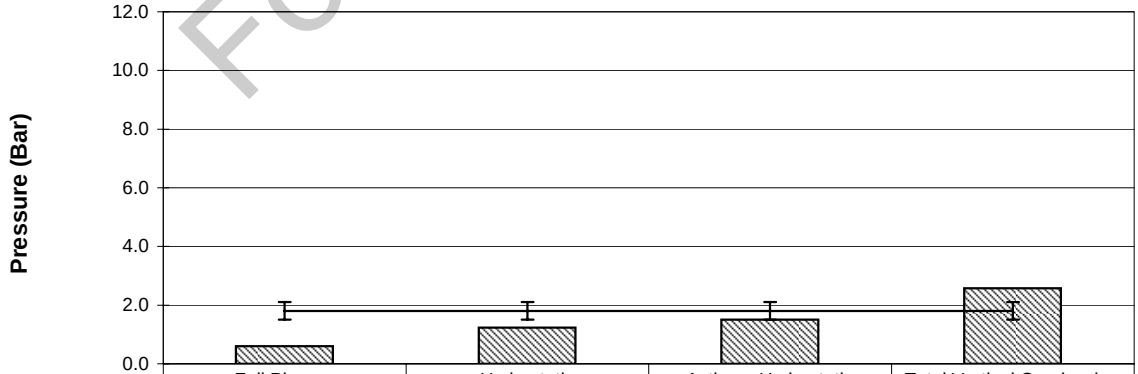
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.5	2.4
Target Pressure, bar	1.8	1.8	1.8	1.8

BH-83 (Sta. 599+83)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.5	2.4
Target Pressure, bar	1.8	1.8	1.8	1.8

BH-86 (Sta. 609+08)



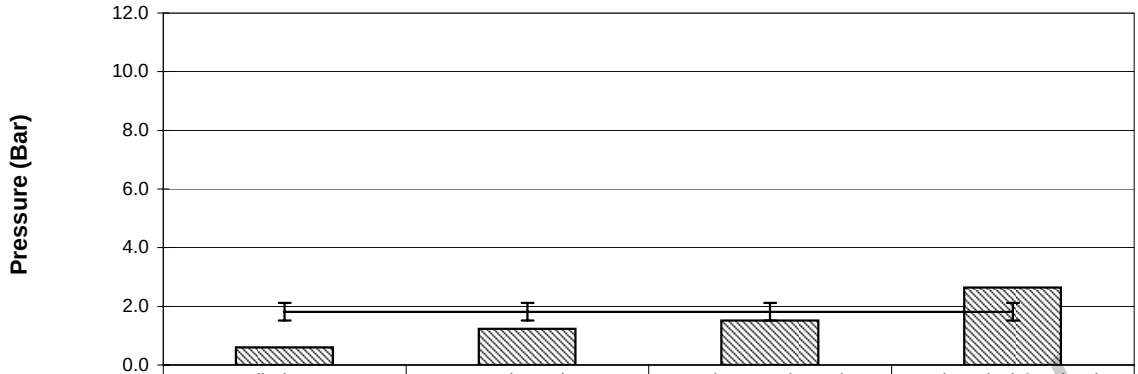
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.5	2.6
Target Pressure, bar	1.8	1.8	1.8	1.8

For Reference Only

See disclaimer on cover page.

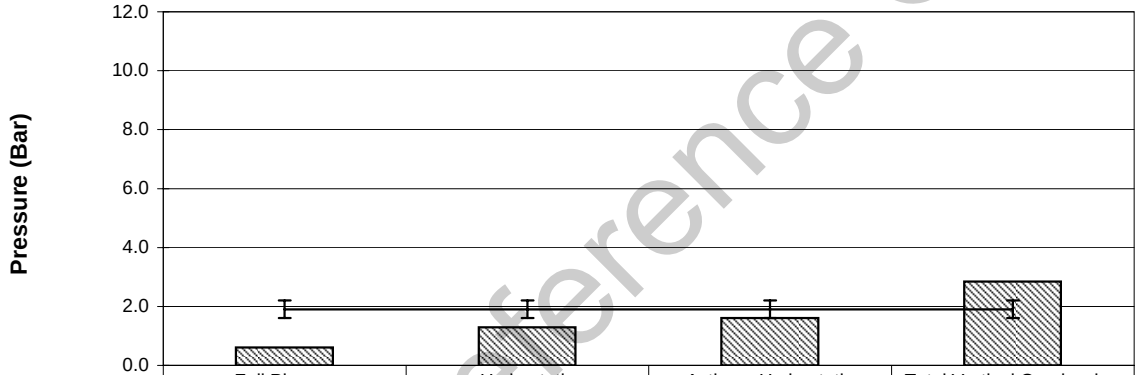
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-07 (Sta. 609+41)



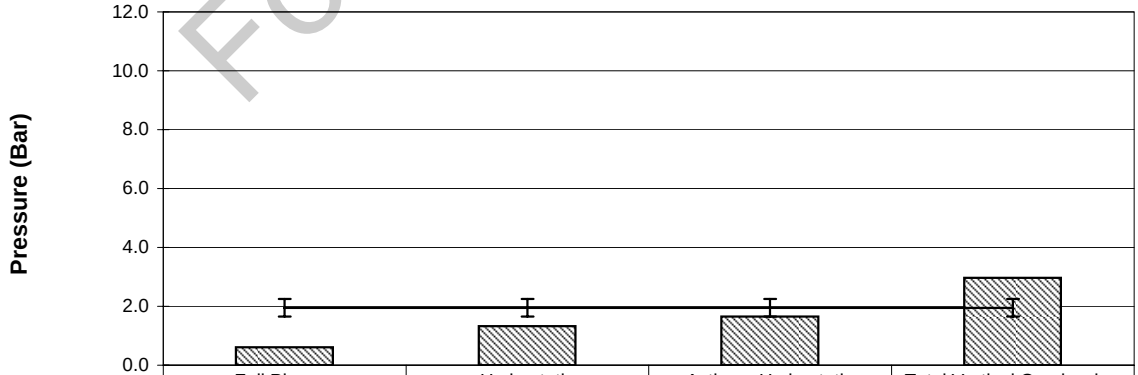
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.5	2.6
Target Pressure, bar	1.8	1.8	1.8	1.8

BH-08 (Sta. 615+76)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.6	2.9
Target Pressure, bar	1.9	1.9	1.9	1.9

BH-09 (Sta. 619+90)



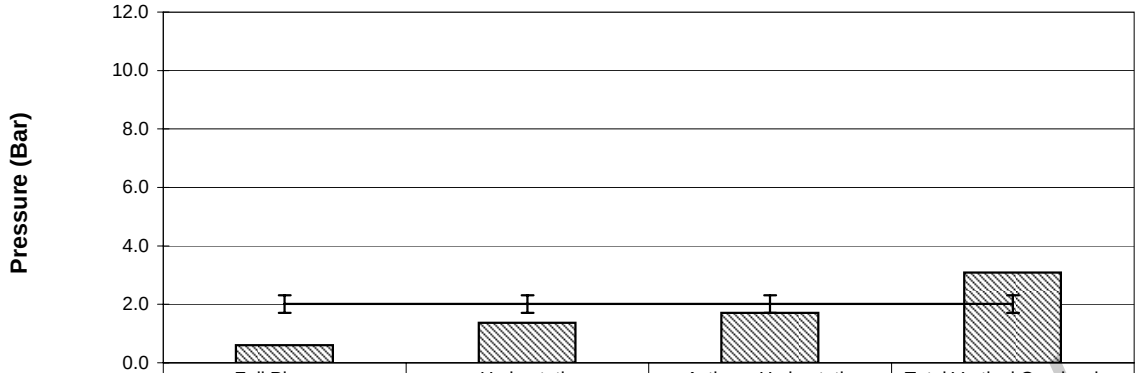
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.6	3.0
Target Pressure, bar	1.9	1.9	1.9	1.9

For Reference Only

See disclaimer on cover page.

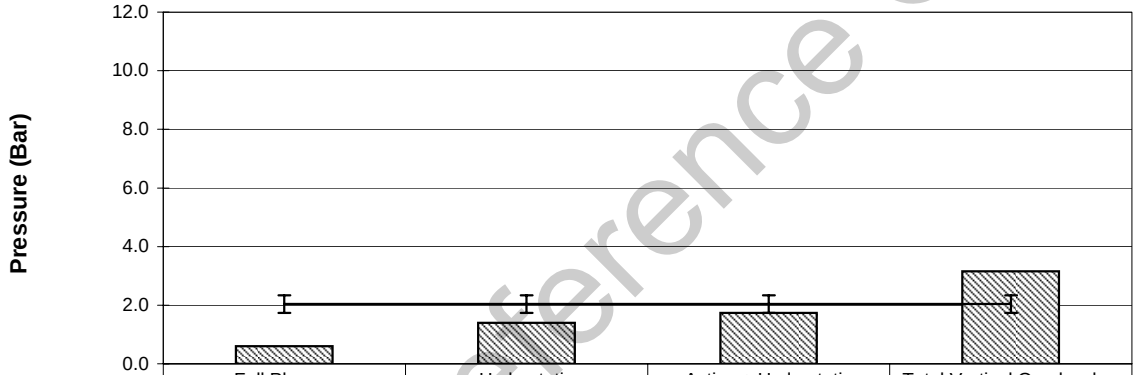
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-10 (Sta. 624+90)



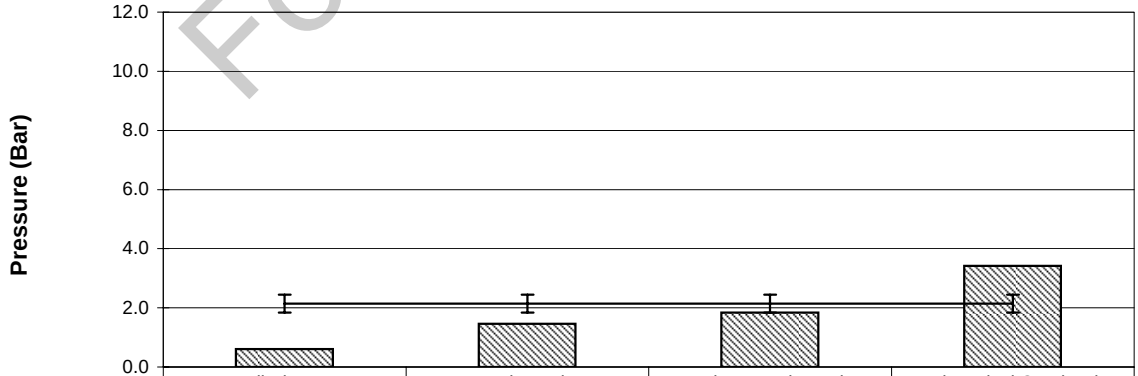
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.4	1.7	3.1
Target Pressure, bar	2.0	2.0	2.0	2.0

BH-11 (Sta. 627+54)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.4	1.7	3.2
Target Pressure, bar	2.0	2.0	2.0	2.0

BH-12 (Sta. 634+68)



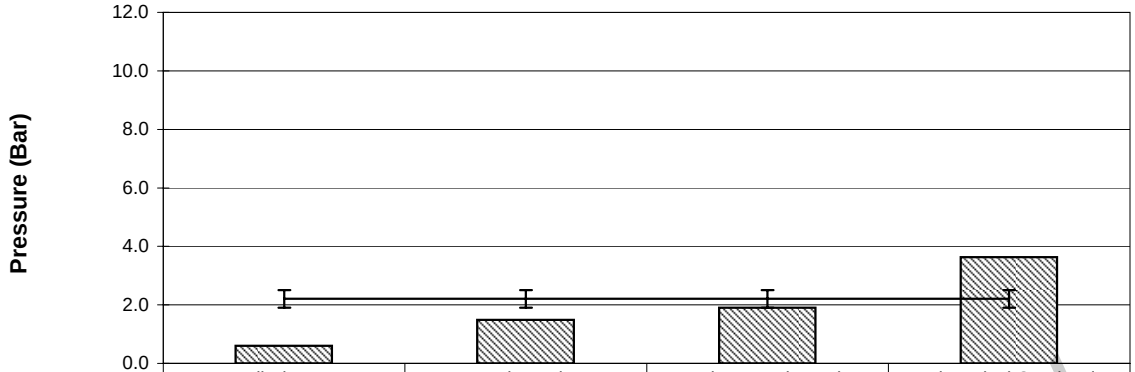
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.5	1.8	3.4
Target Pressure, bar	2.1	2.1	2.1	2.1

For Reference Only

See disclaimer on cover page.

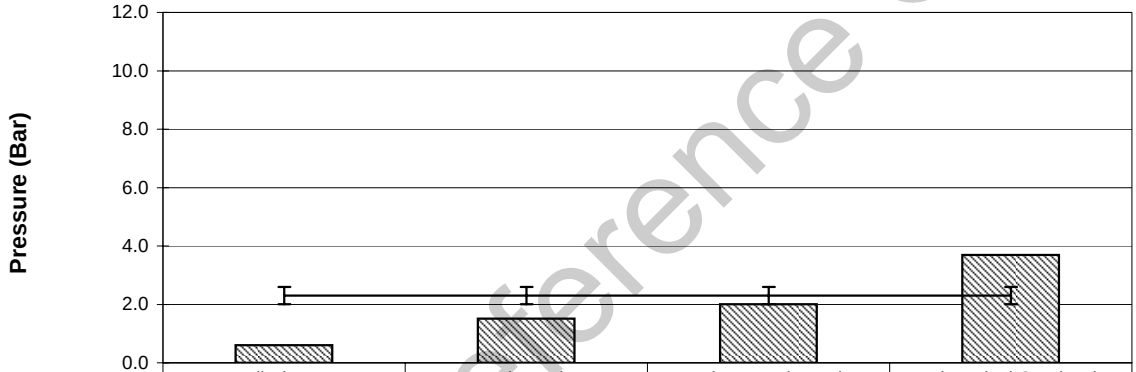
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-13 (Sta. 640+81)



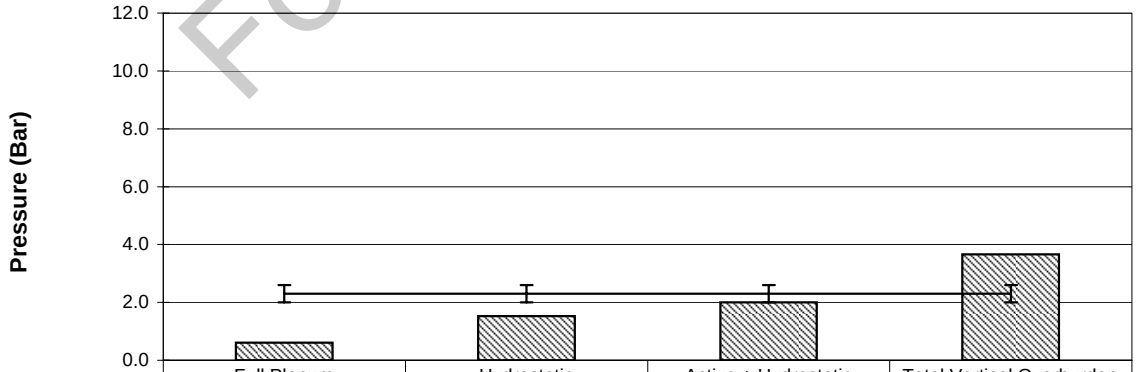
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.5	1.9	3.6
Target Pressure, bar	2.2	2.2	2.2	2.2

BH-14 (Sta. 642+52)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.5	2.0	3.7
Target Pressure, bar	2.3	2.3	2.3	2.3

BH-88 (Sta. 645+04)



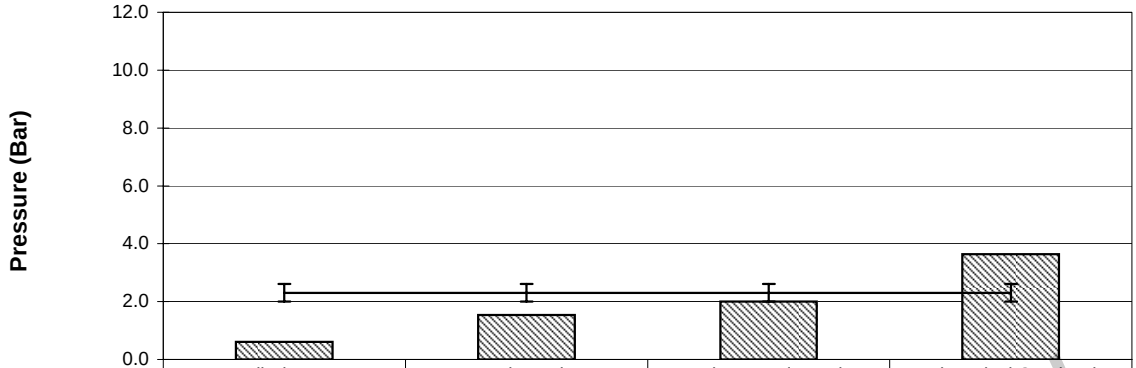
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.5	2.0	3.7
Target Pressure, bar	2.3	2.3	2.3	2.3

For Reference Only

See disclaimer on cover page.

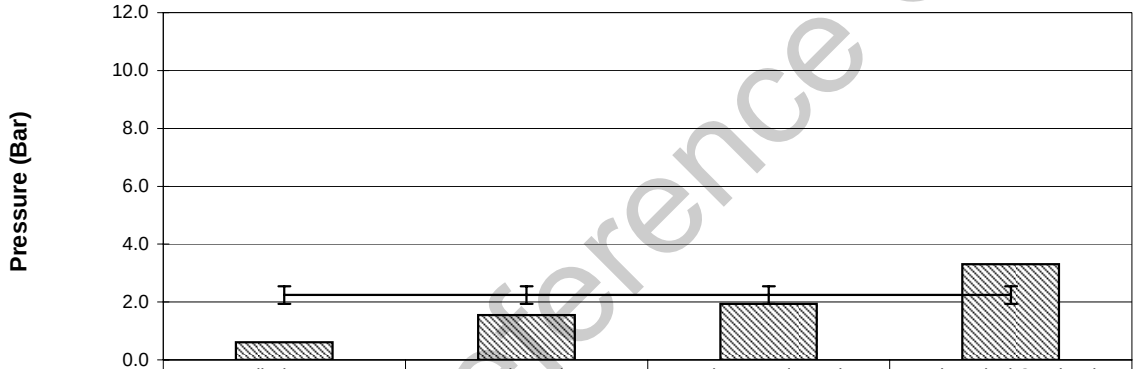
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-15 (Sta. 645+69)



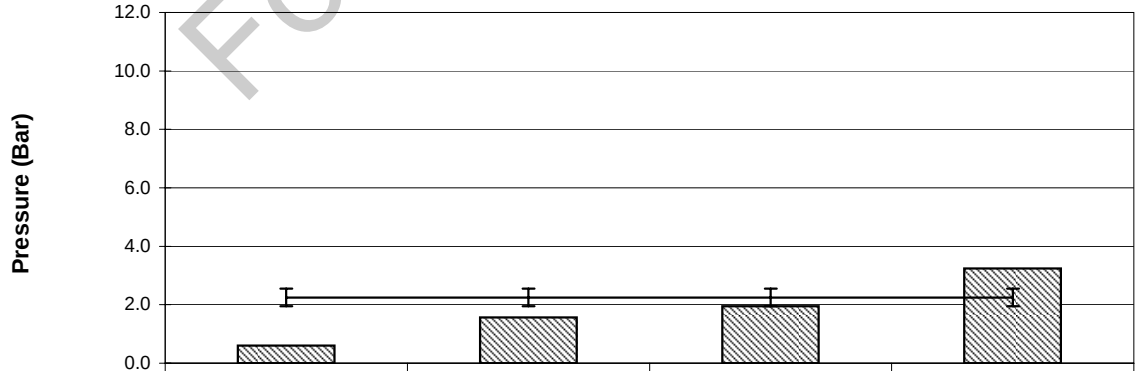
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.5	2.0	3.6
Target Pressure, bar	2.3	2.3	2.3	2.3

BH-87 (Sta. 648+33)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.5	1.9	3.3
Target Pressure, bar	2.2	2.2	2.2	2.2

BH-16 (Sta. 650+32)



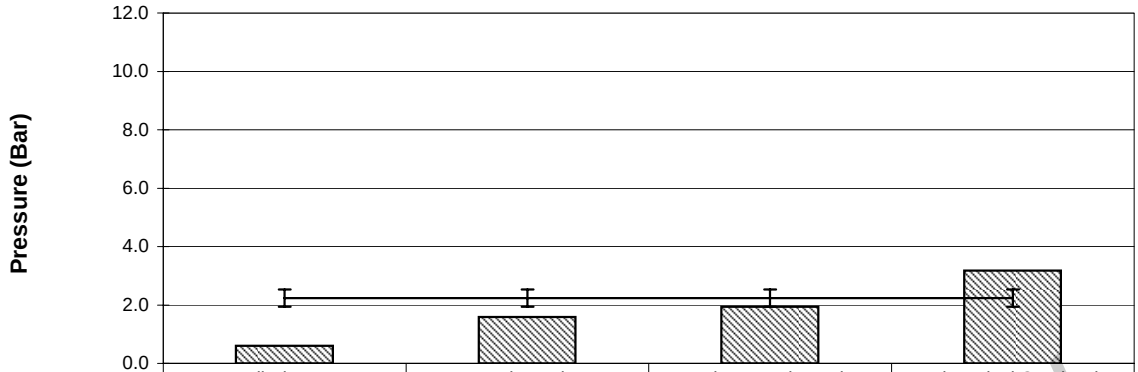
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.6	1.9	3.2
Target Pressure, bar	2.2	2.2	2.2	2.2

For Reference Only

See disclaimer on cover page.

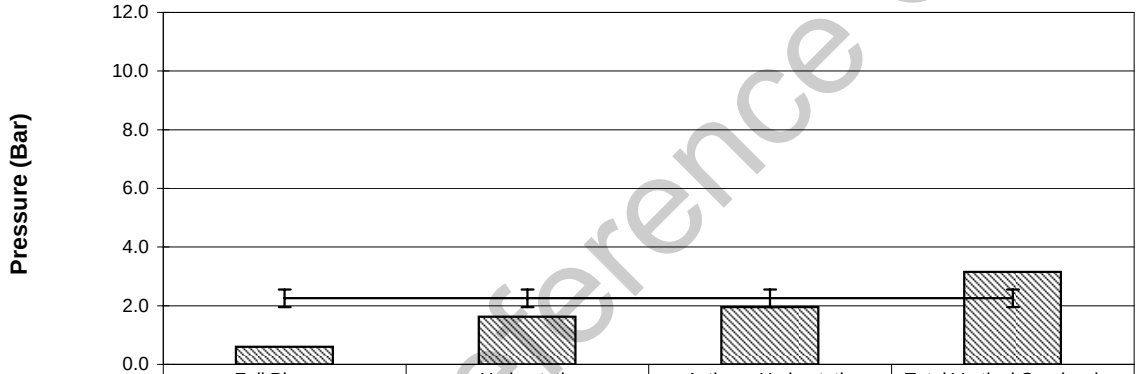
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-17 (Sta. 654+44)



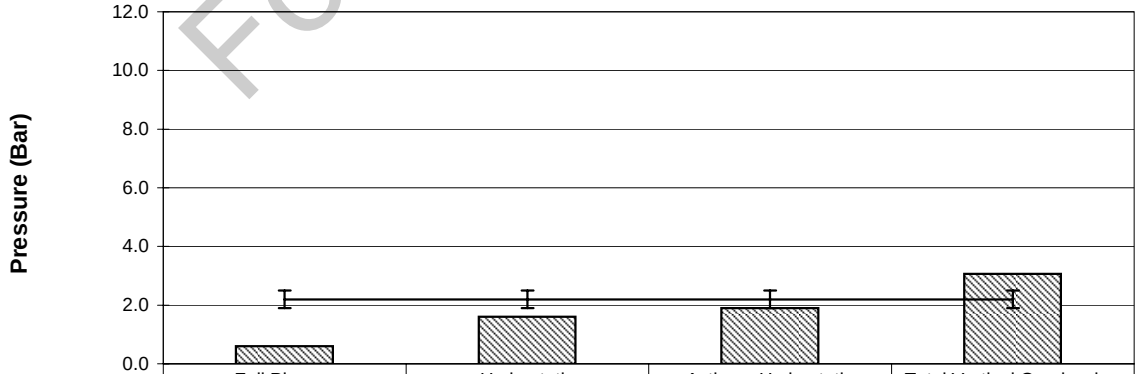
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.6	1.9	3.2
Target Pressure, bar	2.2	2.2	2.2	2.2

BH-18 (Sta. 660+03)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.6	2.0	3.2
Target Pressure, bar	2.3	2.3	2.3	2.3

BH-19 (Sta. 666+26)



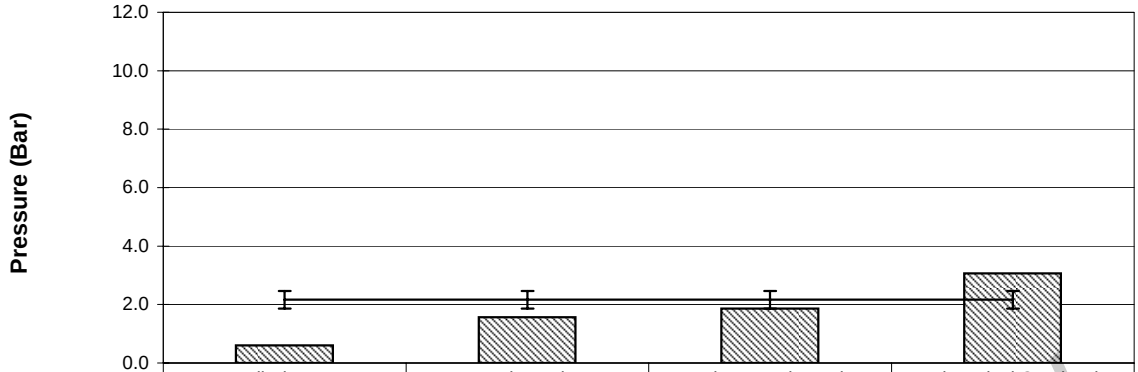
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.6	1.9	3.1
Target Pressure, bar	2.2	2.2	2.2	2.2

For Reference Only

See disclaimer on cover page.

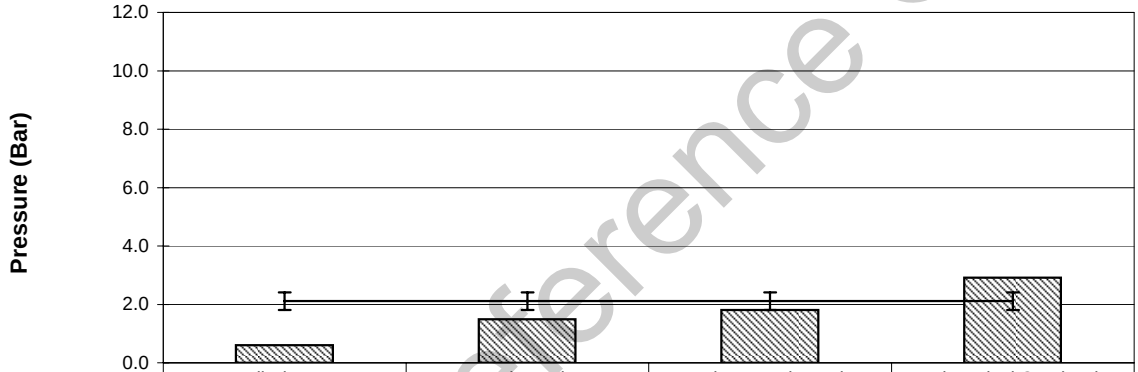
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-20 (Sta. 669+79)



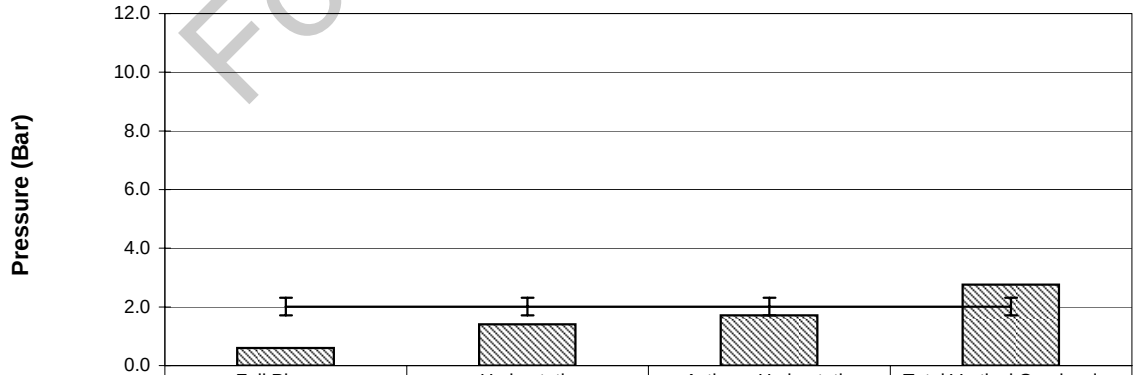
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.6	1.9	3.1
Target Pressure, bar	2.2	2.2	2.2	2.2

BH-21 (Sta. 675+49)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.5	1.8	2.9
Target Pressure, bar	2.1	2.1	2.1	2.1

BH-50 (Sta. 681+71)



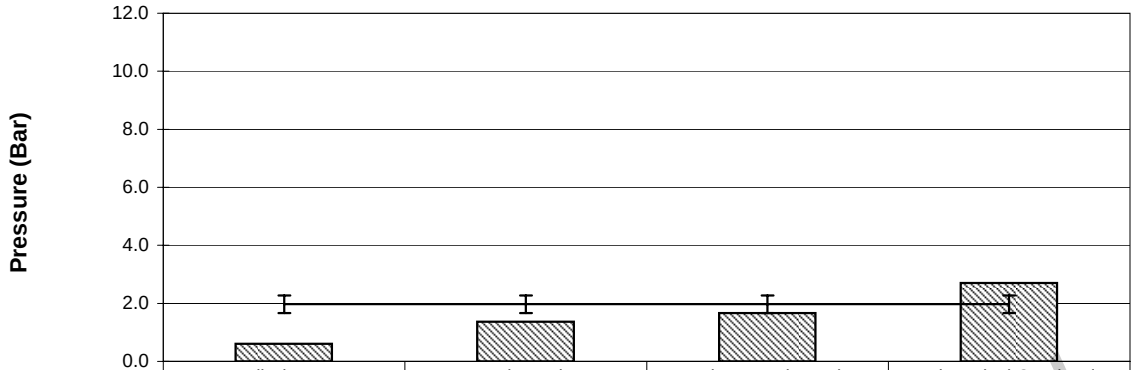
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.4	1.7	2.8
Target Pressure, bar	2.0	2.0	2.0	2.0

For Reference Only

See disclaimer on cover page.

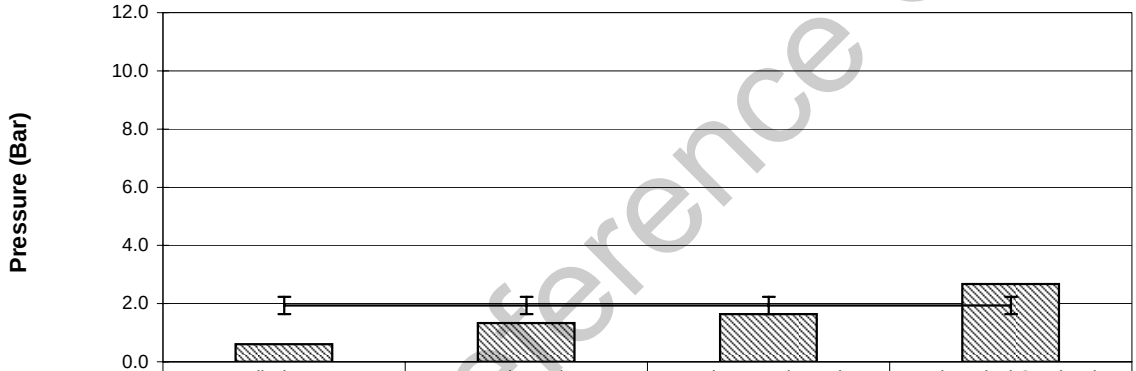
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-52 (Sta. 684+09)



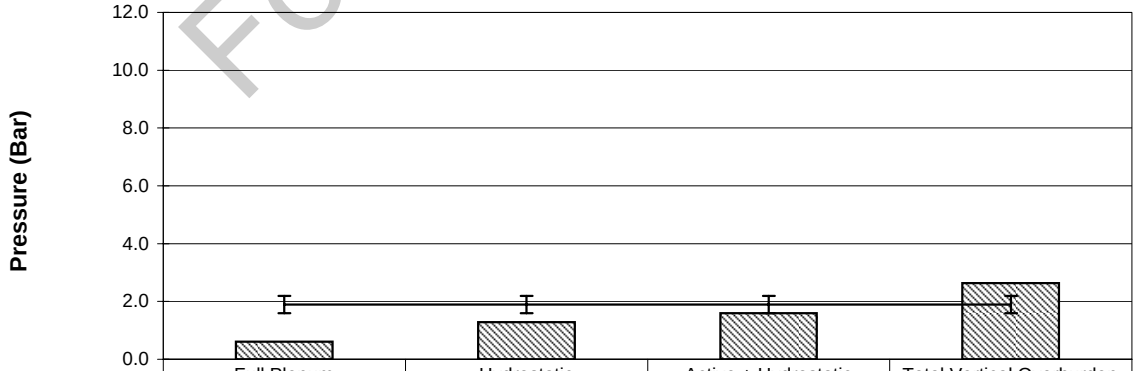
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.4	1.7	2.7
Target Pressure, bar	2.0	2.0	2.0	2.0

BH-53 (Sta. 685+42)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.6	2.7
Target Pressure, bar	1.9	1.9	1.9	1.9

BH-54 (Sta. 687+16)



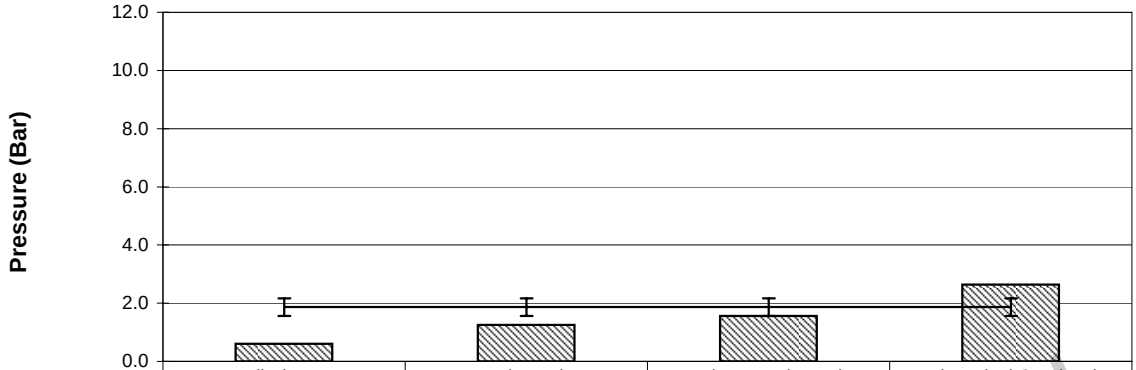
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.6	2.6
Target Pressure, bar	1.9	1.9	1.9	1.9

For Reference Only

See disclaimer on cover page.

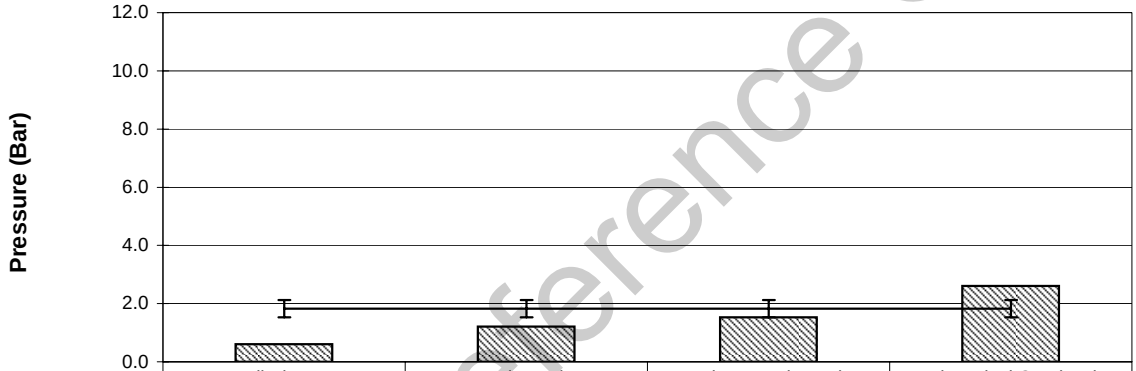
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-55 (Sta. 688+28)



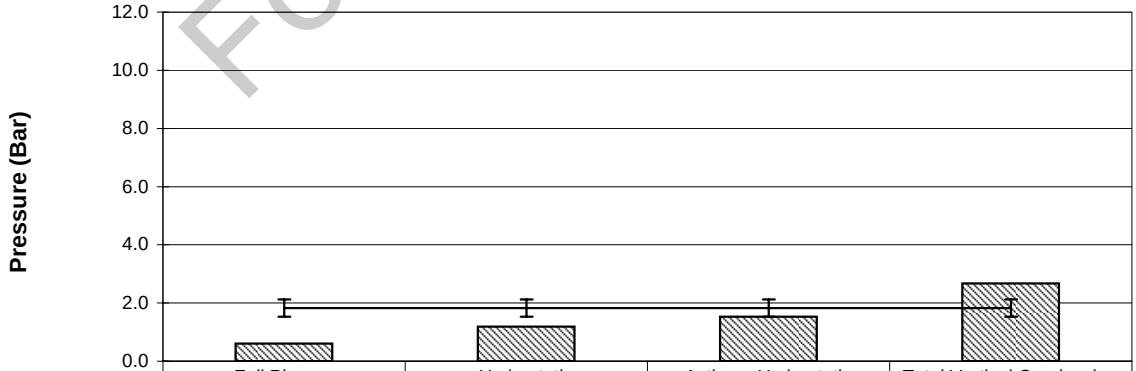
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.6	2.6
Target Pressure, bar	1.9	1.9	1.9	1.9

BH-23 (Sta. 690+02)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.5	2.6
Target Pressure, bar	1.8	1.8	1.8	1.8

BH-64 (Sta. 691+92)



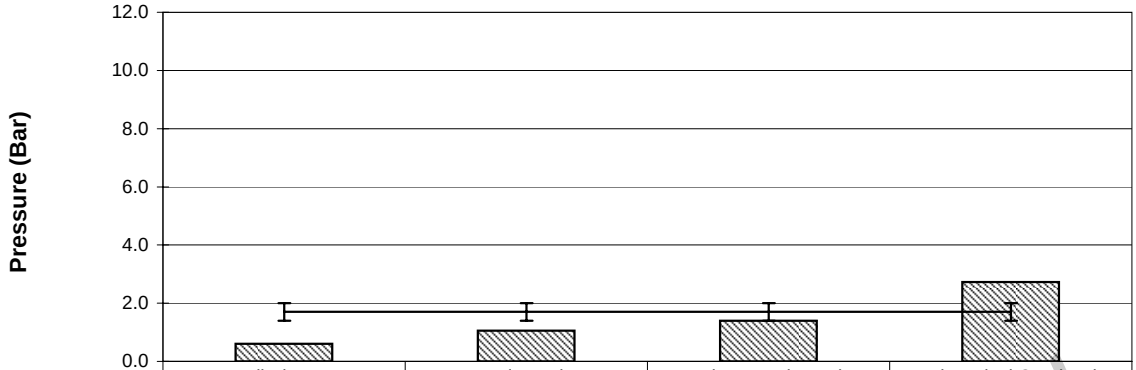
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.5	2.7
Target Pressure, bar	1.8	1.8	1.8	1.8

For Reference Only

See disclaimer on cover page.

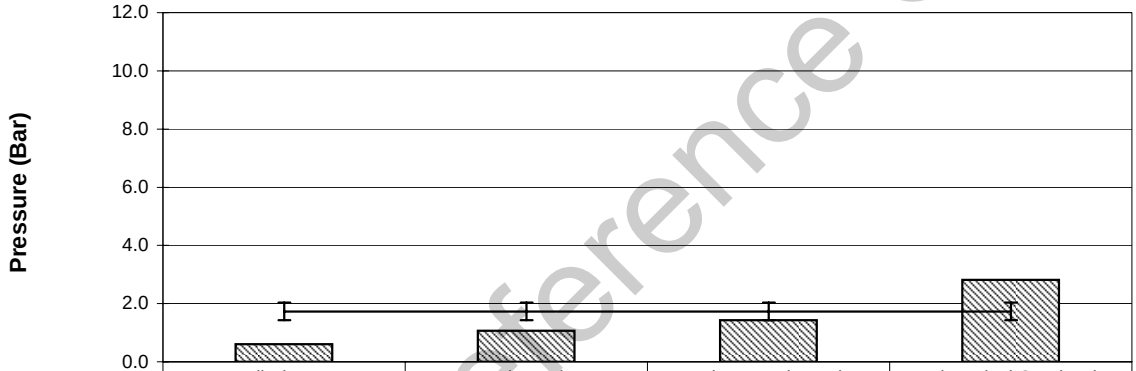
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-70 (Sta. 706+78)



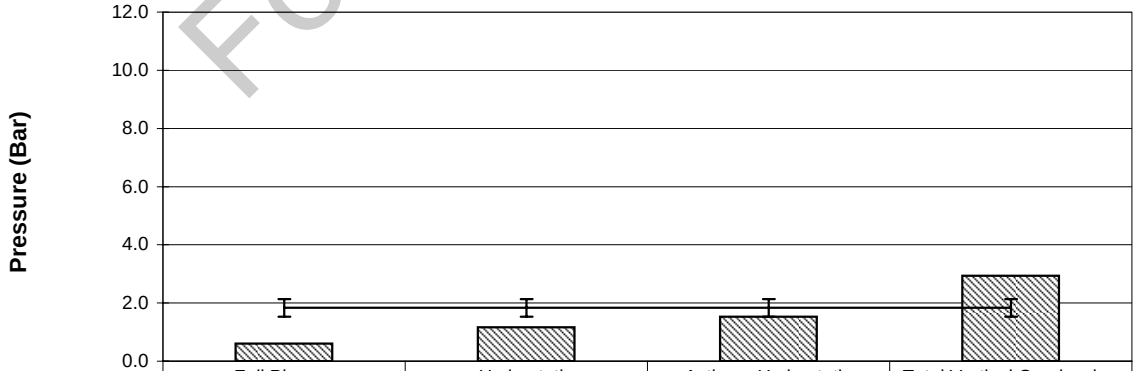
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.4	2.7
Target Pressure, bar	1.7	1.7	1.7	1.7

BH-71 (Sta. 707+62)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.4	2.8
Target Pressure, bar	1.7	1.7	1.7	1.7

BH-72 (Sta. 709+40)



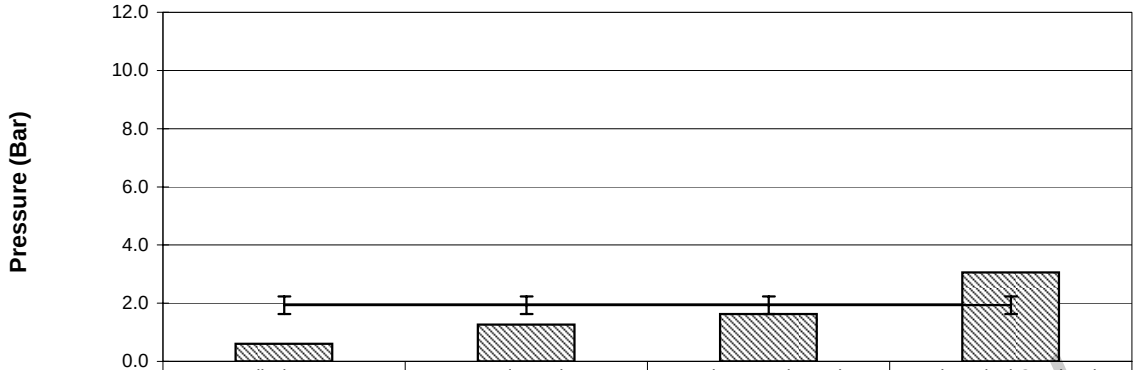
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.5	2.9
Target Pressure, bar	1.8	1.8	1.8	1.8

For Reference Only

See disclaimer on cover page.

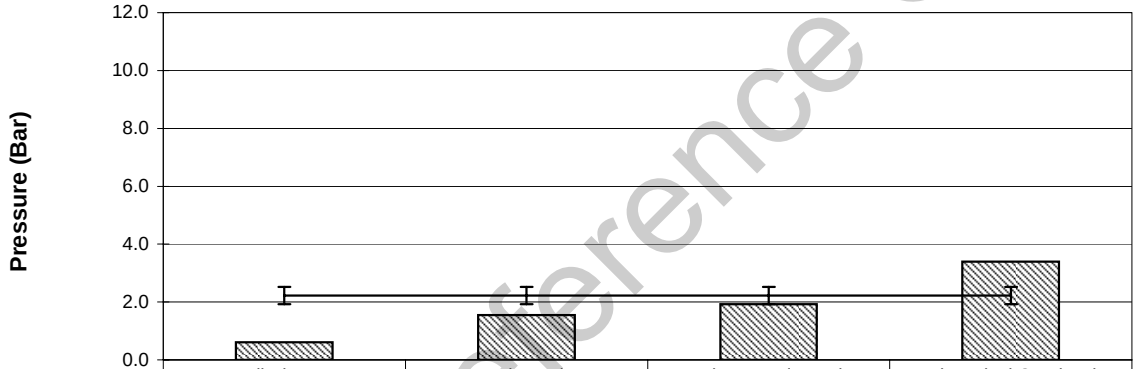
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-26 (Sta. 710+70)



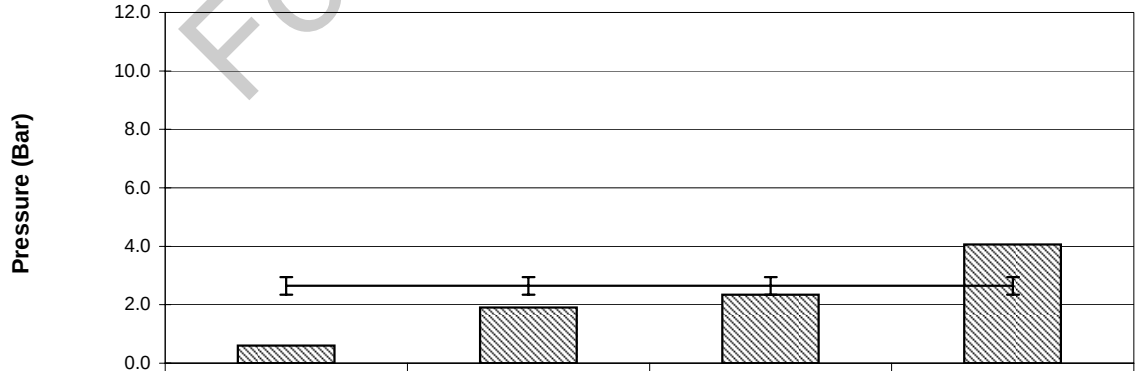
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.6	3.1
Target Pressure, bar	1.9	1.9	1.9	1.9

BH-27 (Sta. 715+00)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.5	1.9	3.4
Target Pressure, bar	2.2	2.2	2.2	2.2

BH-28 (Sta. 720+22)



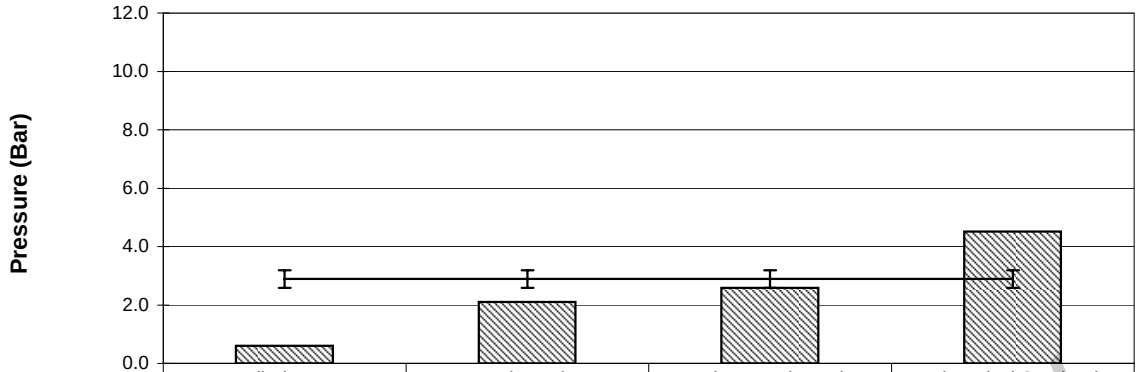
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.9	2.3	4.1
Target Pressure, bar	2.6	2.6	2.6	2.6

For Reference Only

See disclaimer on cover page.

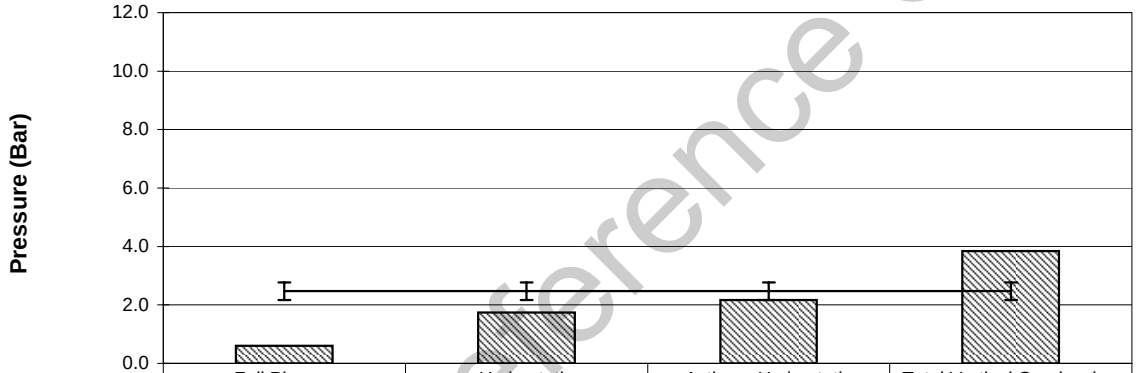
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-29 (Sta. 723+88)



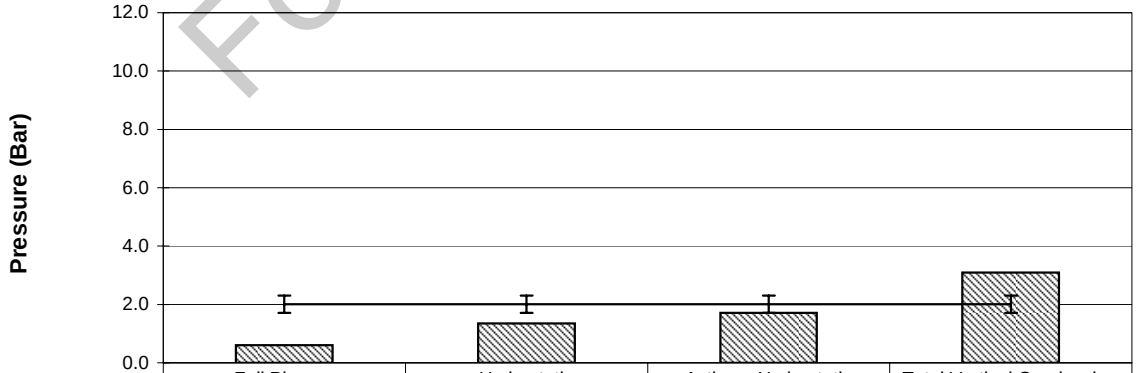
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	2.1	2.6	4.5
Target Pressure, bar	2.9	2.9	2.9	2.9

BH-30 (Sta. 728+01)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.7	2.2	3.8
Target Pressure, bar	2.5	2.5	2.5	2.5

BH-31 (Sta. 731+54)



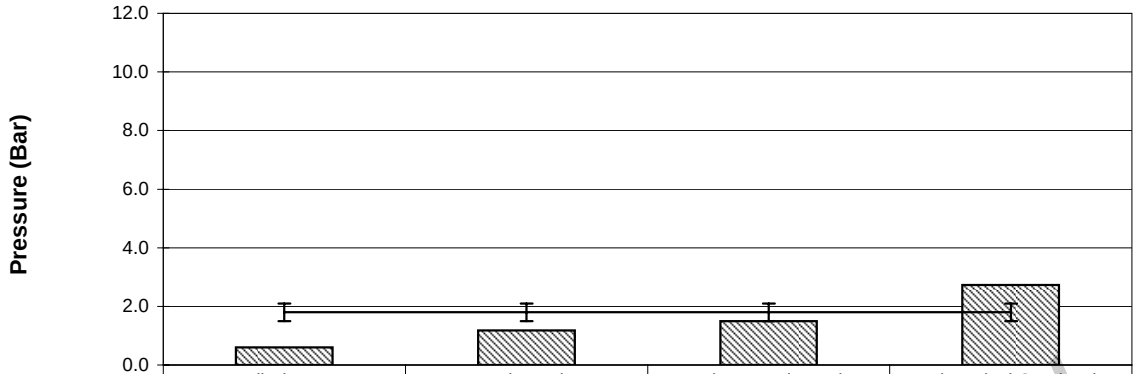
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.7	3.1
Target Pressure, bar	2.0	2.0	2.0	2.0

For Reference Only

See disclaimer on cover page.

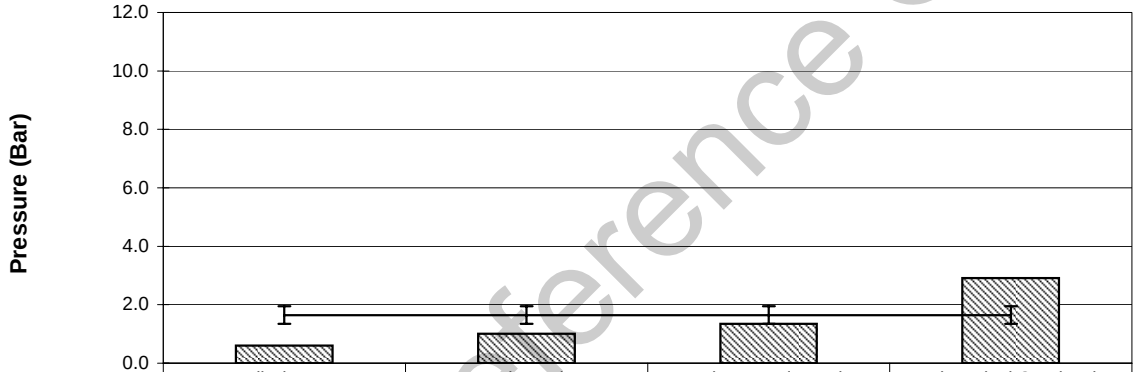
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-32 (Sta. 733+30)



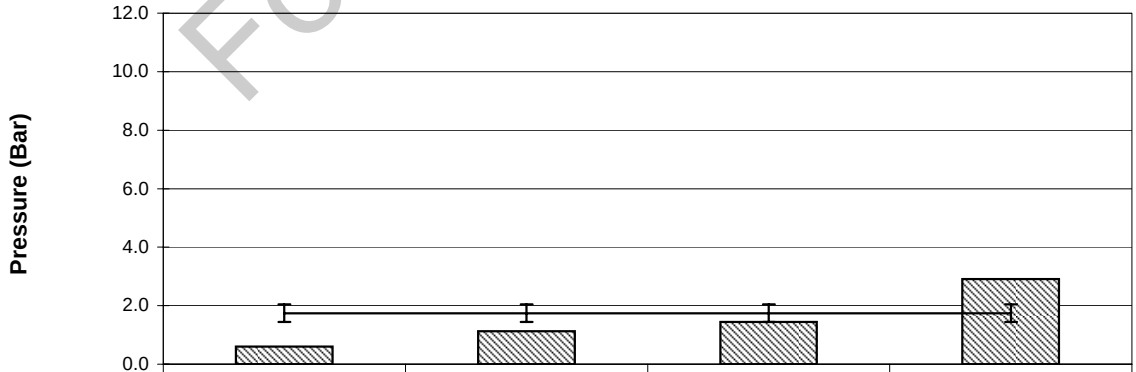
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.5	2.7
Target Pressure, bar	1.8	1.8	1.8	1.8

BH-34 (Sta. 744+65)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.0	1.3	2.9
Target Pressure, bar	1.6	1.6	1.6	1.6

BH-35 (Sta. 750+48)



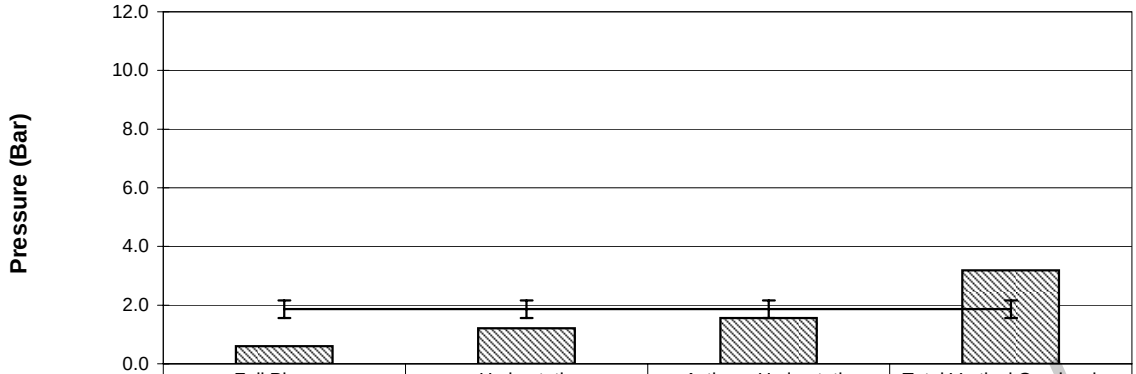
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.4	2.9
Target Pressure, bar	1.7	1.7	1.7	1.7

For Reference Only

See disclaimer on cover page.

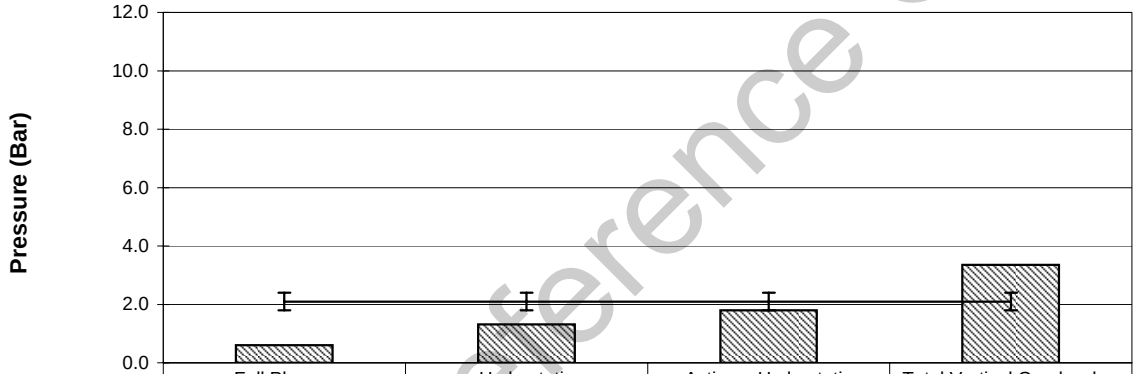
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-36 (Sta. 755+33)



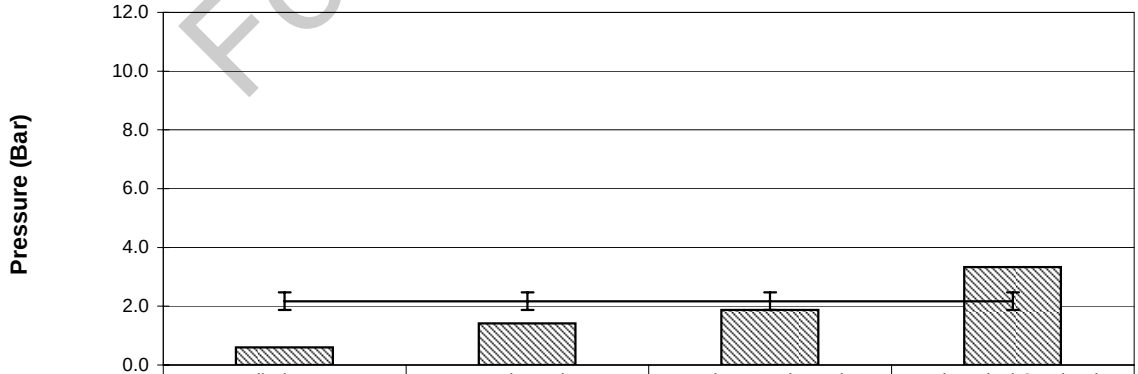
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.6	3.2
Target Pressure, bar	1.9	1.9	1.9	1.9

BH-37 (Sta. 760+59)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.8	3.4
Target Pressure, bar	2.1	2.1	2.1	2.1

BH-38 (Sta. 765+23)



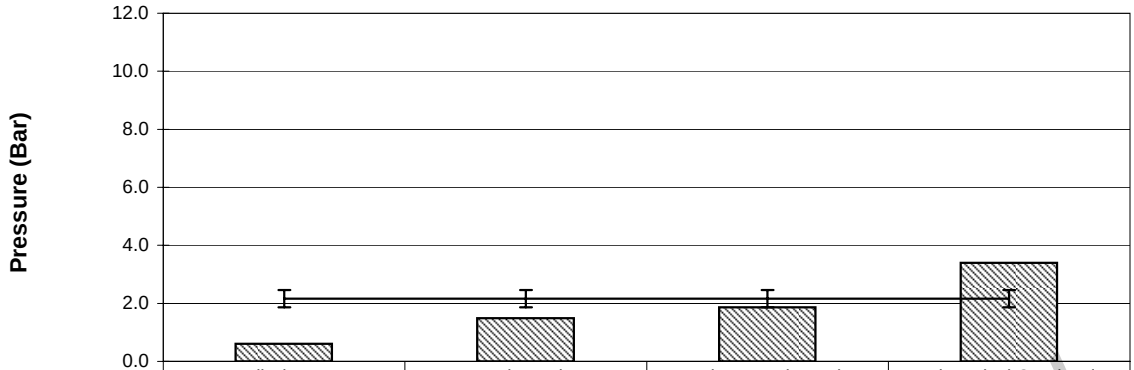
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.4	1.9	3.3
Target Pressure, bar	2.2	2.2	2.2	2.2

For Reference Only

See disclaimer on cover page.

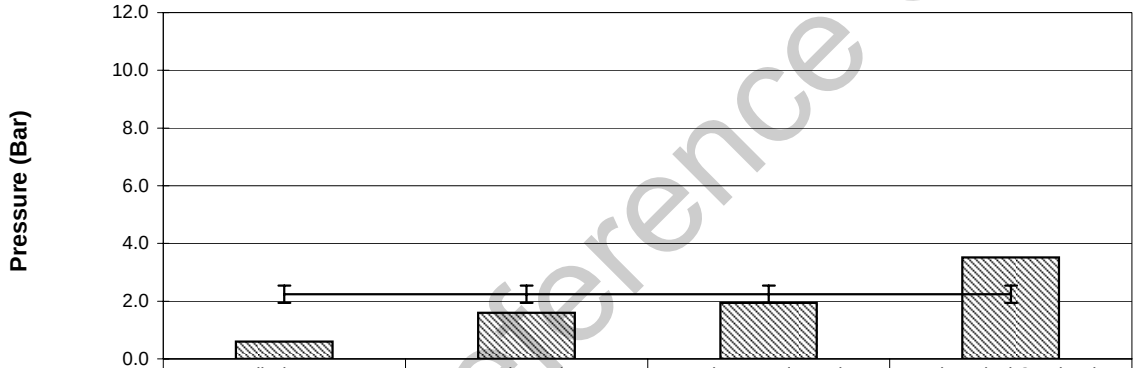
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-39 (Sta. 768+77)



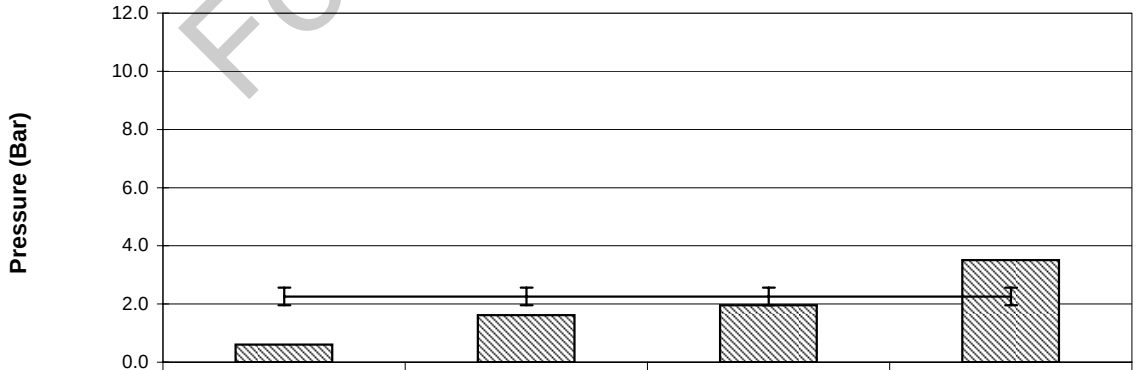
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.5	1.9	3.4
Target Pressure, bar	2.2	2.2	2.2	2.2

BH-95 (Sta. 774+13)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.6	1.9	3.5
Target Pressure, bar	2.2	2.2	2.2	2.2

BH-40 (Sta. 775+76)



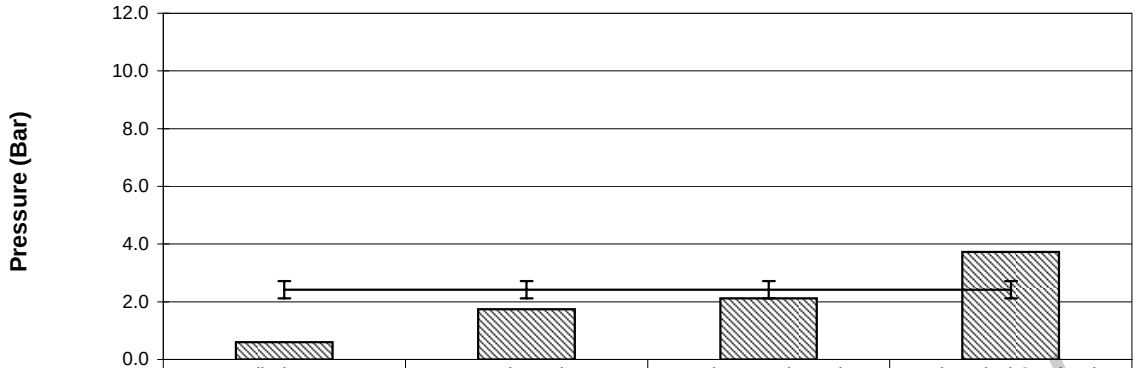
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.6	2.0	3.5
Target Pressure, bar	2.3	2.3	2.3	2.3

For Reference Only

See disclaimer on cover page.

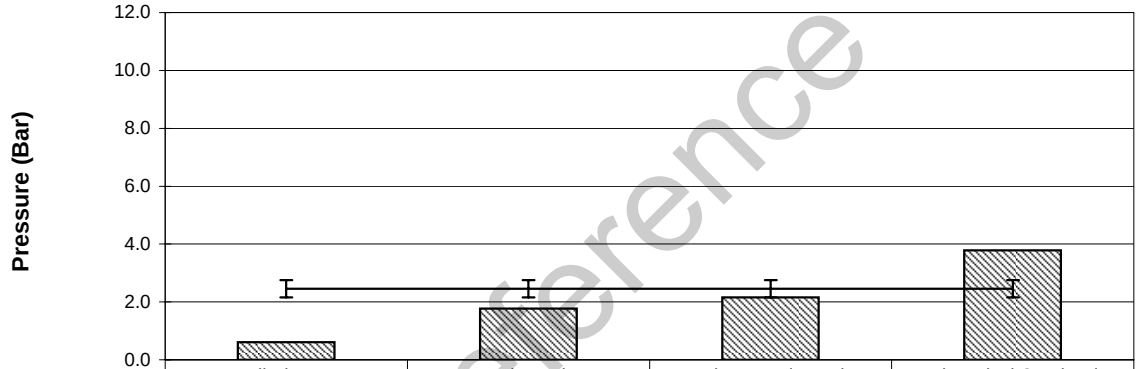
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-41 (Sta. 781+35)



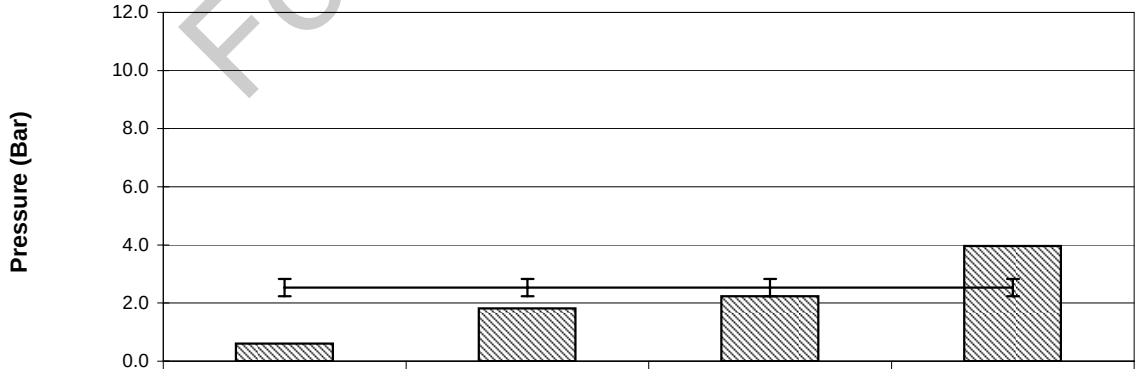
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.7	2.1	3.7
Target Pressure, bar	2.4	2.4	2.4	2.4

BH-79 (Sta. 782+50)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.8	2.2	3.8
Target Pressure, bar	2.5	2.5	2.5	2.5

BH-42 (Sta. 785+37)



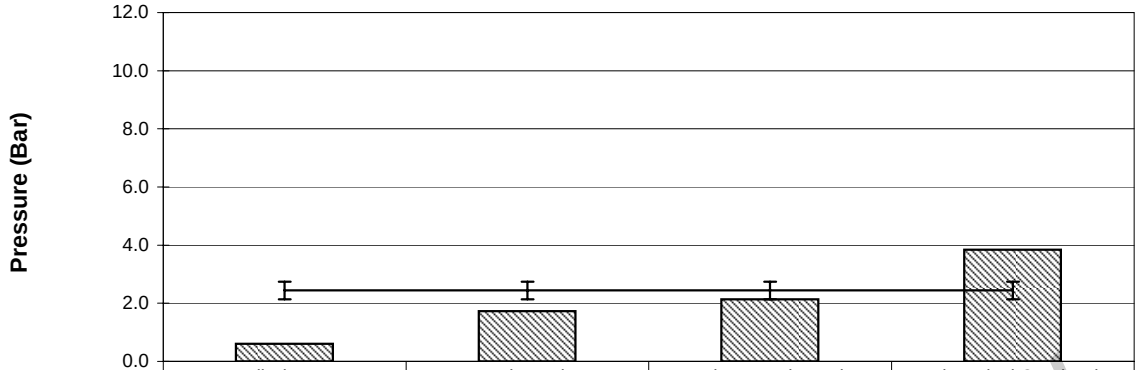
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.8	2.2	4.0
Target Pressure, bar	2.5	2.5	2.5	2.5

For Reference Only

See disclaimer on cover page.

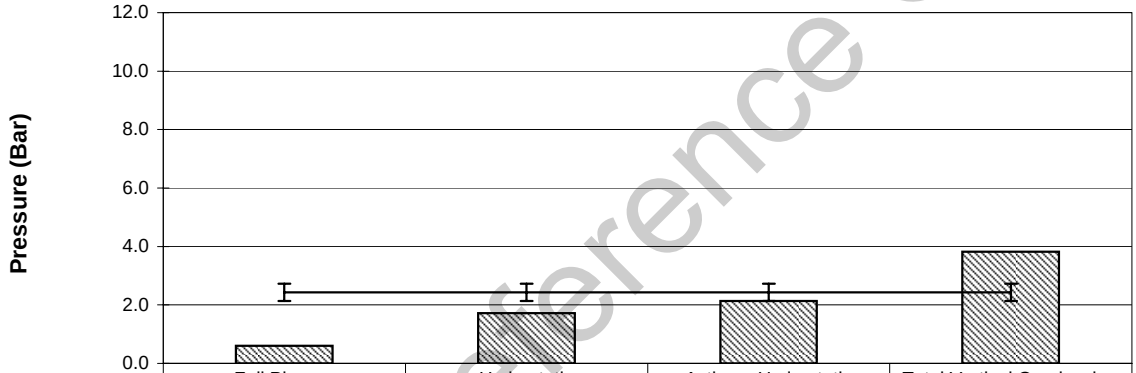
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-81 (Sta. 789+50)



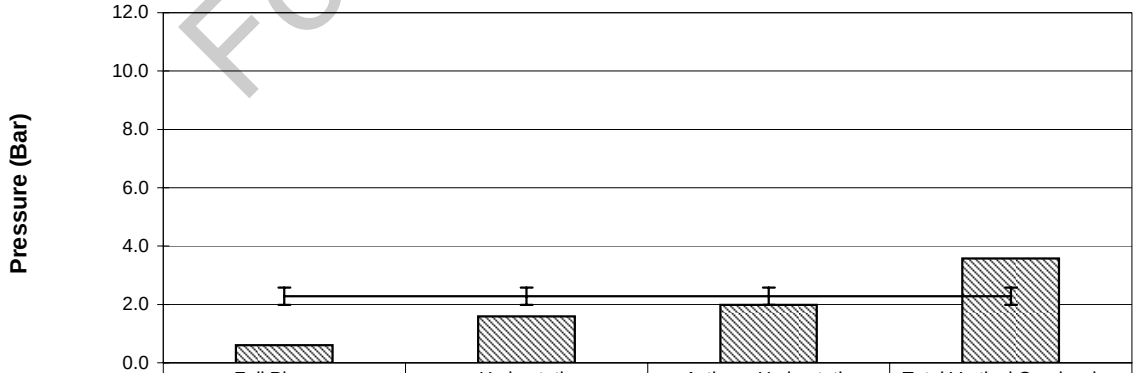
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.7	2.1	3.8
Target Pressure, bar	2.4	2.4	2.4	2.4

BH-43 (Sta. 789+71)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.7	2.1	3.8
Target Pressure, bar	2.4	2.4	2.4	2.4

BH-80 (Sta. 794+39)



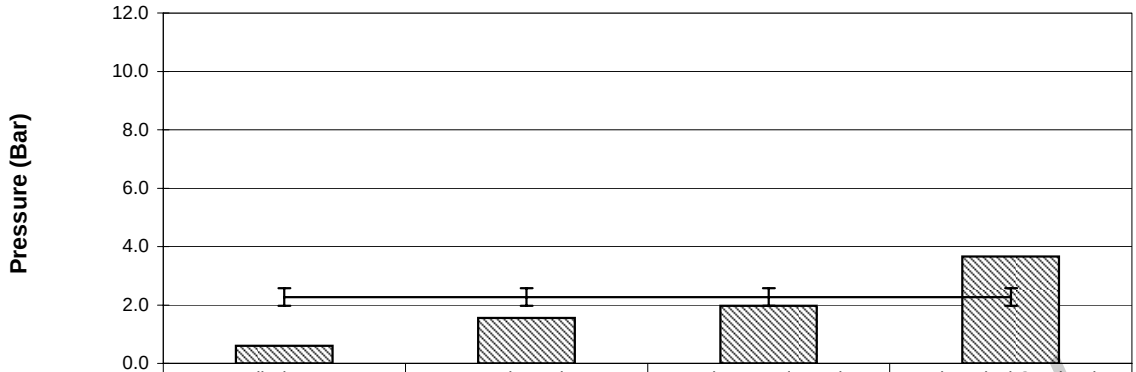
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.6	2.0	3.6
Target Pressure, bar	2.3	2.3	2.3	2.3

For Reference Only

See disclaimer on cover page.

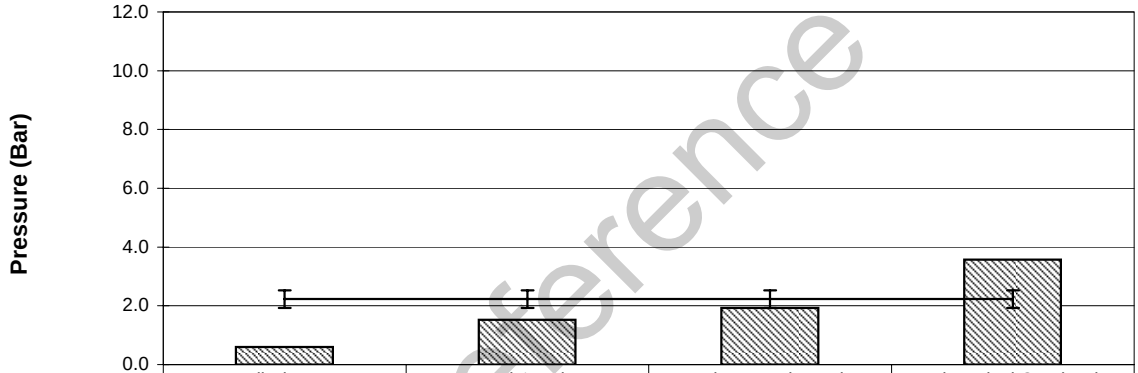
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-102 (Sta. 796+48)



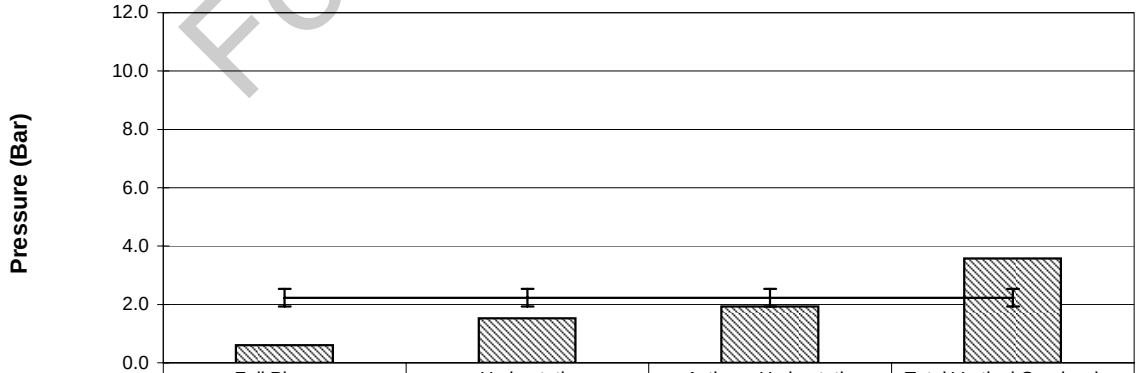
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.6	2.0	3.7
Target Pressure, bar	2.3	2.3	2.3	2.3

BH-103 (Sta. 798+16)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.5	1.9	3.6
Target Pressure, bar	2.2	2.2	2.2	2.2

BH-44 (Sta. 798+27)



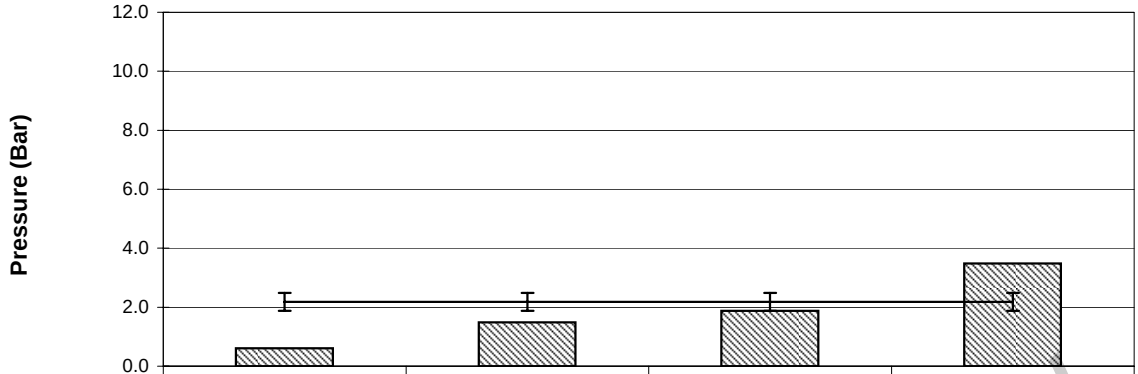
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.5	1.9	3.6
Target Pressure, bar	2.2	2.2	2.2	2.2

For Reference Only

See disclaimer on cover page.

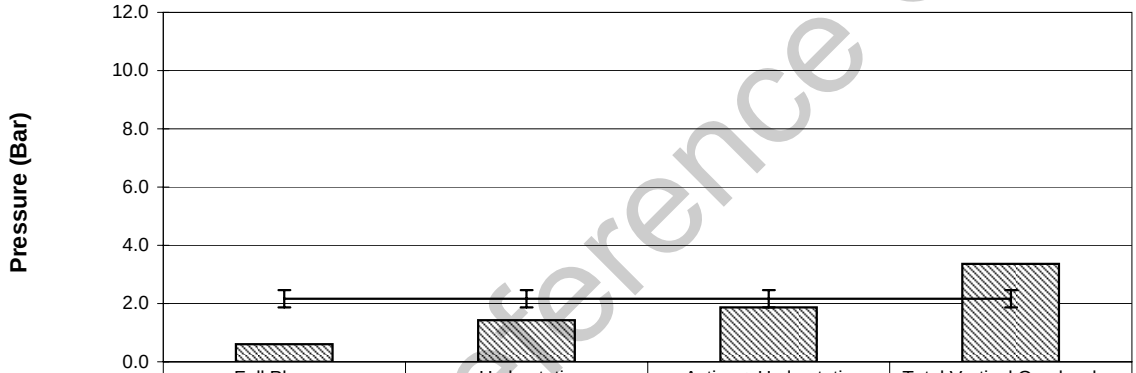
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-106 (Sta. 800+20)



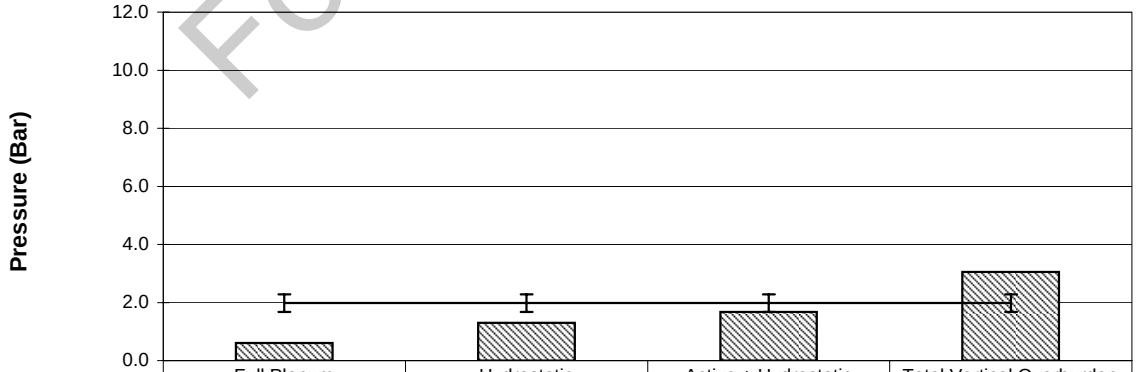
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.5	1.9	3.5
Target Pressure, bar	2.2	2.2	2.2	2.2

BH-45 (Sta. 802+44)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.4	1.9	3.4
Target Pressure, bar	2.2	2.2	2.2	2.2

BH-46 (Sta. 809+36)



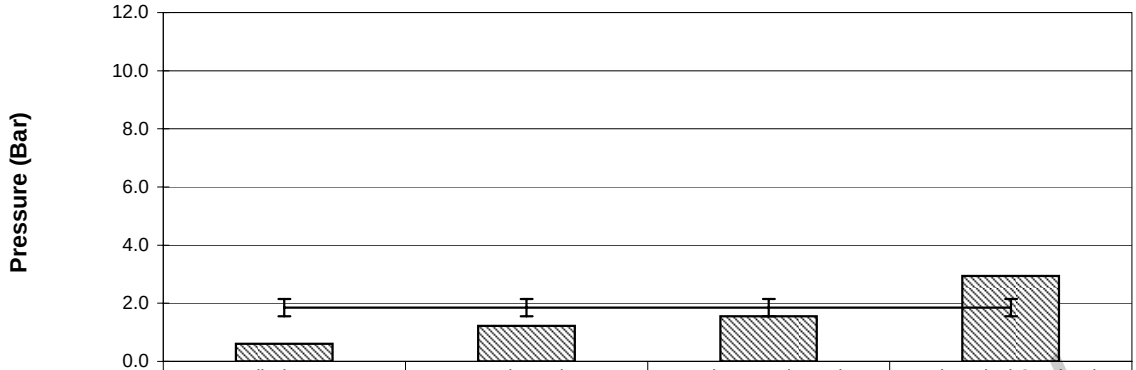
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.7	3.1
Target Pressure, bar	2.0	2.0	2.0	2.0

For Reference Only

See disclaimer on cover page.

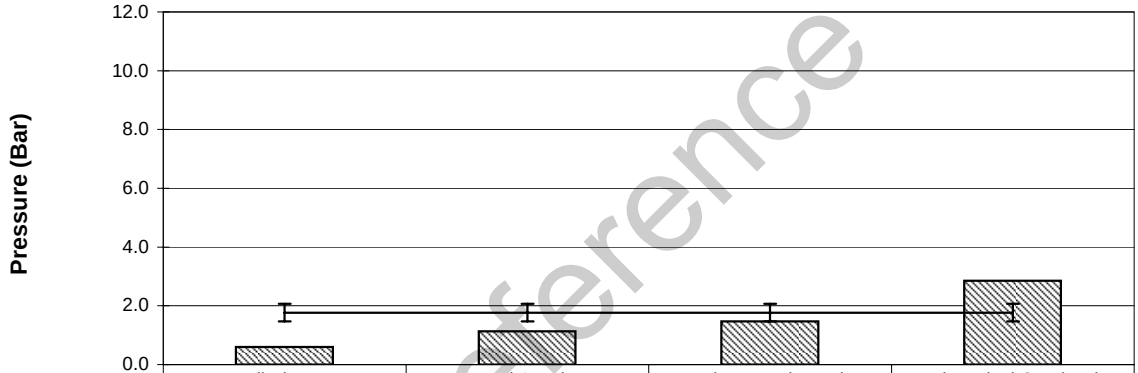
Face Pressure Charts by Borehole
(Maximum Water Level)

BH-47 (Sta. 813+52)



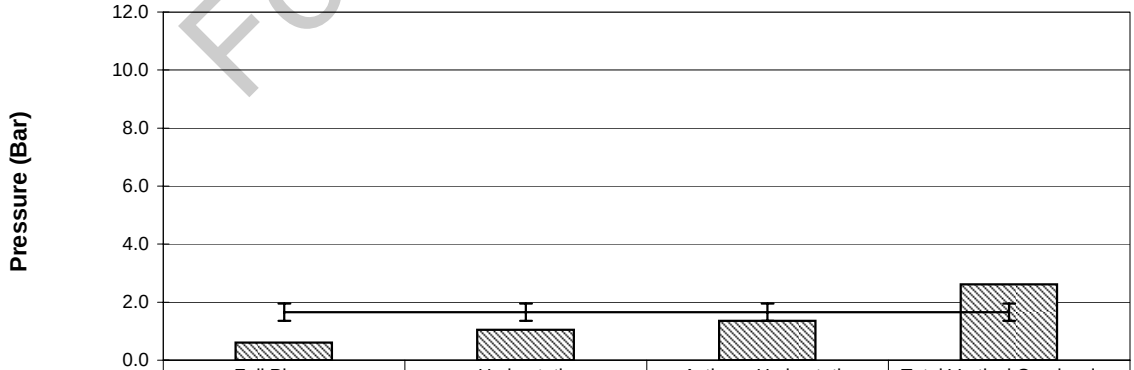
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.5	2.9
Target Pressure, bar	1.8	1.8	1.8	1.8

BH-48 (Sta. 818+34)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.5	2.9
Target Pressure, bar	1.8	1.8	1.8	1.8

BH-49 (Sta. 824+28)

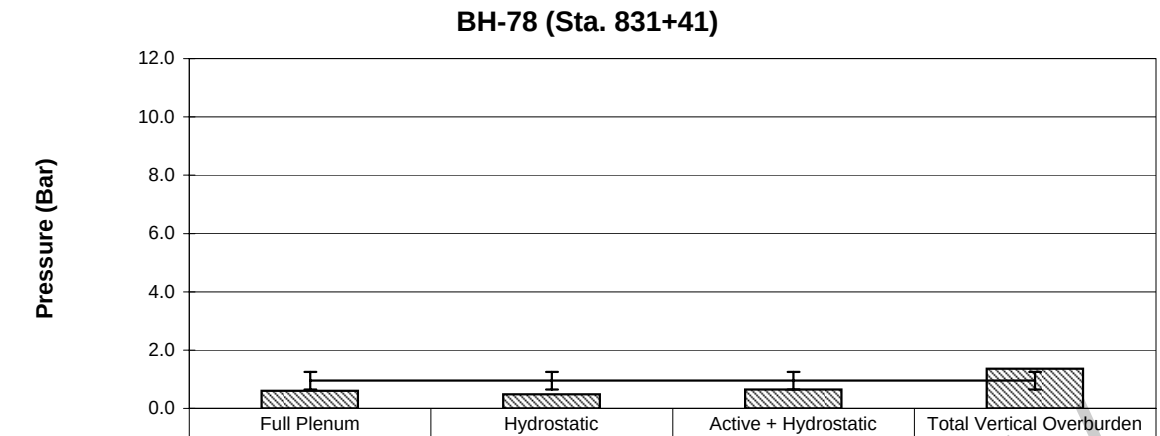


	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.4	2.6
Target Pressure, bar	1.7	1.7	1.7	1.7

For Reference Only

See disclaimer on cover page.

Face Pressure Charts by Borehole
(Maximum Water Level)



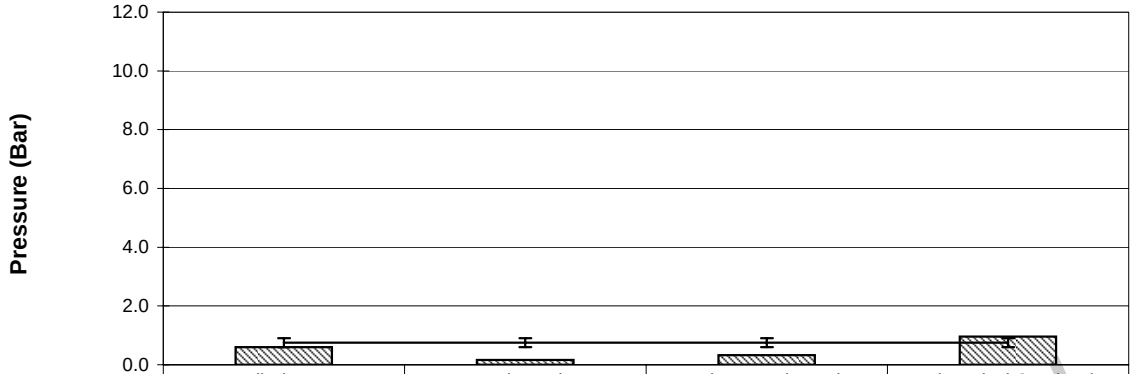
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.5	0.6	1.4
Target Pressure, bar	0.9	0.9	0.9	0.9

For Reference Only

See disclaimer on cover page.

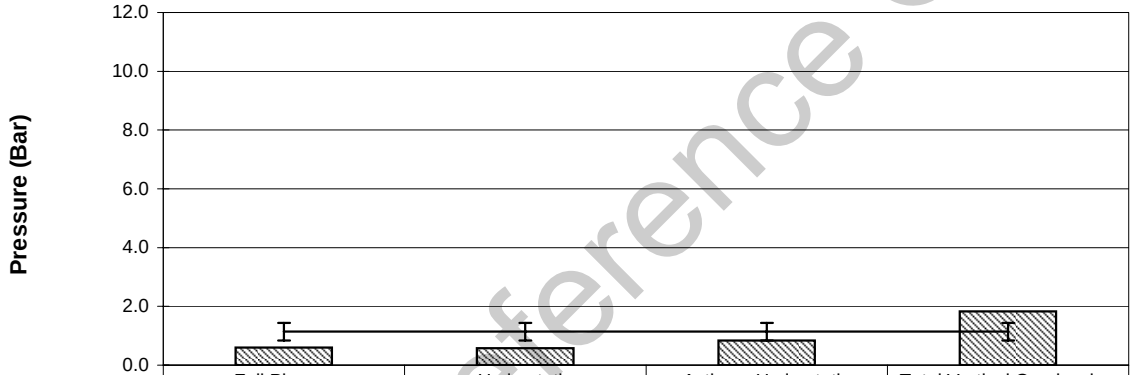
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-57 (Sta. 569+31)



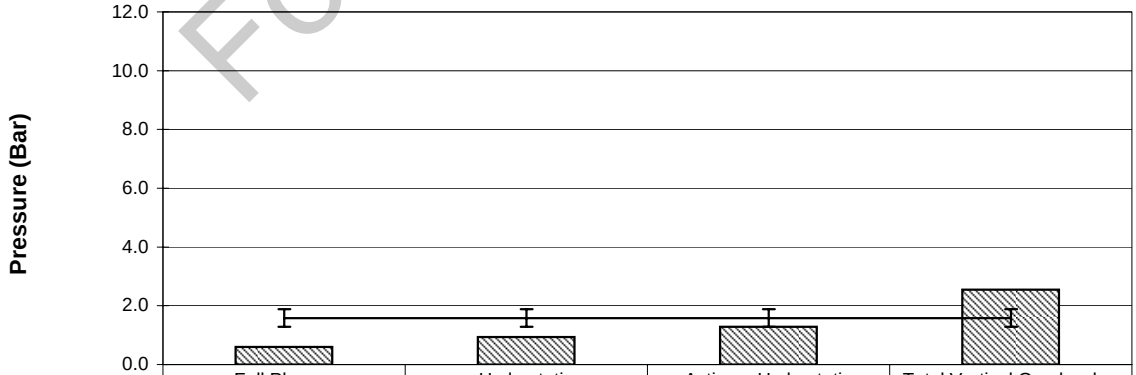
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.2	0.3	1.0
Target Pressure, bar	0.8	0.8	0.8	0.8

BH-01 (Sta. 574+09)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.6	0.8	1.8
Target Pressure, bar	1.1	1.1	1.1	1.1

BH-02 (Sta. 578+10)



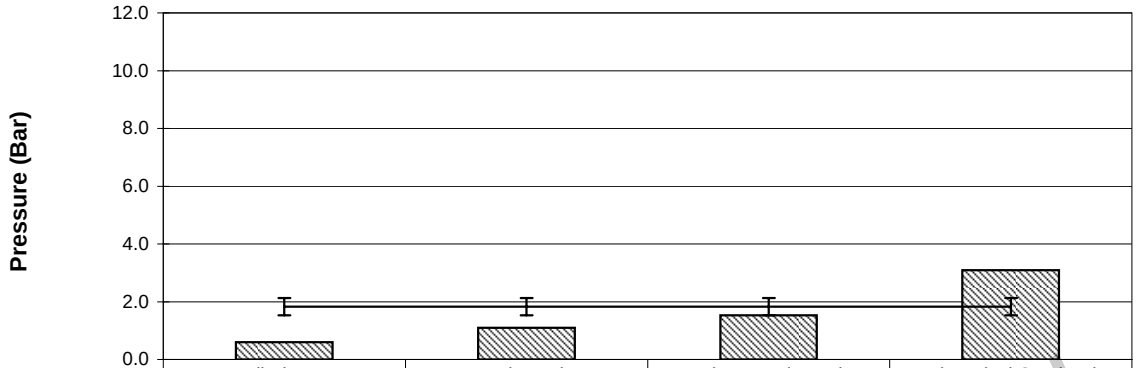
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.9	1.3	2.5
Target Pressure, bar	1.6	1.6	1.6	1.6

For Reference Only

See disclaimer on cover page.

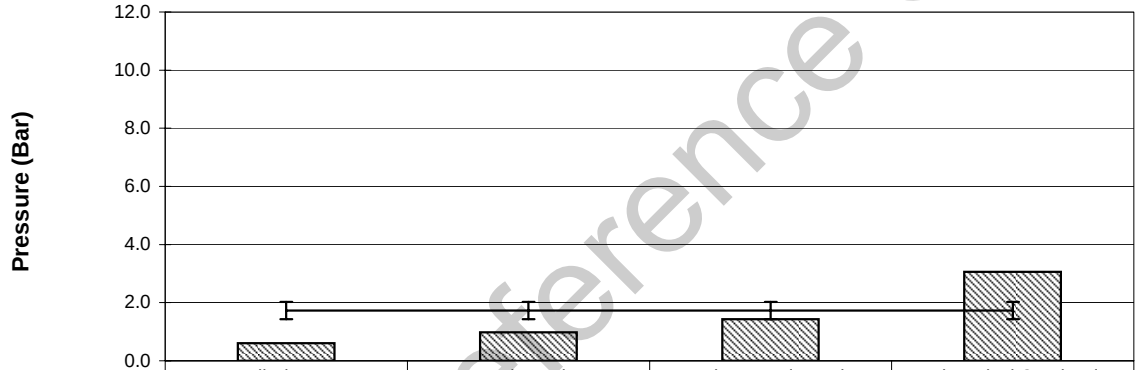
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-03 (Sta. 581+83)



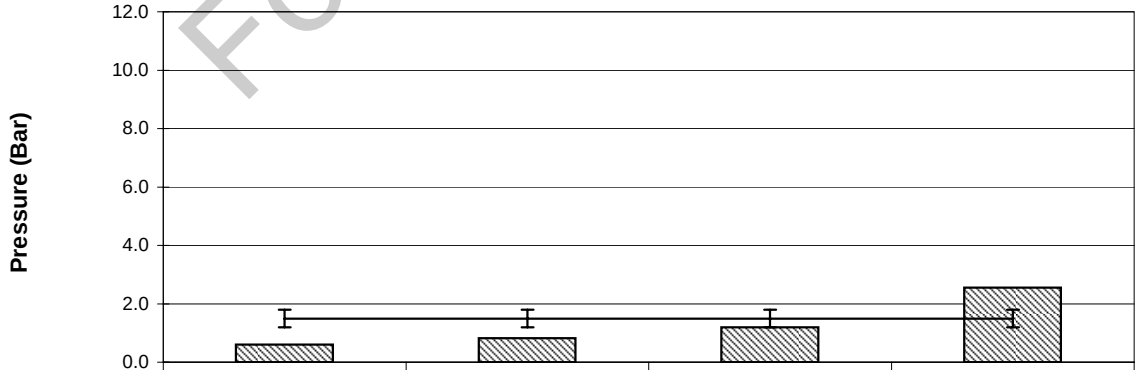
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.5	3.1
Target Pressure, bar	1.8	1.8	1.8	1.8

BH-04 (Sta. 590+53)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.0	1.4	3.1
Target Pressure, bar	1.7	1.7	1.7	1.7

BH-05 (Sta. 598+17)



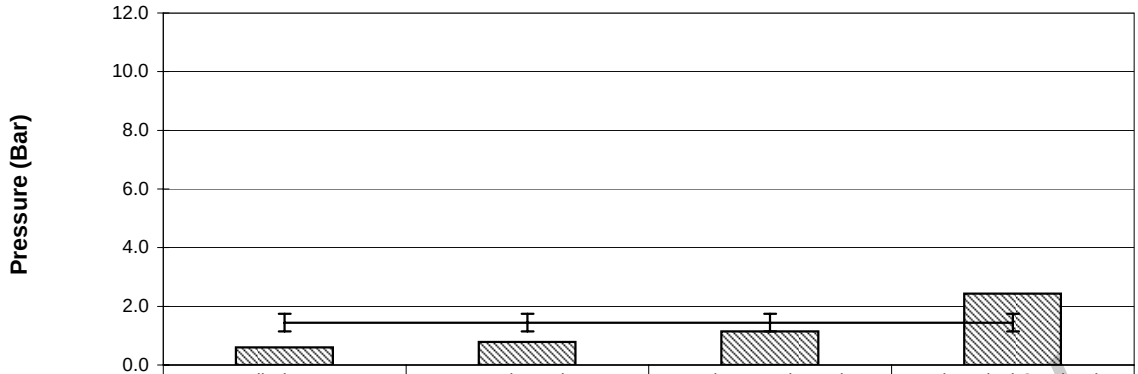
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.8	1.2	2.5
Target Pressure, bar	1.5	1.5	1.5	1.5

For Reference Only

See disclaimer on cover page.

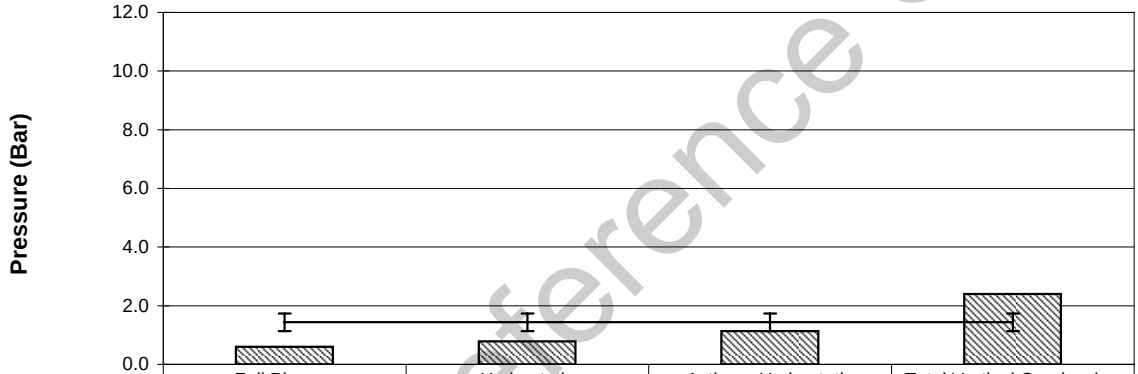
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-06 (Sta. 599+60)



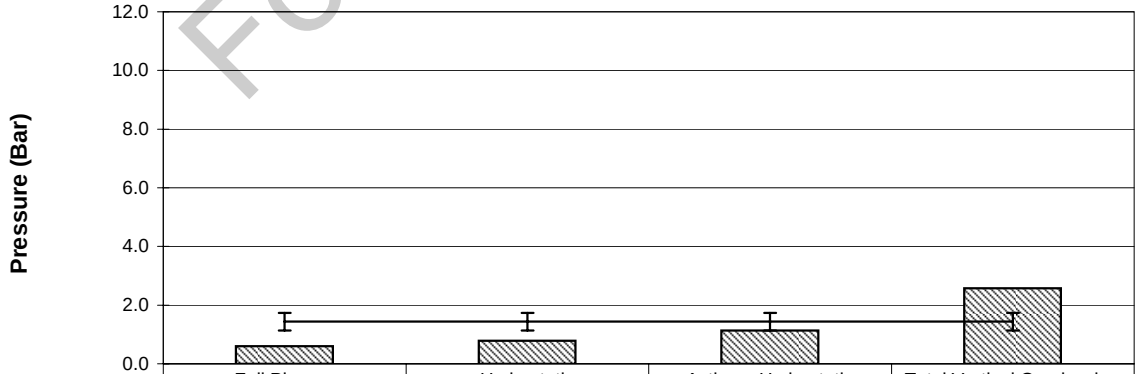
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.8	1.1	2.4
Target Pressure, bar	1.4	1.4	1.4	1.4

BH-83 (Sta. 599+83)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.8	1.1	2.4
Target Pressure, bar	1.4	1.4	1.4	1.4

BH-86 (Sta. 609+08)



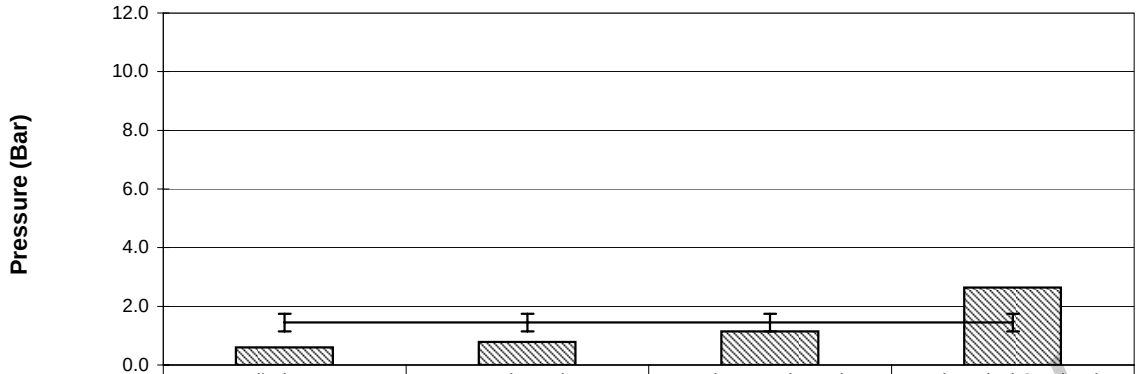
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.8	1.1	2.6
Target Pressure, bar	1.4	1.4	1.4	1.4

For Reference Only

See disclaimer on cover page.

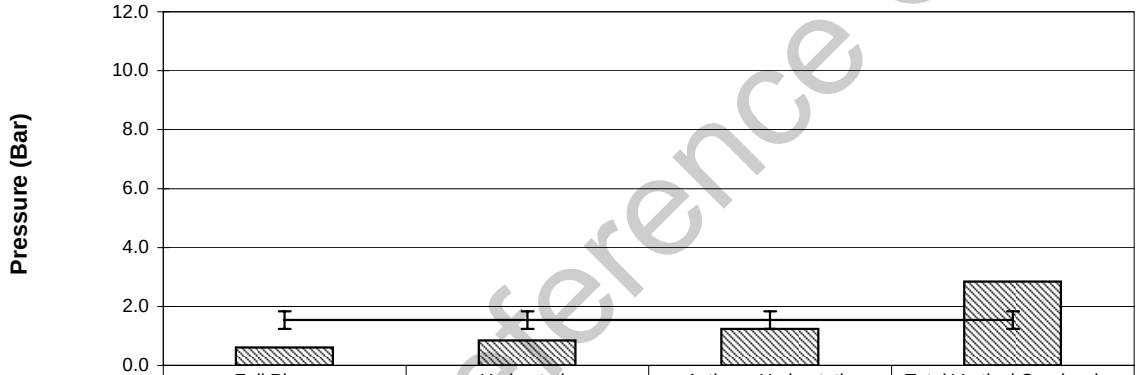
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-07 (Sta. 609+41)



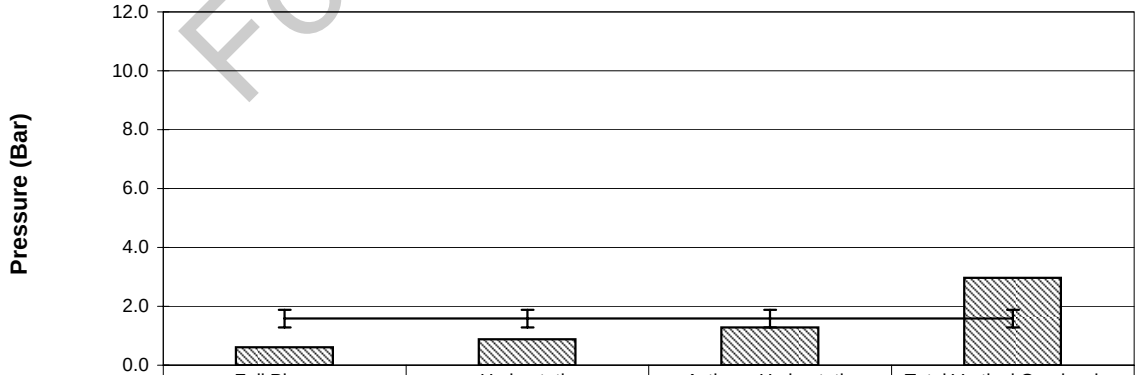
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.8	1.2	2.6
Target Pressure, bar	1.5	1.5	1.5	1.5

BH-08 (Sta. 615+76)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.8	1.2	2.9
Target Pressure, bar	1.5	1.5	1.5	1.5

BH-09 (Sta. 619+90)



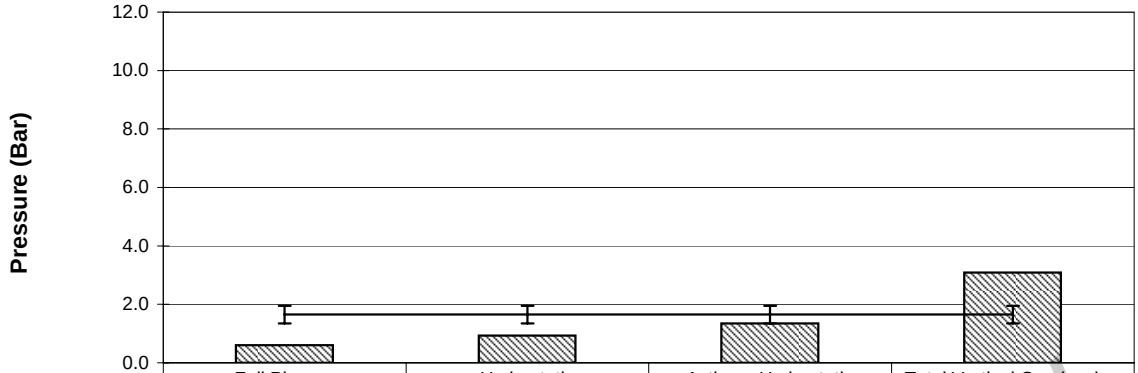
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.9	1.3	3.0
Target Pressure, bar	1.6	1.6	1.6	1.6

For Reference Only

See disclaimer on cover page.

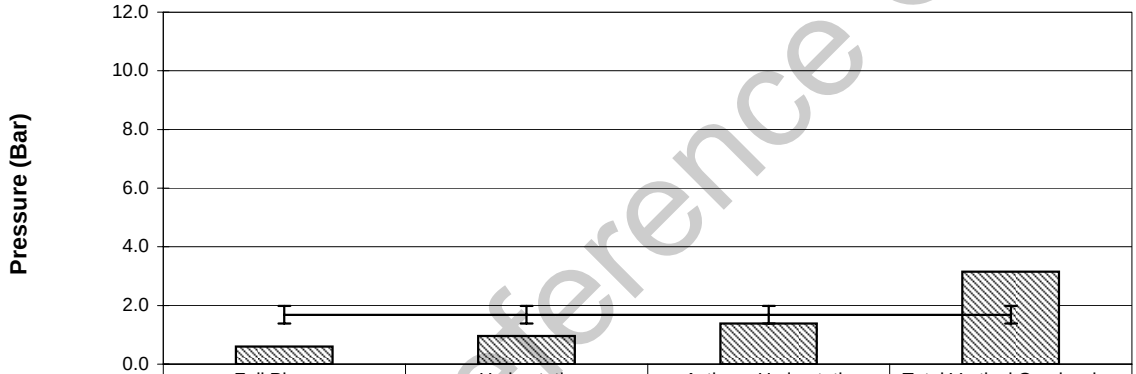
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-10 (Sta. 624+90)



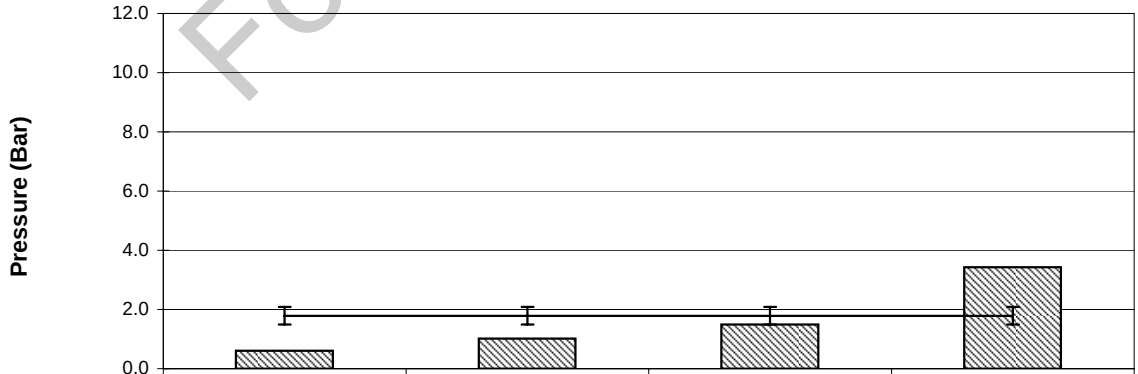
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.9	1.3	3.1
Target Pressure, bar	1.6	1.6	1.6	1.6

BH-11 (Sta. 627+54)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.0	1.4	3.2
Target Pressure, bar	1.7	1.7	1.7	1.7

BH-12 (Sta. 634+68)



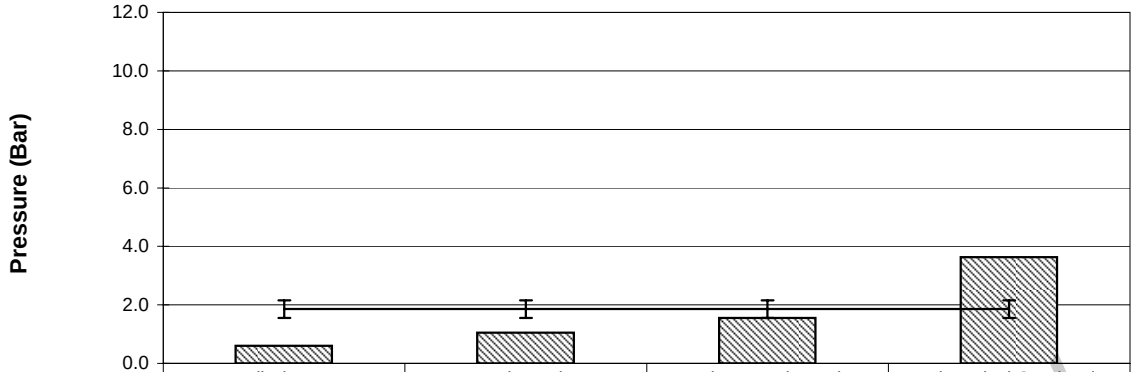
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.0	1.5	3.4
Target Pressure, bar	1.8	1.8	1.8	1.8

For Reference Only

See disclaimer on cover page.

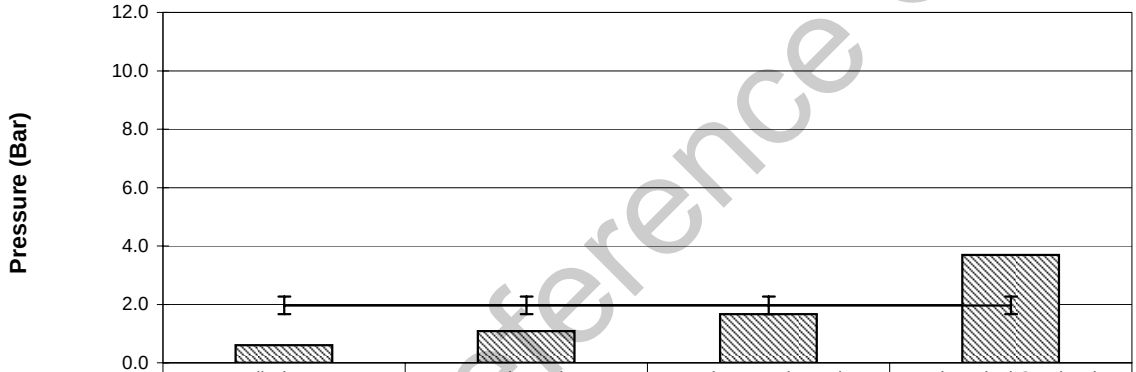
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-13 (Sta. 640+81)



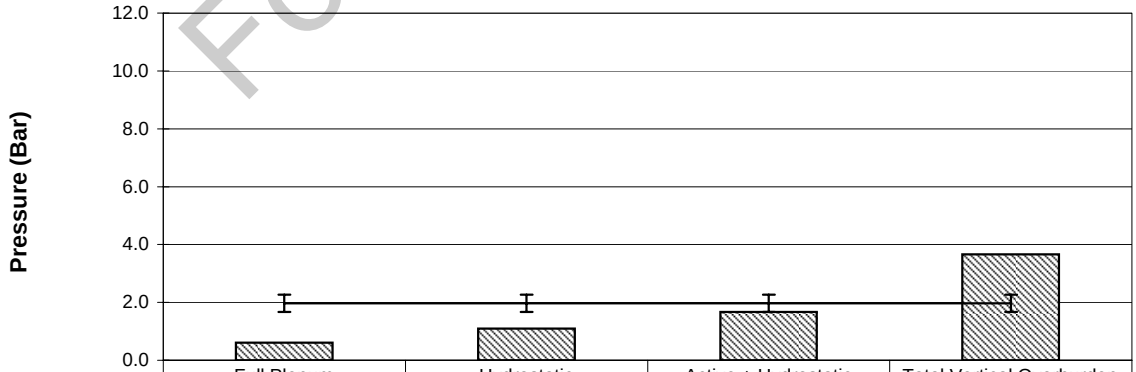
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.6	3.6
Target Pressure, bar	1.9	1.9	1.9	1.9

BH-14 (Sta. 642+52)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.7	3.7
Target Pressure, bar	2.0	2.0	2.0	2.0

BH-88 (Sta. 645+04)



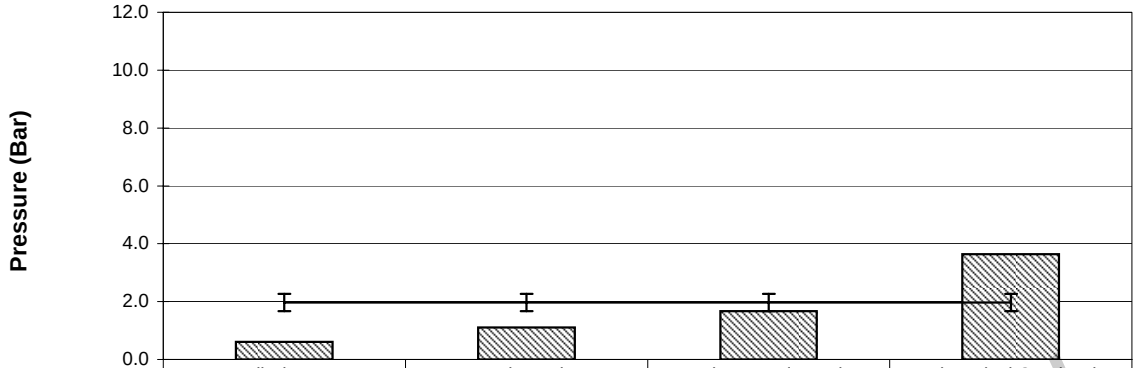
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.7	3.7
Target Pressure, bar	2.0	2.0	2.0	2.0

For Reference Only

See disclaimer on cover page.

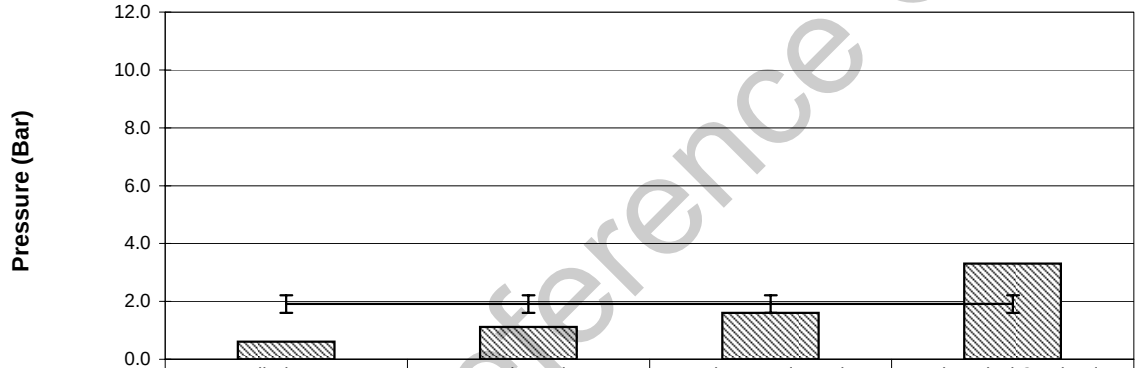
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-15 (Sta. 645+69)



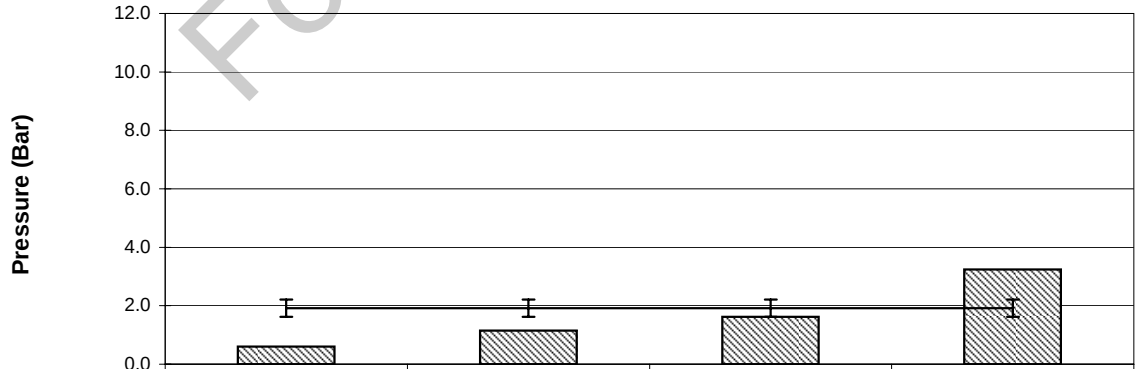
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.7	3.6
Target Pressure, bar	2.0	2.0	2.0	2.0

BH-87 (Sta. 648+33)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.6	3.3
Target Pressure, bar	1.9	1.9	1.9	1.9

BH-16 (Sta. 650+32)



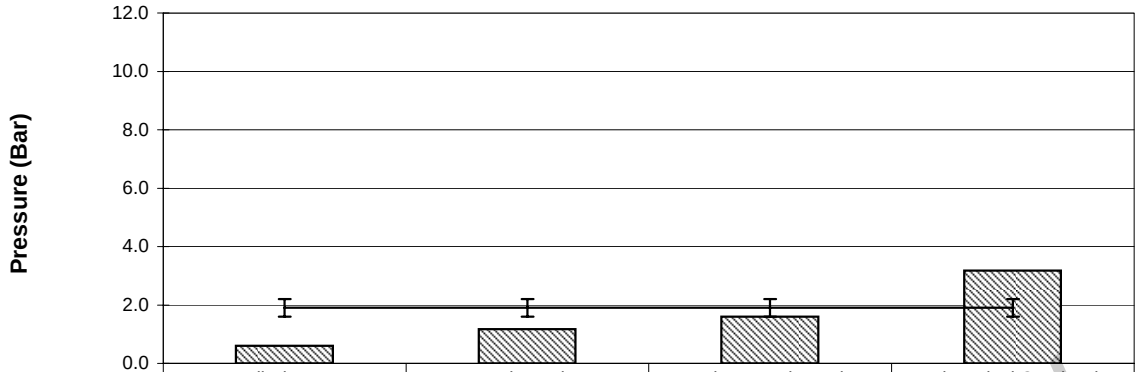
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.6	3.2
Target Pressure, bar	1.9	1.9	1.9	1.9

For Reference Only

See disclaimer on cover page.

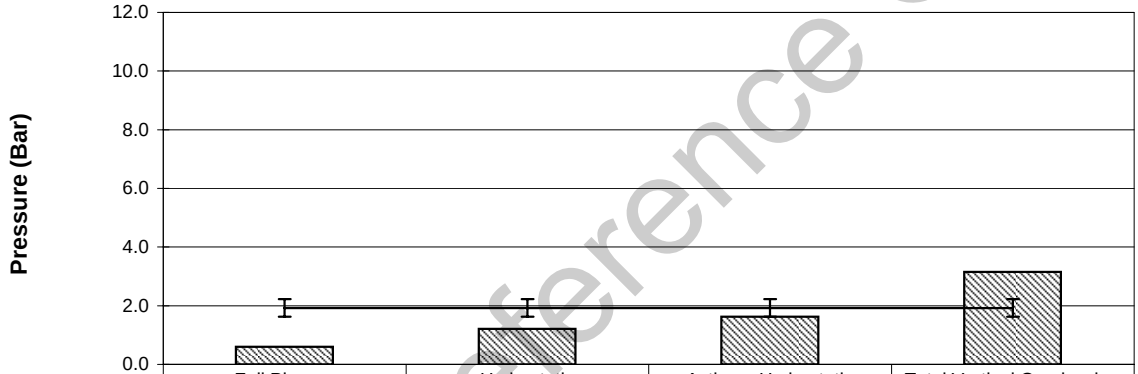
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-17 (Sta. 654+44)



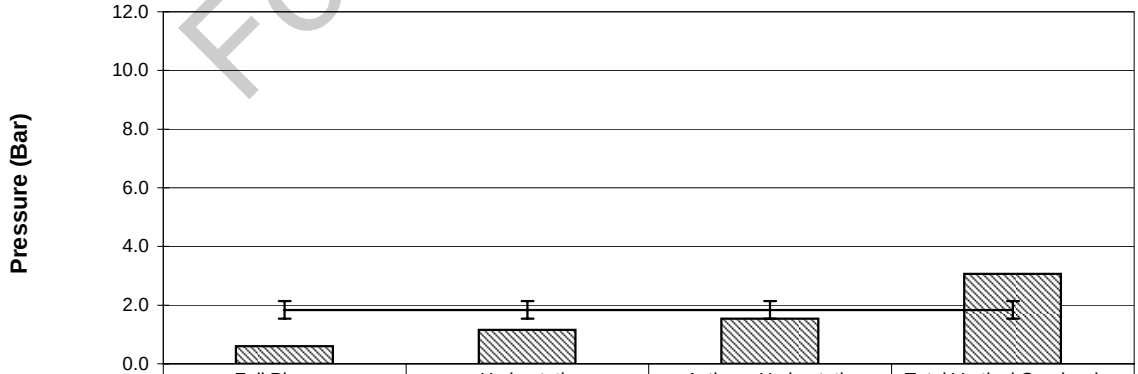
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.6	3.2
Target Pressure, bar	1.9	1.9	1.9	1.9

BH-18 (Sta. 660+03)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.6	3.2
Target Pressure, bar	1.9	1.9	1.9	1.9

BH-19 (Sta. 666+26)



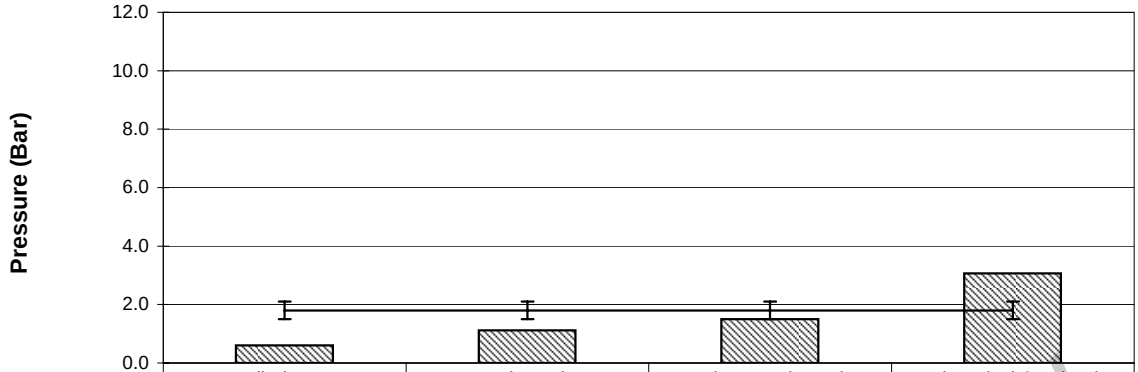
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.5	3.1
Target Pressure, bar	1.8	1.8	1.8	1.8

For Reference Only

See disclaimer on cover page.

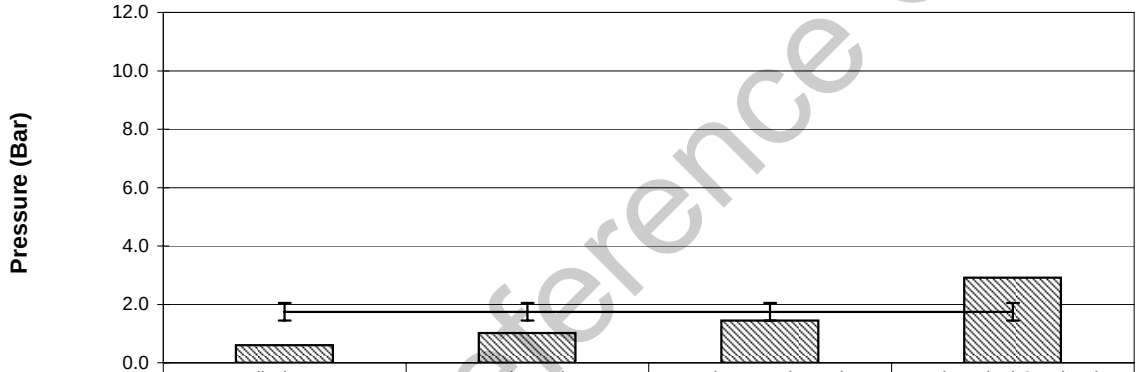
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-20 (Sta. 669+79)



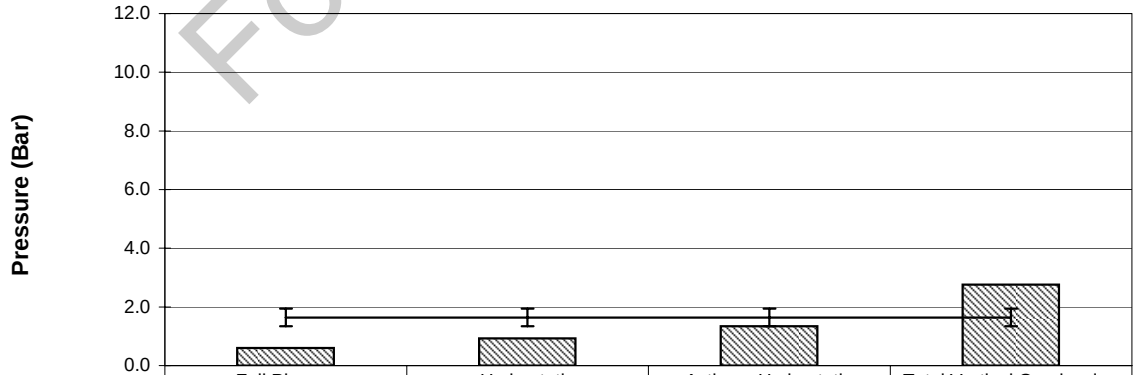
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.5	3.1
Target Pressure, bar	1.8	1.8	1.8	1.8

BH-21 (Sta. 675+49)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.0	1.4	2.9
Target Pressure, bar	1.7	1.7	1.7	1.7

BH-50 (Sta. 681+71)



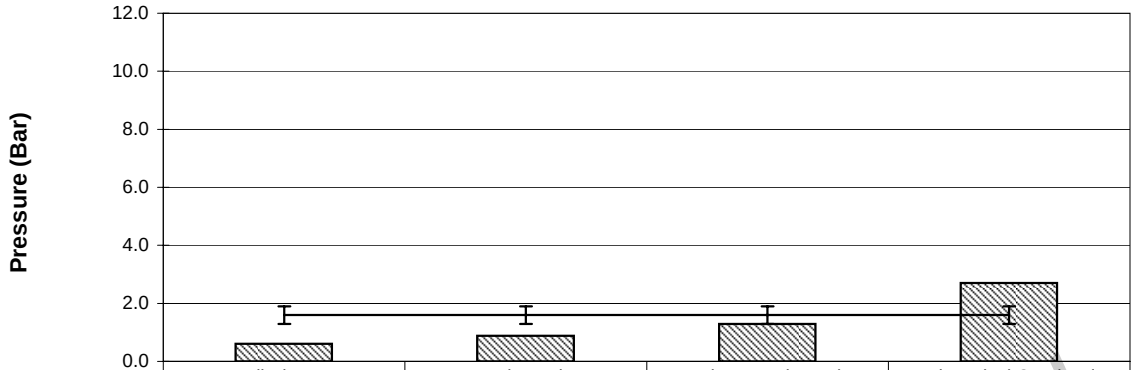
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.9	1.3	2.8
Target Pressure, bar	1.6	1.6	1.6	1.6

For Reference Only

See disclaimer on cover page.

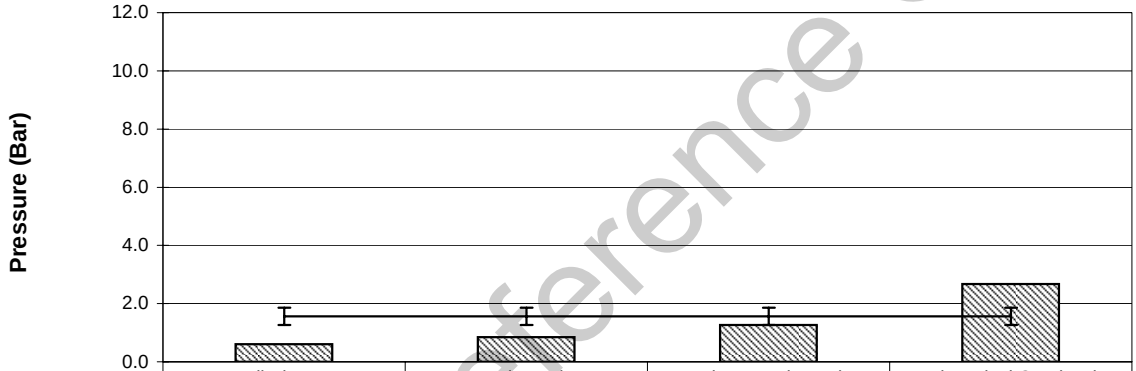
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-52 (Sta. 684+09)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.9	1.3	2.7
Target Pressure, bar	1.6	1.6	1.6	1.6

BH-53 (Sta. 685+42)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.9	1.3	2.7
Target Pressure, bar	1.6	1.6	1.6	1.6

BH-54 (Sta. 687+16)

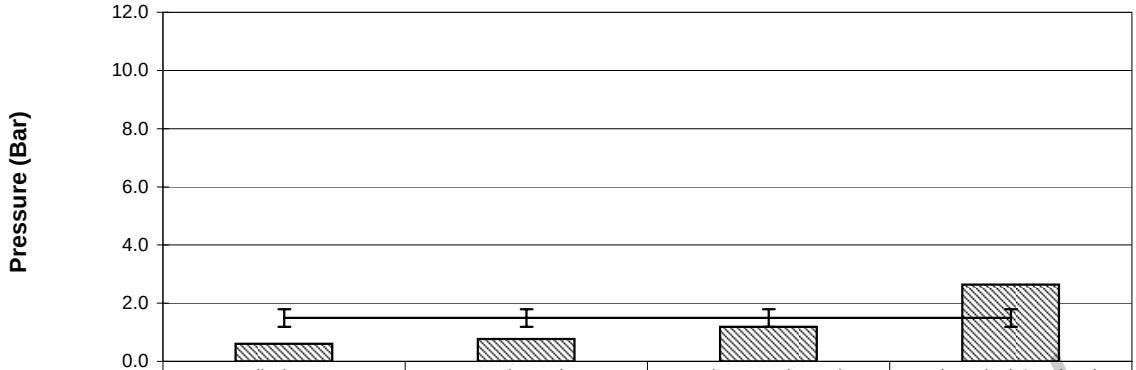
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.8	1.2	2.6
Target Pressure, bar	1.5	1.5	1.5	1.5

For Reference Only

See disclaimer on cover page.

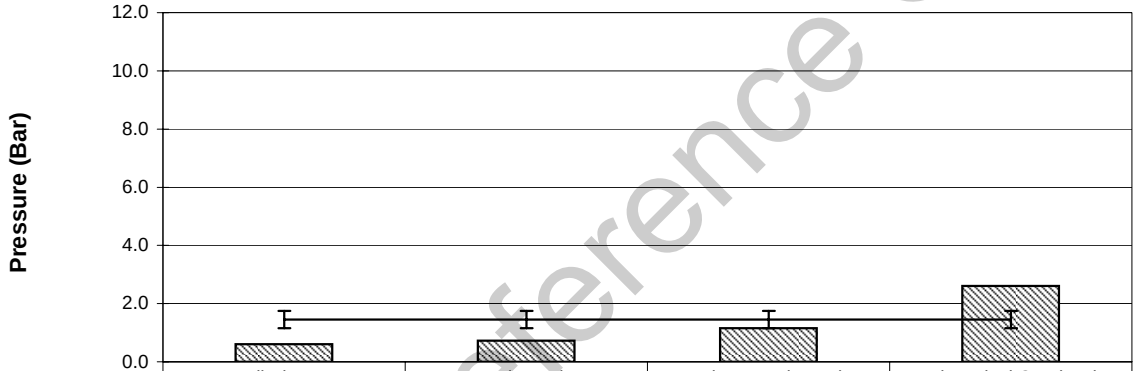
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-55 (Sta. 688+28)



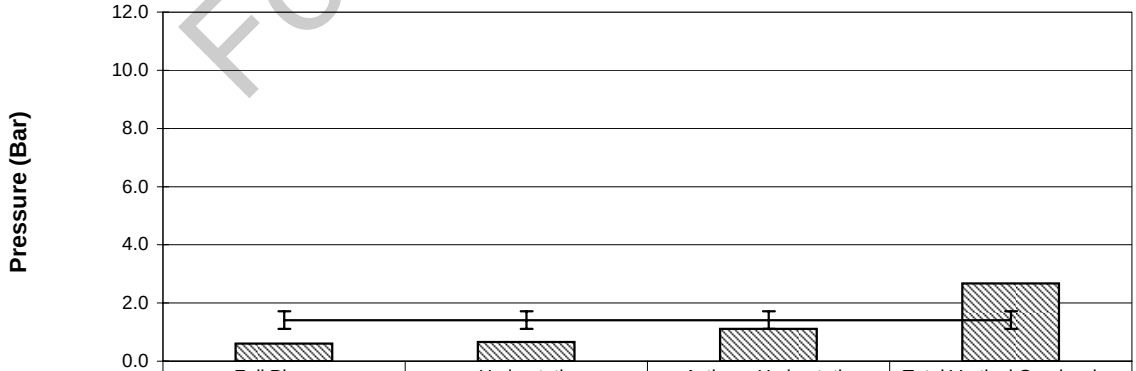
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.8	1.2	2.6
Target Pressure, bar	1.5	1.5	1.5	1.5

BH-23 (Sta. 690+02)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.7	1.1	2.6
Target Pressure, bar	1.4	1.4	1.4	1.4

BH-64 (Sta. 691+92)



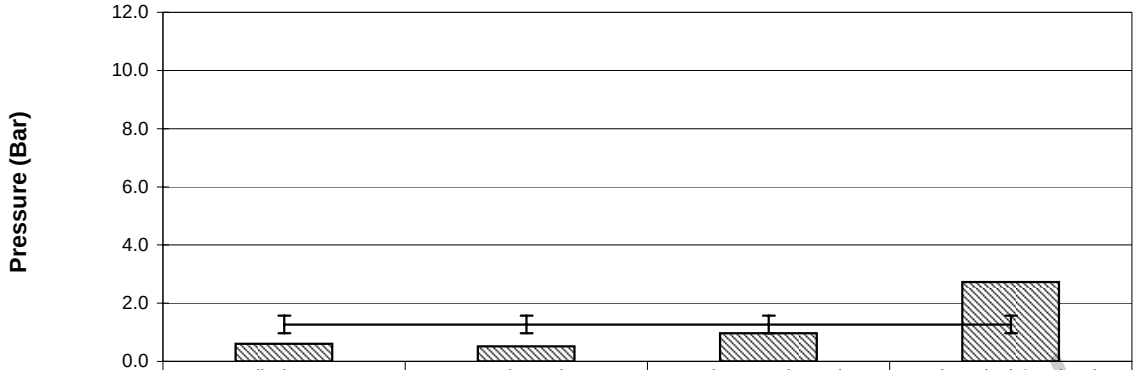
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.7	1.1	2.7
Target Pressure, bar	1.4	1.4	1.4	1.4

For Reference Only

See disclaimer on cover page.

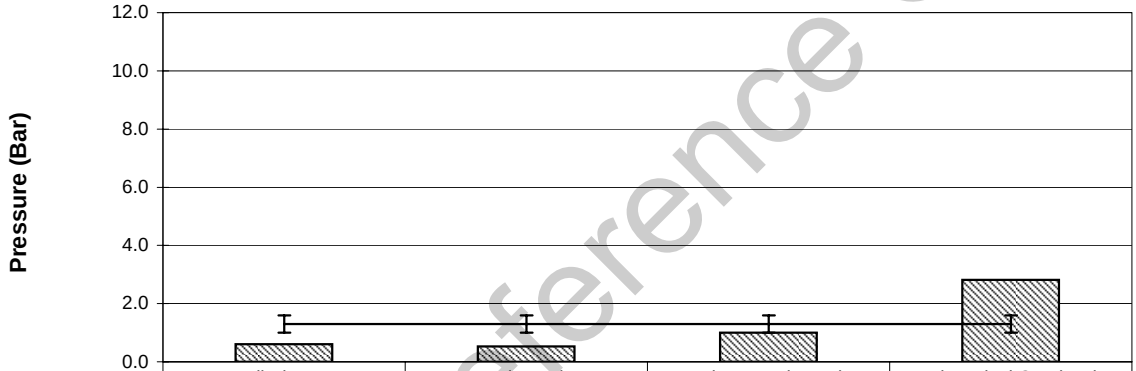
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-70 (Sta. 706+78)



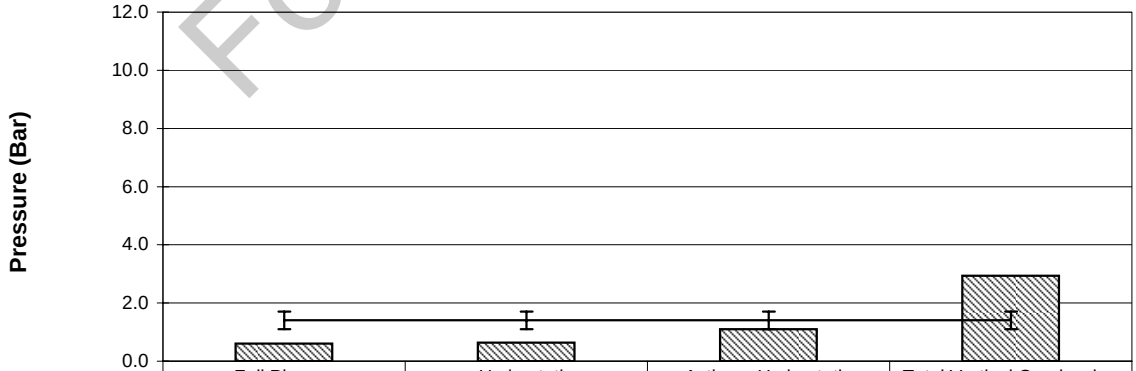
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.5	1.0	2.7
Target Pressure, bar	1.3	1.3	1.3	1.3

BH-71 (Sta. 707+62)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.5	1.0	2.8
Target Pressure, bar	1.3	1.3	1.3	1.3

BH-72 (Sta. 709+40)



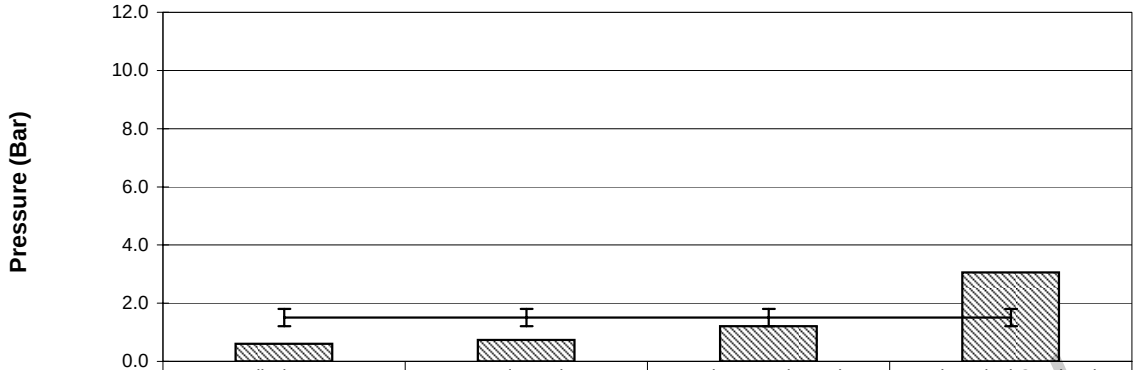
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.6	1.1	2.9
Target Pressure, bar	1.4	1.4	1.4	1.4

For Reference Only

See disclaimer on cover page.

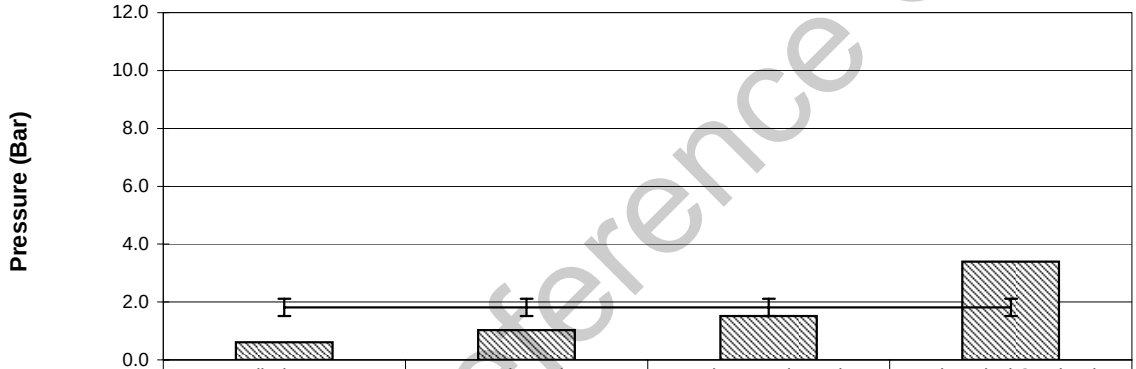
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-26 (Sta. 710+70)



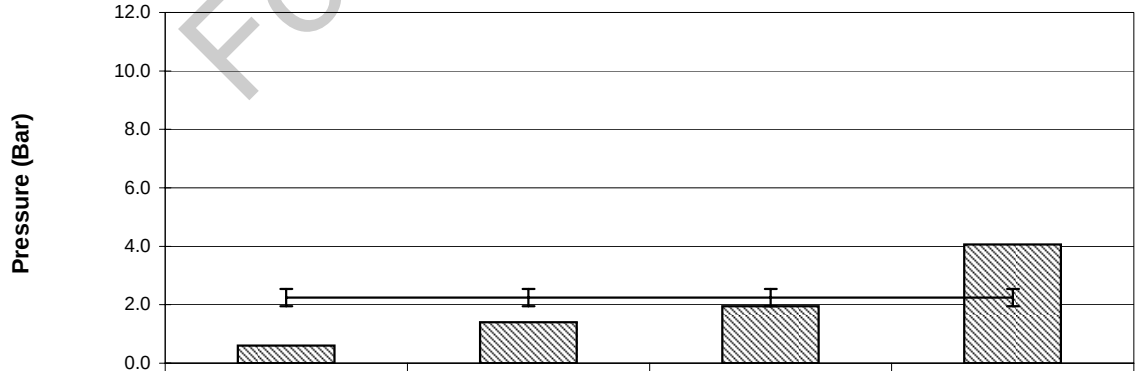
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.7	1.2	3.1
Target Pressure, bar	1.5	1.5	1.5	1.5

BH-27 (Sta. 715+00)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.0	1.5	3.4
Target Pressure, bar	1.8	1.8	1.8	1.8

BH-28 (Sta. 720+22)



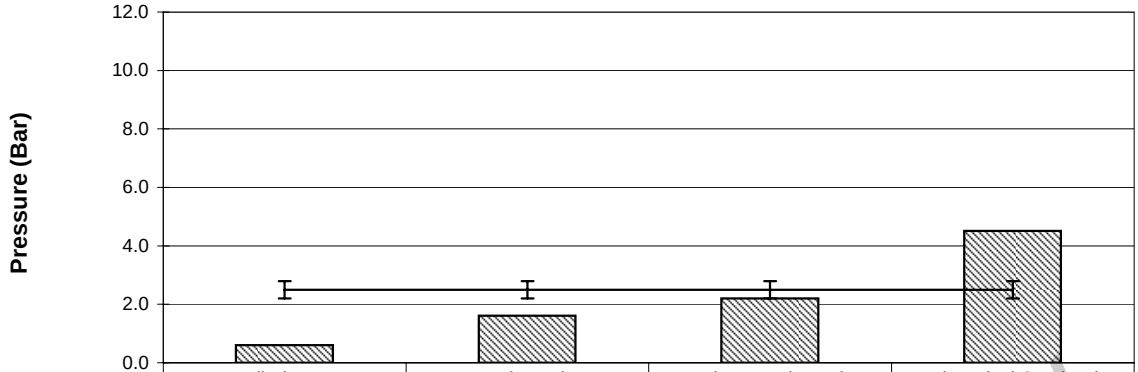
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.4	1.9	4.1
Target Pressure, bar	2.2	2.2	2.2	2.2

For Reference Only

See disclaimer on cover page.

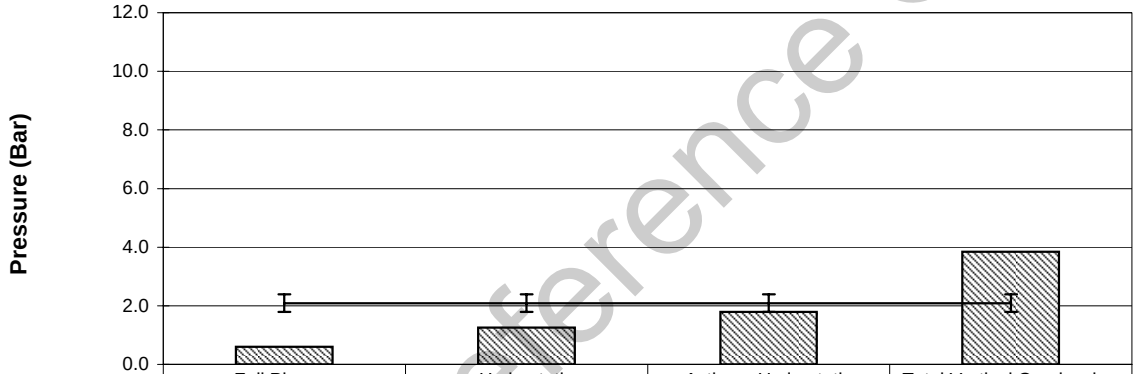
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-29 (Sta. 723+88)



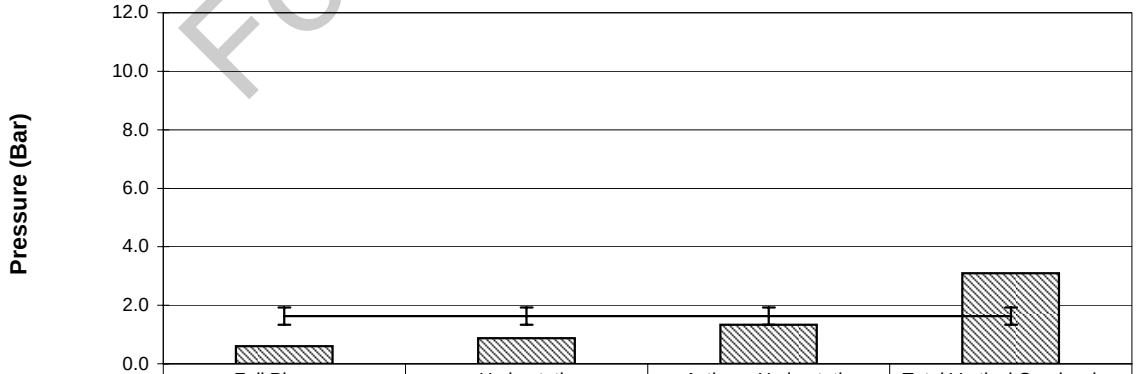
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.6	2.2	4.5
Target Pressure, bar	2.5	2.5	2.5	2.5

BH-30 (Sta. 728+01)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.8	3.8
Target Pressure, bar	2.1	2.1	2.1	2.1

BH-31 (Sta. 731+54)



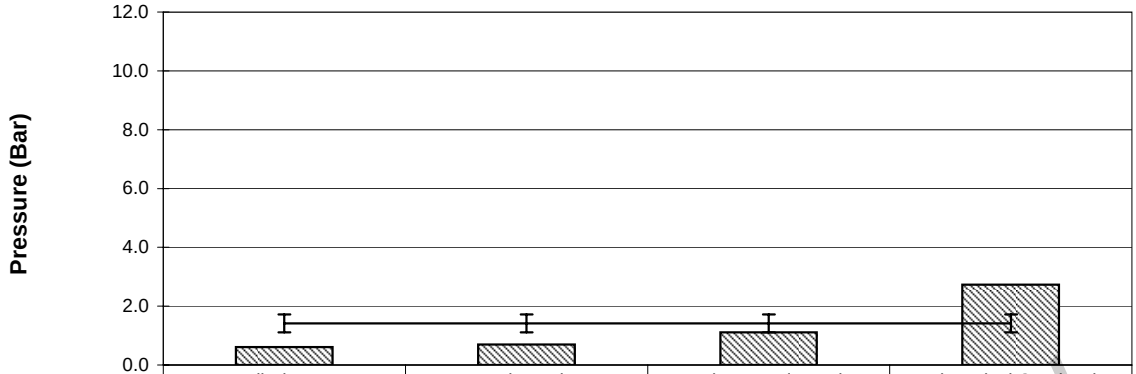
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.9	1.3	3.1
Target Pressure, bar	1.6	1.6	1.6	1.6

For Reference Only

See disclaimer on cover page.

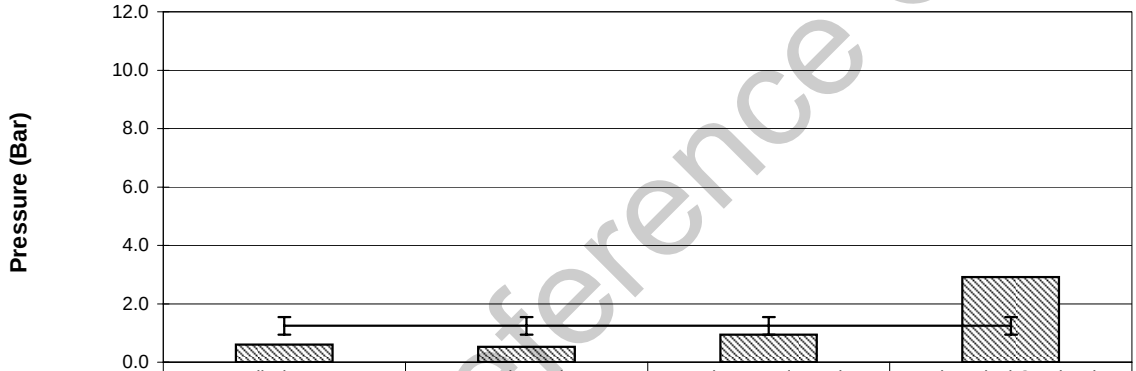
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-32 (Sta. 733+30)



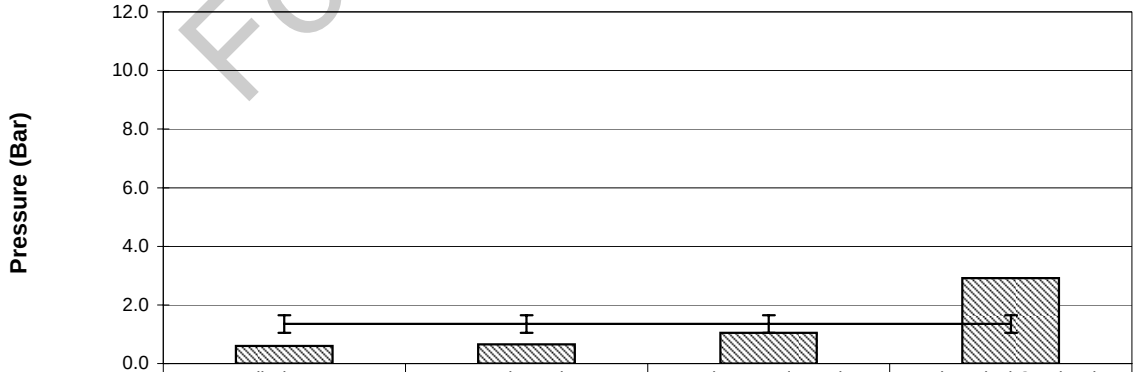
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.7	1.1	2.7
Target Pressure, bar	1.4	1.4	1.4	1.4

BH-34 (Sta. 744+65)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.5	0.9	2.9
Target Pressure, bar	1.2	1.2	1.2	1.2

BH-35 (Sta. 750+48)



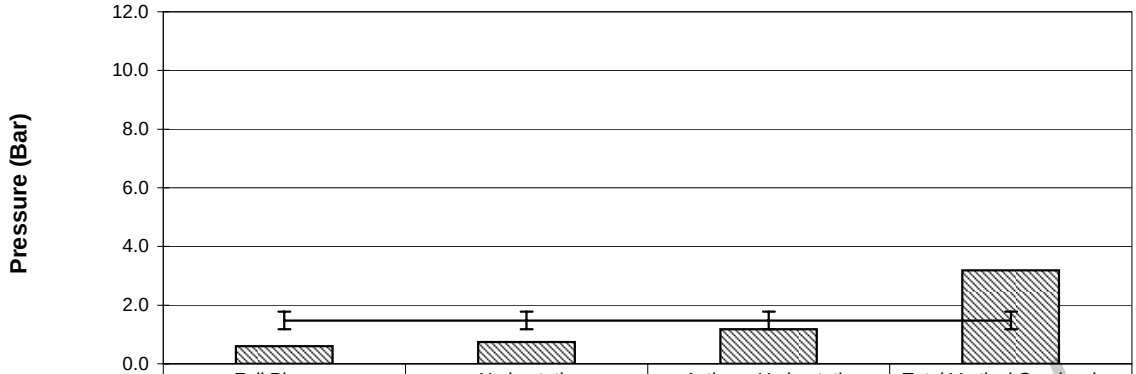
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.6	1.0	2.9
Target Pressure, bar	1.3	1.3	1.3	1.3

For Reference Only

See disclaimer on cover page.

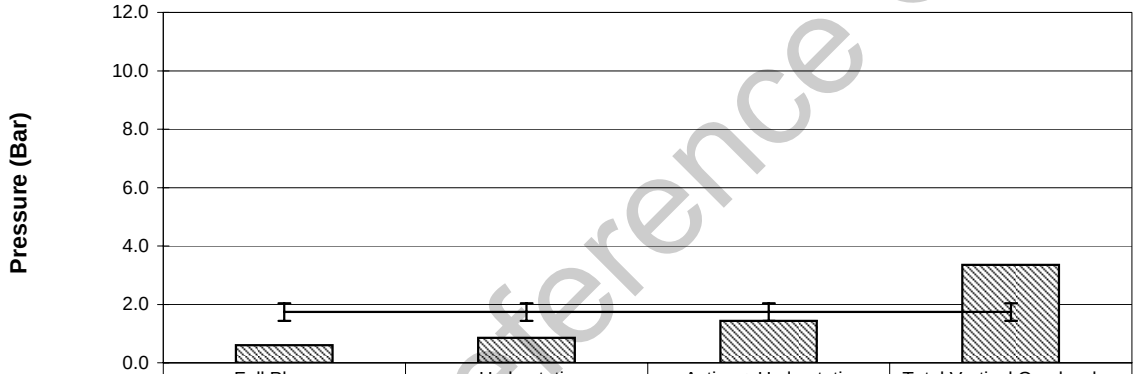
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-36 (Sta. 755+33)



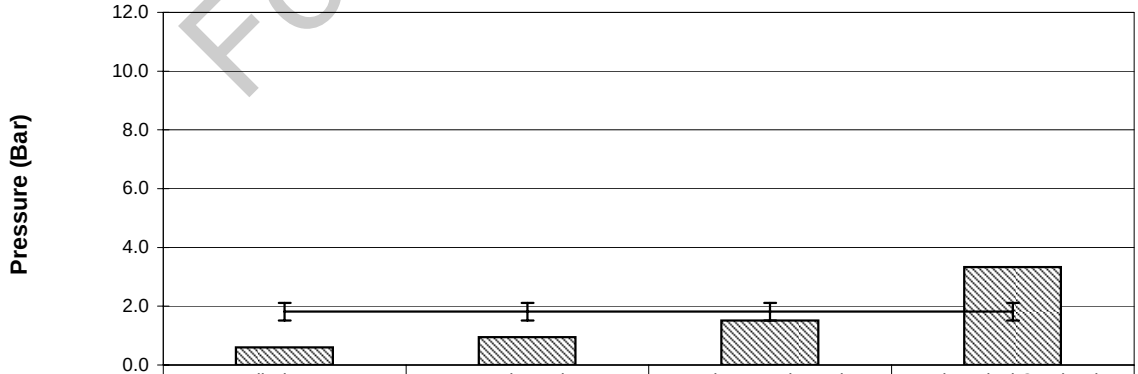
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.7	1.2	3.2
Target Pressure, bar	1.5	1.5	1.5	1.5

BH-37 (Sta. 760+59)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.9	1.4	3.4
Target Pressure, bar	1.7	1.7	1.7	1.7

BH-38 (Sta. 765+23)



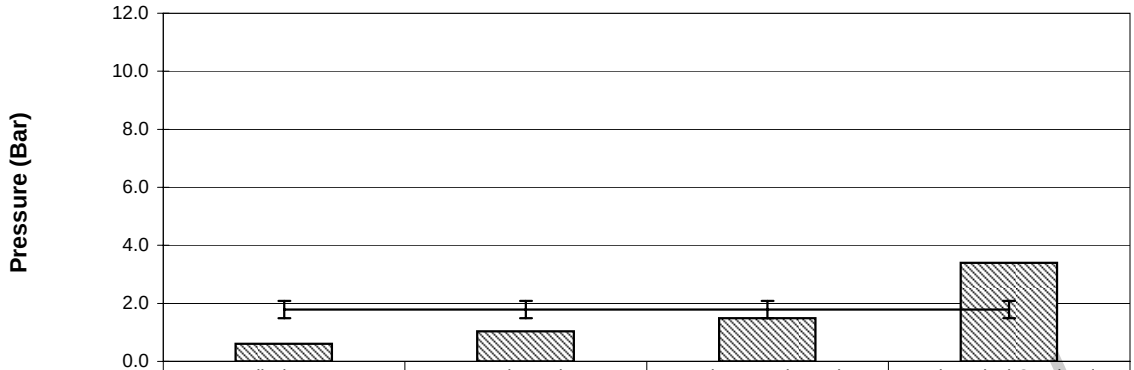
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.9	1.5	3.3
Target Pressure, bar	1.8	1.8	1.8	1.8

For Reference Only

See disclaimer on cover page.

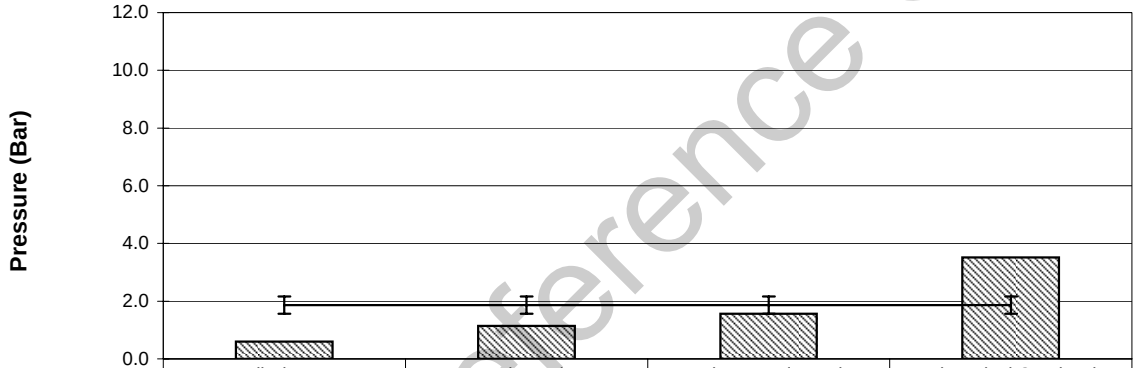
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-39 (Sta. 768+77)



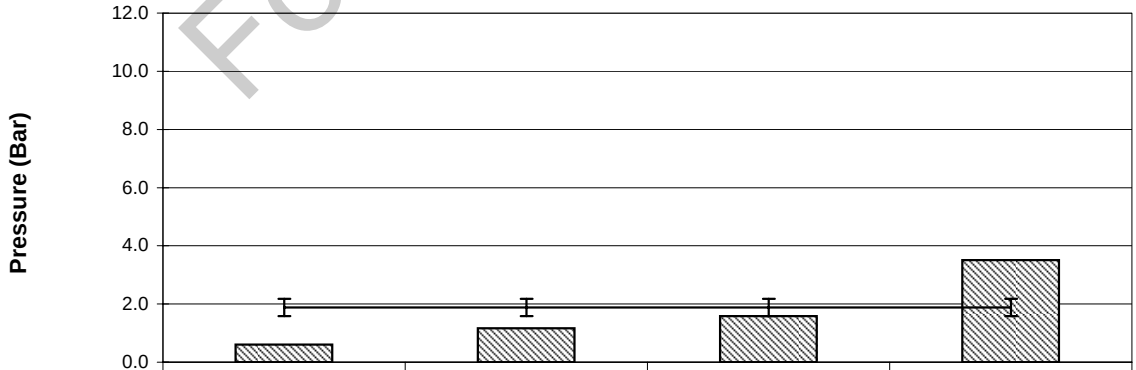
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.0	1.5	3.4
Target Pressure, bar	1.8	1.8	1.8	1.8

BH-95 (Sta. 774+13)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.6	3.5
Target Pressure, bar	1.9	1.9	1.9	1.9

BH-40 (Sta. 775+76)



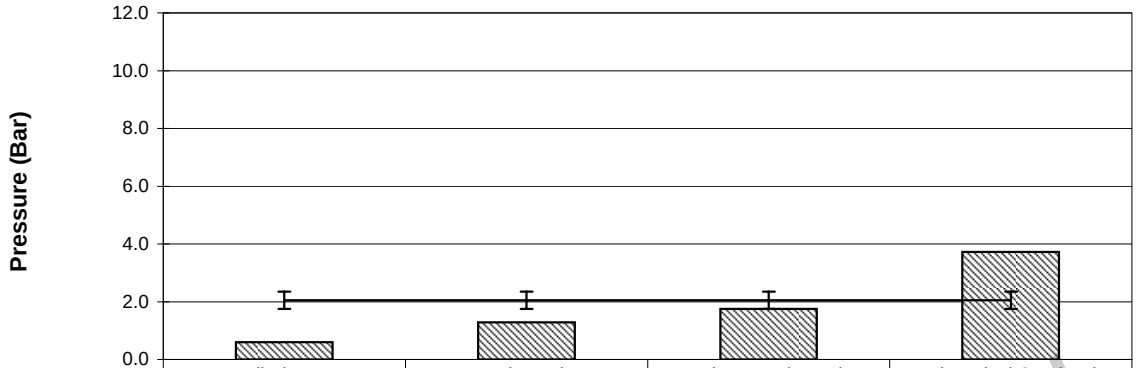
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.6	3.5
Target Pressure, bar	1.9	1.9	1.9	1.9

For Reference Only

See disclaimer on cover page.

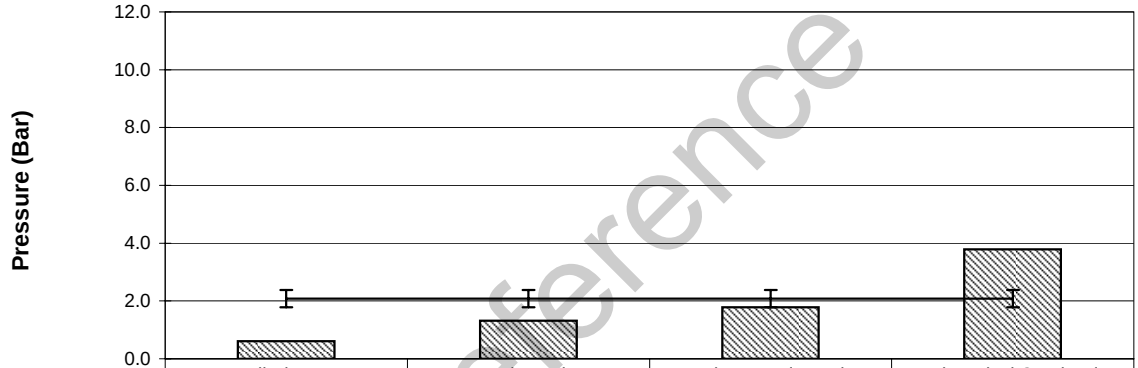
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-41 (Sta. 781+35)



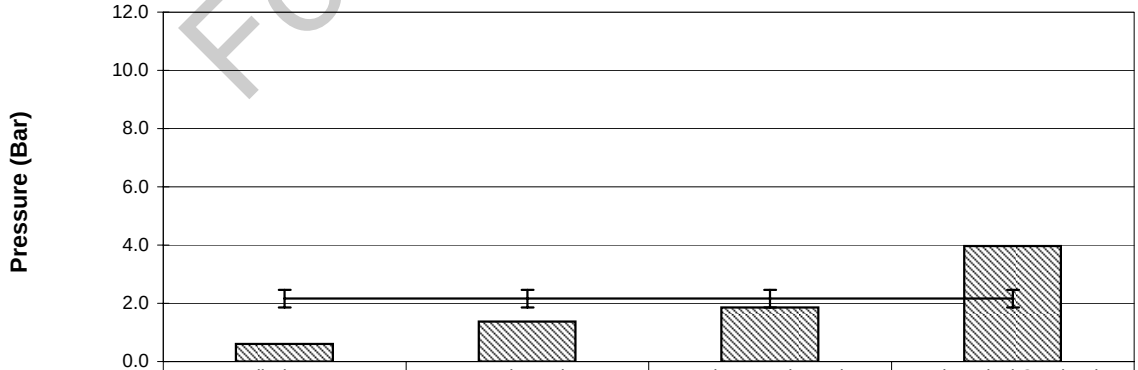
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.8	3.7
Target Pressure, bar	2.1	2.1	2.1	2.1

BH-79 (Sta. 782+50)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.8	3.8
Target Pressure, bar	2.1	2.1	2.1	2.1

BH-42 (Sta. 785+37)



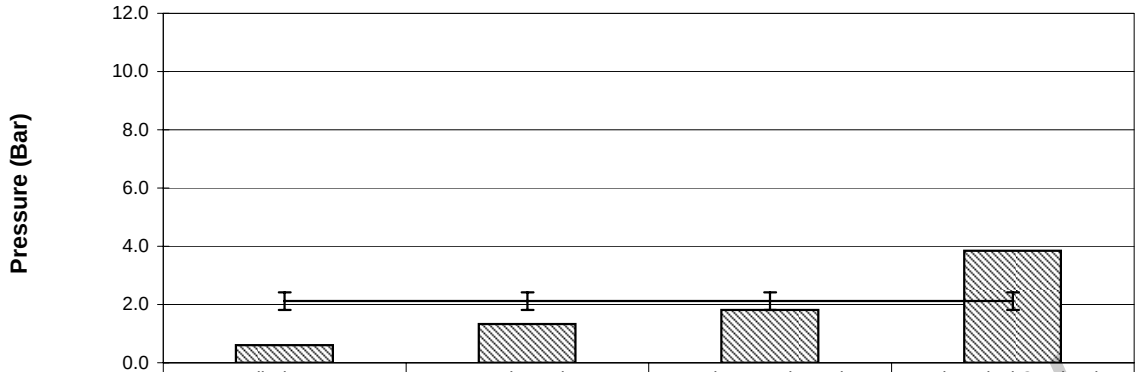
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.4	1.9	4.0
Target Pressure, bar	2.2	2.2	2.2	2.2

For Reference Only

See disclaimer on cover page.

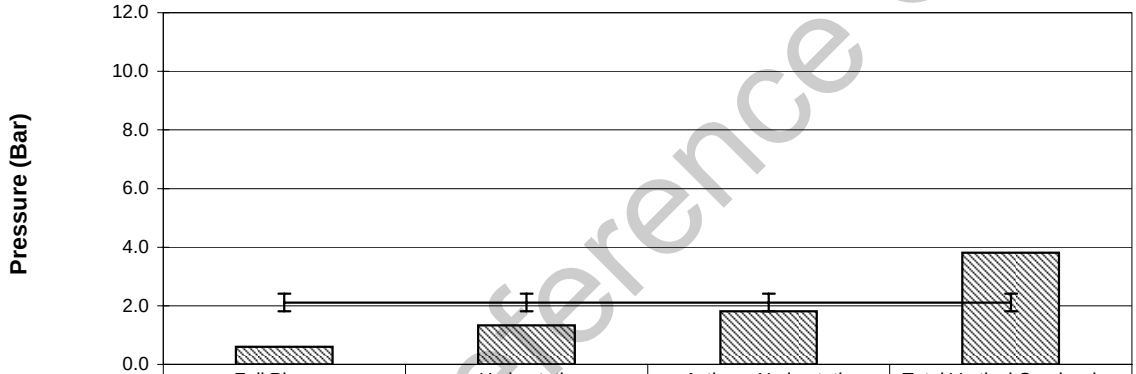
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-81 (Sta. 789+50)



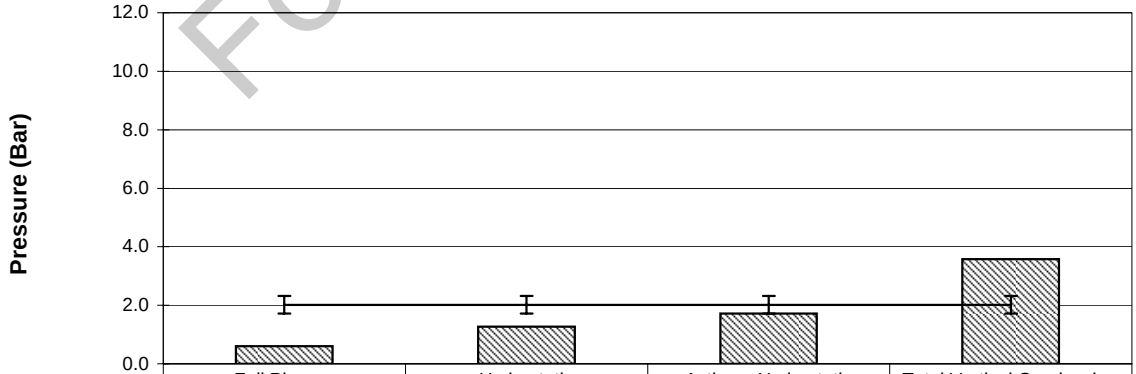
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.8	3.8
Target Pressure, bar	2.1	2.1	2.1	2.1

BH-43 (Sta. 789+71)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.8	3.8
Target Pressure, bar	2.1	2.1	2.1	2.1

BH-80 (Sta. 794+39)



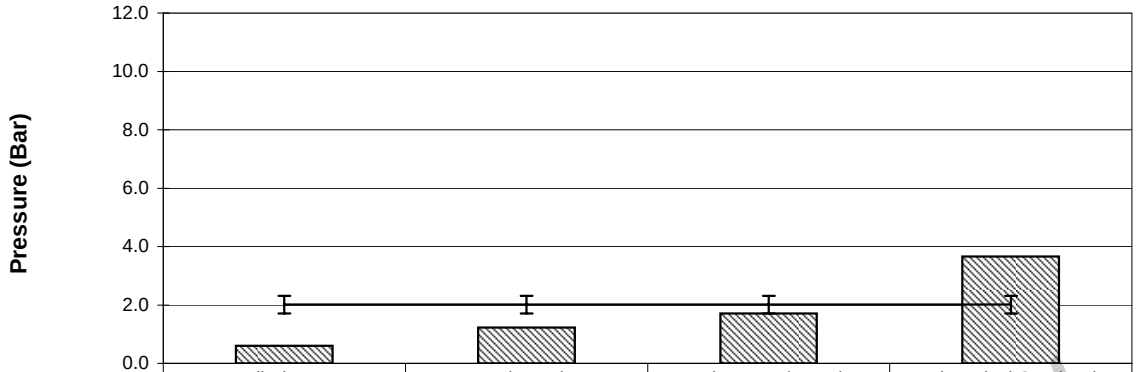
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.3	1.7	3.6
Target Pressure, bar	2.0	2.0	2.0	2.0

For Reference Only

See disclaimer on cover page.

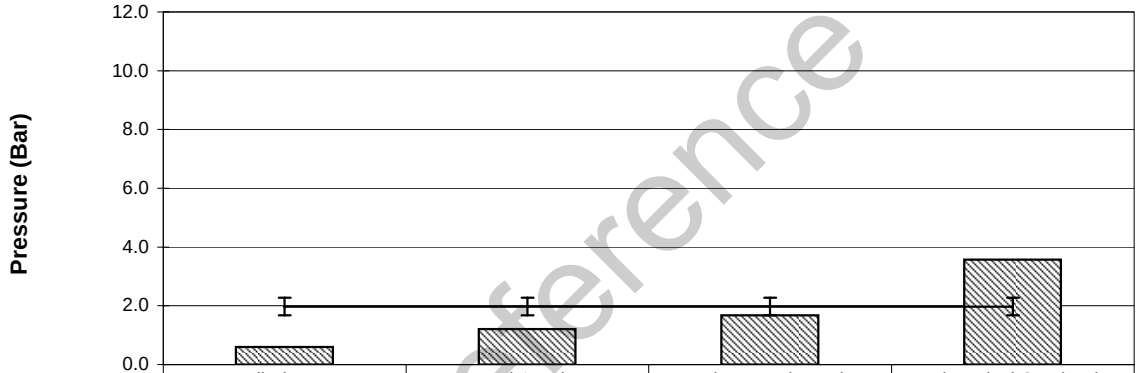
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-102 (Sta. 796+48)



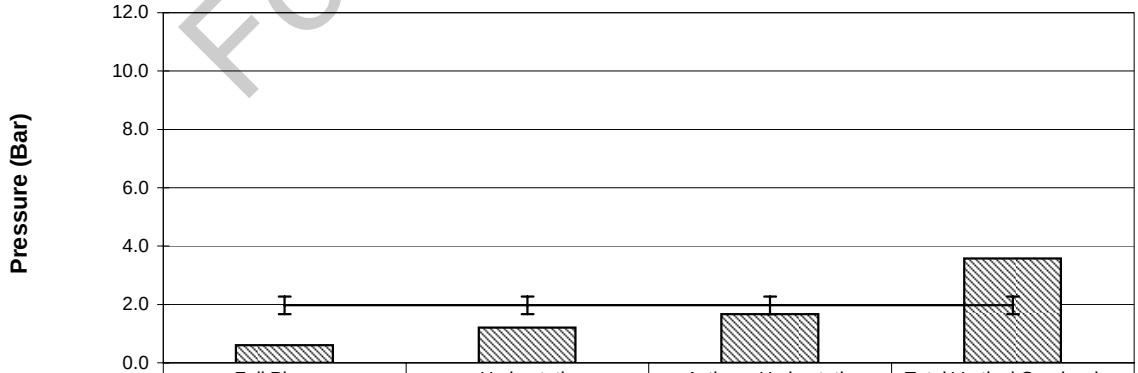
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.7	3.7
Target Pressure, bar	2.0	2.0	2.0	2.0

BH-103 (Sta. 798+16)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.7	3.6
Target Pressure, bar	2.0	2.0	2.0	2.0

BH-44 (Sta. 798+27)



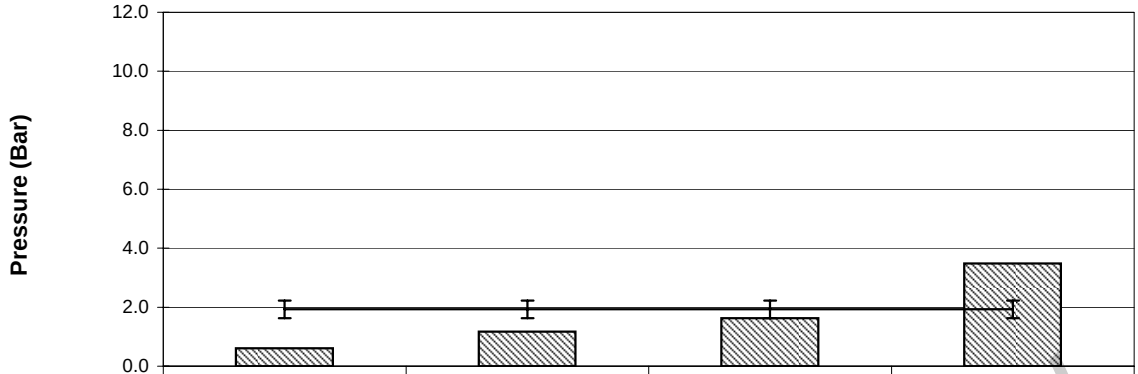
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.7	3.6
Target Pressure, bar	2.0	2.0	2.0	2.0

For Reference Only

See disclaimer on cover page.

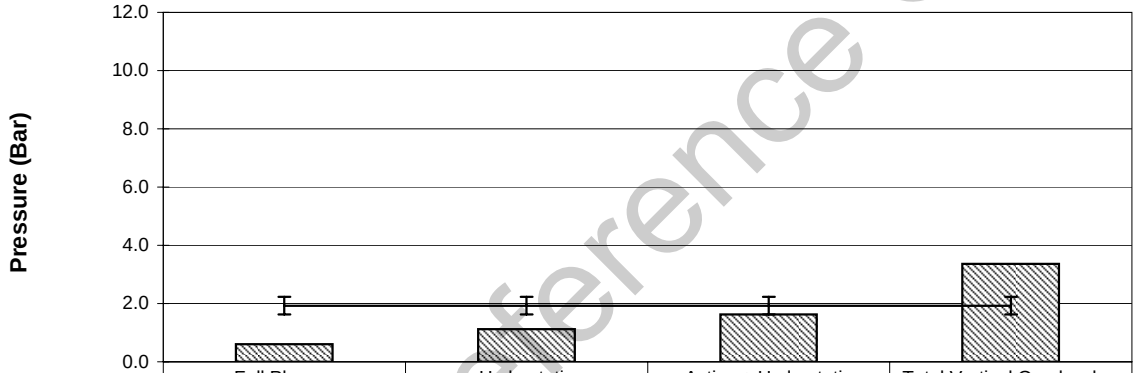
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-106 (Sta. 800+20)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.2	1.6	3.5
Target Pressure, bar	1.9	1.9	1.9	1.9

BH-45 (Sta. 802+44)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.1	1.6	3.4
Target Pressure, bar	1.9	1.9	1.9	1.9

BH-46 (Sta. 809+36)

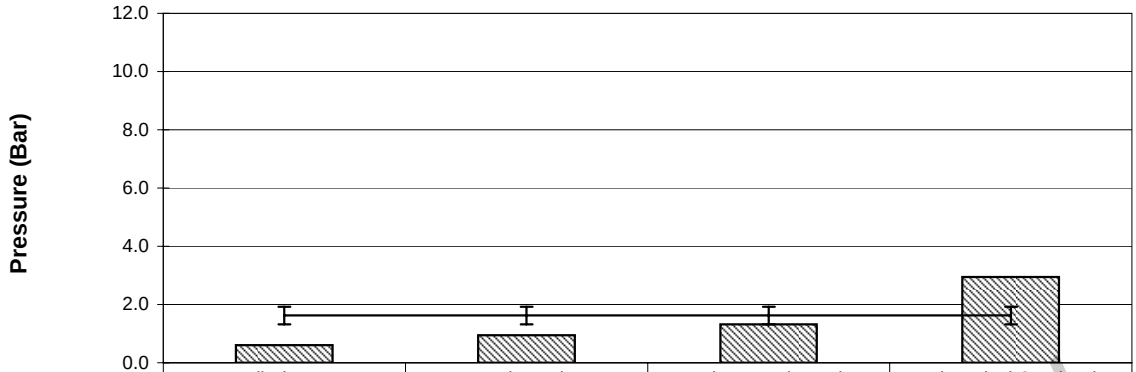
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	1.0	1.5	3.1
Target Pressure, bar	1.8	1.8	1.8	1.8

For Reference Only

See disclaimer on cover page.

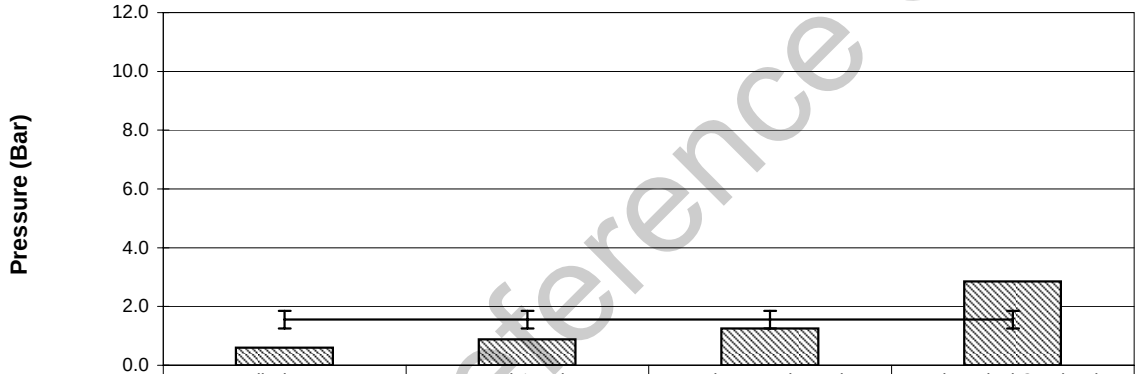
Face Pressure Charts by Borehole
(Minimum Water Level)

BH-47 (Sta. 813+52)



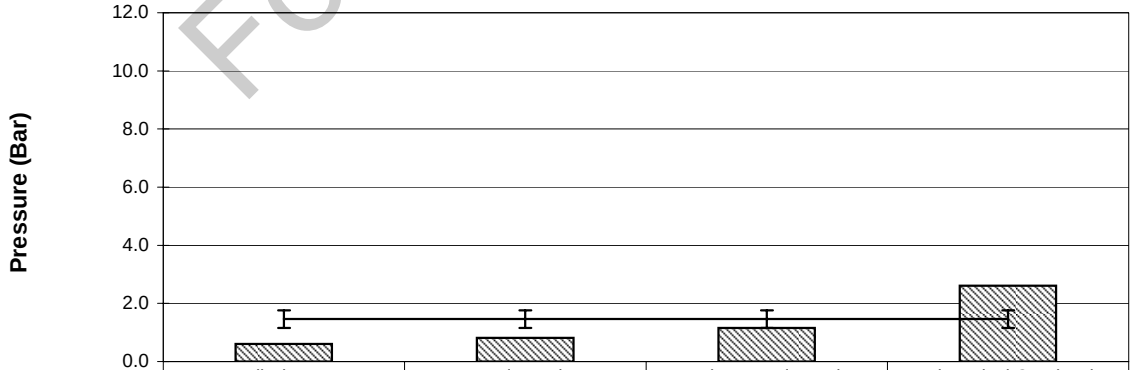
	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.9	1.3	2.9
Target Pressure, bar	1.6	1.6	1.6	1.6

BH-48 (Sta. 818+34)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.9	1.3	2.9
Target Pressure, bar	1.6	1.6	1.6	1.6

BH-49 (Sta. 824+28)

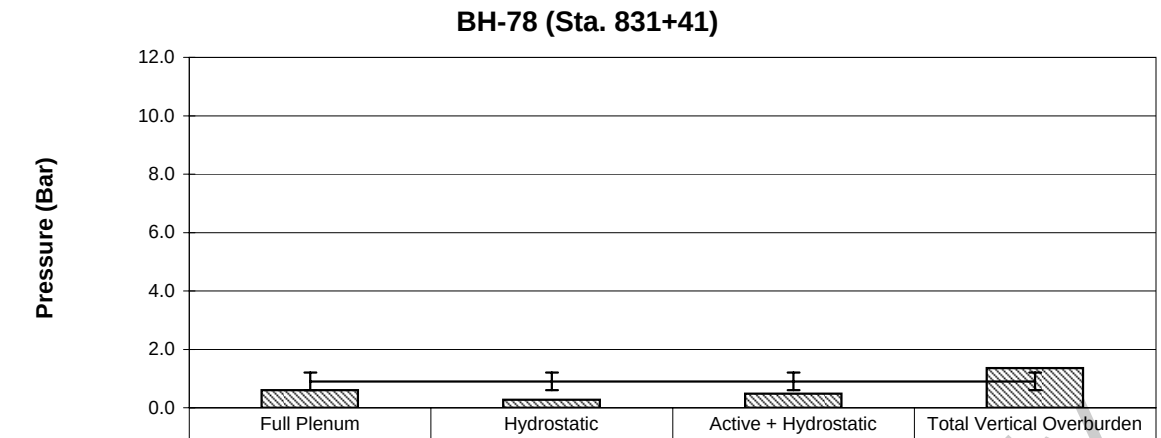


	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.8	1.2	2.6
Target Pressure, bar	1.5	1.5	1.5	1.5

For Reference Only

See disclaimer on cover page.

Face Pressure Charts by Borehole
(Minimum Water Level)



	Full Plenum	Hydrostatic	Active + Hydrostatic	Total Vertical Overburden
Pressure, bar	0.6	0.3	0.5	1.4
Target Pressure, bar	0.9	0.9	0.9	0.9

For Reference Only

See disclaimer on cover page.

Facility/Subject: EPBM Face Pressure Calculations		Prepared by: A. Dean / N. Yu	Date: 08-01-08	Discipline: Tunnel / Civil
Component: TBM GDM		Checked by: D. Young	Date: 12-26-08	
Calculation No: **SBAC19**	Rev. No. **16**			Sheet No.: 1 of 65

Purpose

To estimate target operating pressures for closed-face tunnel boring during normal operations while excavating. These face pressures only apply when TBM excavation and are not indicative of face pressures that may be experienced during delays or work stoppages. Note that since actual soils indicated by borings rather than sands are used for this calculation, this calculation may not be representative of target pressures predicted by the contractor. The Contractor is required to assume the possibility of flowing sand layers throughout. See SBAC60 which assumes sand throughout for comparison.

Method

Estimating face pressures involves a four step process:

1. Calculating and estimating possible upper bound and lower bound face pressures based on soil parameters, hydrogeologic conditions including minimum and maximum expected water table levels during construction, TBM parameters, and ground stability considerations.

2. Establishing controlling upper and lower bound values for a given location.

3. Considering accuracy with which face pressures can be controlled. The face pressure control accuracy is used to estimate the tolerance in operating pressure used in the calculations.

4. Based on the results of the previous steps, establishing the target operating pressures as low as possible while maintaining soil stability. The operating range is the target plus and minus the tolerance. Soil stability was checked against both settlement and heave within the operating range.

Step 1 – Calculations and Estimation of possible upper and lower bound values

The following conditions were considered:

1a. Hydrostatic, including minimum and maximum water table levels anticipated during construction (HMMB, 2008a) and annulus grout pressures, lower bound,

1b. Active lateral earth pressure, lower bound,

For Reference Only

See disclaimer on cover page.

1c. Pressure to maintain a fully charged plenum, lower bound,
1d. At-rest lateral earth pressure, for information,
1e. Vertical overburden pressure, upper bound for sandy soils and fissured cohesive soils[1],
1f. Passive lateral earth pressure, upper bound for non-fissured cohesive soils[2],
1g. Stability ratio assessment, a stability ratio of 4 or less was required for unsupported cohesive materials.

All calculations are shown on the attached spreadsheet and graph.

Step 1a: Hydrostatic and Annulus Grout Pressure

Hydrostatic pressure was estimated using the maximum and minimum construction water table levels shown on profile drawings in the Hydrogeology Report (HMMB, 2008a). The tunnel profile is based on the SVRT April 2008 quarterly update (Rev.G).

Annulus grout pressures are commonly limited in construction specifications to a maximum on the order of 1.1 times the hydrostatic pressure. A maximum pressure difference of 2 psi (0.14 bar) between the annulus grout pressure and hydrostatic pressure has also been suggested.

Face pressures higher than annulus grout pressures minimize the risk of grout migration into the cutterhead and through the TBM. Given the length of the tunnels and the fact that maximum grout pressures can be controlled relatively accurately, grout pressures were estimated for this project using a factor of 1.1 applied to the hydrostatic pressure, with a maximum of 2 psi. This grout pressure rather than the hydrostatic pressure was considered to be a possible lower bound face pressure value.

[1] Pressures above overburden have resulted in surface heave in EPBM tunnels, for example the LNWI Southern Sacramento River Crossing, completed in 2005 (LNWI, 2006).

[2] Mobilizing the passive pressure involves soil deflection, which could cause heave in shallow tunnels. Thus, the upper bound for non-fissured cohesive soil should be set at a value less than passive pressure. This upper bound value depends on the method of estimating passive pressure, and has not been evaluated in this calculation.

For Reference Only

See disclaimer on cover page.

Step 1b: Active Earth Pressure

Active earth pressures were calculated based on the pressure formula developed as part of Rankine's Earth Pressure Theory presented by Rankine for granular materials:

$P_a = K_a \gamma_t H$, where

P_a = Active earth pressure
K_a = Active earth pressure coefficient. This parameter is calculated as described in the "Assumptions" section below.
γ_t = Total unit weight of soil
H = Depth below ground surface

To estimate the active earth pressure plus hydrostatic pressure in granular soil, buoyant unit weights were used and then hydrostatic pressure was added.

$P_{a+u} = K_a \gamma_b H + u$, where

P_{a+u} = Effective active earth pressure plus hydrostatic pressure
γ_b = Buoyant unit weight of soil
u = hydrostatic pressure
other terms as defined above

For cohesive materials, cohesion must be considered and the above equation is modified as follows:

$P_{a,c} = K_a \gamma H - 2c\sqrt{K_a}$, where

$P_{a,c}$ = active earth pressure for cohesive materials
c = cohesion
Other terms as defined above

Calculation of the active plus hydrostatic pressure for cohesive materials involved use of buoyant unit weights in place of total unit weights in the equation above, plus addition of the hydrostatic pressure component. The active pressure plus hydrostatic pressure was more critical than the total active earth pressure, for the purposes of establishing target face pressures.

For Reference Only

See disclaimer on cover page.

$P_{a+u,c} = K_a \gamma_b H - 2c\sqrt{K_a} + u$, where

$P_{a+u,c}$ = Active earth pressure plus hydrostatic pressure for cohesive materials
Other terms as defined above

Pressures included on the attached calculation worksheet are calculated at axis depth.

Step 1c: Pressure Needed to Maintain a Fully Charged Plenum

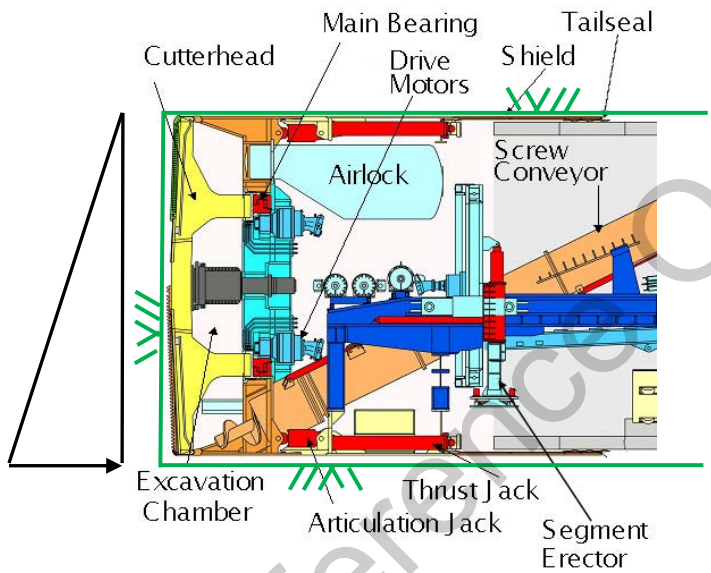
Calculations for pressure needed to maintain a fully charged plenum are shown at the top of the attached spreadsheet. These calculations were based on a fluid pressure caused by conditioned soil in a full plenum. A bulking factor was considered to account for the altered soil density of material in the excavation chamber due to remolding and the application of soil conditioners. Based on construction records from the LNWI project, bulking factors as low as 1.0 are possible (LNWI, 2006). Bulking factors as high as 1.5 can exist in clayey materials (Crow and Holzhauser, 2003, and LNWI, 2006).[3] A bulking factor of 1.0 was used for these calculations, representing a worst case density.

These calculations are based on the assumption that the tunnel boring machine diameter is 20 feet-8 inches, and that fluidized spoil exists in the excavation chamber and cutterhead tool gap for the full excavated diameter. See illustration.

For Reference Only

[3] Soil unit weight in the pressure chamber can be measured during construction by several different methods. This should be done frequently, particularly in low cover areas where the full plenum pressure could be the controlling lower bound pressure for establishing the target pressure. Soil unit weight should be re-evaluated during construction with project data.

See disclaimer on cover page.



Pressure Diagram for Plasticized Soil in Excavation Chamber and Tool Gap

For Reference Only

Step 1d: At-Rest Earth Pressure

At-rest earth pressures were calculated using the at-rest earth pressure coefficient, K_o in the following equation:

$P_o = K_o \gamma_t H$, where

P_o = Total at-rest earth pressure
K_o = At-rest earth pressure coefficient. This parameter is calculated as described in the "Assumptions" section below.
γ_t = Total unit weight of soil
H = Depth below ground surface

Calculation of the at-rest pressure plus hydrostatic pressure involved use of buoyant unit weights in place of total unit weights, plus addition of the hydrostatic pressure component, as shown below:

$P_{o+u} = K_o \gamma_b H + u$, where

See disclaimer on cover page.

P_{o+u} = At-rest earth pressure plus hydrostatic pressure
K_o = as defined above
γ_b = Buoyant unit weight of soil
H = as defined above
U = hydrostatic pressure

Pressures were estimated at axis depth.

Step 1e: Vertical Overburden Pressure

This is a possible upper bound pressure. Exceeding this upper bound pressure with either face pressure or foam injection pressure has the potential to hydrojack (fracture) cohesionless or fissured cohesive soils with potential impact at the ground surface. Unit weights used to estimate vertical overburden pressure are provided in the "Assumptions" section below.

Step 1f: Passive Earth Pressure

For granular materials, passive earth pressures were calculated using the same methodology as the active earth pressures discussed in Step 1b above, but with use of the passive earth pressure coefficient, K_p, instead of the active earth pressure coefficient, K_a.

For cohesive materials, cohesion needs to be considered and the passive pressure equation is as follows:

$P_p = K_p \gamma H + 2c\sqrt{K_p}$, where

P_p = Passive earth pressure
K_p = Passive earth pressure coefficient. This parameter is calculated as described in the "Assumptions" section below.
γ = Unit weight of soil
H = Depth below ground surface
c = cohesion

Estimation of the passive pressure plus hydrostatic pressure involved use of buoyant unit weights in place of total unit weights, plus addition of the hydrostatic pressure component.

For Reference Only

See disclaimer on cover page.

Passive earth plus hydrostatic pressures are potential upper bound pressures. Pressures were estimated at axis depth.

Step 1g: Stability Ratio

The concept of a stability ratio was developed by Broms and Bennermark (1967), who referred to it as a failure ratio. The failure ratio was subsequently refined by Davis et al (1980), and renamed the stability ratio. The stability ratio is commonly expressed as the following equation:

$$N = [\sigma_S - \sigma_T + \gamma(C + D/2)]/c_u$$

Where N = Stability ratio

σ_S = surface pressure (surcharge)
σ_T = tunnel pressure (face pressure)
γ_t = total soil unit weight
C = depth of cover to tunnel crown
D = tunnel diameter
C_u = undrained shear strength

The stability ratio was developed for use in cohesive materials only. Materials at the tunnel face were classified based on available geotechnical data according to the Unified Soil Classification System (USCS) within the GDR (HMMB, 2005d). In this calculation, stability ratios were estimated where the tunnel face was composed primarily of materials classified as CL, CH, ML, or MH. An argument could also be made for using the stability ratio method in some clayey and silty sands. The stability ratio for the clays and silts at the tunnel face for each borehole is assigned based on the nearest available CPT to each borehole, based on s_u and depth data provided with each CPT log.

Intervals of cohesive soil and stability numbers for each CPT are shown in the "Average Stability Ratio by CPT" calculation (SBAC54, 2008). Stability ratios were not estimated from boing data because s_u data for borings comes from the generalized ShanSep Method.

Lower stability ratios indicate materials that are more stable at a tunnel heading. Broms and Bennermark (1967) indicated that stable openings in clays theoretically have a stability ratio

For Reference Only

See disclaimer on cover page.

(N) less than 6.28 for a finite opening. In practice, stable openings are typically expected in tunnels for N less than 4.

To be conservative, the maximum stability ratio in the tunnel face of the CPT is used rather than the average, when estimating face pressure required to provide sufficient support at the face. This was coupled with minimum undrained shear strength values within the same CPT, which may occur at different depths than the maximum stability ratio in the same interval. Together, the maximum stability ratio and minimum shear strength were used to estimate required face pressures assuming they occurred together at the tunnel springline. This assumption results in a tolerance of about 10 feet of additional soil pressure, which could be 0.2 to 0.3 bar at most.

The following relationship was used to derive a required face pressure for unstable cohesive soil.

$$\text{if } N > 4, \text{ then } \sigma_T = \sigma_S + \gamma (c + D/2) - 4\, c_u$$

Upon examination of the results, there are no locations where face pressures would be controlled by the stability ratio within this tolerance resulting from the application of the calculation at the springline.

Step 2 – Controlling Lower Bound and Upper Bound Face Pressure Values

The controlling lower bound value is the highest of the lower bound values estimated at a given tunnel station. The target pressure is set as low as possible without allowing normal fluctuations in pressure within the tolerance to dip below the lower bound. The tolerance is evaluated in Step 3 below. The upper bound values and risk of heave are checked once target and tolerance are established.

Step 3 – Face Pressure Control Accuracy

Face pressure control accuracy is typically specified to the Contractor. For this project, face pressure accuracy is required to be better than +/-4.35 psi, or 0.3 bar. This tolerance was added to the controlling lower bound value for the purpose of developing the recommended target face pressure. The recommended target face pressure was then used to check against the risk of heave by comparing against the controlling upper bound pressure.

Step 4 – Target Face Pressure Recommendations

For Reference Only

See disclaimer on cover page.

Recommended target face pressures as well as the potential upper and lower bound values are shown on the attached calculation spreadsheet and graphs. Target face pressure values were estimated by selecting the largest lower bound value and adding the face pressure control tolerance to this value. At the same time, a check was run to ensure that operating within the tolerance of the target face pressure would not present a risk of heave or upper bound face pressure values.

Assumptions

Geotechnical Parameters Used to Estimate Face Pressure

Geotechnical parameters used in the analysis are shown on the table below. Most of these parameters are from the technical memorandum entitled "Geotechnical Parameters for Design of Structures: Central Area Guideway" (HMMB, 2007a). Additional geotechnical information was obtained from the Geotechnical Data Report (HMMB, 2005d) and the General / Static Geotechnical Parameters calculation (HMMB, 2007b).

While the friction angle and unit weight are parameters resulting from the analysis of geotechnical laboratory testing data provided by HMMB (2007a), the active and passive earth pressure coefficients K_a and K_p are calculated parameters. The active and passive earth pressure coefficients were calculated using the following equations developed by Rankine:

$$K_a = \tan^2 (45 - \phi/2)$$

$$K_p = \tan^2 (45 + \phi/2) = 1/ K_a$$

Where ϕ = friction angle of the material.

For cohesive materials, the duration of loading becomes a concern. During tunnel boring, the TBM passes through the ground in a relatively short period of time that precludes the presence of fully drained conditions. This results in an undrained friction angle of 0, and passive and active earth pressure coefficients of 1. If the machine is shut down for a period of time such that drained conditions are allowed to develop in cohesive materials, then the drained friction angle is used; however this would not occur during normal operation. The duration of loading is indicated for each case on the table below.

For Reference Only

See disclaimer on cover page.

Geotechnical Parameters Used to Estimate Face Pressure

Load Duration	Soil type	Friction Angle, ϕ, degrees	Active Earth Pressure Coefficient, K_a	Passive Earth Pressure Coefficient, K_p
All	Sands and gravels	Varies by location (HMMB, 2008c)		
Short Term	High plasticity clay	0 degrees[1]	1	1
Short Term	Average plasticity clay	0 degrees[1]	1	1
Long Term	High plasticity clay	28 degrees[2]	0.36	2.77
Long Term	Average plasticity clay	32 degrees[2]	0.31	3.25

[1] HMMB, 2007a
[2] HMMB, 2007b, Section 6.0

Unit weights used in these calculations are presented on the table below.

Unit Weights Used to Estimate Face Pressure

Depth Below Ground Surface	Unit Weight, γ, pcf
Ground surface to 35 feet	122 pcf
Below 35 feet	126 pcf

Source: HMMB (2007a), Section 3.0

Discussion of design values for at-rest earth pressure coefficients (K_o) is provided in HMMB (2007a). The study states that the recommended design value for K_o is 1.0 above a depth of twenty feet and an average of 0.6 below a depth of twenty feet at all locations, with an average plus one standard deviation value of 0.7 below a depth of twenty feet at all locations. For this calculation, the average value of 0.6 is used rather than the more conservative average-plus-one-standard-deviation value of 0.7.

Groundwater Level

Groundwater levels vary seasonally and with pumping and recharge in the groundwater basin. These calculations include groundwater levels provided by the Hydrogeology Report (HMMB, 2008a) labeled "Maximum Water Level for Construction (based on historic maximum)" And "Minimum Water Level for Construction". Groundwater levels should be

For Reference Only

See disclaimer on cover page.

verified as future additional piezometer readings are made. Changes in groundwater pressure estimated at tunnel depth for whatever reason, including significant artesian pressure, will influence the target pressures estimated in this calculation; however the artesian pressures encountered during the SR87 HOV project CIDH construction (HMMB, 2006b) are consistent with the water levels used in this calculation.

Tunnel Depth and Stationing

Tunnel depth and stationing was established based on the Geotechnical Plan and Profile Drawings, Rev. G released April 2008 for the quarterly update (HMMB, 2008).

TBM Parameters

TBM dimensions were obtained from Appendix C2, Tunnel Lining Geometric Design and Installation Plan of the Tunneling Methodology Report (HMMB, 2005e). This gave an external tunnel diameter of 19 feet–6 inches, and an internal diameter of 17 feet-10 inches. After inclusion of an overcut of 7-inches on radius, the machine diameter is expected to be 20 feet – 8 inches. This diameter corresponds to the maximum overcut at the cutterhead per drawing number Z001 (HMMB, 2005g).

Calculations

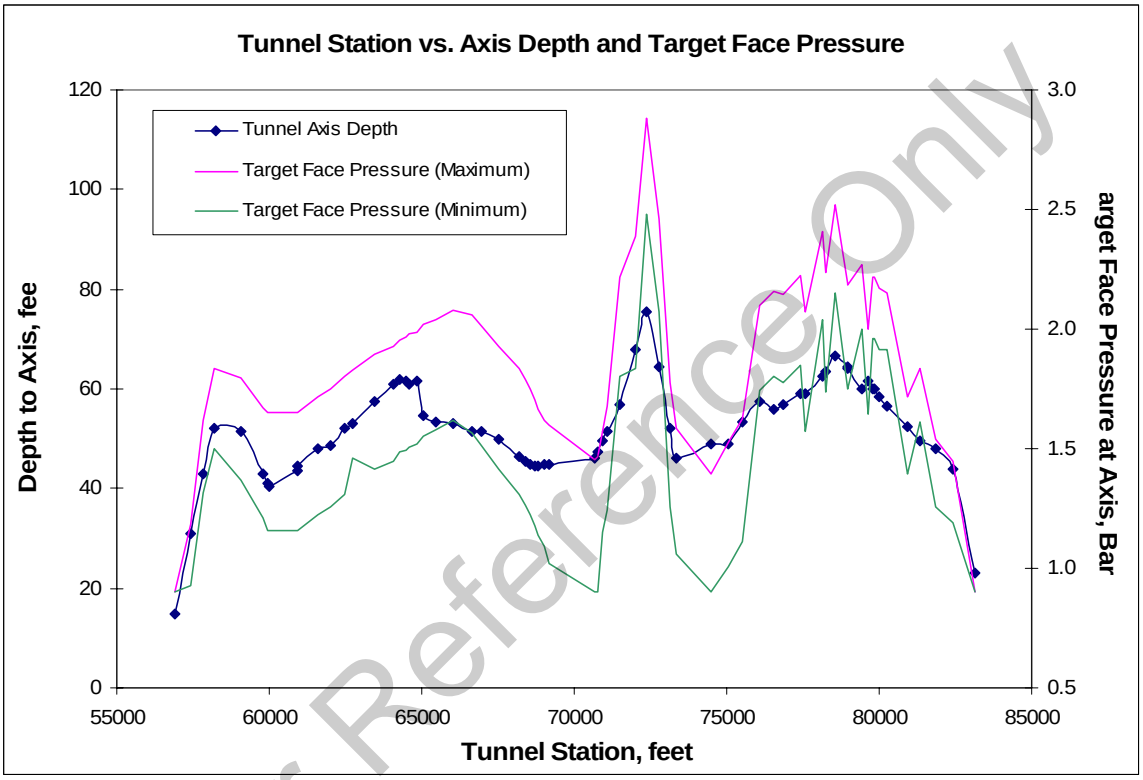
Calculations are shown on the attached pages.

Results and Conclusions

Results are summarized for each boring on the attached charts. Based on the method described above, and the calculations presented on the attached spreadsheet, minimum and maximum face pressures can be established along with target face pressures. Target pressures were generally controlled by either the maximum annulus grouting pressure or the active earth + hydrostatic pressure. Controlling minima are provided on the attached Table for each boring location where analyses were conducted. The annulus grout pressure is assumed to be up to 2 psi above local hydrostatic pressure. The target pressure near the portals are sometimes controlled by the face pressure needed to maintain a charged plenum, however these calculations show that this is not the case for the conditions analyzed. Target face pressures by station are shown on the chart below.

For Reference Only

See disclaimer on cover page.

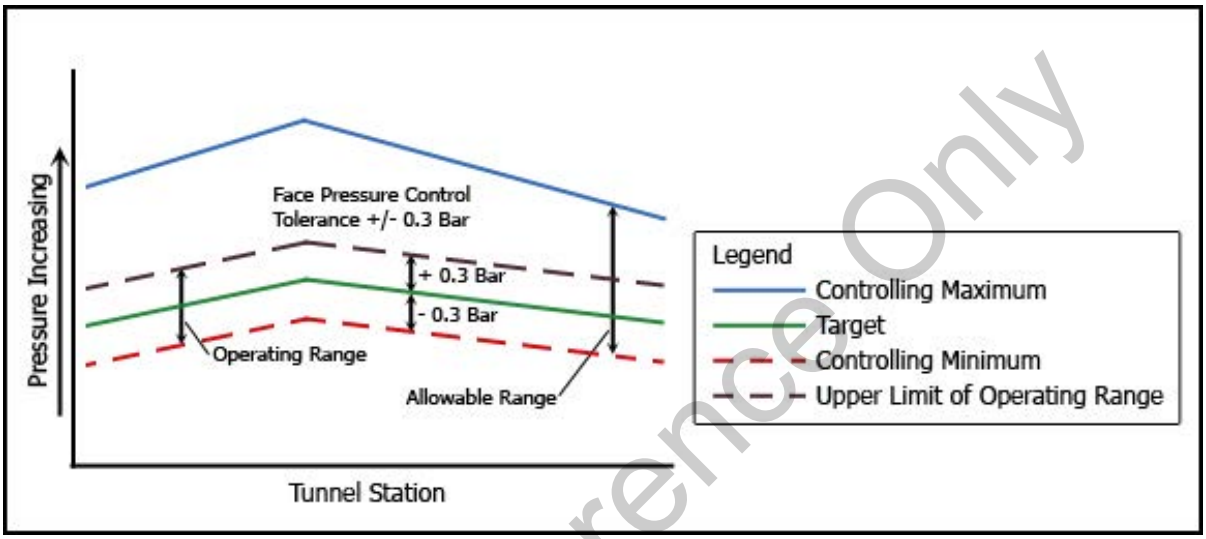


For Reference Only

Target Face Pressures by Station

It has been assumed that, through control of the TBM with soil conditioning, face pressures can be maintained within a range of plus or minus 0.3 bar (4.35 psi) of the target pressure. This tolerance was added to the controlling minimum face pressure to develop a proposed target face pressure. This resulted in a maximum target face pressure needed to control face stability of 3.1 bar. An illustration of the relationship between the tolerance and target face pressure is presented on the figure below.

See disclaimer on cover page.



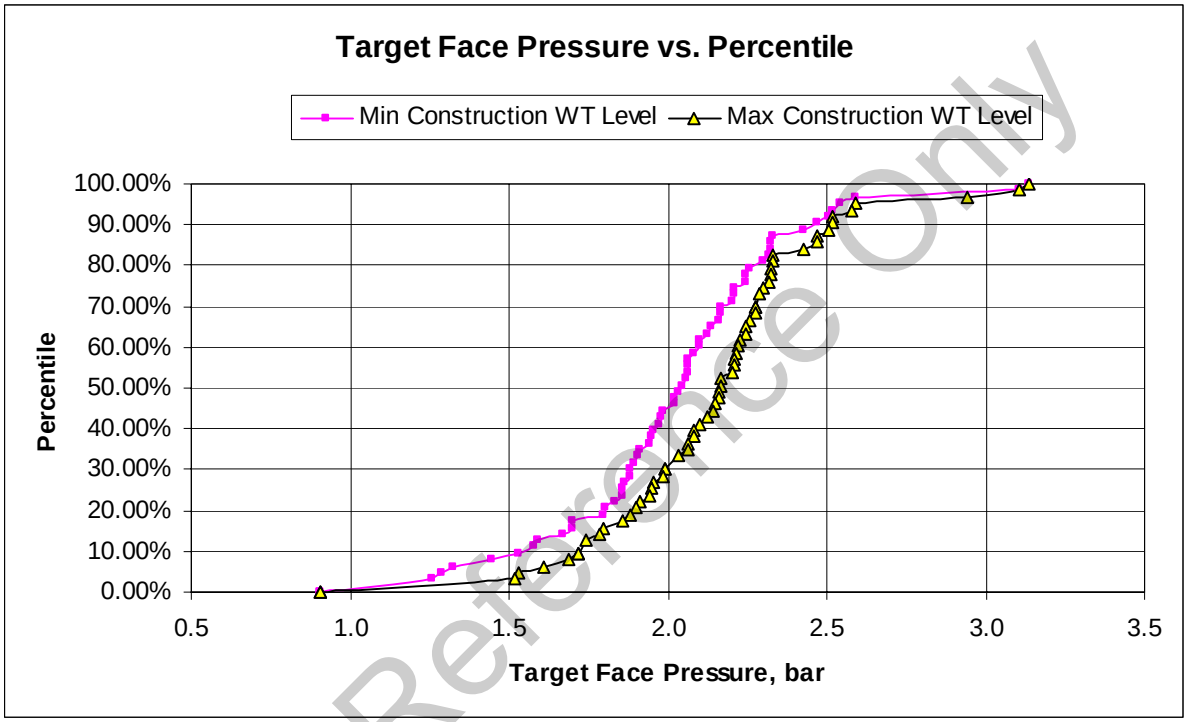
Relationship between Target Face Pressure and Tolerance

Although both short and long term loading conditions were considered when estimating lateral earth pressures, these considerations did not control selection of target face pressures. Face pressures are summarized in a percentile format for both minimum and maximum water table levels anticipated during construction, based on data points at each of the geotechnical borings evaluated in the calculations, on the figure below.

During construction, actual water table elevations will be used to establish target face pressures. Water table levels will be measured ahead of the TBM with piezometers that will be monitored throughout the mining phase of the project. This calculation shows the expected range of anticipated face pressures based on encountered soil conditions.

For Reference Only

See disclaimer on cover page.



Summary of Face Pressures by Percentile

For Reference Only

From a face pressure and soil conditioning standpoint, the alignment from about Station 569+00 to 625+00 represents the most favorable ground for tunneling, and would be a good place to commence tunneling.

If the stability ratio of cohesive soils were the only consideration, face pressure would rarely be needed in cohesive soils, as the stability ratios are almost always less than 4 without application of a tunnel face pressure. However, fissures in the clays and the presence of water bearing sand zones at unpredictable locations within the clays reduce the influence of the stability ratio on face pressure estimates for this project. Fissuring is typically expected in approximately the top 20 feet or so of the upper cohesive unit in the soil profile, and is therefore above the bored tunnel horizon along nearly all of the alignment. As a result, the primary concern regarding face pressures is inclusions of flowing sand within zones that are otherwise cohesive.

See disclaimer on cover page.

If the extent of flowing sand zones could be determined with confidence, then zones composed of cohesive soils could be delineated with confidence. Pressures required within the cohesive soils would be zero in most cases. Since potential flowing sand zones within cohesive units cannot be estimated with sufficient accuracy at this time, face pressures should be developed with the assumption that flowing sand could be encountered at any time. As a result, the potential to encounter flowing sand along the alignment will generally control the target face pressures along the tunnel rather than use of stability number estimates in cohesive materials.

Target face pressure values were compared against the pressure needed to keep the plenum of the machine charged, as recommended in BTS, 2005. The pressure required to maintain a fully charged plenum was relatively low, and as a result, did not control the target face pressure.

The target pressure plus the tolerance was also checked against the total vertical overburden pressure and was found to be less than the vertical overburden pressure in all cases except at BH-01 and BH-57 where there is a risk of heave or fracout if granular soils are encountered. This needs to be studied further and additional calculations may be required to estimate face pressures in these areas.

Changes Since Last Issue

- A revision log is available in a separate document.
- "Relationship between Target Face Pressure and Tolerance" Figure updated

Next Steps

Develop estimate of target pressures assuming sand could be encountered at all times. Confirm that those estimated upper and lower limits and targets are more conservative than those estimated here.

For Reference Only

See disclaimer on cover page.

References

Broms, B.B. and Bennermark, H. (1967). Stability of Clay in Vertical Openings. *J Soil Mech Fdns Div, ASCE*, 193, SM1, 71-94.

British Tunneling Society (BTS), 2005. Closed Face Tunneling Machines and Ground Stability- A guideline for best practice, Thomas Telford.

Crow, M. and Holzhauser, J. (2003). Performance of Four EPB-TBMs Above and Below the Groundwater Table on the ECIS Project, Los Angeles, CA, USA, Proceedings of the Rapid Excavation and Tunneling Conference.

Davis, E.H., Gunn, M.J., Mair, R.J., and Senevirante, H.N. (1980). The stability of shallow tunnels and underground openings in cohesive material. *Geotechnique* 30, No. 4, 397-416.

HMM/Bechtel, 2005 (HMMB, 2005a). Tunnel Segment Geotechnical Design Memorandum for Tunnel Lining Design, Document # P0503-D300-RPT-DE-034, Rev. A. December 21, 2005.

HMM/Bechtel, 2005 (HMMB, 2005b). Technical Memorandum No. 14, "Aquifer Degassing and Artesian Conditions." August 2005.

HMM/Bechtel, 2005 (HMMB, 2005c). Geotechnical Plan and Profile with TBM Design Water Levels, Rev. A. Sheet numbers C026 – C047. October 14, 2005.

HMM/Bechtel, 2005 (HMMB, 2005d). Geotechnical Data Report, First Draft – Internal Review Only, Document # P0503-D300-RPT-DE-021. July 1, 2005.

HMM/Bechtel, 2005 (HMMB, 2005e). Tunneling Methodology Report, Document #P0503-D300-RPT-DE-009, Rev. 0. July 5, 2005.

HMM/Bechtel, 2005 (HMMB, 2005f). Technical Memorandum, "Undrained Shear Strength and Stability Factor Along Bored Tunnel Alignment," Document # P0503-D300-DBM-GE-004, Rev. 0. December 12, 2005.

HMM/Bechtel, 2005 (HMMB, 2005g). TBM Technical Reference Report, Document # P0503-D300-RPT-DE-018, Rev. A, "TBM Tail Seal and Grout Annulus Study," Sheet Number Z001. June 12, 2007.

HMM/Bechtel, 2006 (HMMB, 2006b). Technical Memorandum, "Geotechnical Input on Coyote Creek Bridge Options," Document # 0503-D300-TM-DE-035, Rev. 0. May 24, 2006.

For Reference Only

See disclaimer on cover page.

HMM/Bechtel, 2007 (HMMB, 2007a). Technical Memorandum, "Geotechnical Parameters for Design of Structures, Central Area Guideway, Tunnel Segment, SVRT Project," Document # 0503-D300-TM-DE-033, Rev. 1. July 12, 2007.

HMM/Bechtel (2007b). Calculation SOOG04 – General / Static Geotechnical Parameters. December 03, 2007.

HMM/Bechtel, 2008 (HMMB, 2008a). Hydrogeology Report. P0503-D300-RPT-DE-020, Rev. 1.

HMM/Bechtel, 2008 (HMMB, 2008b). Calculation SBAC54. Stability Ratio and Cohesion by CPT. May 6, 2008.

HMM/Bechtel, 2008 (HMMB, 2008c). Calculation SBAC55. Friction Angle by Borehole. May 6, 2008.

Lower Northwest Interceptor (LNWI), 2006. Sacramento River Crossing Tunnels Construction Report, internal report, unpublished draft.

Sowers, G. F. (1979). Introductory Soil Mechanics and Foundations: Geotechnical Engineering, 4th ed., McMillan, NY.

For Reference Only

See disclaimer on cover page.

Development of Target TBM Face Pressures
Short Term Loading

	A	B	C	D	E	F	G	H	I	J
1	**INPUT DATA**									
2	**Machine Diameter**		20.7	feet					**Lower Bound, pressure for**	
3	**Face Pressure Control**								**charged plenum at SL**	
4	**Accuracy (Tolerance)**		4.35	psi	0.30	bar			psf	bar
5	**Bulking Factor**		1						1261	0.6
6	**Max Grout Overpressure**		2	psi	0.14	bar				
7	**Unit Weight**									
8	from sfc to 35 ft depth		122	pcf						
9	below 35 ft depth		126	pcf						

10	Study Section	Borehole Number	Station (ft)	Tunnel Crown Depth (fbgs)	Tunnel Invert Depth (fbgs)	Tunnel Springline Depth (fbgs)	Water Surface Depth for Construction (fbgs)	Friction Angle	Unit Weight at Springline pcf	Ground Conditions
11										
12										
13	1	BH-57	569+31	5.0	25.0	15.0	3	0	122	CL & CH
14	1	BH-01	574+09	21.0	41.0	31.0	4.1	0	122	CL
15	1	BH-02	578+10	33.0	53.0	43.0	2.9	0	126	CL-CH, some GW-GC/GM
16	1	BH-03	581+83	42.0	62.0	52.0	5.4	0	126	CL-CH, GW
17	1	BH-04	590+53	41.5	61.5	51.5	6	0	126	CL-ML, CL, & CH
18	1	BH-05	598+17	33.0	53.0	43.0	1.3	0	126	CL
19	1	BH-06	599+60	31.0	51.0	41.0	0	0	126	CL & CH
20	1	BH-83	599+83	30.5	50.5	40.5	-0.5	0	126	CL & SC
21	3	BH-86	609+08	33.5	53.5	43.5	2.5	0	126	CL & CH
22	3	BH-07	609+40	34.5	54.5	44.5	3.5	0	126	CL & CH
23	3	BH-08	615+76	38.0	58.0	48.0	5	0	126	CL
24	3	BH-09	619+91	38.5	58.5	48.5	4.5	0	126	CH
25	3	BH-10	624+90	42.0	62.0	52.0	6.4	0	126	CL
26	3	BH-11	627+53	43.0	63.0	53.0	6.6	0	126	CH, some CL
27	3	BH-12	634+65	47.5	67.5	57.5	9.1	0	126	nterlayered CH & CL with some ML/CI
28	3	BH-13	640+82	51.0	71.0	61.0	11.6	0	126	CL with some SM
29	3	BH-14	642+55	52.0	72.0	62.0	11.7	0	126	CH
30	3	BH-88	645+03	51.5	71.5	61.5	10.9	0	126	CL with ML, SM
31	3	BH-15	645+73	51.0	71.0	61.0	10	0	126	CH
32	3	BH-87	648+42	51.5	71.5	61.5	10.2	0	126	CL
33	3	BH-16	650+43	44.5	64.5	54.5	2.3	0	126	CH with some CL
34	3	BH-17	654+53	43.5	63.5	53.5	0.6	0	126	CH with some ML at invert
35	3	BH-18	660+09	43.0	63.0	53.0	-1.1	0	126	H/CL at crown, SP-SM, SW-SM at inv
36	3	BH-19	666+26	41.5	61.5	51.5	-2	0	126	CH, some CL, GP & SM at invert
37	3	BH-20	669+79	41.5	61.5	51.5	-0.6	0	126	CH, CL lens, GW-GC at invert
38	3	BH-21	675+49	40.0	60.0	50.0	0.6	0	126	CL, ML lens, SW-SC at invert
39	3	BH-50	681+71	36.5	56.5	46.5	-0.1	0	126	CH, CL, SP-SM invert
40	3	BH-52	684+09	35.5	55.5	45.5	0.2	0	126	CH, CL, ML zone, & CL/SC at invert
41	3	BH-53	685+42	35.0	55.0	45.0	0.9	0	126	MH/CH, CL, ML
42	3	BH-54	687+16	34.5	54.5	44.5	1.9	0	126	CH & CL
43	3	BH-55	688+35	34.5	54.5	44.5	3	0	126	CH-MH & CL
44	3	BH-23	690+02	35.0	55.0	45.0	5	0	126	CL
45	3	BH-64	691+93	35.0	55.0	45.0	5.5	0	126	CL, SW-SM, ML
46	5	BH-70	706+78	36.0	56.0	46.0	11	0	126	CL, SM, SP-SM, CL-ML
47	5	BH-71	707+62	37.5	57.5	47.5	12	0	126	CH, SW-SM, CL
48	5	BH-72	709+39	39.5	59.5	49.5	10.8	0	126	CL
49	5	BH-26	710+66	41.5	61.5	51.5	9.6	0	126	CL, SC, ML
50	5	BH-27	715+00	47.0	67.0	57.0	5.6	42	126	CL, SM & SW-SM
51	5	BH-28	720+22	58.0	78.0	68.0	4.5	0	126	CL, SM & SC
52	5	BH-29	723+88	65.5	85.5	75.5	5.6	42	126	CL, SP, CL-ML, & GW
53	5	BH-30	728+01	54.5	74.5	64.5	6.6	42	126	GW-GC & CL
54	5	BH-31	731+54	42.0	62.0	52.0	7.2	0	126	CL, SC, & GW-GM?
55	5	BH-32	733+30	36.0	56.0	46.0	7	0	126	CL, ML, & SM
56	7	BH-34	744+65	39.0	59.0	49.0	15.7	0	126	CL, ML, SM
57	7	BH-35	750+48	39.0	59.0	49.0	11.8	0	126	CL
58	7	BH-36	755+33	43.5	63.5	53.5	13.2	0	126	SW-SM & CL
59	7	BH-37	760+59	47.5	67.5	57.5	13.7	39	126	CL, SM, & SW
60	7	BH-38	765+23	46.0	66.0	56.0	9.1	39	126	SM, ML, & SP-SC?
61	7	BH-39	768+77	47.0	67.0	57.0	7.4	43	126	GW & GP-GC?
62	7	BH-95	774+14	49.0	69.0	59.0	5.9	45	126	CL, CL-ML, SP-SM, GW, GM, SM
63	7	BH-40	775+76	49.0	69.0	59.0	5.1	0	126	CL & SM
64	7	BH-41	781+35	52.5	72.5	62.5	4.6	44	126	SP/no data
65	7	BH-79	782+50	53.5	73.5	63.5	4.7	0	126	CL
66	7	BH-42	785+37	56.5	76.5	66.5	5.9	44	126	CL/no data
67	7	BH-81	789+62	54.5	74.5	64.5	7.1	0	126	CL, SC
68	7	BH-43	789+71	54.0	74.0	64.0	6.7	0	126	CL/no data
69	7	BH-80	794+39	50.0	70.0	60.0	7	43	126	CL, SP-SM, & SM
70	7	BH-102	796+49	51.5	71.5	61.5	9.8	0	126	CL, ML, SW, GW-GM
71	7	BH-103	798+17	50.0	70.0	60.0	9.4	43	126	GW-GM, CL, ML, GW, SP, SW, ML
72	7	BH-44	798+27	50.0	70.0	60.0	9.4	43	126	GW-GM & GP-GM
73	7	BH-106	800+21	48.5	68.5	58.5	9.2	43	126	GW-GM & ML
74	7	BH-45	802+44	46.5	66.5	56.5	9	40	126	SW, SW-SM & CL
75	7	BH-46	809+36	42.5	62.5	52.5	9.5	0	126	GW-GC? & CL-ML
76	7	BH-47	813+52	39.5	59.5	49.5	9.1	44	126	SP-SM & CL
77	7	BH-48	818+34	38.0	58.0	48.0	10.3	0	126	CL & SM
78	7	BH-49	824+28	34.0	54.0	44.0	9.1	0	126	ML, CL, & SM
79	7	BH-78	831+41	13.0	33.0	23.0	7	0	122	CH, SC, GW, GM

10	Vertical Overburden Pressure at Springline	Kp	Passive Earth Pressure, avg	Passive + Hydrostatic, avg	Ko	At Rest Earth Pressure, avg	At Rest Earth Pressure + Hydrostatic, avg	Ka	Active Earth Pressure, avg	Hydrostatic Pressure at SL	Active + hydrostatic	Maximum Annulus Grouting Pressure at SL	Corresponding CPT	Stability Ratio (N)	Undrained shear strength	Stability of Cohesive Soil	Target Face Pressure at Springline	Difference Between Target and Heave Pressure [2]
11	bar		bar	bar		bar	bar		bar	bar	bar	bar			bar	bar	bar	bar
12																		
13	0.9	1.00	2.4	2.4	1.0	0.9	0.9	1.00	0.0	0.4	0.4	0.4	001	5.5	0.77	0.0	0.9	-0.3
14	1.8	1.00	3.3	3.3	0.6	1.1	1.4	1.00	0.3	0.8	0.8	0.9	001	5.5	0.77	0.0	1.2	0.3
15	2.5	1.00	5.1	5.1	0.6	1.6	2.0	1.00	0.1	1.2	1.2	1.3	004	3.3	1.25	0.0	1.6	0.6
16	3.1	1.00	5.6	5.6	0.6	1.9	2.4	1.00	0.6	1.4	1.4	1.5	004	3.3	1.25	0.0	1.8	0.9
17	3.0	1.00	5.9	5.9	0.6	1.9	2.4	1.00	0.3	1.4	1.4	1.5	009	5.2	1.40	0.0	1.8	0.9
18	2.5	1.00	4.3	4.3	0.6	1.6	2.1	1.00	0.9	1.2	1.2	1.4	010	4.1	0.84	0.0	1.7	0.6
19	2.4	1.00	4.4	4.4	0.6	1.5	2.0	1.00	0.5	1.2	1.2	1.3	112	3.9	0.98	0.0	1.6	0.5
20	2.4	1.00	4.4	4.4	0.6	1.5	2.0	1.00	0.5	1.2	1.2	1.3	112	3.9	0.98	0.0	1.6	0.4
21	2.6	1.00	4.6	4.6	0.6	1.6	2.1	1.00	0.7	1.2	1.2	1.3	123	3.8	0.97	0.0	1.6	0.6
22	2.6	1.00	4.9	4.9	0.6	1.6	2.1	1.00	0.5	1.2	1.2	1.3	124	3.2	1.10	0.0	1.6	0.7
23	2.8	1.00	5.8	5.8	0.6	1.7	2.3	1.00	0.0	1.3	1.3	1.4	013	3.4	1.43	0.0	1.7	0.8
24	2.9	1.00	5.3	5.3	0.6	1.8	2.3	1.00	0.5	1.3	1.3	1.4	015	3.7	1.20	0.0	1.7	0.8
25	3.1	1.00	5.6	5.6	0.6	1.9	2.4	1.00	0.6	1.4	1.4	1.5	018	4.4	1.25	0.0	1.8	1.0
26	3.1	1.00	5.2	5.2	0.6	1.9	2.5	1.00	1.2	1.4	1.4	1.5	019	4.5	1.02	0.0	1.8	1.0
27	3.4	1.00	6.1	6.1	0.6	2.1	2.7	1.00	0.8	1.4	1.4	1.6	023	4.4	1.32	0.0	1.9	1.2
28	3.6	1.00	9.2	9.2	0.6	2.2	2.8	1.00	0.0	1.5	1.5	1.6	026	2.8	2.74	0.0	1.9	1.4
29	3.7	1.00	6.3	6.3	0.6	2.2	2.8	1.00	1.2	1.5	1.5	1.6	165	3.7	1.28	0.0	2.0	1.4
30	3.6	1.00	7.5	7.5	0.6	2.2	2.8	1.00	0.0	1.5	1.5	1.7	027	3.5	1.88	0.0	2.0	1.4
31	3.6	1.00	7.4	7.4	0.6	2.2	2.8	1.00	0.0	1.5	1.5	1.7	027	3.5	1.88	0.0	2.0	1.3
32	3.6	1.00	6.7	6.7	0.6	2.2	2.8	1.00	0.8	1.5	1.5	1.7	029	3.7	1.47	0.0	2.0	1.4
33	3.2	1.00	5.5	5.5	0.6	2.0	2.6	1.00	1.0	1.6	1.6	1.7	166	3.8	1.13	0.0	2.0	0.9
34	3.2	1.00	5.9	5.9	0.6	1.9	2.6	1.00	0.5	1.6	1.6	1.7	031	4.3	1.36	0.0	2.0	0.8
35	3.1	1.00	5.9	5.9	0.6	1.9	2.6	1.00	0.5	1.6	1.6	1.8	033	3.4	1.35	0.0	2.1	0.8
36	3.0	1.00	5.1	5.1	0.6	1.9	2.5	1.00	1.1	1.6	1.6	1.7	036	3.8	1.00	0.0	2.1	0.7
37	3.0	1.00	5.9	5.9	0.6	1.9	2.5	1.00	0.3	1.6	1.6	1.7	037	2.5	1.42	0.0	2.0	0.7
38	3.0	1.00	6.2	6.2	0.6	1.8	2.4	1.00	0.0	1.5	1.5	1.6	040	3.2	1.58	0.0	1.9	0.7
39	2.7	1.00	5.0	5.0	0.6	1.7	2.2	1.00	0.6	1.4	1.4	1.5	096	4.3	1.08	0.0	1.8	0.6
40	2.7	1.00	5.1	5.1	0.6	1.6	2.2	1.00	0.4	1.4	1.4	1.5	098	4.3	1.16	0.0	1.8	0.6
41	2.6	1.00	4.7	4.7	0.6	1.6	2.2	1.00	0.8	1.3	1.3	1.5	102	4.2	0.97	0.0	1.8	0.6
42	2.6	1.00	4.9	4.9	0.6	1.6	2.1	1.00	0.5	1.3	1.3	1.4	103	4.5	1.09	0.0	1.7	0.6
43	2.6	1.00	4.8	4.8	0.6	1.6	2.1	1.00	0.6	1.2	1.2	1.4	105	3.9	1.07	0.0	1.7	0.7
44	2.6	1.00	4.9	4.9	0.6	1.6	2.1	1.00	0.6	1.2	1.2	1.3	105	3.9	1.07	0.0	1.6	0.7
45	2.6	1.00	4.7	4.7	0.6	1.6	2.1	1.00	0.7	1.2	1.2	1.3	043	4.2	1.01	0.0	1.6	0.7
46	2.7	1.00	5.7	5.7	0.6	1.7	2.1	1.00	0.0	1.0	1.0	1.2	142	4.0	1.44	0.0	1.5	1.0
47	2.8	1.00	5.4	5.4	0.6	1.7	2.1	1.00	0.3	1.1	1.1	1.2	143	4.8	1.26	0.0	1.5	1.0
48	2.9	1.00	5.1	5.1	0.6	1.8	2.3	1.00	0.9	1.2	1.2	1.3	145A	5.3	1.07	0.0	1.6	1.0
49	3.0	1.00	5.3	5.3	0.6	1.9	2.4	1.00	0.9	1.3	1.3	1.4	145	4.6	1.08	0.0	1.7	1.1
50	3.4	5.04	17.4	11.1	0.6	2.1	2.7	0.20	0.7	1.5	1.9	1.7	050	NA	0.00		2.2	0.9
51	4.0	1.00	7.6	7.6	0.6	2.5	3.2	1.00	0.6	1.9	1.9	2.0	052	4.3	1.76	0.0	2.4	1.3
52	4.5	5.04	23.0	14.5	0.6	2.7	3.6	0.20	0.9	2.1	2.6	2.2	053	NA	0.00		2.9	1.3
53	3.8	5.04	19.6	12.6	0.6	2.3	3.0	0.20	0.8	1.7	2.2	1.9	055	NA	0.00		2.5	1.1
54	3.1	1.00	6.2	6.2	0.6	1.9	2.4	1.00	0.1	1.3	1.3	1.5	057	4.8	1.54	0.0	1.8	1.0
55	2.7	1.00	5.9	5.9	0.6	1.7	2.1	1.00	0.0	1.2	1.2	1.3	057	4.8	1.54	0.0	1.6	0.8
56	2.9	1.00	5.3	5.3	0.6	1.8	2.2	1.00	0.6	1.0	1.0	1.1	060	4.0	1.18	0.0	1.4	1.2
57	2.9	1.00	5.6	5.6	0.6	1.8	2.2	1.00	0.3	1.1	1.1	1.2	061	3.9	1.31	0.0	1.5	1.1
58	3.2	1.00	6.1	6.1	0.6	1.9	2.4	1.00	0.4	1.2	1.2	1.3	063	3.8	1.44	0.0	1.6	1.2
59	3.4	4.40	15.3	10.8	0.6	2.1	2.6	0.23	0.8	1.3	1.8	1.4	066	NA	0.00		2.1	1.0
60	3.3	4.34	14.7	10.0	0.6	2.0	2.6	0.23	0.8	1.4	1.9	1.5	068	NA	0.00		2.2	0.9
61	3.4	5.37	18.5	12.0	0.6	2.1	2.7	0.19	0.6	1.5	1.8	1.6	070	NA	0.00		2.1	0.9
62	3.5	5.83	20.8	13.1	0.6	2.1	2.8	0.17	0.6	1.6	1.9	1.7	073	NA	0.00		2.2	1.0
63	3.5	1.00	6.5	6.5	0.6	2.1	2.8	1.00	0.6	1.6	1.6	1.7	073	3.1	1.47	0.0	2.1	1.1
64	3.7	5.42	20.4	12.8	0.6	2.3	3.0	0.18	0.7	1.7	2.1	1.9	076	NA	0.00		2.4	1.0
65	3.8	1.00	7.1	7.1	0.6	2.3	3.0	1.00	0.6	1.8	1.8	1.9	076	3.6	1.61	0.0	2.2	1.2
66	3.9	5.42	21.7	13.7	0.6	2.4	3.1	0.18	0.7	1.8	2.2	1.9	077	NA	0.00		2.5	1.1
67	3.8	1.00	6.3	6.3	0.6	2.3	3.0	1.00	1.4	1.7	1.7	1.9	079	6.5	1.23	0.0	2.2	1.3
68	3.8	1.00	6.3	6.3	0.6	2.3	3.0	1.00	1.4	1.7	1.7	1.9	079	6.5	1.23	0.0	2.2	1.3
69	3.6	5.29	19.2	12.4	0.6	2.2	2.8	0.19	0.7	1.6	2.0	1.7	171	NA	0.00		2.3	1.0
70	3.6	1.00	7.3	7.3	0.6	2.2	2.8	1.00	0.1	1.5	1.5	1.7	171	4.1	1.81	0.0	2.0	1.3
71	3.6	5.21	18.9	12.5	0.6	2.2	2.8	0.19	0.7	1.5	1.9	1.7	082	NA	0.00		2.2	1.0
72	3.6	5.21	18.9	12.5	0.6	2.2	2.8	0.19	0.7	1.5	1.9	1.7	082	NA	0.00		2.2	1.0
73	3.5	5.21	18.4	12.2	0.6	2.1	2.7	0.19	0.7	1.5	1.9	1.6	082	NA	0.00		2.2	1.0
74	3.3	4.60	15.7	10.6	0.6	2.0	2.6	0.22	0.7	1.4	1.9	1.6	083	NA	0.00		2.2	0.9
75	3.1	1.00	6.4	6.4	0.6	1.9	2.4	1.00	0.0	1.3	1.3	1.4	086	3.5	1.61	0.0	1.7	1.1
76	2.9	5.42	16.2	10.9	0.6	1.8	2.3	0.18	0.6	1.2	1.5	1.3	087	NA	0.00		1.8	0.8
77	2.8	1.00	5.9	5.9	0.6	1.7	2.2	1.00	0.0	1.1	1.1	1.2	089	4.7	1.49	0.0	1.5	1.0
78	2.6	1.00	5.7	5.7	0.6	1.6	2.0	1.00	0.0	1.0	1.0	1.1	092	3.5	1.55	0.0	1.4	0.8
79	1.3	1.00	3.9	3.9	0.6	0.8	1.0	1.00	0.0	0.5	0.5	0.5	093	1.9	1.25	0.0	0.9	0.1

Notes:

1. The uncorrected blowcount was used with the Sowers correlation in one case. Other equivalent stability numbers were estimated using field test data (such as pocket penetrometer tests).
2. This column is used to check to see that there is an adequate factor of safety against heave. Heave pressure conservatively assumed to be vertical overburden pressure.
3. This column shows the stability number recalculated using the target face pressure in the stability number equation as described in the text.

For Reference Only

See disclaimer on cover page.

Development of Target TBM Face Pressures
Short Term Loading

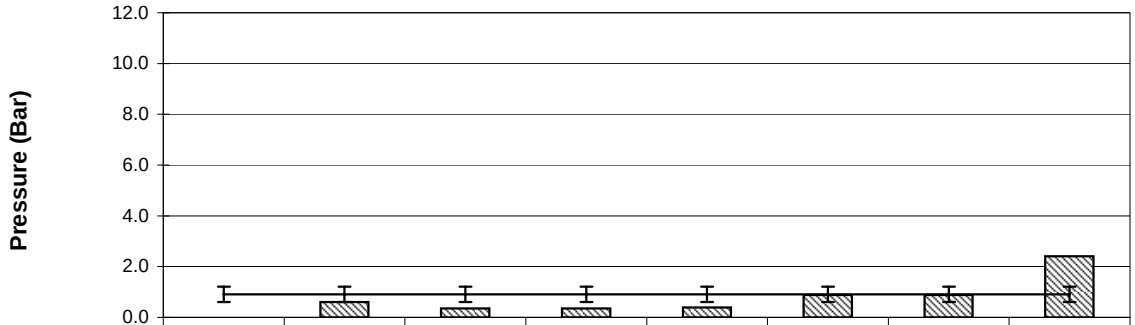
	A	B	C	D	E	F	G	H	I	J
1	**INPUT DATA**									
2	**Machine Diameter**		20.7	feet					**Lower Bound, pressure for**	
3	**Face Pressure Control**								**charged plenum at SL**	
4	**Accuracy (Tolerance)**		4.35	psi	0.30	bar			psf	bar
5	**Bulking Factor**		1						1261	0.6
6	**Max Grout Overpressure**		2	psi	0.14	bar				
7	**Unit Weight**									
8	from sfc to 35 ft depth		122	pcf						
9	below 35 ft depth		126	pcf						

10	Study Section	Borehole Number	Station (ft)	Tunnel Crown Depth (fbgs)	Tunnel Invert Depth (fbgs)	Tunnel Springline Depth (fbgs)	Water Surface Depth for Construction (fbgs)	Friction Angle	Unit Weight at Springline pcf	Ground Conditions	Vertical Overburden Pressure at Springline bar	Kp	Passive Earth Pressure, avg bar	Passive + Hydrostatic, avg bar	Ko	At Rest Earth Pressure, avg bar	At Rest Earth Pressure + Hydrostatic, avg bar	Ka	Active Earth Pressure, avg bar	Hydrostatic Pressure at SL bar	Active + hydrostatic bar	Maximum Annulus Grouting Pressure at SL bar	Corresponding CPT	Stability Ratio (N)	Undrained shear strength bar	Stability of Cohesive Soil bar	Target Face Pressure at Springline bar	Difference Between Target and Heave Pressure [2] bar
11																												
12																												
13	1	BH-57	569+31	5.0	25.0	15.0	10	0	122	CL & CH	0.9	1.00	2.4	2.4	1	0.9	0.9	1.00	0.0	0.1	0.1	0.2	001	5.5	0.77	0.0	0.9	-0.3
14	1	BH-01	574+07	21.0	41.0	31.0	11.9	0	122	CL	1.8	1.00	3.3	3.3	0.6	1.1	1.3	1.00	0.3	0.6	0.6	0.6	001	5.5	0.77	0.0	0.9	0.6
15	1	BH-02	578+10	33.0	53.0	43.0	12.1	0	126	CL-CH, some GW-GC/GM	2.5	1.00	5.1	5.1	0.6	1.6	1.9	1.00	0.1	0.9	0.9	1.0	004	3.3	1.25	0.0	1.3	0.9
16	1	BH-03	581+83	42.0	62.0	52.0	15.6	0	126	CL-CH, GW	3.1	1.00	5.6	5.6	0.6	1.9	2.3	1.00	0.6	1.1	1.1	1.2	004	3.3	1.25	0.0	1.5	1.3
17	1	BH-04	590+53	41.5	61.5	51.5	19	0	126	CL-ML, CL, & CH	3.0	1.00	5.9	5.9	0.6	1.9	2.3	1.00	0.3	1.0	1.0	1.1	009	5.2	1.40	0.0	1.4	1.4
18	1	BH-05	598+17	33.0	53.0	43.0	15.9	0	126	CL	2.5	1.00	4.3	4.3	0.6	1.6	1.9	1.00	0.9	0.8	0.9	0.9	010	4.1	0.84	0.0	1.2	1.0
19	1	BH-06	599+58	31.0	51.0	41.0	14.9	0	126	CL & CH	2.4	1.00	4.4	4.4	0.6	1.5	1.8	1.00	0.5	0.8	0.8	0.9	112	3.9	0.98	0.0	1.2	0.9
20	1	BH-83	599+83	30.5	50.5	40.5	14.5	0	126	CL & SC	2.4	1.00	4.4	4.4	0.6	1.5	1.8	1.00	0.5	0.8	0.8	0.9	112	3.9	0.98	0.0	1.2	0.9
21	3	BH-86	609+08	33.5	53.5	43.5	17.5	0	126	CL & CH	2.6	1.00	4.6	4.6	0.6	1.6	1.9	1.00	0.7	0.8	0.8	0.9	123	3.8	0.97	0.0	1.2	1.1
22	3	BH-07	609+40	34.5	54.5	44.5	18.5	0	126	CL & CH	2.6	1.00	4.9	4.9	0.6	1.6	1.9	1.00	0.5	0.8	0.8	0.9	124	3.2	1.10	0.0	1.2	1.2
23	3	BH-08	615+76	38.0	58.0	48.0	20	0	126	CL	2.8	1.00	5.8	5.8	0.6	1.7	2.1	1.00	0.0	0.8	0.8	0.9	013	3.4	1.43	0.0	1.2	1.3
24	3	BH-09	619+91	38.5	58.5	48.5	19.5	0	126	CH	2.9	1.00	5.3	5.3	0.6	1.8	2.1	1.00	0.5	0.9	0.9	1.0	015	3.7	1.20	0.0	1.3	1.3
25	3	BH-10	624+90	42.0	62.0	52.0	21.3	0	126	CL	3.1	1.00	5.6	5.6	0.6	1.9	2.3	1.00	0.6	0.9	0.9	1.0	018	4.4	1.25	0.0	1.3	1.5
26	3	BH-11	627+53	43.0	63.0	53.0	21.4	0	126	CH, some CL	3.1	1.00	5.2	5.2	0.6	1.9	2.3	1.00	1.2	0.9	1.2	1.0	019	4.5	1.02	0.0	1.5	1.4
27	3	BH-12	634+65	47.5	67.5	57.5	23.7	0	126	nterlayered CH & CL with some ML/CI	3.4	1.00	6.1	6.1	0.6	2.1	2.5	1.00	0.8	1.0	1.0	1.1	023	4.4	1.32	0.0	1.4	1.7
28	3	BH-13	640+82	51.0	71.0	61.0	26.1	0	126	CL with some SM	3.6	1.00	9.2	9.2	0.6	2.2	2.6	1.00	0.0	1.0	1.0	1.1	026	2.8	2.74	0.0	1.4	1.9
29	3	BH-14	642+55	52.0	72.0	62.0	26.1	0	126	CH	3.7	1.00	6.3	6.3	0.6	2.2	2.7	1.00	1.2	1.1	1.2	1.2	165	3.7	1.28	0.0	1.5	1.9
30	3	BH-88	645+03	51.5	71.5	61.5	25.3	0	126	CL with ML, SM	3.6	1.00	7.5	7.5	0.6	2.2	2.7	1.00	0.0	1.1	1.1	1.2	027	3.5	1.88	0.0	1.5	1.9
31	3	BH-15	645+73	51.0	71.0	61.0	24.3	0	126	CH	3.6	1.00	7.4	7.4	0.6	2.2	2.6	1.00	0.0	1.1	1.1	1.2	027	3.5	1.88	0.0	1.5	1.8
32	3	BH-87	648+42	51.5	71.5	61.5	24.4	0	126	CL	3.6	1.00	6.7	6.7	0.6	2.2	2.7	1.00	0.8	1.1	1.1	1.2	029	3.7	1.47	0.0	1.5	1.8
33	3	BH-16	650+43	44.5	64.5	54.5	16.5	0	126	CH with some CL	3.2	1.00	5.5	5.5	0.6	2.0	2.4	1.00	1.0	1.1	1.1	1.2	166	3.8	1.13	0.0	1.5	1.4
34	3	BH-17	654+53	43.5	63.5	53.5	14.7	0	126	CH with some ML at invert	3.2	1.00	5.9	5.9	0.6	1.9	2.4	1.00	0.5	1.2	1.2	1.3	031	4.3	1.36	0.0	1.6	1.3
35	3	BH-18	660+09	43.0	63.0	53.0	13	0	126	H/CL at crown, SP-SM, SW-SM at inv	3.1	1.00	5.9	5.9	0.6	1.9	2.4	1.00	0.5	1.2	1.2	1.3	033	3.4	1.35	0.0	1.6	1.2
36	3	BH-19	666+26	41.5	61.5	51.5	13	0	126	CH, some CL, GP & SM at invert	3.0	1.00	5.1	5.1	0.6	1.9	2.3	1.00	1.1	1.2	1.2	1.3	036	3.8	1.00	0.0	1.6	1.2
37	3	BH-20	669+79	41.5	61.5	51.5	14.6	0	126	CH, CL lens, GW-GC at invert	3.0	1.00	5.9	5.9	0.6	1.9	2.3	1.00	0.3	1.1	1.1	1.2	037	2.5	1.42	0.0	1.5	1.2
38	3	BH-21	675+49	40.0	60.0	50.0	16.2	0	126	CL, ML lens, SW-SC at invert	3.0	1.00	6.2	6.2	0.6	1.8	2.2	1.00	0.0	1.0	1.0	1.1	040	3.2	1.58	0.0	1.4	1.2
39	3	BH-50	681+71	36.5	56.5	46.5	15.8	0	126	CH, CL, SP-SM invert	2.7	1.00	5.0	5.0	0.6	1.7	2.1	1.00	0.6	0.9	0.9	1.0	096	4.3	1.08	0.0	1.3	1.1
40	3	BH-52	684+09	35.5	55.5	45.5	16.2	0	126	CH, CL, ML zone, & CL/SC at invert	2.7	1.00	5.1	5.1	0.6	1.6	2.0	1.00	0.4	0.9	0.9	1.0	098	4.3	1.16	0.0	1.3	1.1
41	3	BH-53	685+42	35.0	55.0	45.0	16.9	0	126	MH/CH, CL, ML	2.6	1.00	4.7	4.7	0.6	1.6	2.0	1.00	0.8	0.8	0.8	0.9	102	4.2	0.97	0.0	1.2	1.1
42	3	BH-54	687+16	34.5	54.5	44.5	17.9	0	126	CH & CL	2.6	1.00	4.9	4.9	0.6	1.6	1.9	1.00	0.5	0.8	0.8	0.9	103	4.5	1.09	0.0	1.2	1.1
43	3	BH-55	688+35	34.5	54.5	44.5	19	0	126	CH-MH & CL	2.6	1.00	4.8	4.8	0.6	1.6	1.9	1.00	0.6	0.8	0.8	0.8	105	3.9	1.07	0.0	1.1	1.2
44	3	BH-23	690+02	35.0	55.0	45.0	21	0	126	CL	2.6	1.00	4.9	4.9	0.6	1.6	1.9	1.00	0.6	0.7	0.7	0.8	105	3.9	1.07	0.0	1.1	1.3
45	3	BH-64	691+93	35.0	55.0	45.0	23.2	0	126	CL, SW-SM, ML	2.6	1.00	4.7	4.7	0.6	1.6	1.9	1.00	0.7	0.7	0.7	0.7	043	4.2	1.01	0.0	1.0	1.3
46	5	BH-70	706+78	36.0	56.0	46.0	29	0	126	CL, SM, SP-SM, CL-ML	2.7	1.00	5.7	5.7	0.6	1.7	1.9	1.00	0.0	0.5	0.5	0.6	142	4.0	1.44	0.0	0.9	1.5
47	5	BH-71	707+62	37.5	57.5	47.5	30	0	126	CH, SW-SM, CL	2.8	1.00	5.4	5.4	0.6	1.7	1.9	1.00	0.3	0.5	0.5	0.6	143	4.8	1.26	0.0	0.9	1.6
48	5	BH-72	709+39	39.5	59.5	49.5	28.6	0	126	CL	2.9	1.00	5.1	5.1	0.6	1.8	2.0	1.00	0.9	0.6	0.9	0.7	145A	5.3	1.07	0.0	1.2	1.5
49	5	BH-26	710+66	41.5	61.5	51.5	27.3	0	126	CL, SC, ML	3.0	1.00	5.3	5.3	0.6	1.9	2.2	1.00	0.9	0.7	0.9	0.8	145	4.6	1.08	0.0	1.2	1.5
50	5	BH-27	714+96	47.0	67.0	57.0	22.9	42	126	CL, SM & SW-SM	3.4	5.04	17.4	13.2	0.6	2.1	2.5	0.20	0.7	1.0	1.5	1.1	050	NA	0.00		1.8	1.3
51	5	BH-28	720+22	58.0	78.0	68.0	21.3	0	126	CL, SM & SC	4.0	1.00	7.6	7.6	0.6	2.5	3.0	1.00	0.6	1.4	1.4	1.5	052	4.3	1.76	0.0	1.8	1.9
52	5	BH-29	723+88	65.5	85.5	75.5	22.1	42	126	CL, SP, CL-ML, & GW	4.5	5.04	23.0	16.5	0.6	2.7	3.4	0.20	0.9	1.6	2.2	1.7	053	NA	0.00		2.5	1.7
53	5	BH-30	728+01	54.5	74.5	64.5	22.7	42	126	GW-GC & CL	3.8	5.04	19.6	14.6	0.6	2.3	2.8	0.20	0.8	1.2	1.8	1.4	055	NA	0.00		2.1	1.5
54	5	BH-31	731+54	42.0	62.0	52.0	23	0	126	CL, SC, & GW-GM?	3.1	1.00	6.2	6.2	0.6	1.9	2.2	1.00	0.1	0.9	0.9	1.0	057	4.8	1.54	0.0	1.3	1.5
55	5	BH-32	733+30	36.0	56.0	46.0	23	0	126	CL, ML, & SM	2.7	1.00	5.9	5.9	0.6	1.7	1.9	1.00	0.0	0.7	0.7	0.8	057	4.8	1.54	0.0	1.1	1.4
56	7	BH-34	744+65	39.0	59.0	49.0	31.7	0	126	CL, ML, SM	2.9	1.00	5.3	5.3	0.6	1.8	2.0	1.00	0.6	0.5	0.6	0.6	060	4.0	1.18	0.0	0.9	1.7
57	7	BH-35	750+48	39.0	59.0	49.0	27.6	0	126	CL	2.9	1.00	5.6	5.6	0.6	1.8	2.0	1.00	0.3	0.6	0.6	0.7	061	3.9	1.31	0.0	1.0	1.6
58	7	BH-36	755+33	43.5	63.5	53.5	28.9	0	126	SW-SM & CL	3.2	1.00	6.1	6.1	0.6	1.9	2.2	1.00	0.4	0.7	0.7	0.8	063	3.8	1.44	0.0	1.1	1.8
59	7	BH-37	760+59	47.5	67.5	57.5	29.3	39	126	CL, SM, & SW	3.4	4.40	15.3	12.4	0.6	2.1	2.4	0.23	0.8	0.8	1.4	0.9	066	NA	0.00		1.7	1.4
60	7	BH-38	765+23	46.0	66.0	56.0	24.6	39	126	SM, ML, & SP-SC?	3.3	4.34	14.7	11.5	0.6	2.0	2.4	0.23	0.8	0.9	1.5	1.0	068	NA	0.00		1.8	1.2
61	7	BH-39	768+77	47.0	67.0	57.0	22.8	43	126	GW & GP-GC?	3.4	5.37	18.5	14.0	0.6	2.1	2.5	0.19	0.6	1.0	1.5	1.1	070	NA	0.00		1.8	1.3
62	7	BH-95	774+14	49.0	69.0	59.0	21.2	45	126	CL, CL-ML, SP-SM, GW, GM, SM	3.5	5.83	20.8	15.3	0.6	2.1	2.6	0.17	0.6	1.1	1.5	1.2	073	NA	0.00		1.8	1.3
63	7	BH-40	775+76	49.0	69.0	59.0	20.3	0	126	CL & SM	3.5	1.00	6.5	6.5	0.6	2.1	2.6	1.00	0.6	1.2	1.2	1.3	073	3.1	1.47	0.0	1.6	1.6
64	7	BH-41	781+35	52.5	72.5	62.5	19.7	44	126	SP/no data	3.7	5.42	20.4	14.8	0.6	2.3	2.8	0.18	0.7	1.3	1.7	1.4	076	NA	0.00		2.0	1.4
65	7	BH-79	782+50	53.5	73.5	63.5	19.8	0	126	CL	3.8	1.00	7.1	7.1	0.6	2.3	2.8	1.00	0.6	1.3	1.3	1.4	076	3.6	1.61	0.0	1.7	1.7
66	7	BH-42	785+37	56.5	76.5	66.5	21	44	126	CL/no data	3.9	5.42	21.7	15.7	0.6	2.4	3.0	0.18	0.7	1.4	1.9	1.5	077	NA	0.00		2.2	1.5
67	7	BH-81	789+62	54.5	74.5	64.5	20.4	0	126	CL, SC	3.8	1.00	6.3	6.3	0.6	2.3	2.9	1.00	1.4	1.3	1.4	1.5	079	6.5	1.23	0.0	1.8	1.8
68	7	BH-43	789+71	54.0	74.0	64.0	19.9	0	126	CL/no data	3.8	1.00	6.3	6.3	0.6	2.3	2.8	1.00	1.4	1.3	1.4	1.5	079	6.5	1.23	0.0	1.8	1.7
69	7	BH-80	794+39	50.0	70.0	60.0	18	43	126	CL, SP-SM, & SM	3.6	5.29	19.2	13.8	0.6	2.2	2.7	0.19	0.7	1.3	1.7	1.4	171	NA	0.00		2.0	1.3
70	7	BH-102	796+49	51.5	71.5	61.5	20.6	0	126	CL, ML, SW, GW-GM	3.6	1.00	7.3	7.3	0.6	2.2	2.7	1.00	0.1	1.2	1.2	1.3	171	4.1	1.81	0.0	1.6	1.7
71	7	BH-103	798+17	50.0	70.0	60.0	20	43	126	GW-GM, CL, ML, GW, SP, SW, ML	3.6	5.21	18.9	13.8	0.6	2.2	2.7	0.19	0.7	1.2	1.7	1.3	082	NA	0.00		2.0	1.3
72	7	BH-44	798+27	50.0	70.0	60.0	20.1	43	126	GW-GM & GP-GM	3.6	5.21	18.9	13.9	0.6	2.2	2.7	0.19	0.7	1.2	1.7	1.3	082	NA	0.00		2.0	1.3
73	7	BH-106	800+21	48.5	68.5	58.5	19.6	43	126	GW-GM & ML	3.5	5.21	18.4	13.5	0.6	2.1	2.6	0.19	0.7	1.2	1.6	1.3	082	NA	0.00		1.9	1.2
74	7	BH-45	802+44	46.5	66.5	56.5	19.2	40	126	SW, SW-SM & CL	3.3	4.60	15.7	11.7	0.6	2.0	2.5	0.22	0.7	1.1	1.6	1.2	083	NA	0.00		1.9	1.1
75	7	BH-46	809+36	42.5	62.5	52.5	19.2	0	126	GW-GC? & CL-ML	3.1	1.00	6.4	6.4	0.6	1.9	2.3	1.00	0.0	1.0	1.0	1.1	086	3.5	1.61	0.0	1.4	1.4
76	7	BH-47	813+52	39.5	59.5	49.5	18.3	44	126	SP-SM & CL	2.9	5.42	16.2	12.1	0.6	1.8	2.2	0.18	0.6	0.9	1.3	1.0	087	NA	0.00		1.6	1.0
77	7	BH-48	818+34	38.0	58.0	48.0	19	0	126	CL & SM	2.8	1.00	5.9	5.9	0.6	1.7	2.1	1.00	0.0	0.9	0.9	1.0	089	4.7	1.49	0.0	1.3	1.3
78	7	BH-49	824+28	34.0	54.0	44.0	17	0	126	ML, CL, & SM	2.6	1.00	5.7	5.7	0.6	1.6	1.9	1.00	0.0	0.8	0.8	0.9	092	3.5	1.55	0.0	1.2	1.1
79	7	BH-78	831+41	13.0	33.0	23.0	14	0	122	CH, GW, GW-GM	1.3	1.00	3.9	3.9	0.6	0.8	0.9	1.00	0.0	0.3	0.3	0.3	093	1.9	1.25	0.0	0.9	0.1
80										
81										
82										
83	Notes:									
84	1	The uncorrected blowcount was used with the Sowers correlation in one case. Other equivalent stability numbers were estimated using field test data (such as pocket penetrometer tests).								
85	2	This column is used to check to see that there is an adequate factor of safety against heave. Heave pressure conservatively assumed to be vertical overburden pressure.								
86	3	This column shows the stability number recalculated using the target face pressure in the stability number equation as described in the text.								

For Reference Only

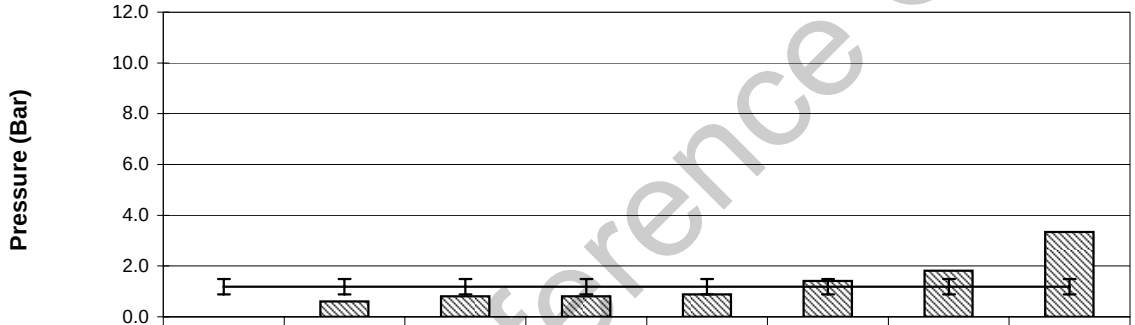
See disclaimer on cover page.

BH-57 (Sta. 569+31)



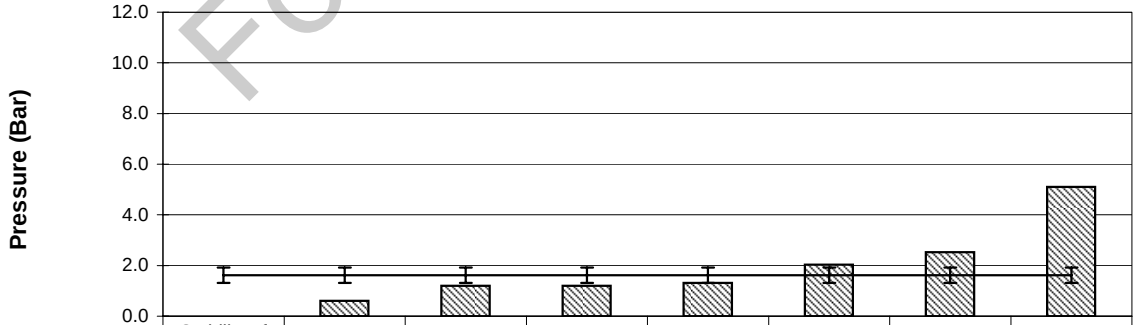
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.4	0.4	0.4	0.9	0.9	2.4
Target Pressure, bar	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

BH-01 (Sta. 574+30)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.8	0.8	0.9	1.4	1.8	3.3
Target Pressure, bar	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2

BH-02 (Sta. 578+10)

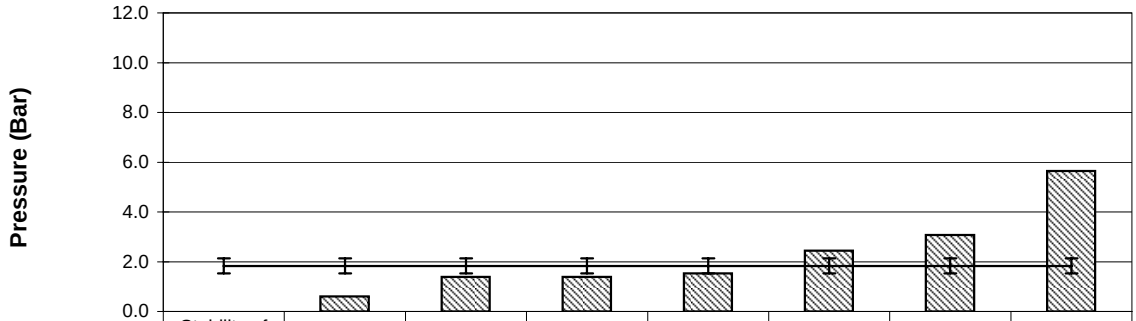


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.0	2.5	5.1
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

For Reference Only

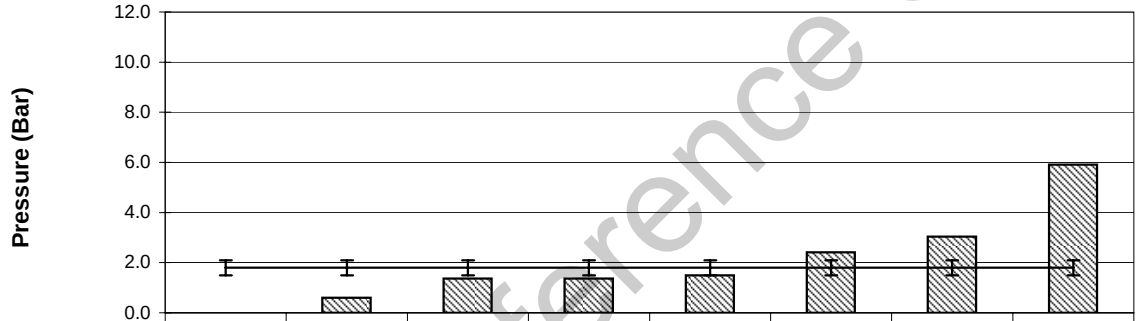
See disclaimer on cover page.

BH-03 (Sta. 581+83)



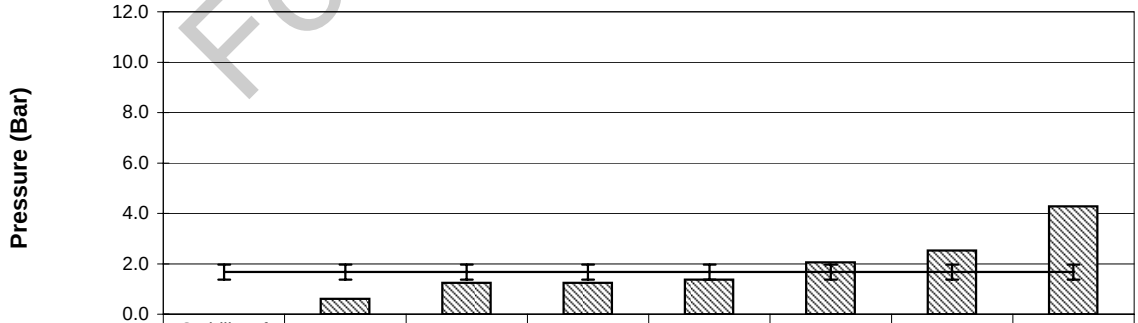
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.4	1.4	1.5	2.4	3.1	5.6
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

BH-04 (Sta. 590+53)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.4	1.4	1.5	2.4	3.0	5.9
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

BH-05 (Sta. 598+17)

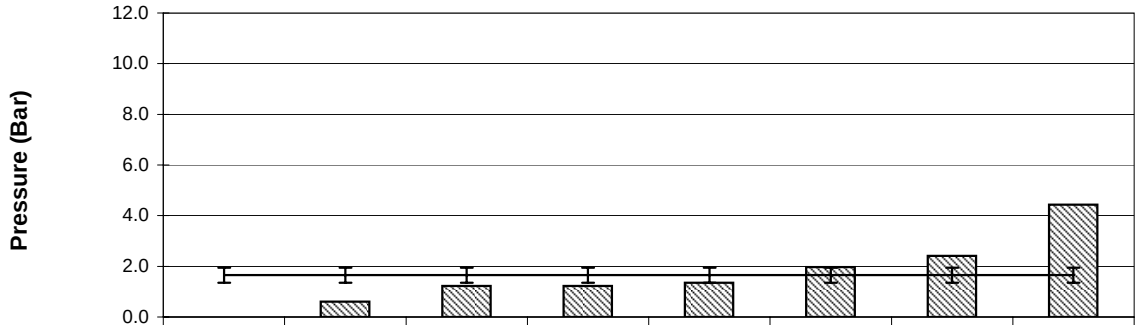


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.4	2.1	2.5	4.3
Target Pressure, bar	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7

For Reference Only

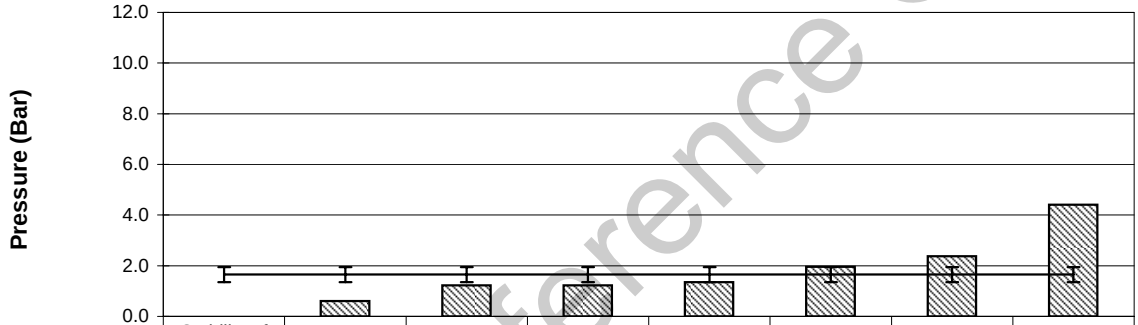
See disclaimer on cover page.

BH-06 (Sta. 599+58)



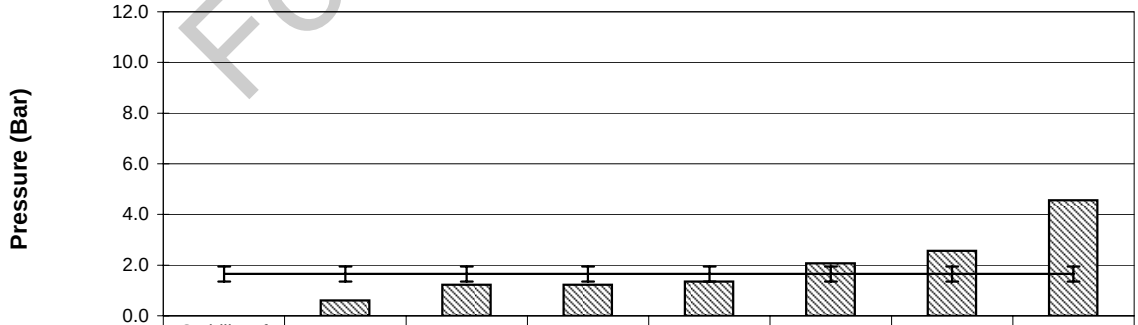
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.0	2.4	4.4
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

BH-83 (Sta. 599+83)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.0	2.4	4.4
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

BH-86 (Sta. 609+08)

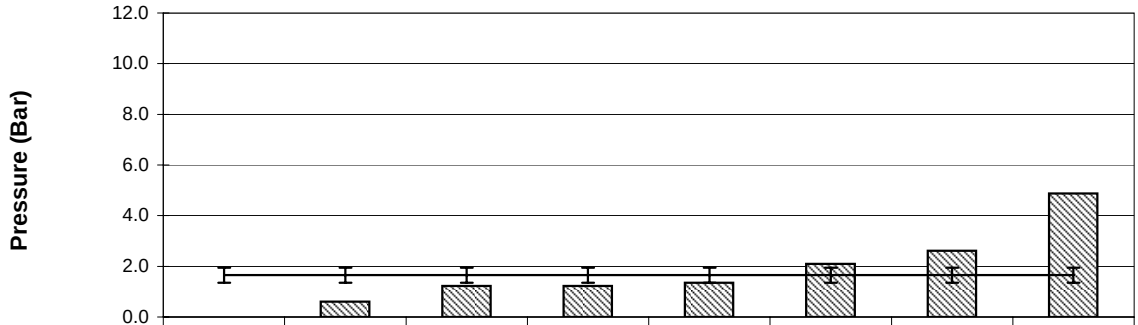


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.1	2.6	4.6
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

For Reference Only

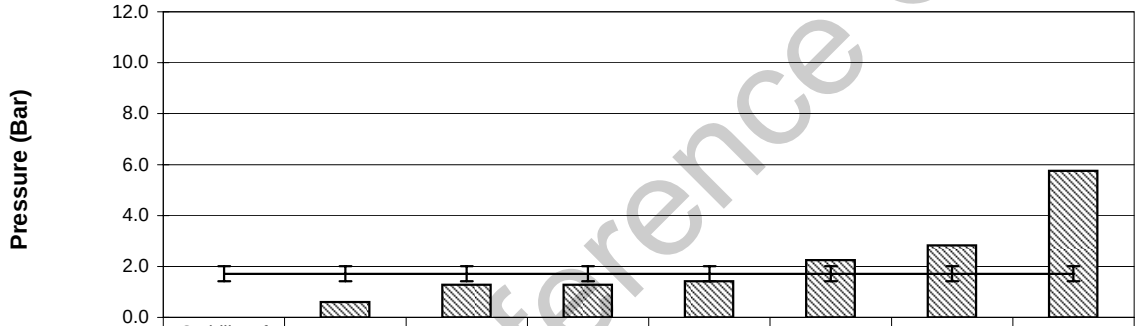
See disclaimer on cover page.

BH-07 (Sta. 609+41)



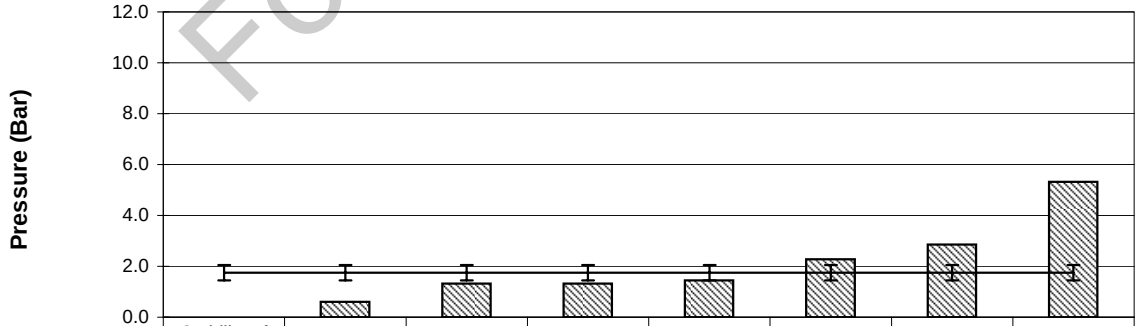
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.1	2.6	4.9
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

BH-08 (Sta. 615+76)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.3	1.3	1.4	2.3	2.8	5.8
Target Pressure, bar	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7

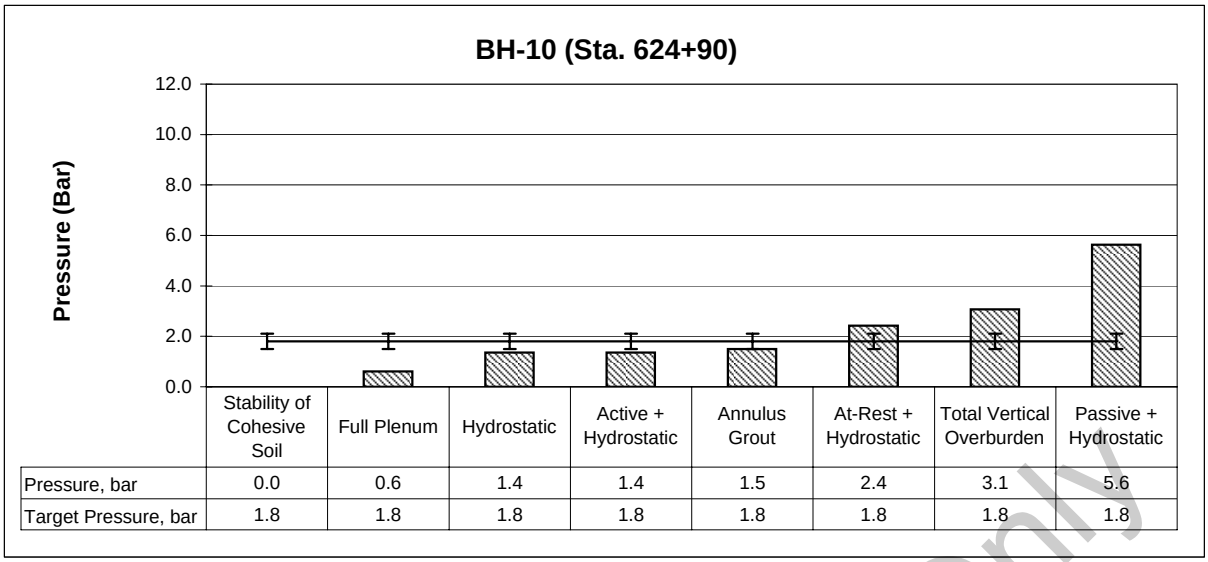
BH-09 (Sta. 619+90)



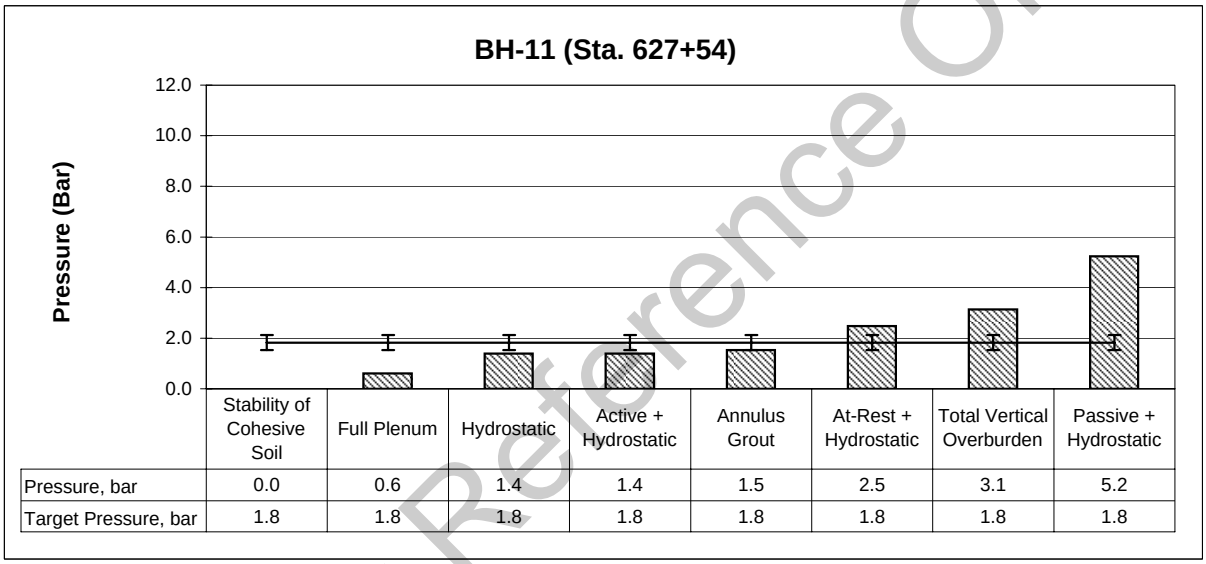
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.3	1.3	1.4	2.3	2.9	5.3
Target Pressure, bar	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7

For Reference Only

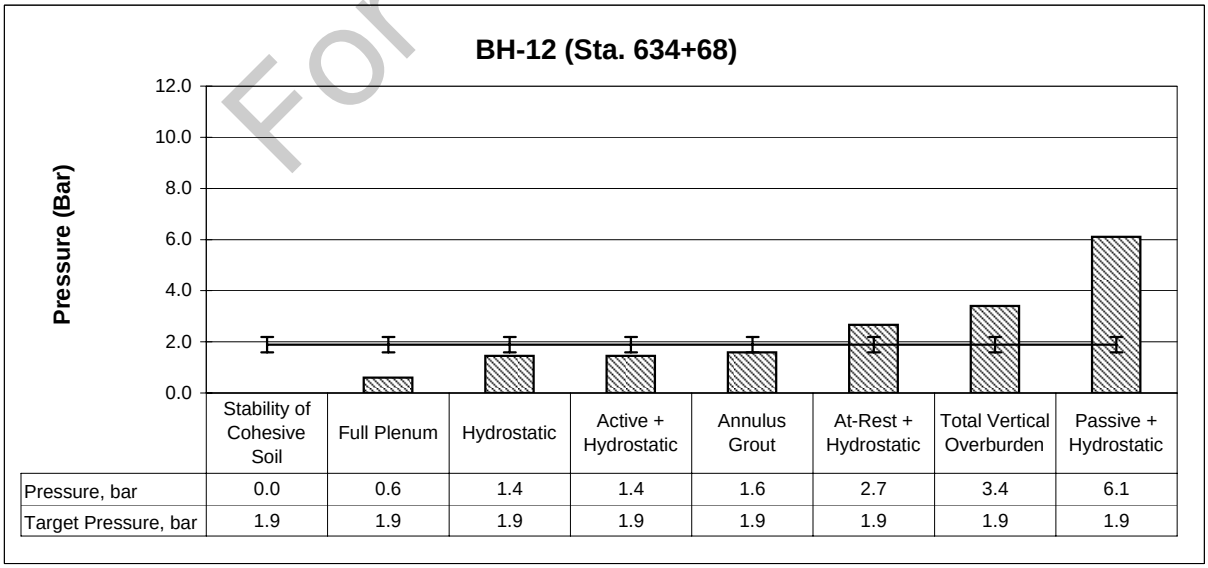
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.4	1.4	1.5	2.4	3.1	5.6
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8



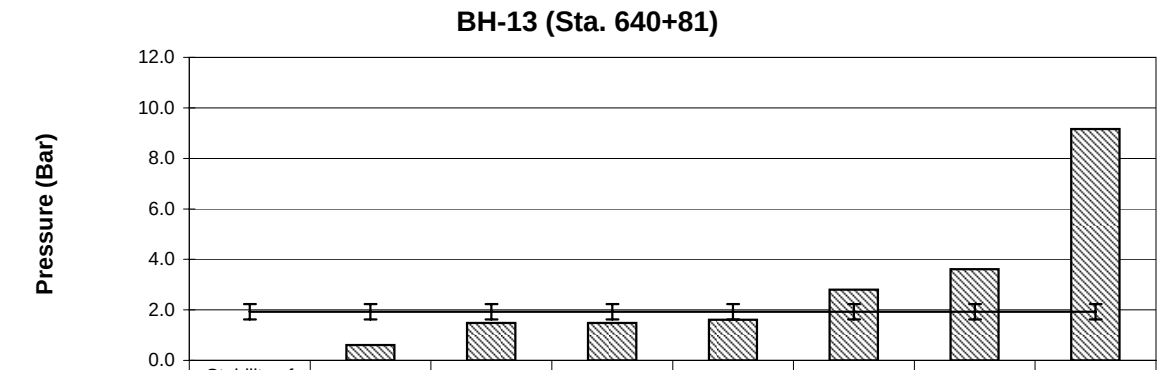
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.4	1.4	1.5	2.5	3.1	5.2
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8



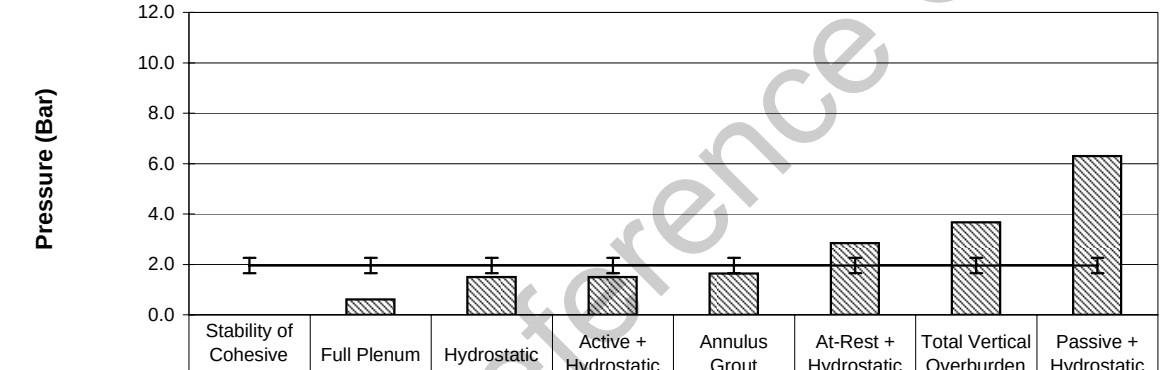
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.4	1.4	1.6	2.7	3.4	6.1
Target Pressure, bar	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9

For Reference Only

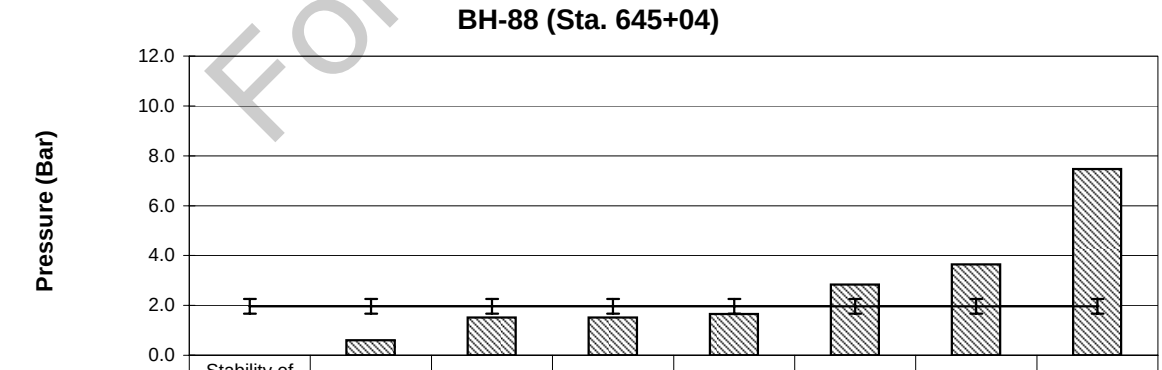
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.5	1.5	1.6	2.8	3.6	9.2
Target Pressure, bar	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9



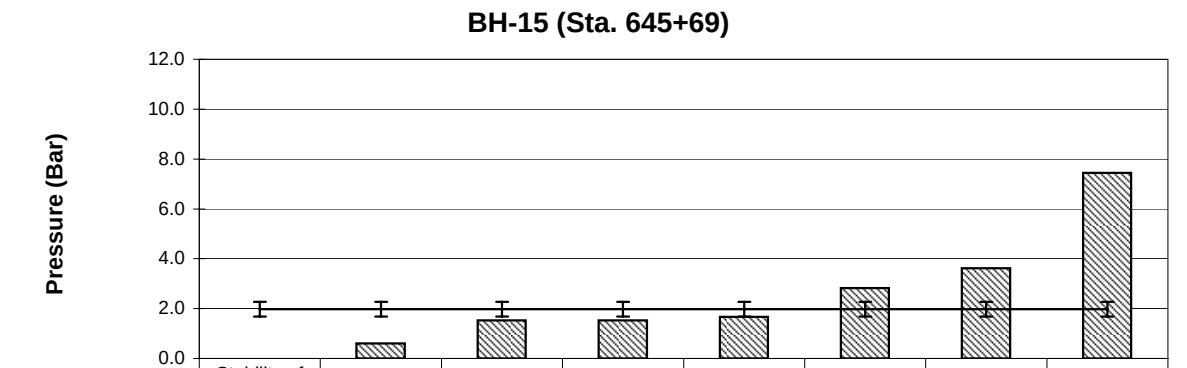
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.5	1.5	1.6	2.8	3.7	6.3
Target Pressure, bar	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0



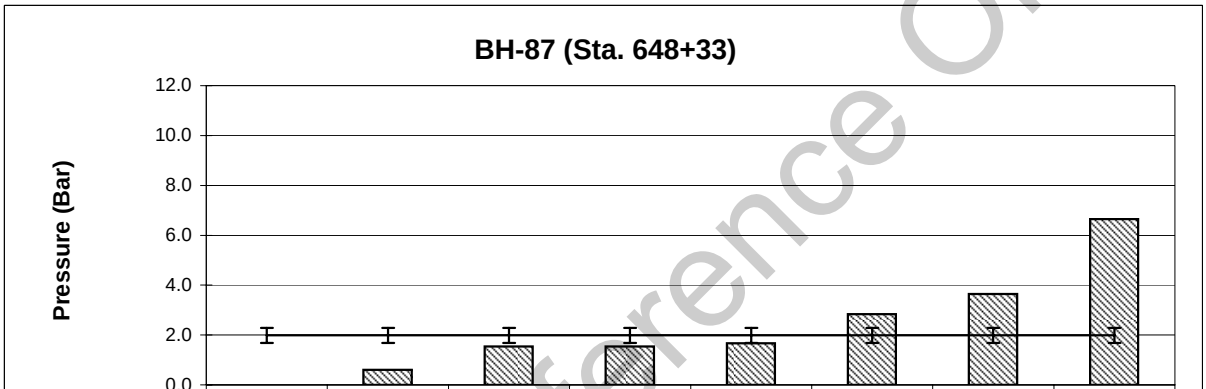
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.5	1.5	1.7	2.8	3.6	7.5
Target Pressure, bar	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

For Reference Only

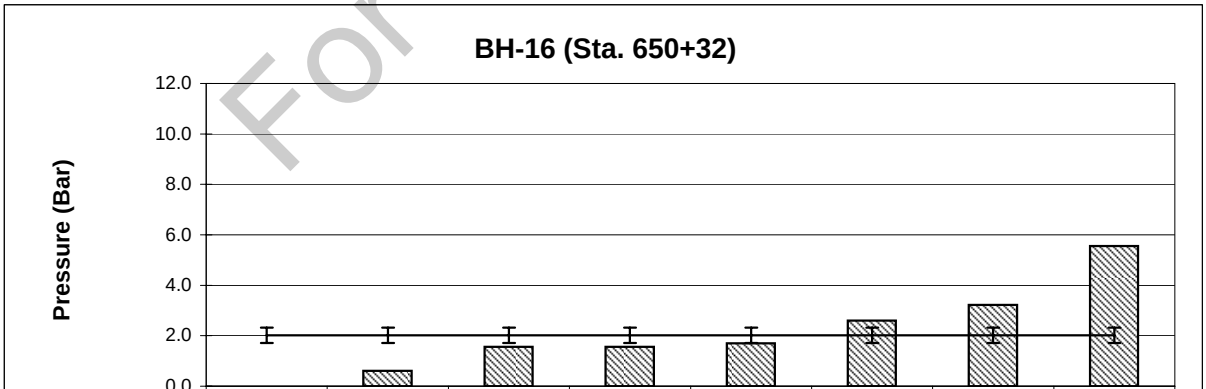
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.5	1.5	1.7	2.8	3.6	7.4
Target Pressure, bar	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.5	1.5	1.7	2.8	3.6	6.7
Target Pressure, bar	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

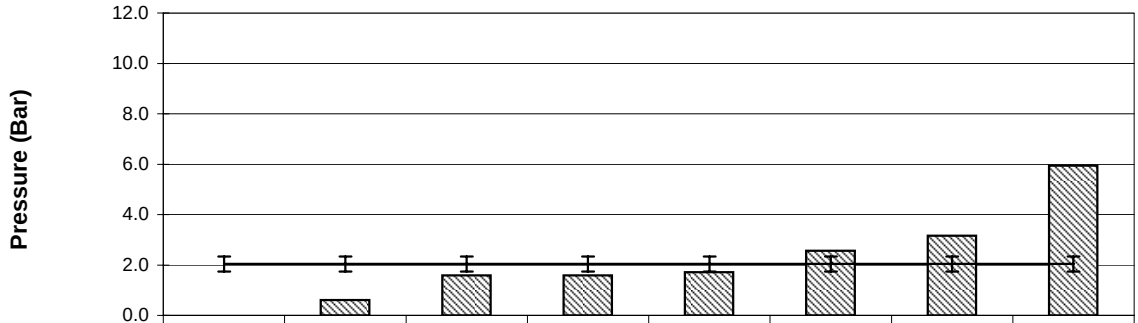


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.6	1.6	1.7	2.6	3.2	5.5
Target Pressure, bar	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

For Reference Only

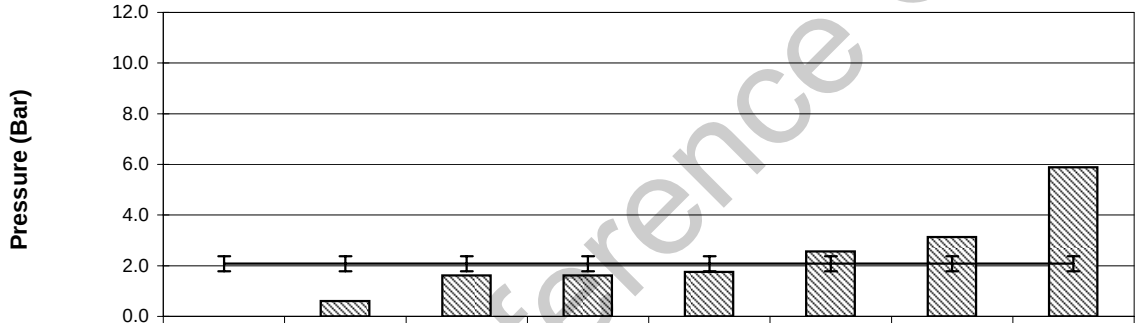
See disclaimer on cover page.

BH-17 (Sta. 654+44)



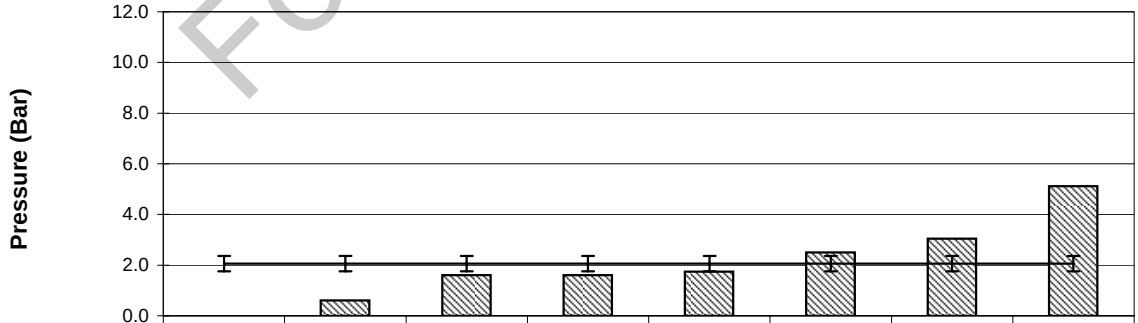
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.6	1.6	1.7	2.6	3.2	5.9
Target Pressure, bar	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

BH-18 (Sta. 660+03)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.6	1.6	1.8	2.6	3.1	5.9
Target Pressure, bar	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1

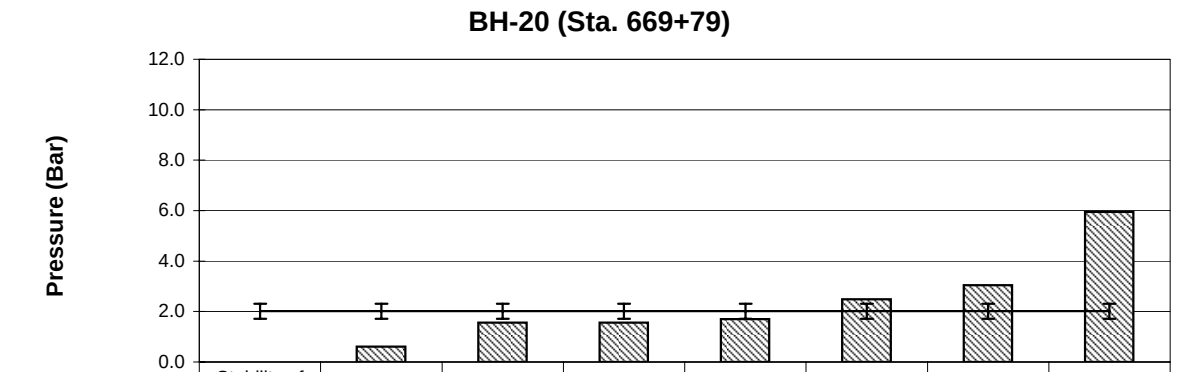
BH-19 (Sta. 666+26)



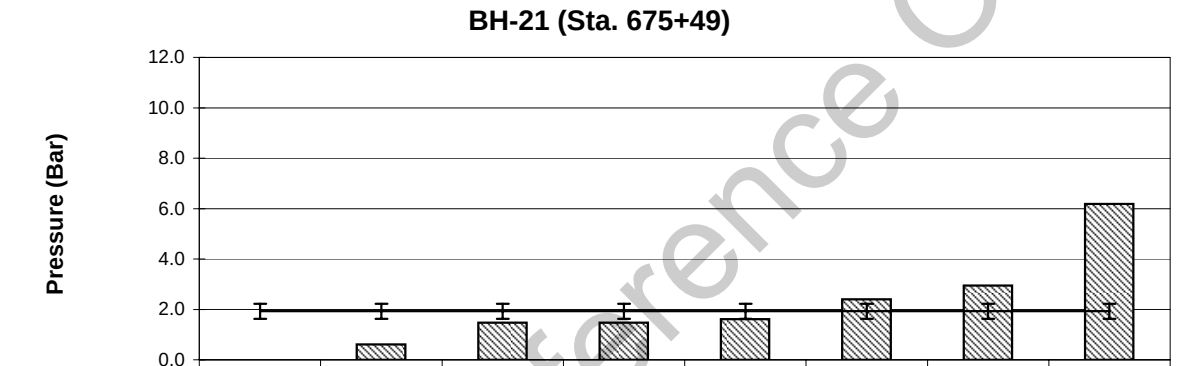
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.6	1.6	1.7	2.5	3.0	5.1
Target Pressure, bar	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1

For Reference Only

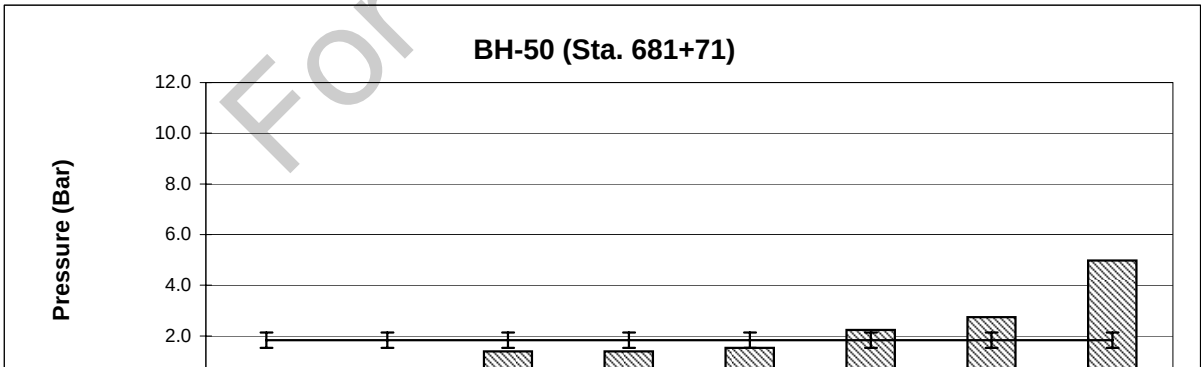
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.6	1.6	1.7	2.5	3.0	5.9
Target Pressure, bar	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.5	1.5	1.6	2.4	3.0	6.2
Target Pressure, bar	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9

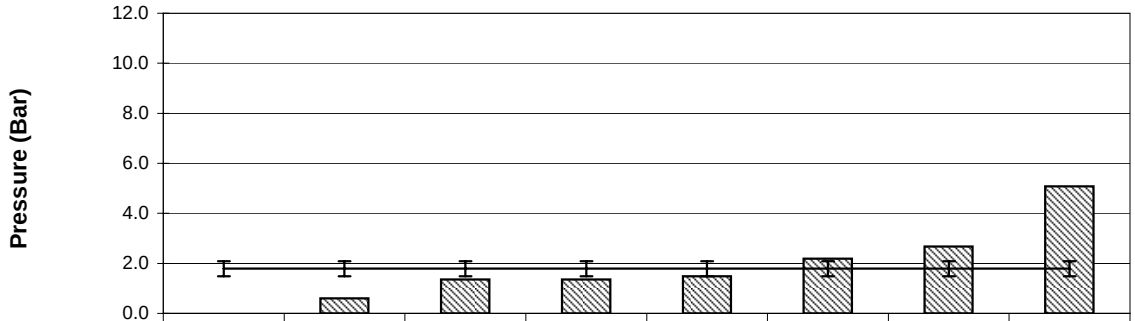


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.4	1.4	1.5	2.2	2.7	5.0
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

For Reference Only

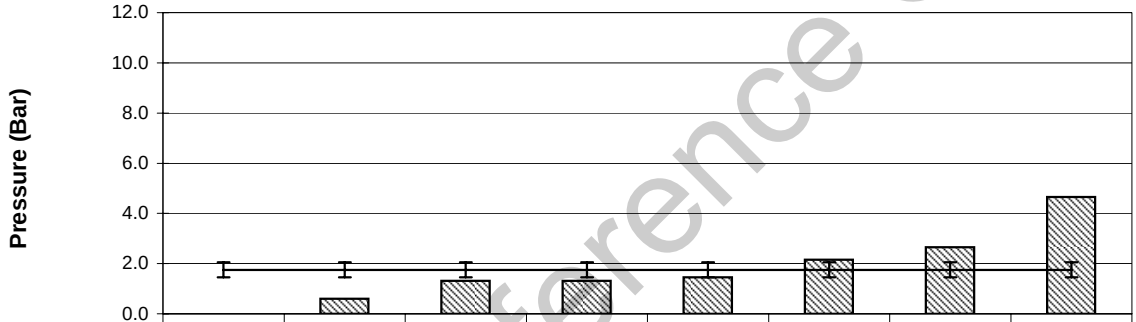
See disclaimer on cover page.

BH-52 (Sta. 684+09)



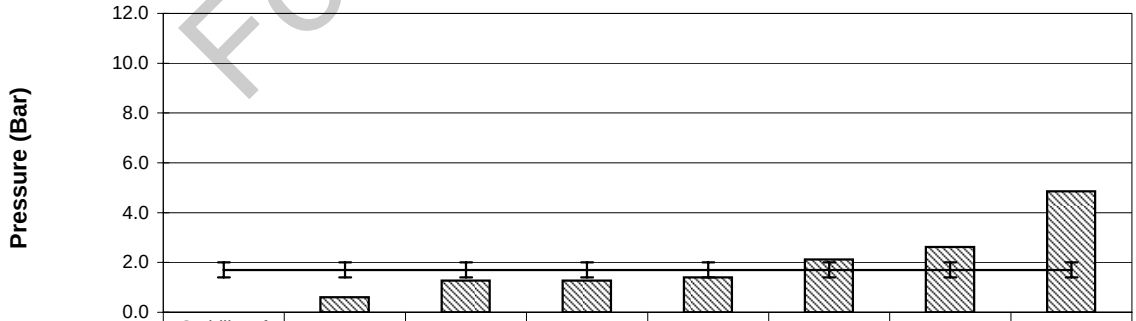
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.4	1.4	1.5	2.2	2.7	5.1
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

BH-53 (Sta. 685+42)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.3	1.3	1.5	2.2	2.6	4.7
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

BH-54 (Sta. 687+16)

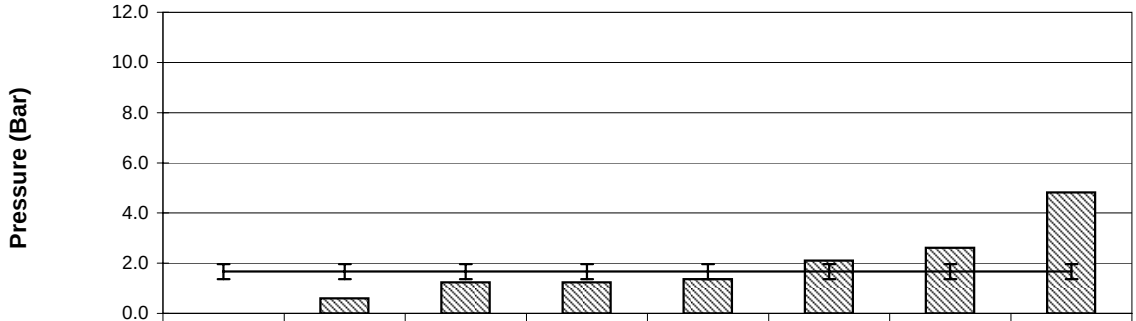


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.3	1.3	1.4	2.1	2.6	4.9
Target Pressure, bar	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7

For Reference Only

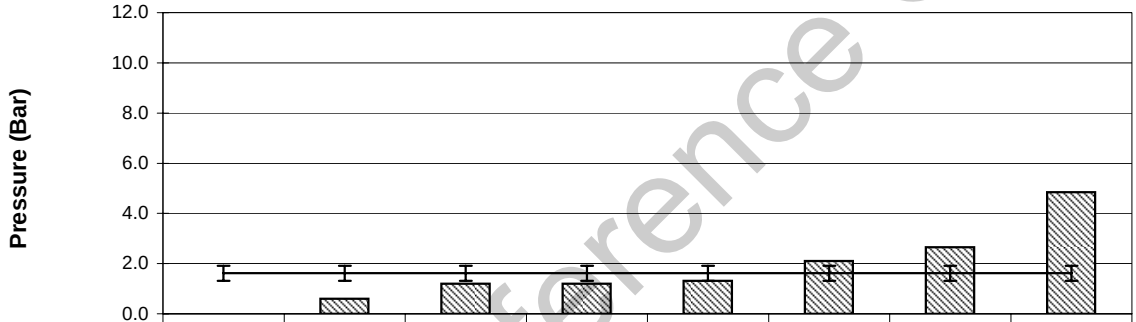
See disclaimer on cover page.

BH-55 (Sta. 688+28)



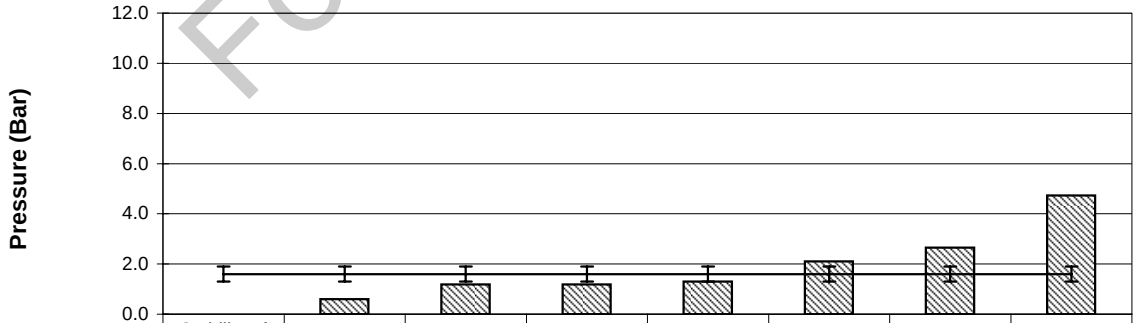
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.4	2.1	2.6	4.8
Target Pressure, bar	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7

BH-23 (Sta. 690+02)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.1	2.6	4.9
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

BH-64 (Sta. 691+92)

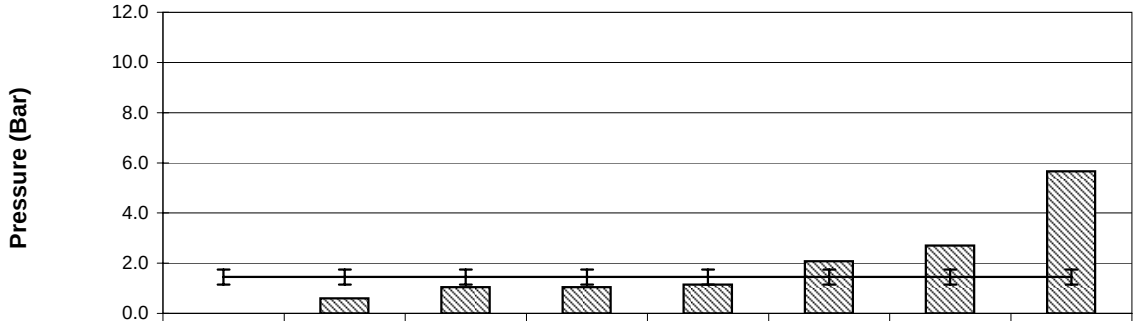


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.1	2.6	4.7
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

For Reference Only

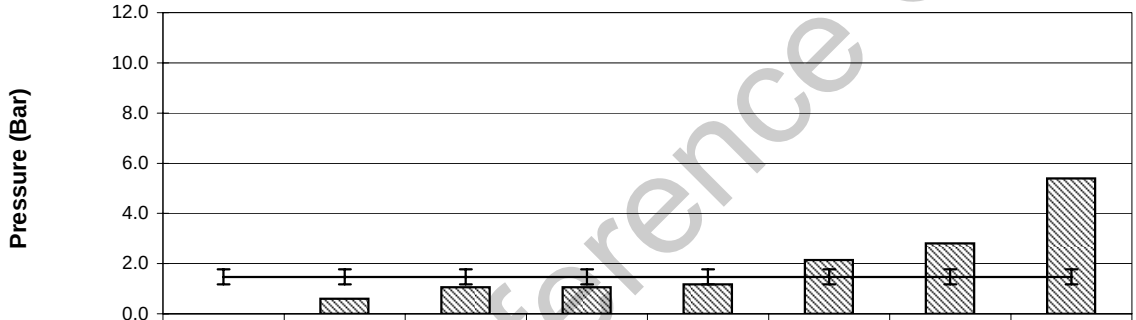
See disclaimer on cover page.

BH-70 (Sta. 706+78)



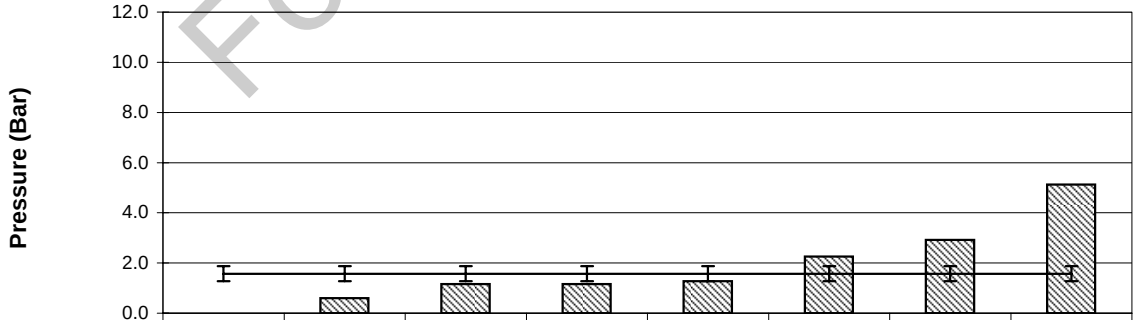
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.0	1.0	1.2	2.1	2.7	5.7
Target Pressure, bar	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5

BH-71 (Sta. 707+62)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.1	1.1	1.2	2.1	2.8	5.4
Target Pressure, bar	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5

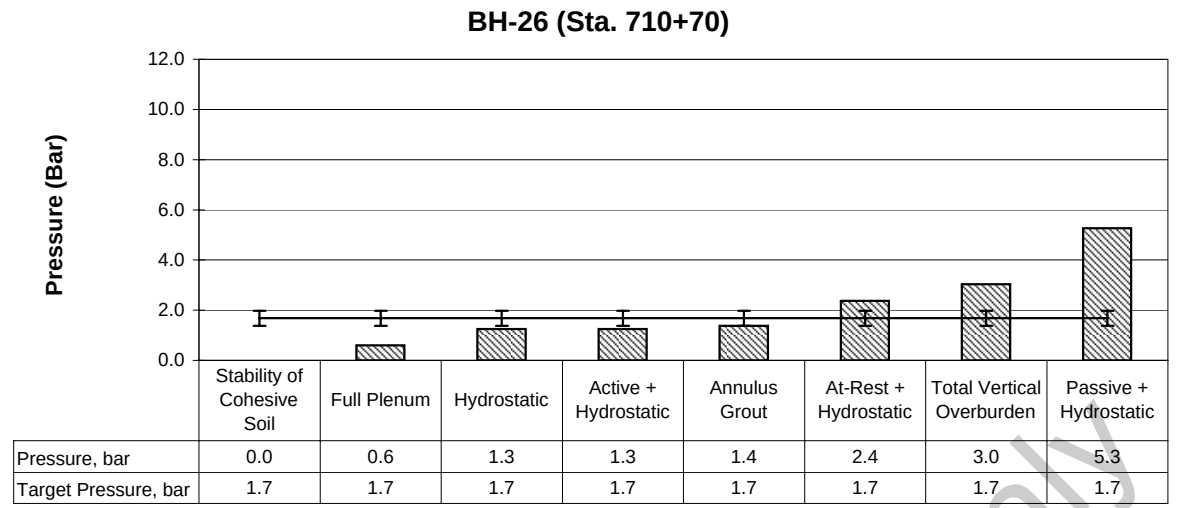
BH-72 (Sta. 709+40)



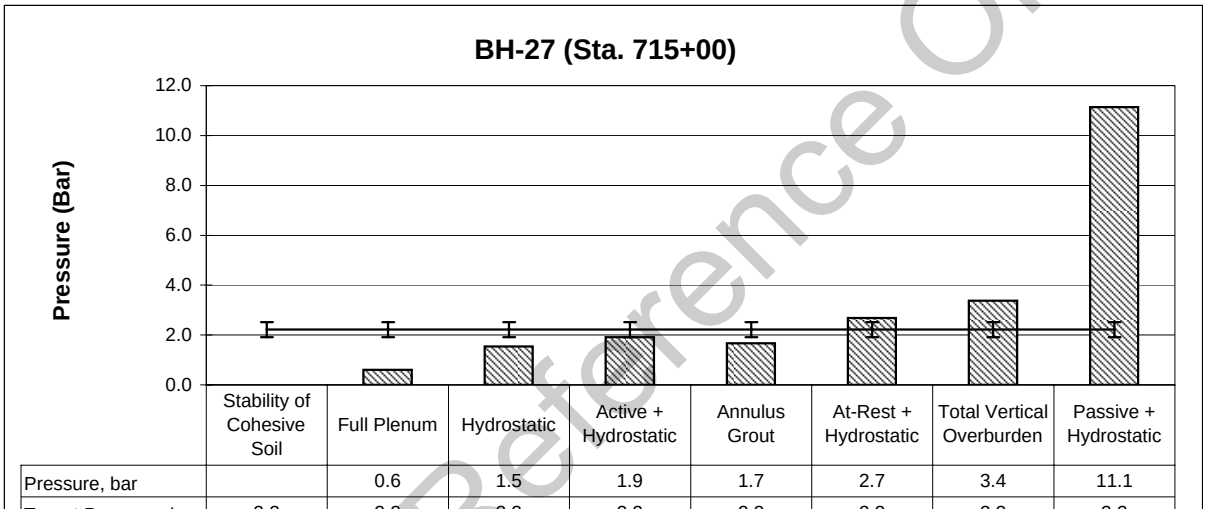
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.3	2.9	5.1
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

For Reference Only

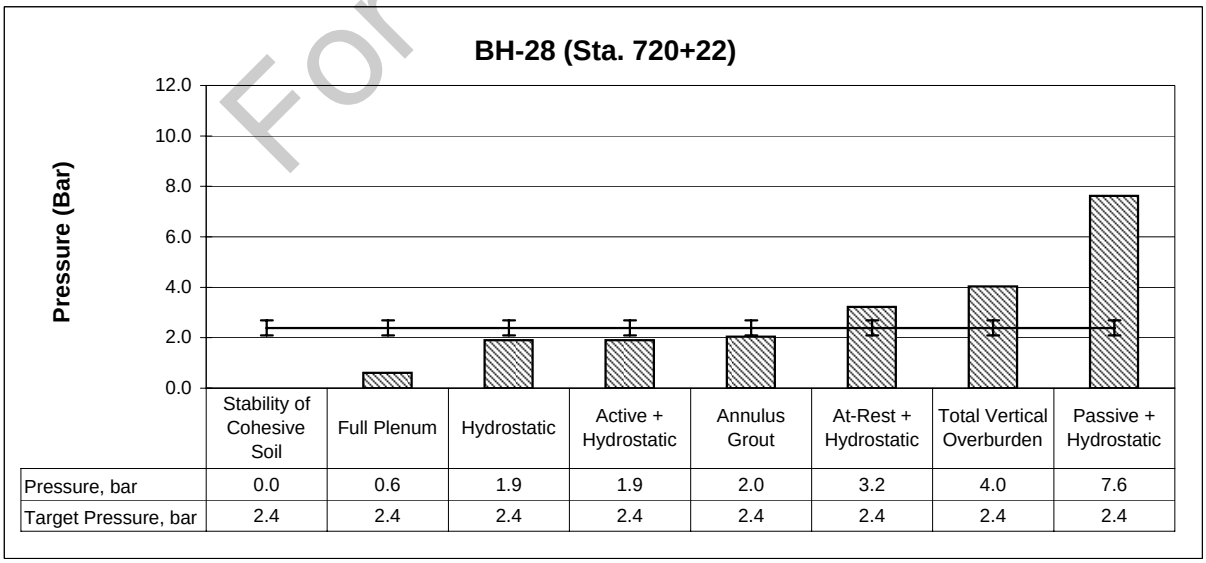
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.3	1.3	1.4	2.4	3.0	5.3
Target Pressure, bar	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.5	1.9	1.7	2.7	3.4	11.1
Target Pressure, bar	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2

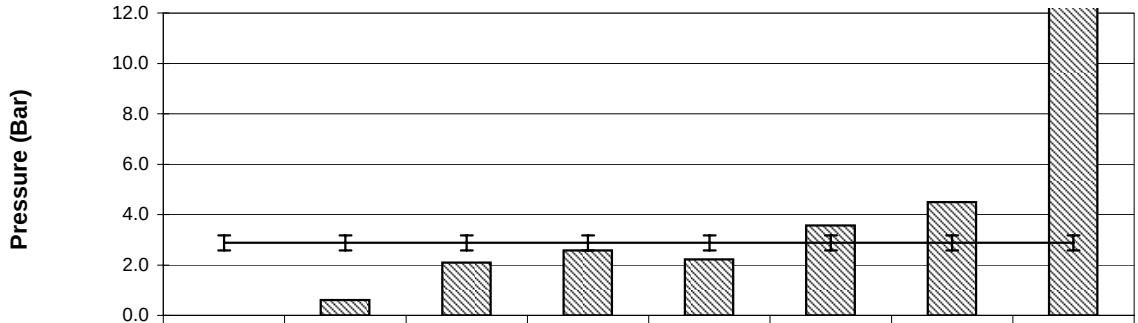


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.9	1.9	2.0	3.2	4.0	7.6
Target Pressure, bar	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4

For Reference Only

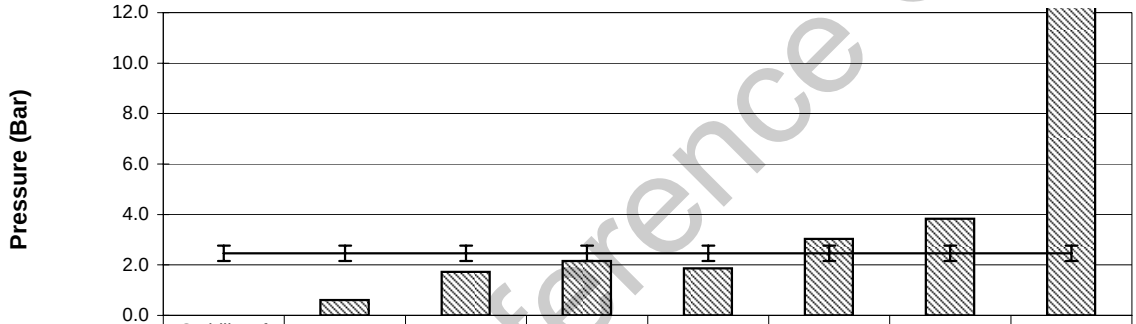
See disclaimer on cover page.

BH-29 (Sta. 723+88)



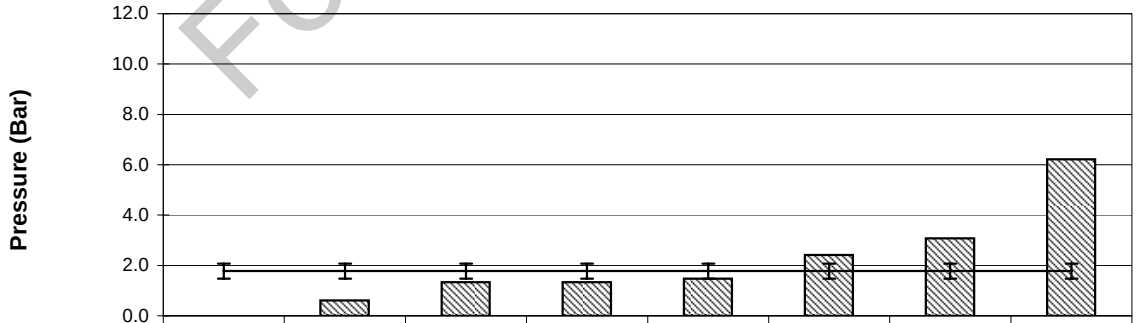
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	2.1	2.6	2.2	3.6	4.5	14.5
Target Pressure, bar	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9

BH-30 (Sta. 728+01)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.7	2.2	1.9	3.0	3.8	12.6
Target Pressure, bar	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5

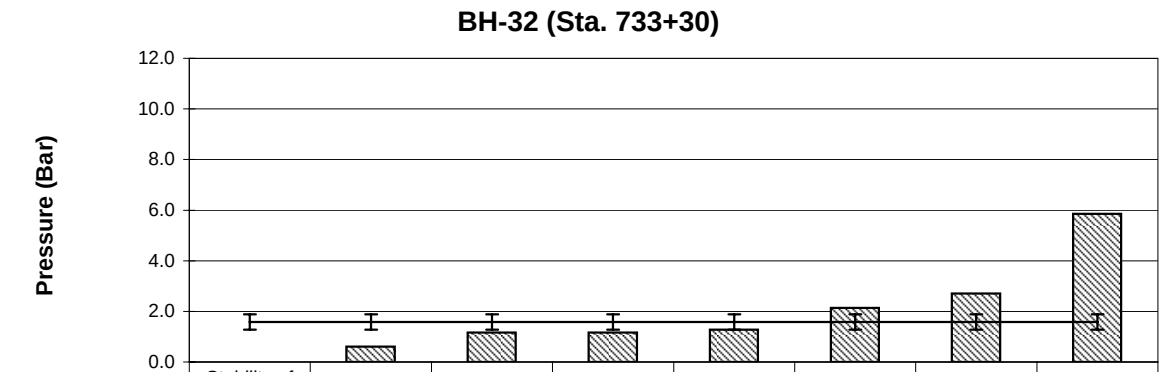
BH-31 (Sta. 731+54)



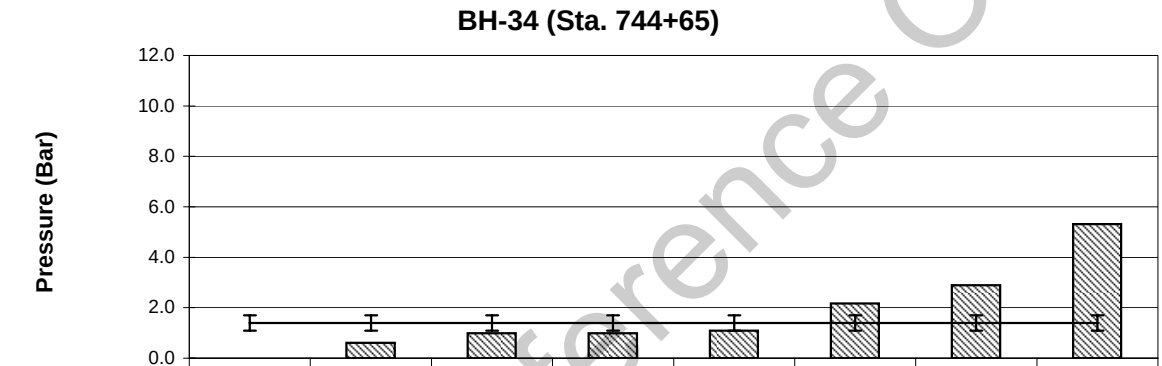
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.3	1.3	1.5	2.4	3.1	6.2
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

For Reference Only

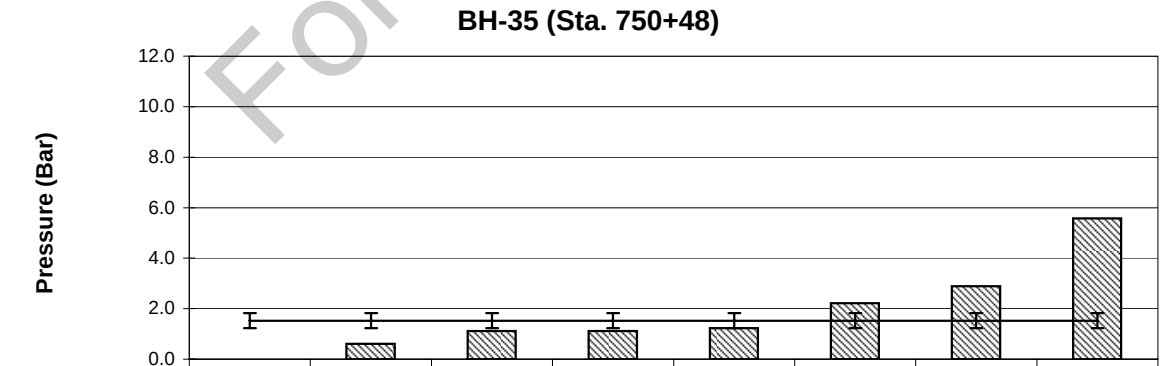
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.1	2.7	5.9
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6



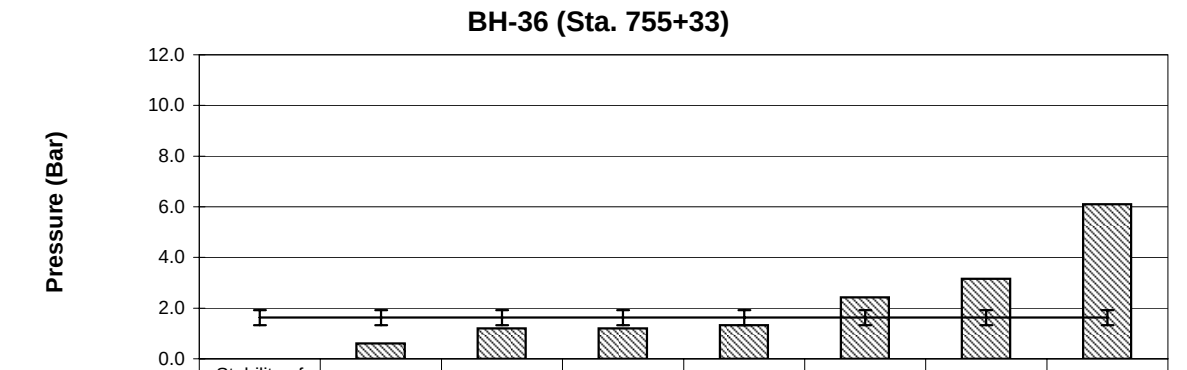
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.0	1.0	1.1	2.2	2.9	5.3
Target Pressure, bar	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4



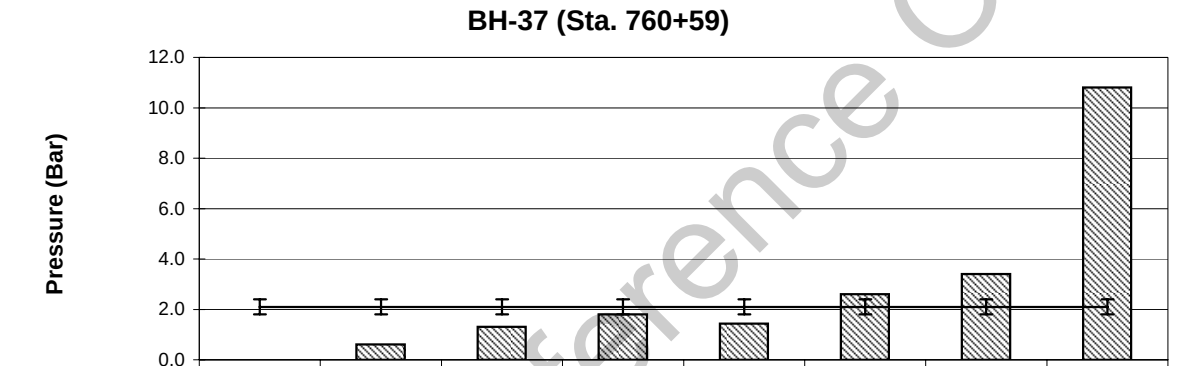
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.1	1.1	1.2	2.2	2.9	5.6
Target Pressure, bar	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5

For Reference Only

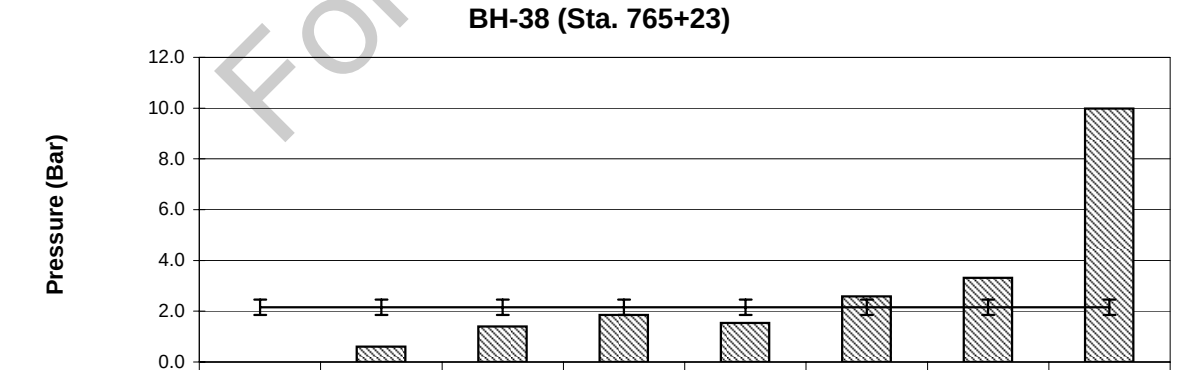
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.4	3.2	6.1
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6



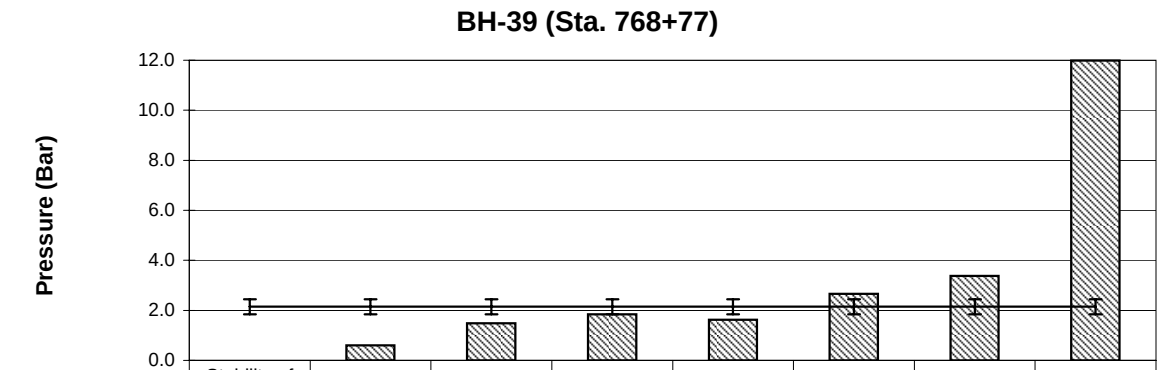
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.3	1.8	1.4	2.6	3.4	10.8
Target Pressure, bar	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1



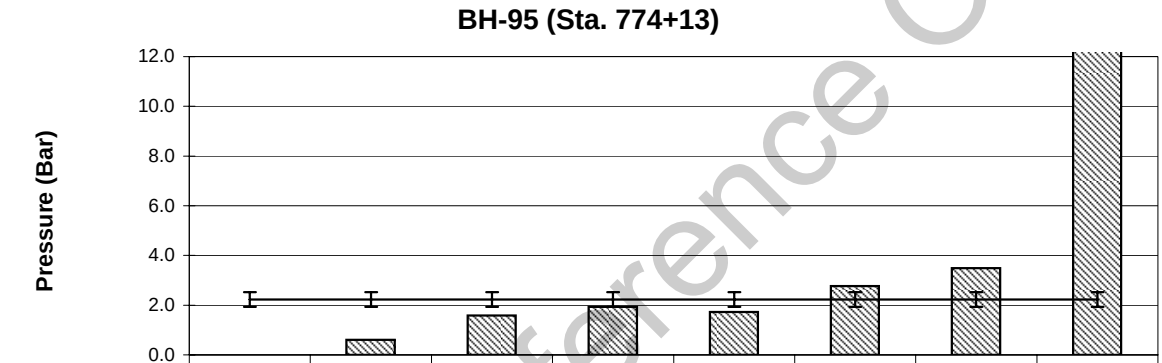
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.4	1.9	1.5	2.6	3.3	10.0
Target Pressure, bar	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2

For Reference Only

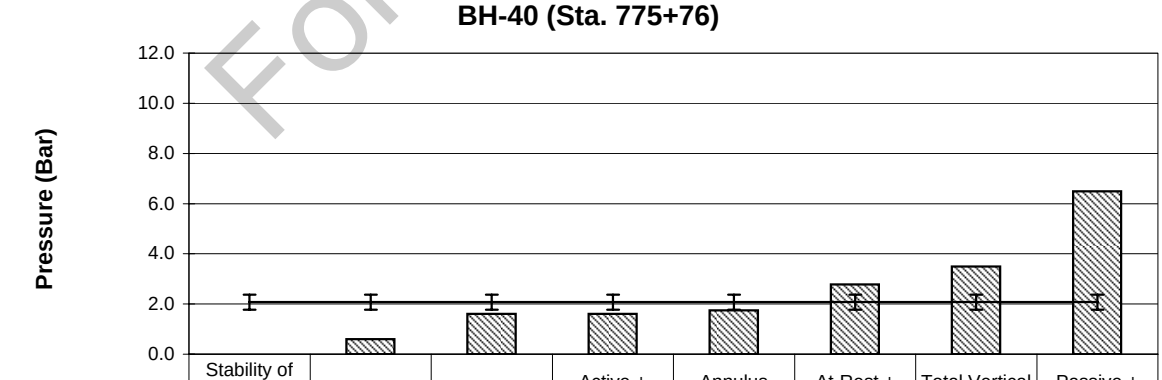
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.5	1.8	1.6	2.7	3.4	12.0
Target Pressure, bar	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.6	1.9	1.7	2.8	3.5	13.1
Target Pressure, bar	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2

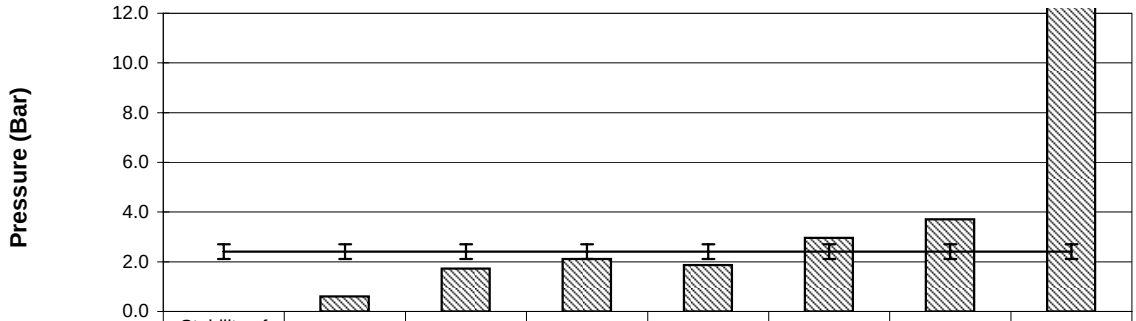


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.6	1.6	1.7	2.8	3.5	6.5
Target Pressure, bar	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1

For Reference Only

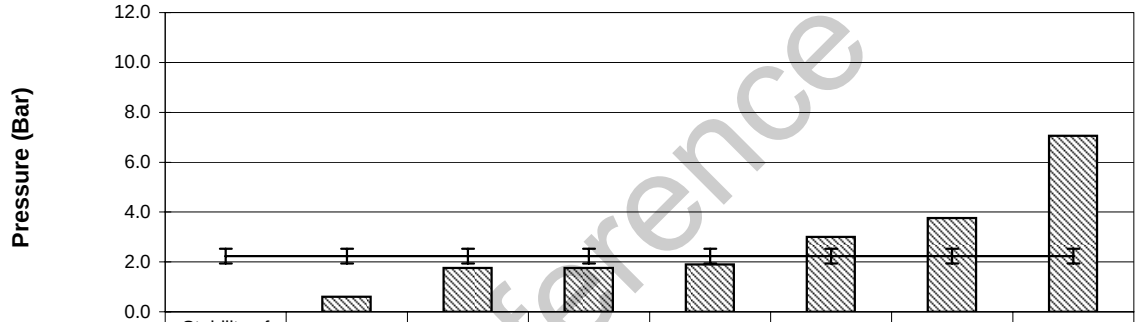
See disclaimer on cover page.

BH-41 (Sta. 781+35)



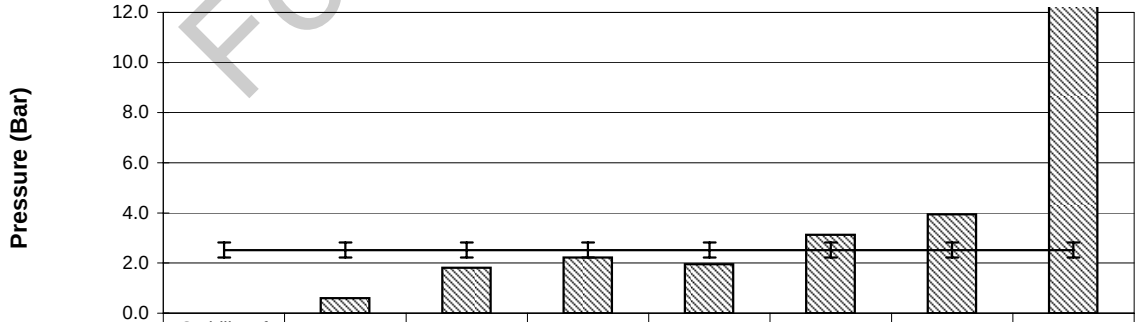
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.7	2.1	1.9	3.0	3.7	12.8
Target Pressure, bar	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4

BH-79 (Sta. 782+50)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.8	1.8	1.9	3.0	3.8	7.1
Target Pressure, bar	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2

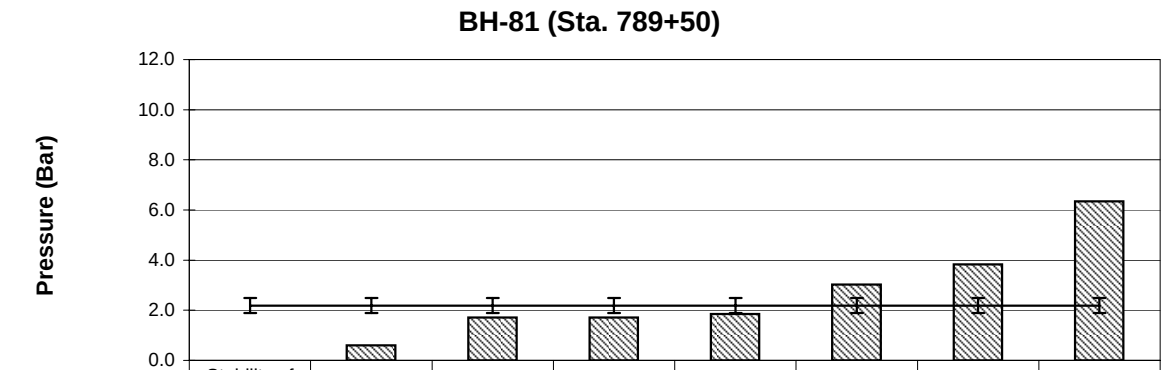
BH-42 (Sta. 785+37)



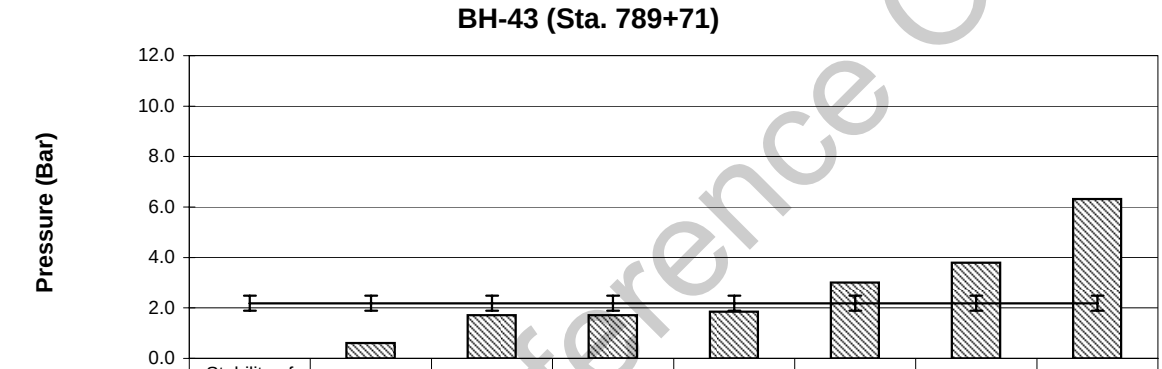
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.8	2.2	1.9	3.1	3.9	13.7
Target Pressure, bar	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5

For Reference Only

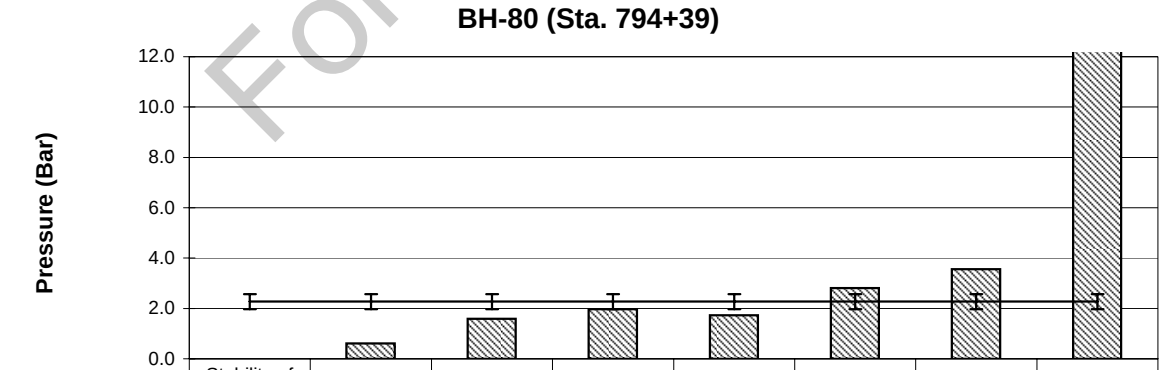
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.7	1.7	1.9	3.0	3.8	6.3
Target Pressure, bar	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.7	1.7	1.9	3.0	3.8	6.3
Target Pressure, bar	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2

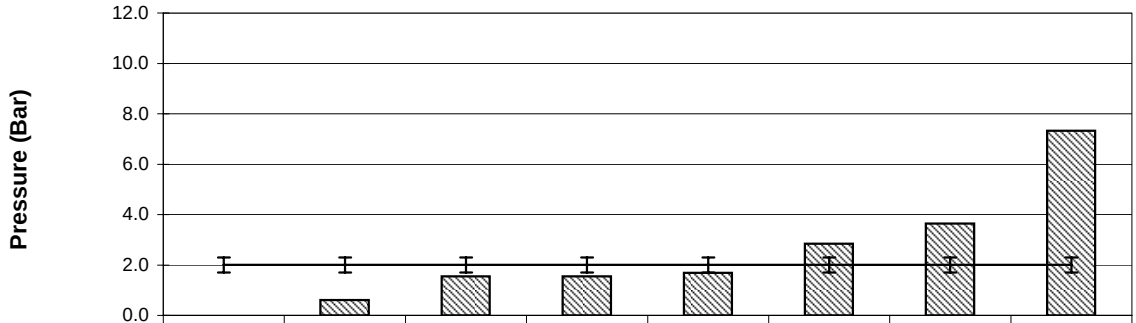


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.6	2.0	1.7	2.8	3.6	12.4
Target Pressure, bar	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3

For Reference Only

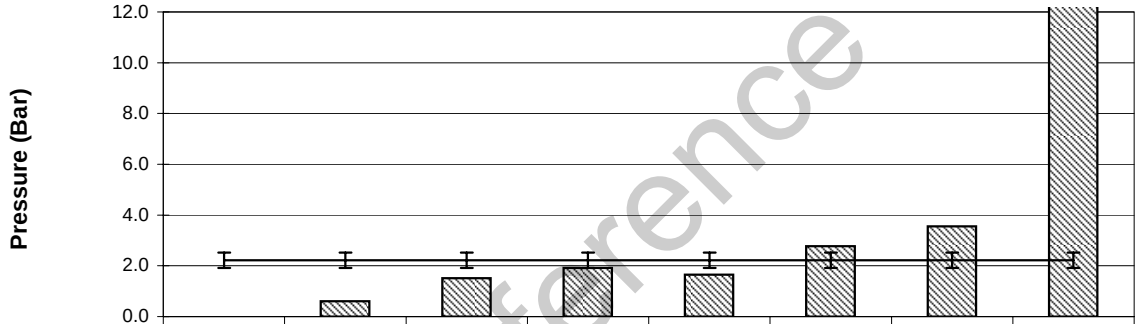
See disclaimer on cover page.

BH-102 (Sta. 796+48)



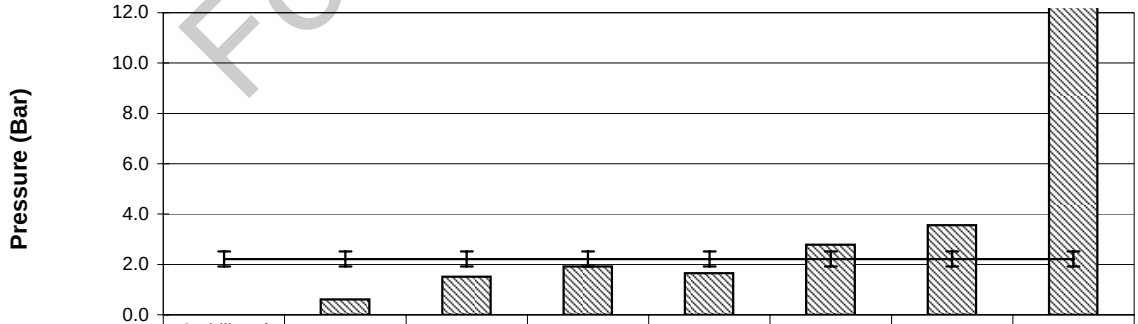
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.5	1.5	1.7	2.8	3.6	7.3
Target Pressure, bar	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

BH-103 (Sta. 798+16)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.5	1.9	1.7	2.8	3.6	12.5
Target Pressure, bar	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2

BH-44 (Sta. 798+27)

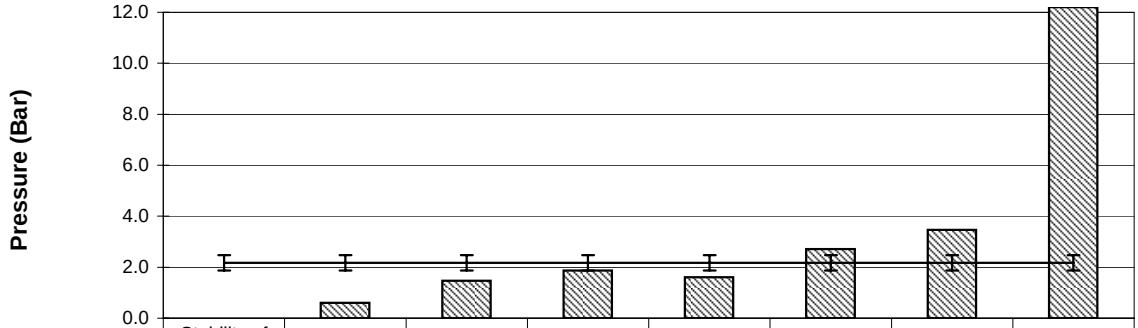


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.5	1.9	1.7	2.8	3.6	12.5
Target Pressure, bar	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2

For Reference Only

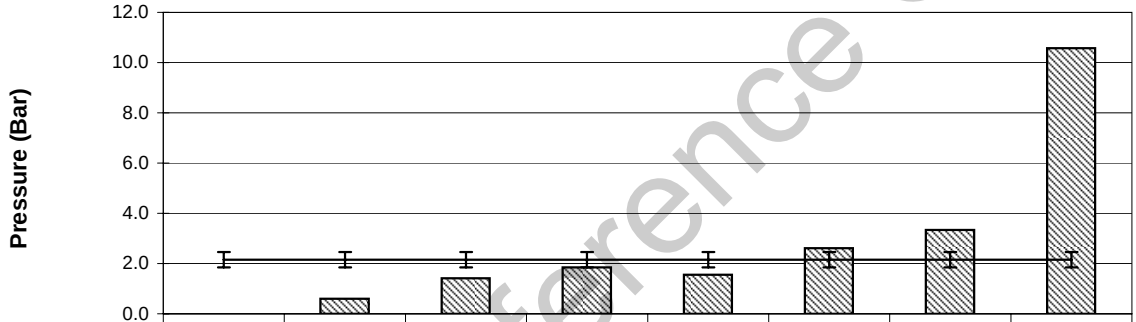
See disclaimer on cover page.

BH-106 (Sta. 800+20)



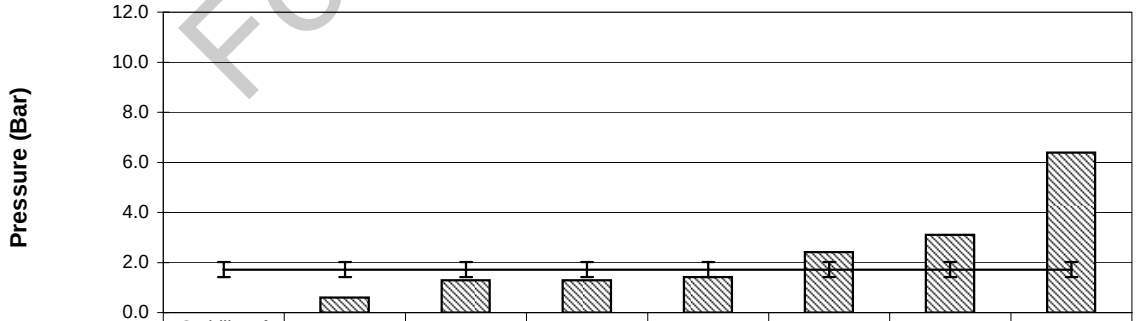
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.5	1.9	1.6	2.7	3.5	12.2
Target Pressure, bar	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2

BH-45 (Sta. 802+44)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.4	1.9	1.6	2.6	3.3	10.6
Target Pressure, bar	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2

BH-46 (Sta. 809+36)

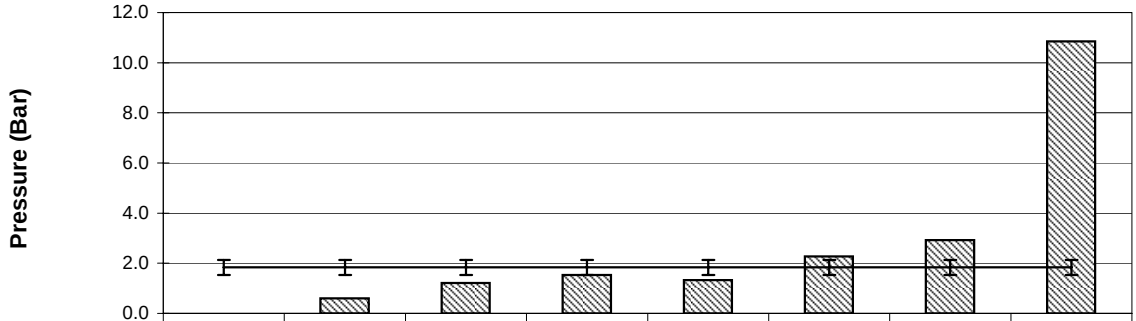


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.3	1.3	1.4	2.4	3.1	6.4
Target Pressure, bar	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7

For Reference Only

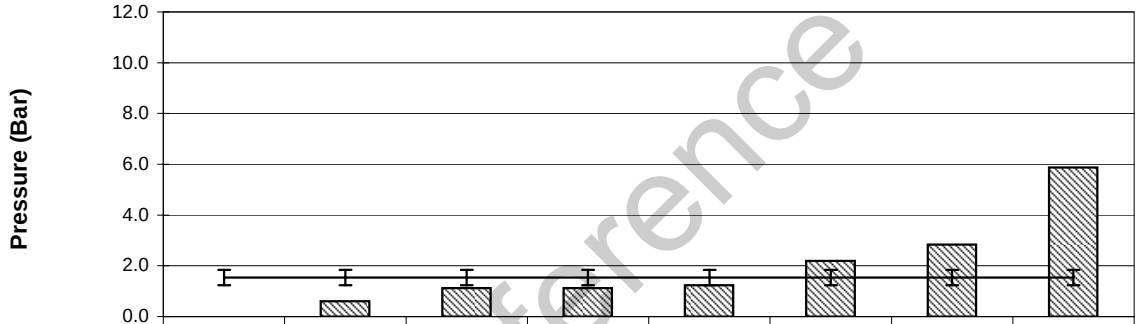
See disclaimer on cover page.

BH-47 (Sta. 813+52)



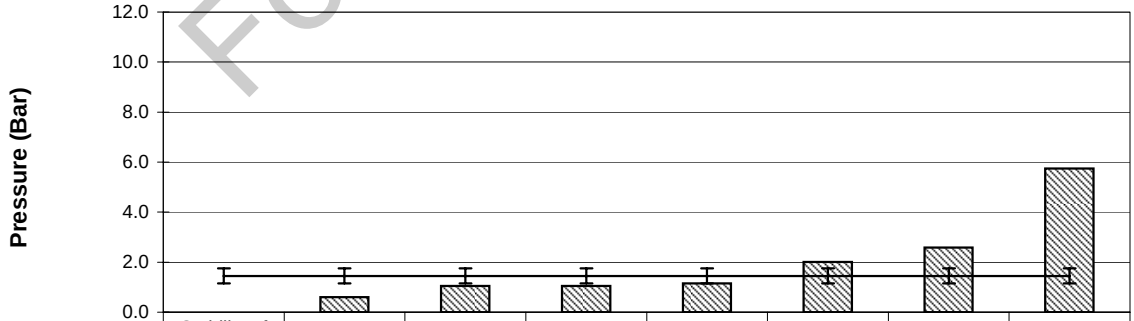
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.2	1.5	1.3	2.3	2.9	10.9
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

BH-48 (Sta. 818+34)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.1	1.1	1.2	2.2	2.8	5.9
Target Pressure, bar	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5

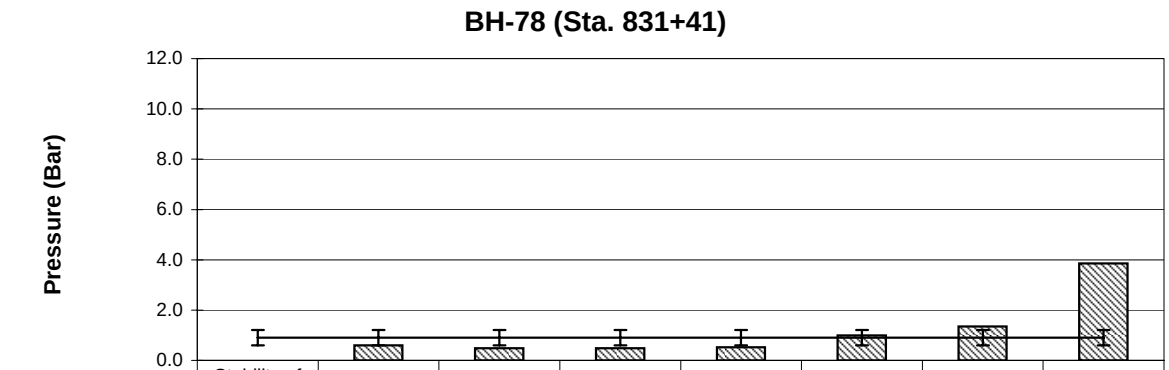
BH-49 (Sta. 824+28)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.0	1.0	1.1	2.0	2.6	5.7
Target Pressure, bar	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4

For Reference Only

See disclaimer on cover page.

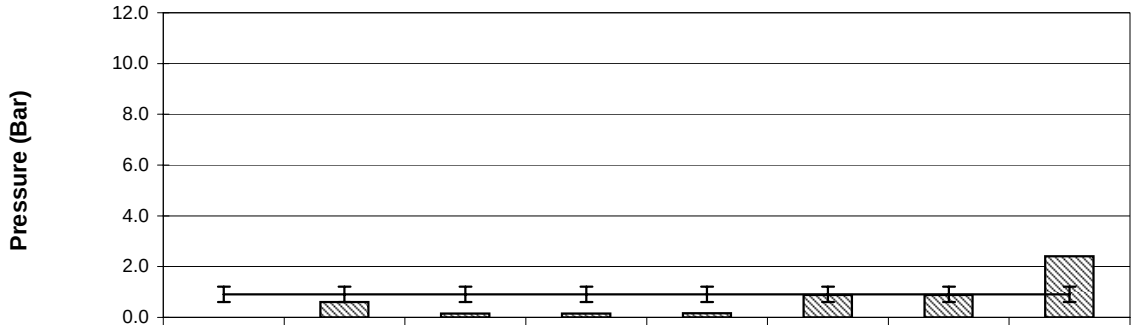


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.5	0.5	0.5	1.0	1.3	3.9
Target Pressure, bar	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

For Reference Only

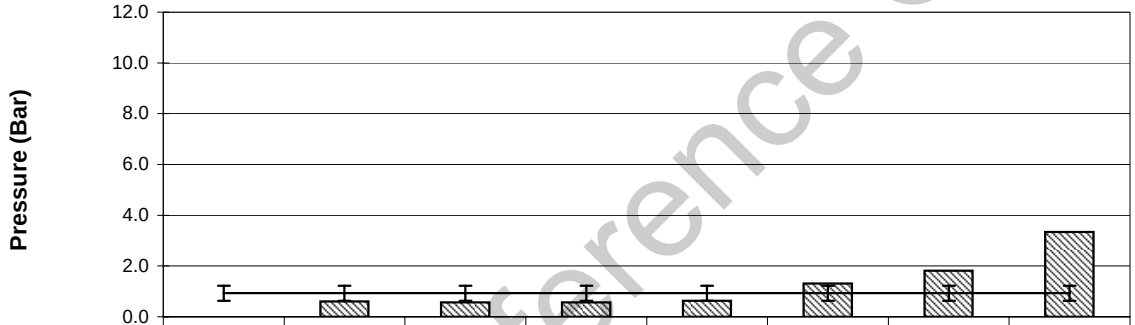
See disclaimer on cover page.

BH-57 (Sta. 569+31)



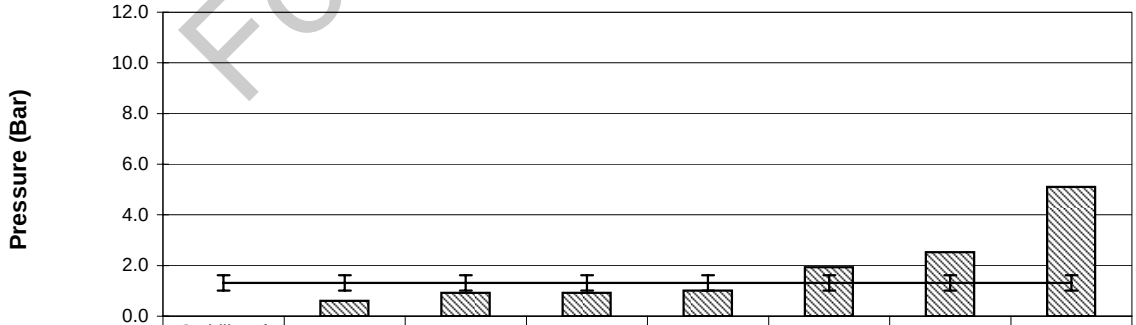
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.1	0.1	0.2	0.9	0.9	2.4
Target Pressure, bar	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

BH-01 (Sta. 574+30)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.6	0.6	0.6	1.3	1.8	3.3
Target Pressure, bar	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

BH-02 (Sta. 578+10)

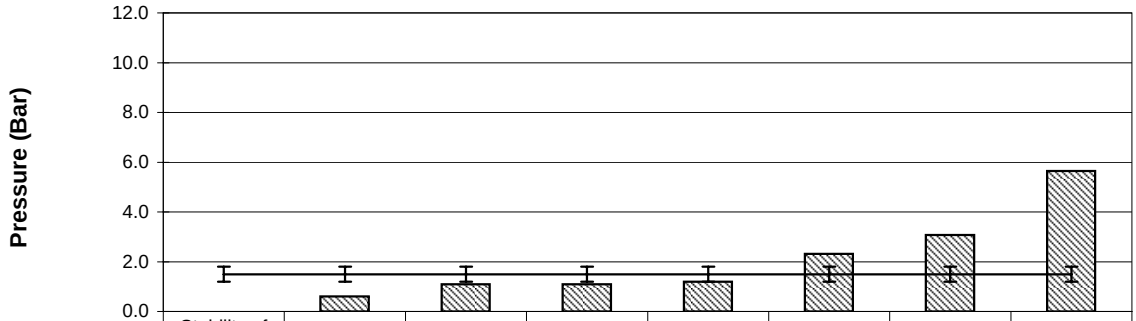


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.9	0.9	1.0	1.9	2.5	5.1
Target Pressure, bar	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3

For Reference Only

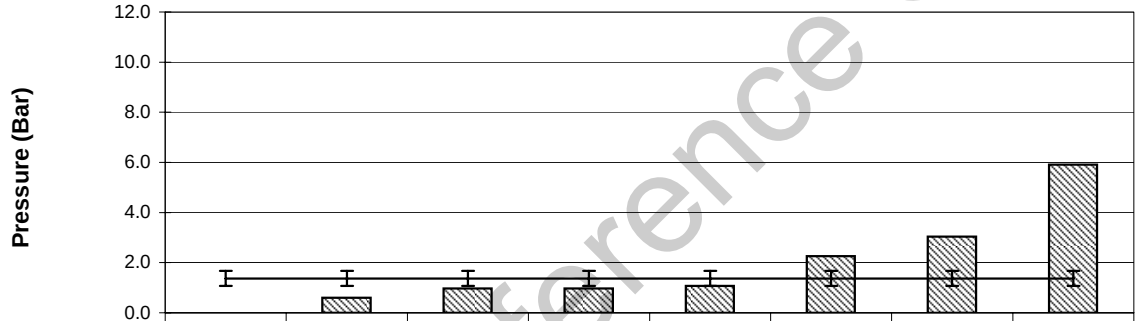
See disclaimer on cover page.

BH-03 (Sta. 581+83)



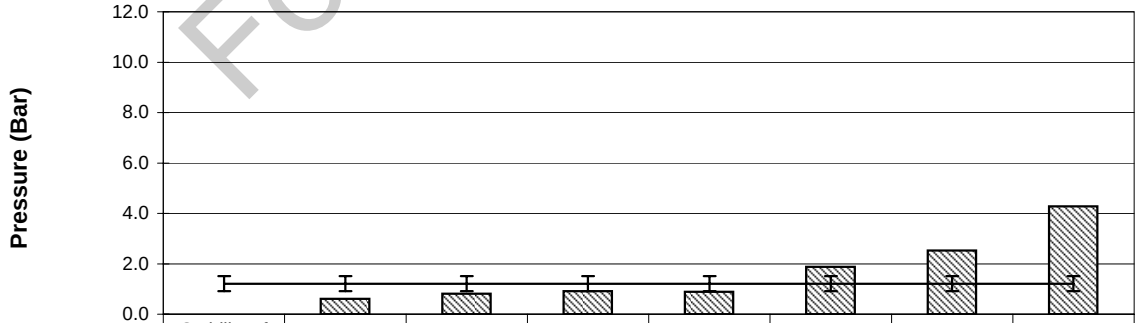
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.1	1.1	1.2	2.3	3.1	5.6
Target Pressure, bar	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5

BH-04 (Sta. 590+53)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.0	1.0	1.1	2.3	3.0	5.9
Target Pressure, bar	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4

BH-05 (Sta. 598+17)

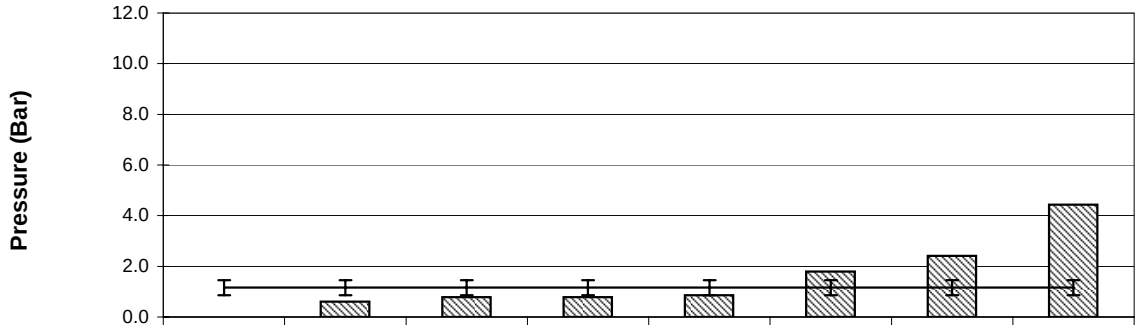


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.8	0.9	0.9	1.9	2.5	4.3
Target Pressure, bar	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2

For Reference Only

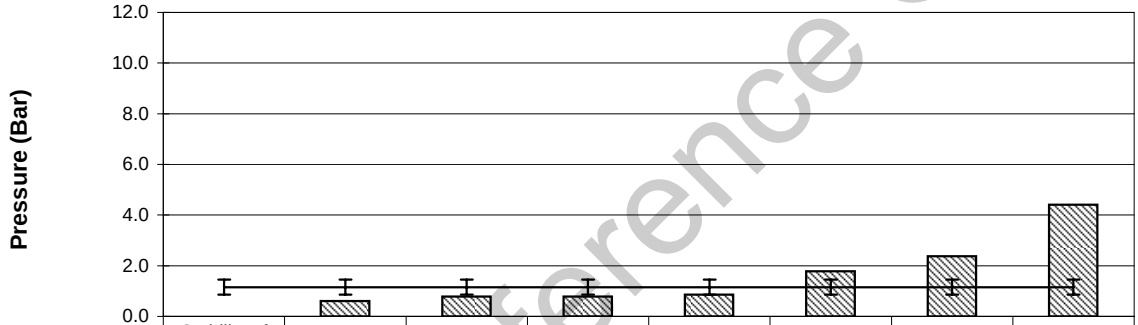
See disclaimer on cover page.

BH-06 (Sta. 599+60)



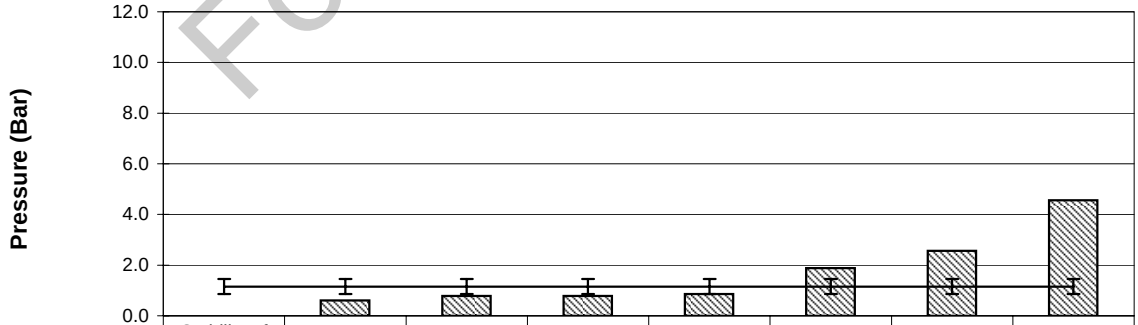
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.8	0.8	0.9	1.8	2.4	4.4
Target Pressure, bar	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2

BH-83 (Sta. 599+83)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.8	0.8	0.9	1.8	2.4	4.4
Target Pressure, bar	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2

BH-86 (Sta. 609+08)

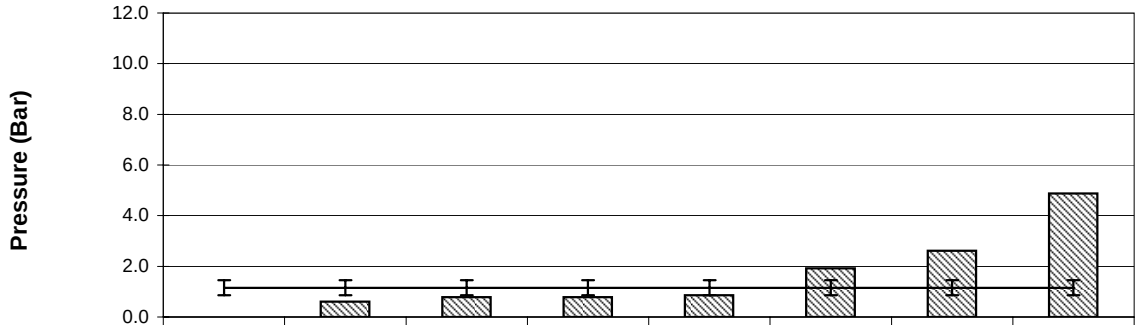


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.8	0.8	0.9	1.9	2.6	4.6
Target Pressure, bar	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2

For Reference Only

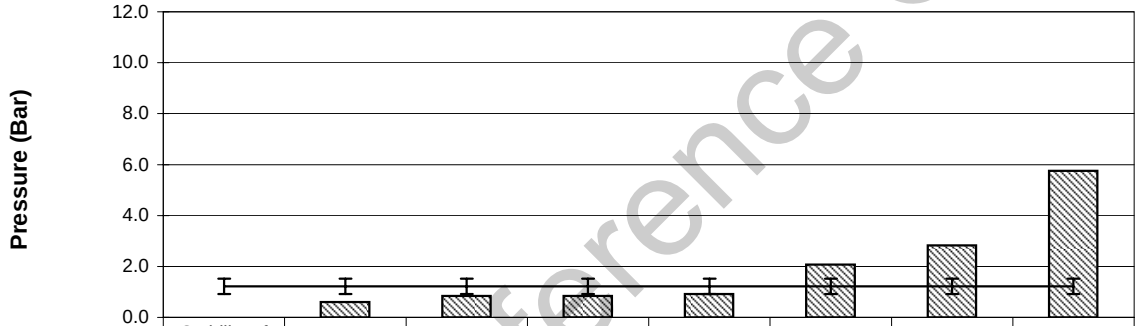
See disclaimer on cover page.

BH-07 (Sta. 609+41)



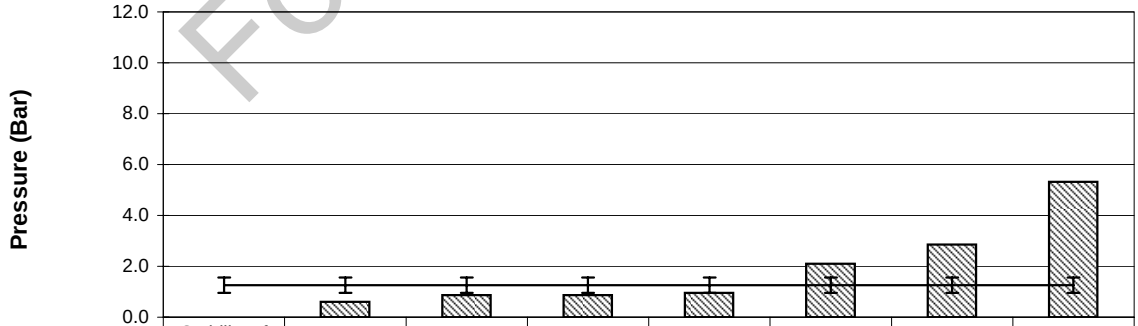
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.8	0.8	0.9	1.9	2.6	4.9
Target Pressure, bar	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2

BH-08 (Sta. 615+76)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.8	0.8	0.9	2.1	2.8	5.8
Target Pressure, bar	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2

BH-09 (Sta. 619+90)

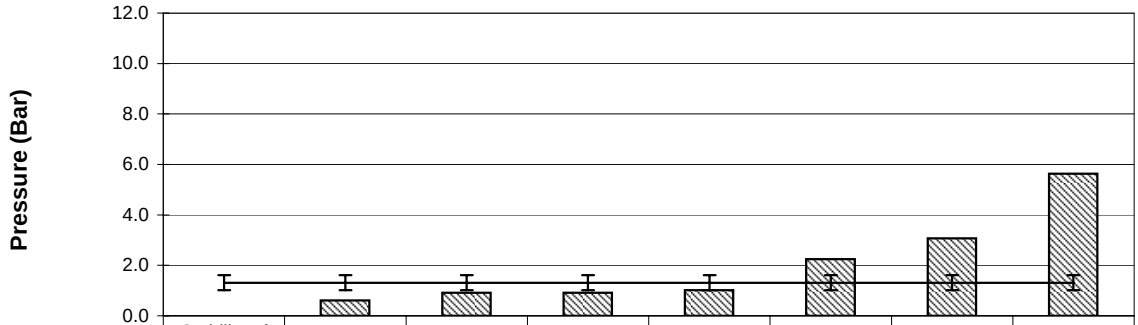


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.9	0.9	1.0	2.1	2.9	5.3
Target Pressure, bar	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3

For Reference Only

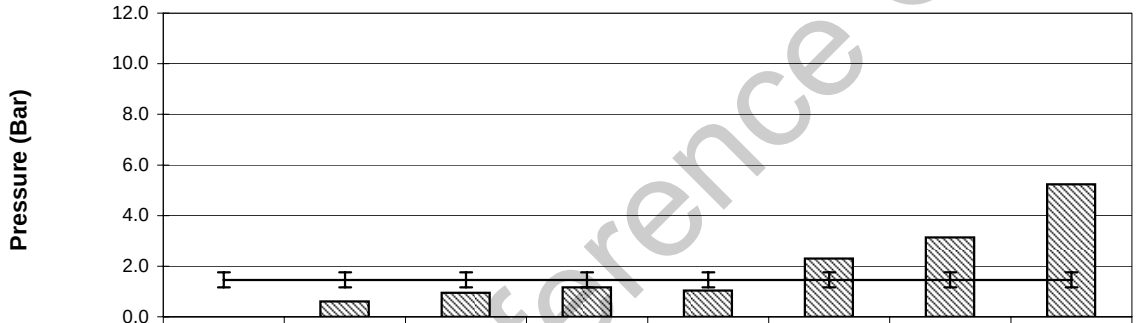
See disclaimer on cover page.

BH-10 (Sta. 624+90)



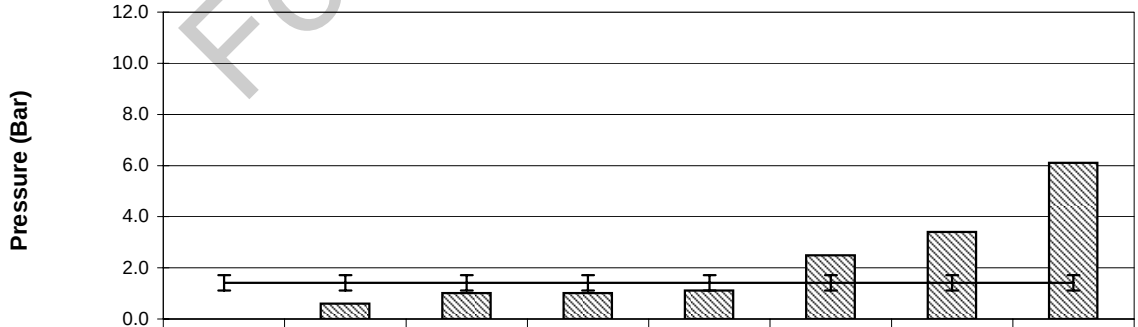
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.9	0.9	1.0	2.3	3.1	5.6
Target Pressure, bar	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3

BH-11 (Sta. 627+54)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.9	1.2	1.0	2.3	3.1	5.2
Target Pressure, bar	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5

BH-12 (Sta. 634+68)

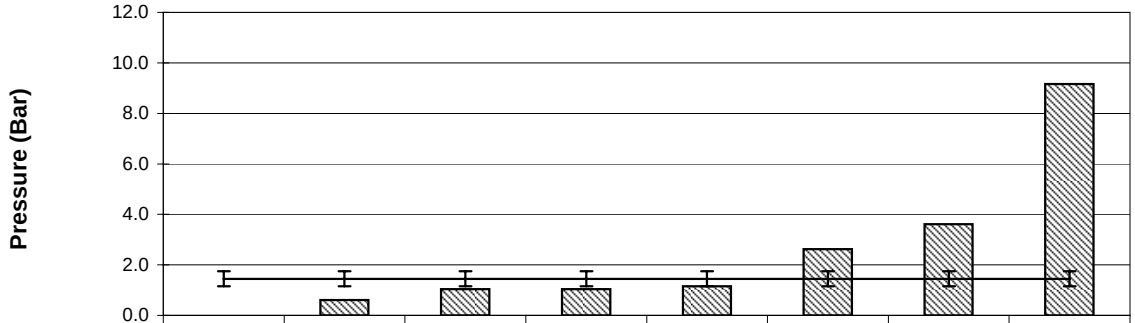


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.0	1.0	1.1	2.5	3.4	6.1
Target Pressure, bar	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4

For Reference Only

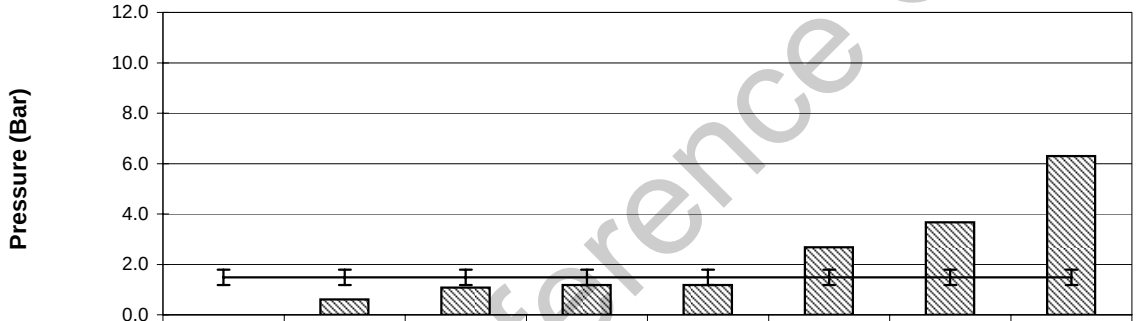
See disclaimer on cover page.

BH-13 (Sta. 640+81)



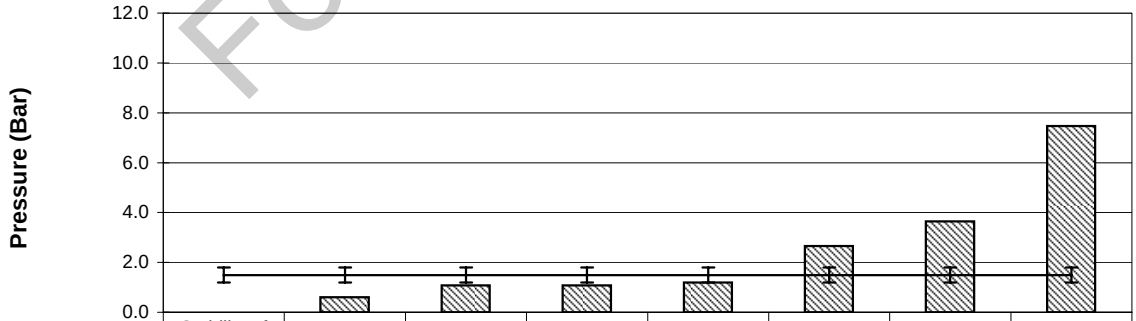
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.0	1.0	1.1	2.6	3.6	9.2
Target Pressure, bar	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4

BH-14 (Sta. 642+52)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.1	1.2	1.2	2.7	3.7	6.3
Target Pressure, bar	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5

BH-88 (Sta. 645+04)

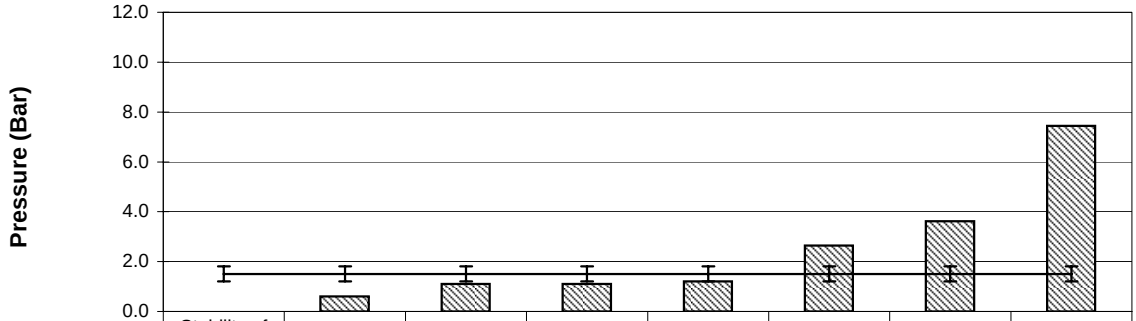


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.1	1.1	1.2	2.7	3.6	7.5
Target Pressure, bar	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5

For Reference Only

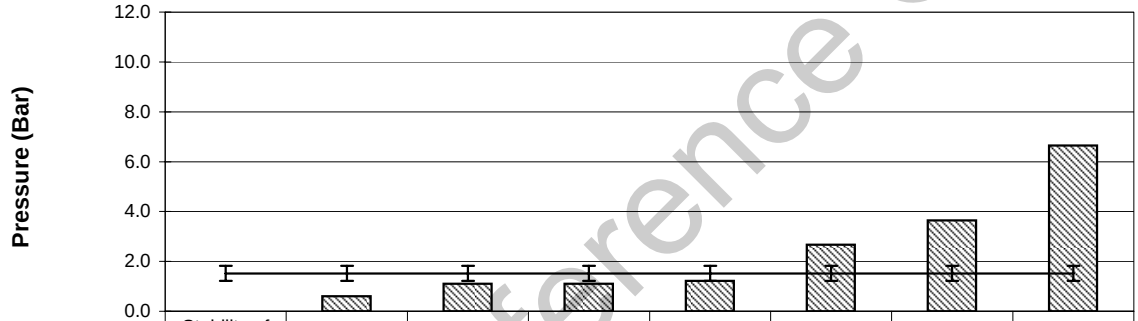
See disclaimer on cover page.

BH-15 (Sta. 645+69)



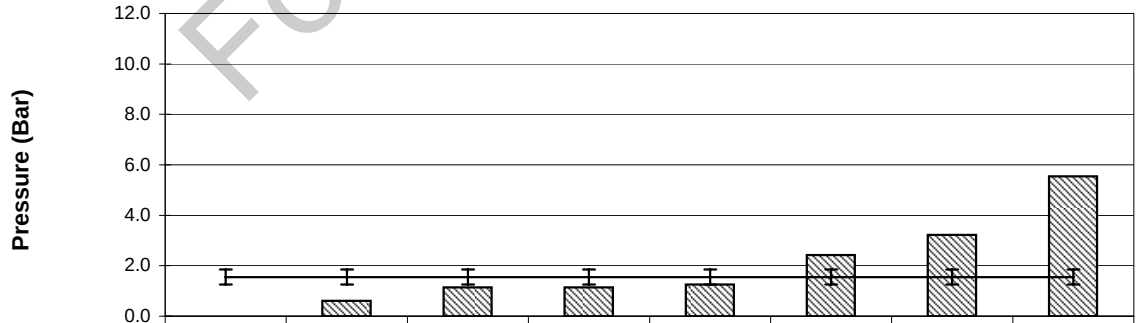
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.1	1.1	1.2	2.6	3.6	7.4
Target Pressure, bar	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5

BH-87 (Sta. 648+33)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.1	1.1	1.2	2.7	3.6	6.7
Target Pressure, bar	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5

BH-16 (Sta. 650+32)

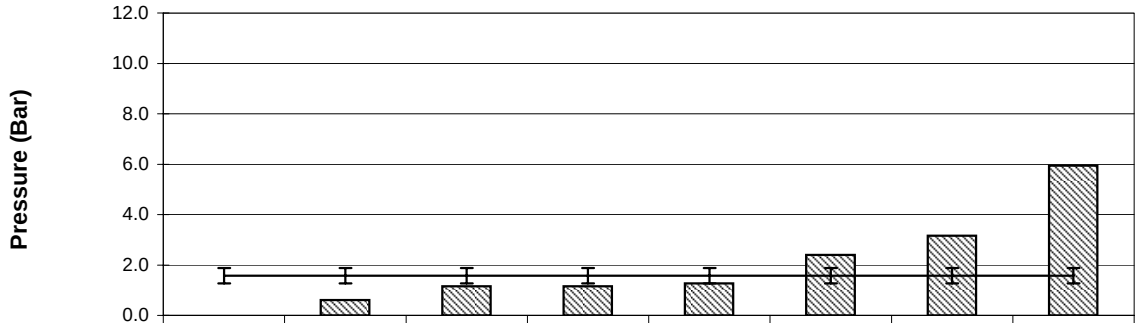


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.1	1.1	1.2	2.4	3.2	5.5
Target Pressure, bar	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5

For Reference Only

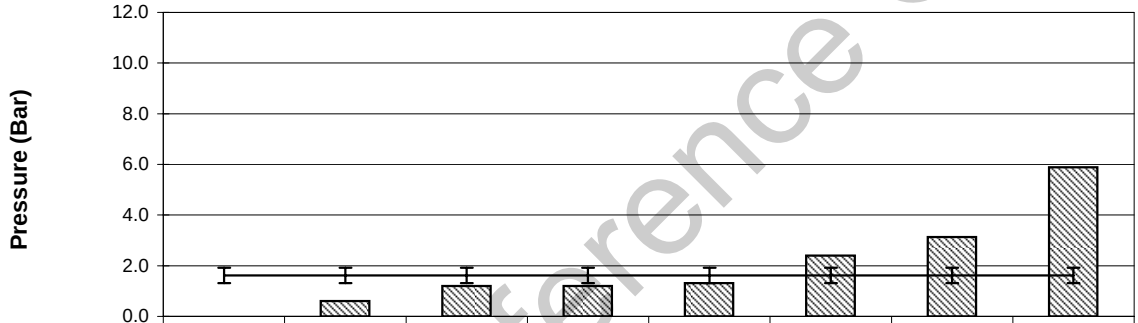
See disclaimer on cover page.

BH-17 (Sta. 654+44)



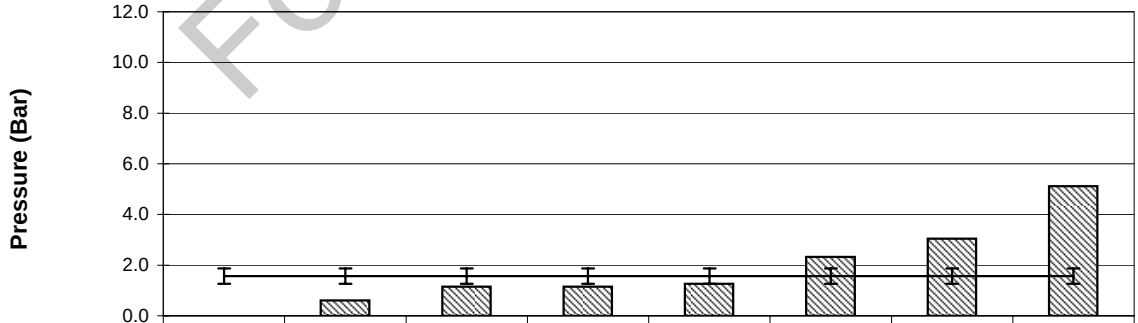
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.4	3.2	5.9
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

BH-18 (Sta. 660+03)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.4	3.1	5.9
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

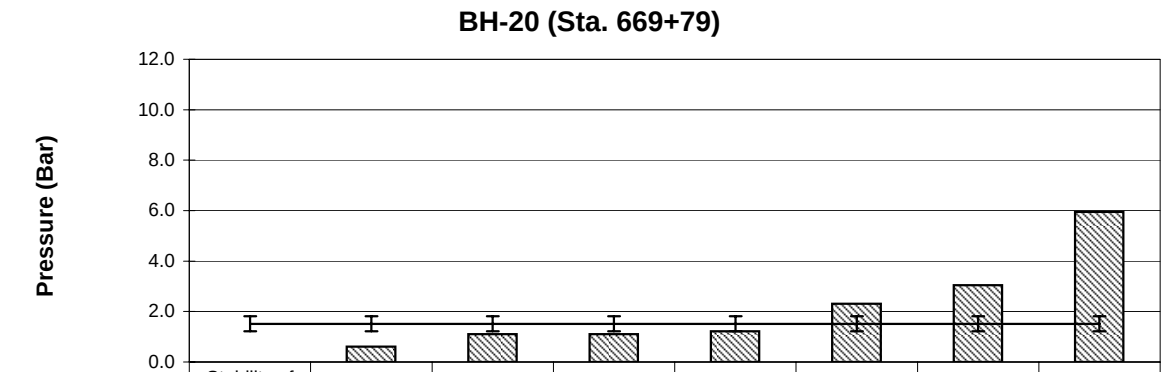
BH-19 (Sta. 666+26)



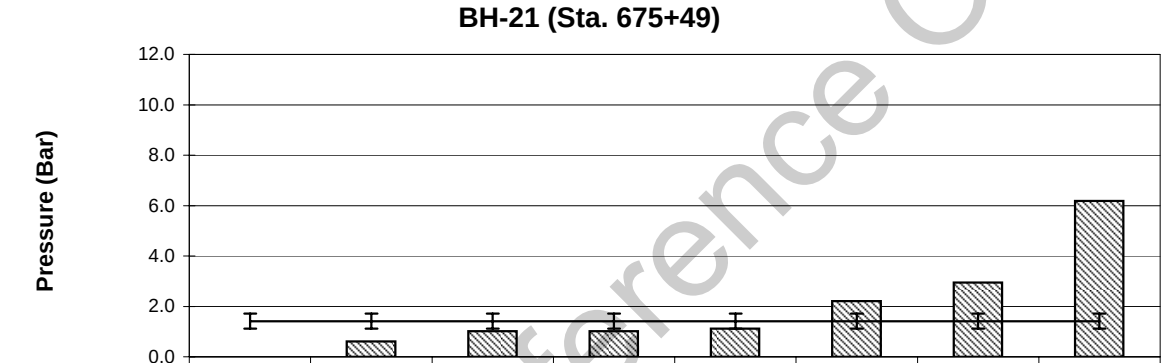
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.3	3.0	5.1
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

For Reference Only

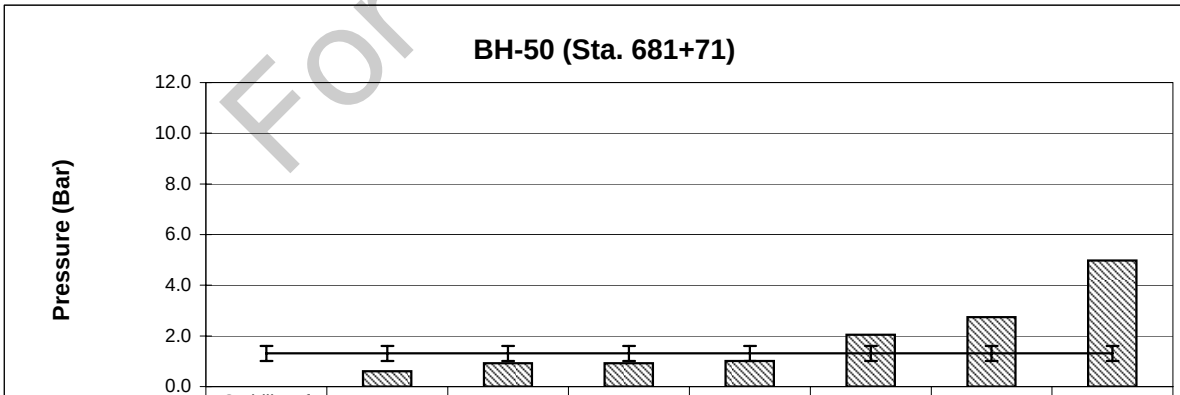
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.1	1.1	1.2	2.3	3.0	5.9
Target Pressure, bar	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.0	1.0	1.1	2.2	3.0	6.2
Target Pressure, bar	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4

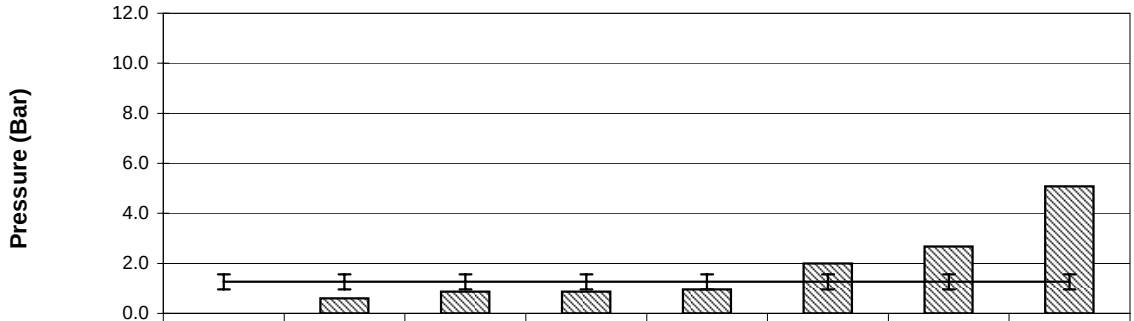


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.9	0.9	1.0	2.1	2.7	5.0
Target Pressure, bar	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3

For Reference Only

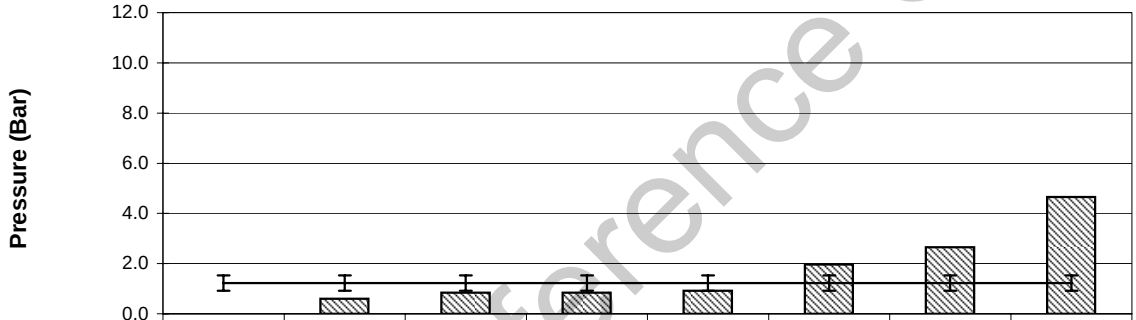
See disclaimer on cover page.

BH-52 (Sta. 684+09)



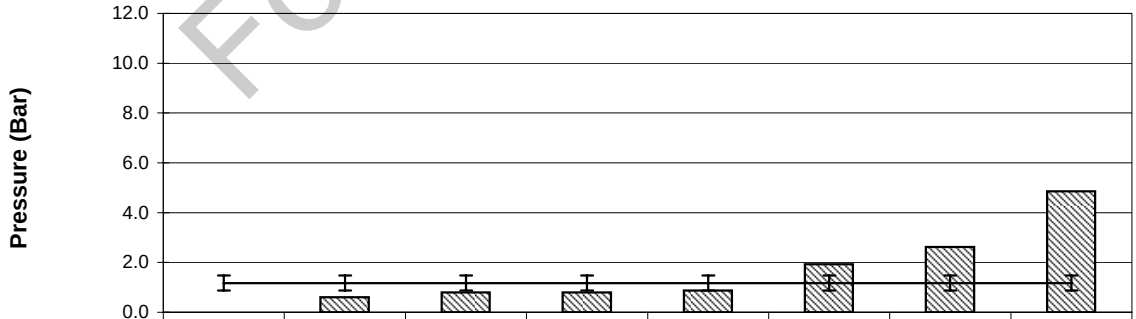
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.9	0.9	1.0	2.0	2.7	5.1
Target Pressure, bar	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3

BH-53 (Sta. 685+42)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.8	0.8	0.9	2.0	2.6	4.7
Target Pressure, bar	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2

BH-54 (Sta. 687+16)

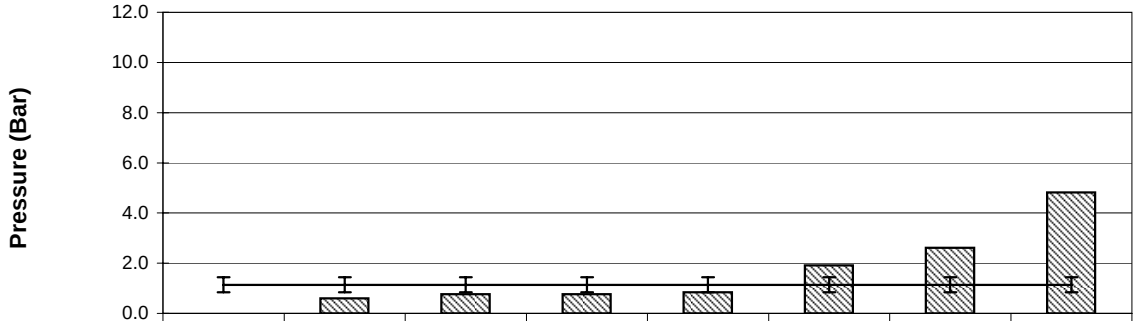


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.8	0.8	0.9	1.9	2.6	4.9
Target Pressure, bar	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2

For Reference Only

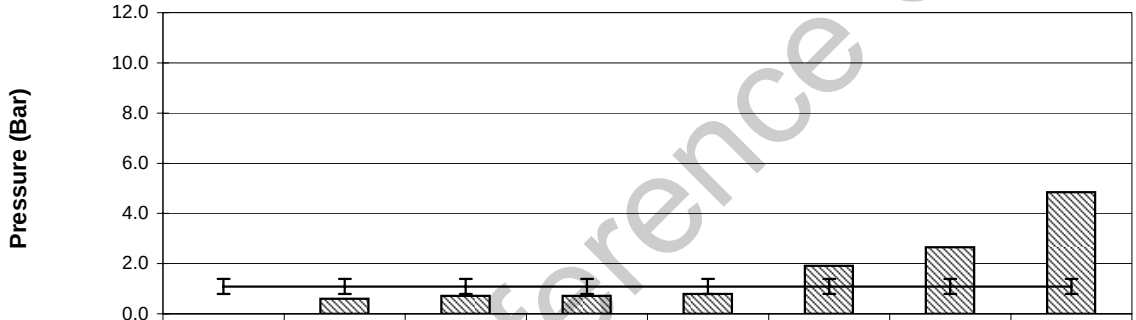
See disclaimer on cover page.

BH-55 (Sta. 688+28)



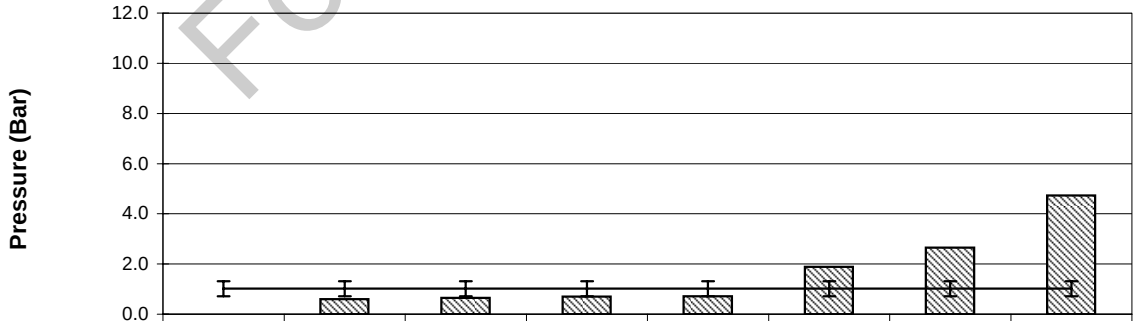
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.8	0.8	0.8	1.9	2.6	4.8
Target Pressure, bar	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1

BH-23 (Sta. 690+02)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.7	0.7	0.8	1.9	2.6	4.9
Target Pressure, bar	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1

BH-64 (Sta. 691+92)

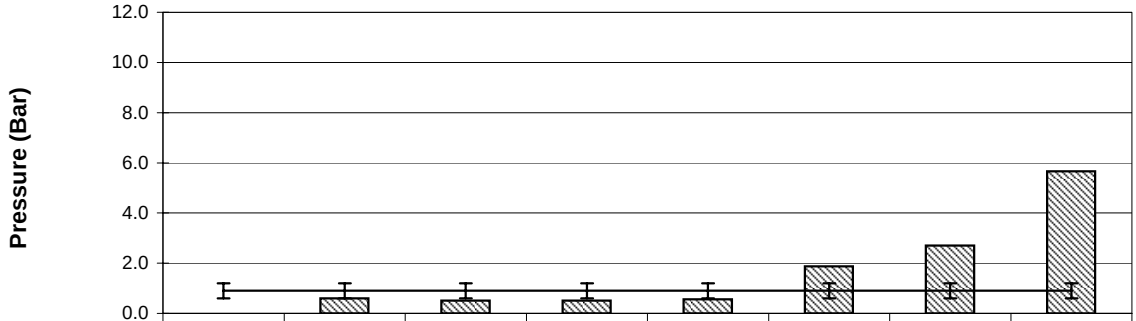


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.7	0.7	0.7	1.9	2.6	4.7
Target Pressure, bar	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

For Reference Only

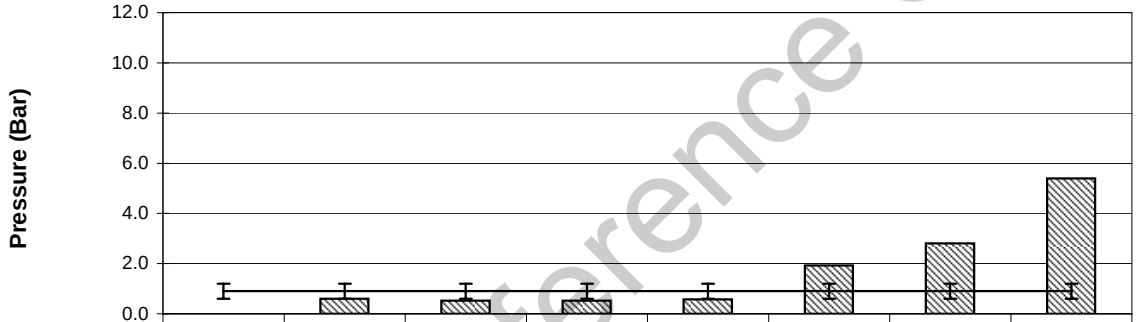
See disclaimer on cover page.

BH-70 (Sta. 706+78)



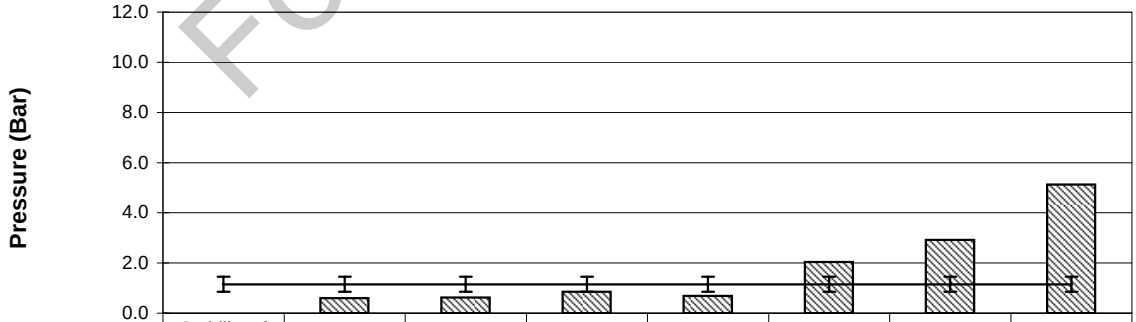
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.5	0.5	0.6	1.9	2.7	5.7
Target Pressure, bar	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

BH-71 (Sta. 707+62)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.5	0.5	0.6	1.9	2.8	5.4
Target Pressure, bar	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

BH-72 (Sta. 709+40)

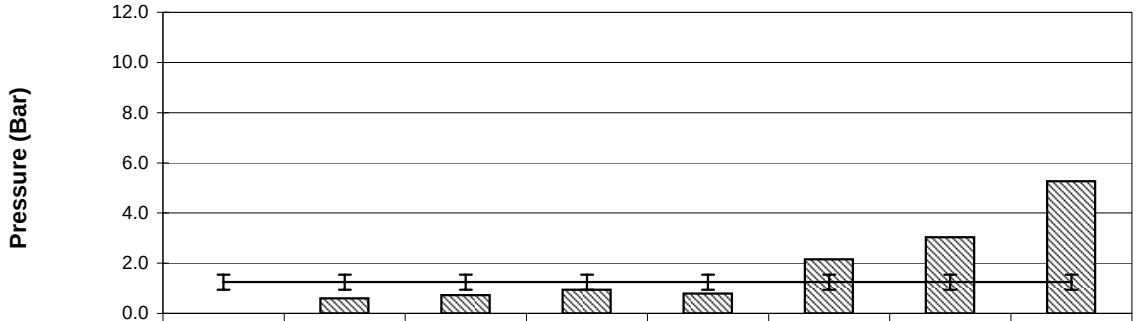


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.6	0.9	0.7	2.0	2.9	5.1
Target Pressure, bar	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2

For Reference Only

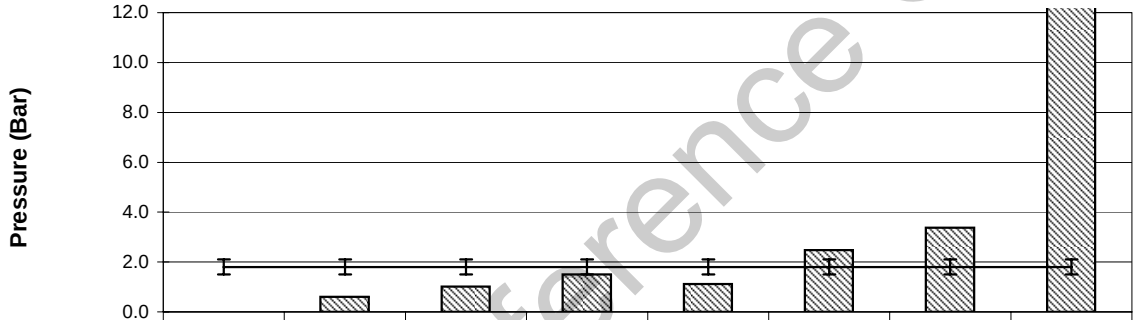
See disclaimer on cover page.

BH-26 (Sta. 710+70)



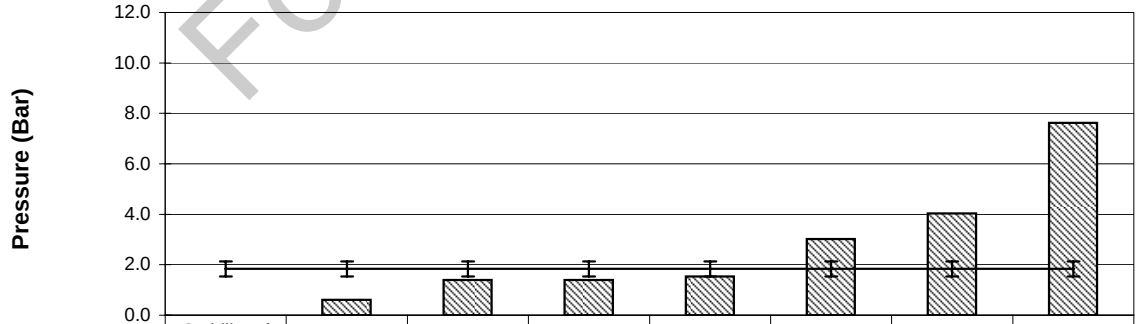
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.7	0.9	0.8	2.2	3.0	5.3
Target Pressure, bar	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2

BH-27 (Sta. 715+00)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.0	1.5	1.1	2.5	3.4	13.2
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

BH-28 (Sta. 720+22)

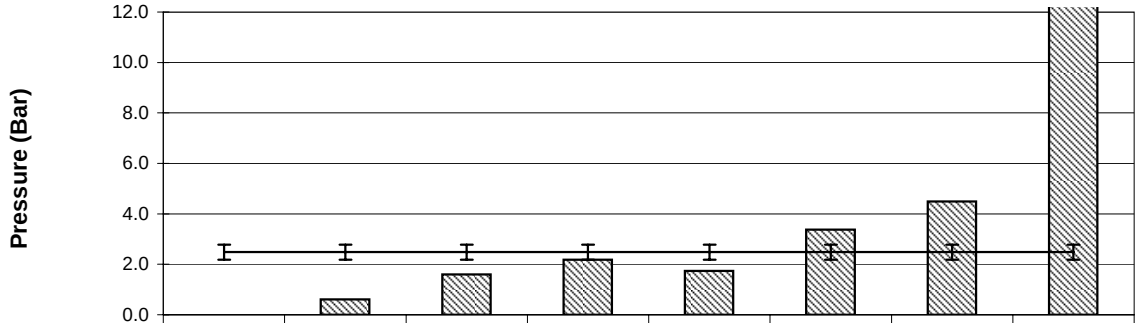


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.4	1.4	1.5	3.0	4.0	7.6
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

For Reference Only

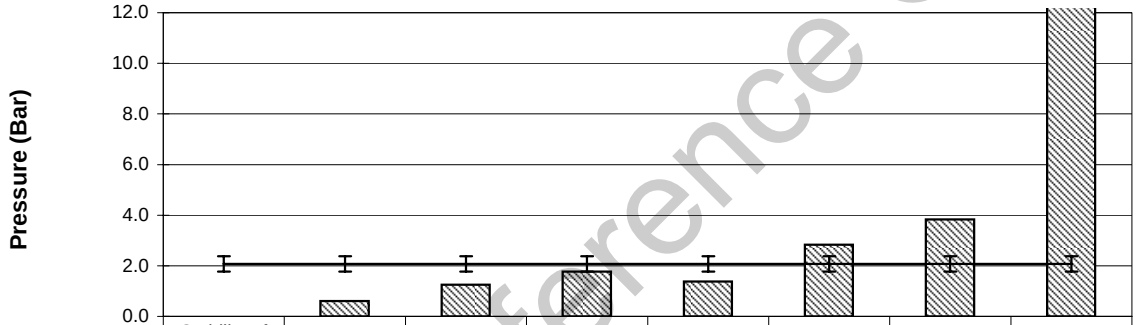
See disclaimer on cover page.

BH-29 (Sta. 723+88)



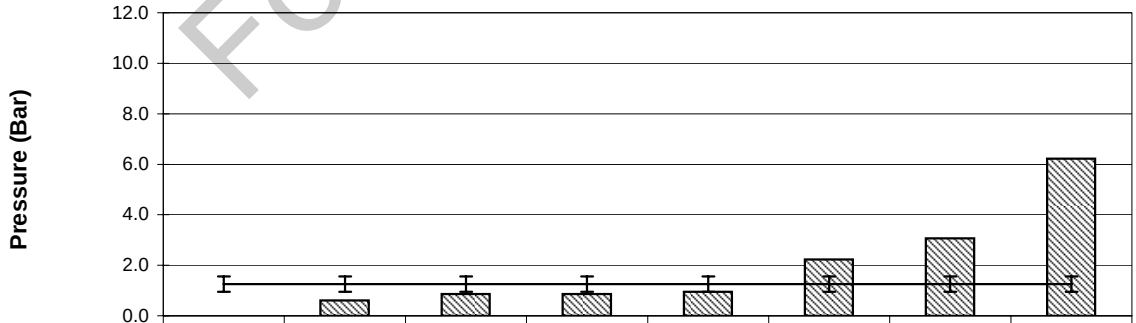
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.6	2.2	1.7	3.4	4.5	16.5
Target Pressure, bar	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5

BH-30 (Sta. 728+01)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.2	1.8	1.4	2.8	3.8	14.6
Target Pressure, bar	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1

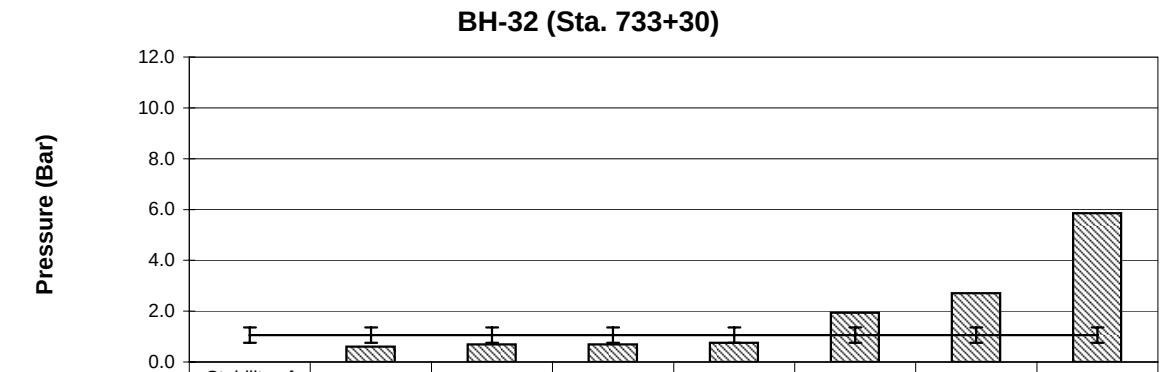
BH-31 (Sta. 731+54)



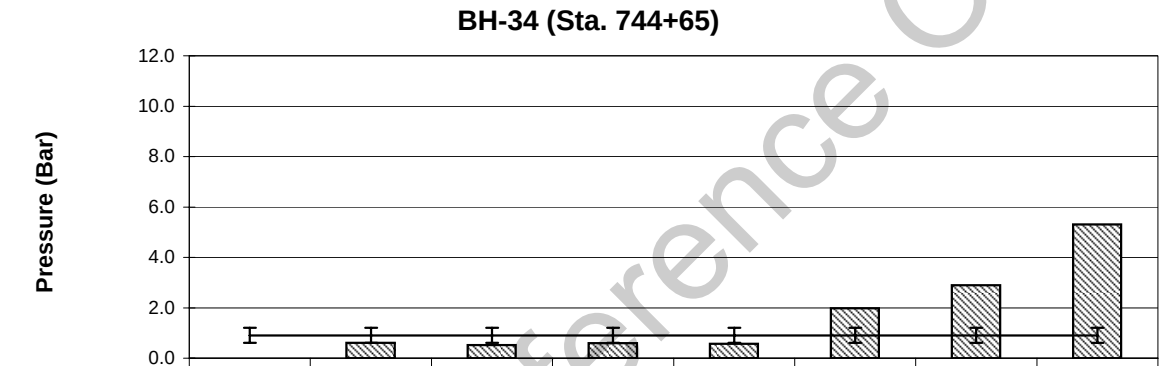
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.9	0.9	1.0	2.2	3.1	6.2
Target Pressure, bar	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3

For Reference Only

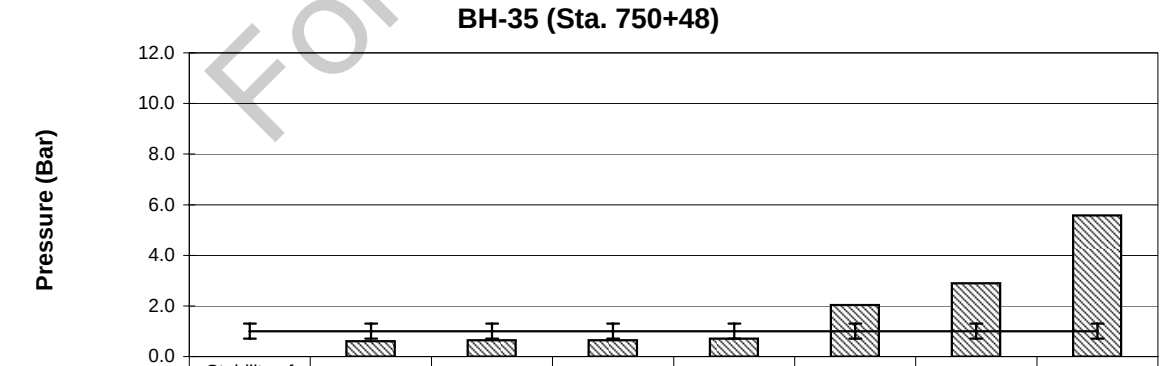
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.7	0.7	0.8	1.9	2.7	5.9
Target Pressure, bar	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1



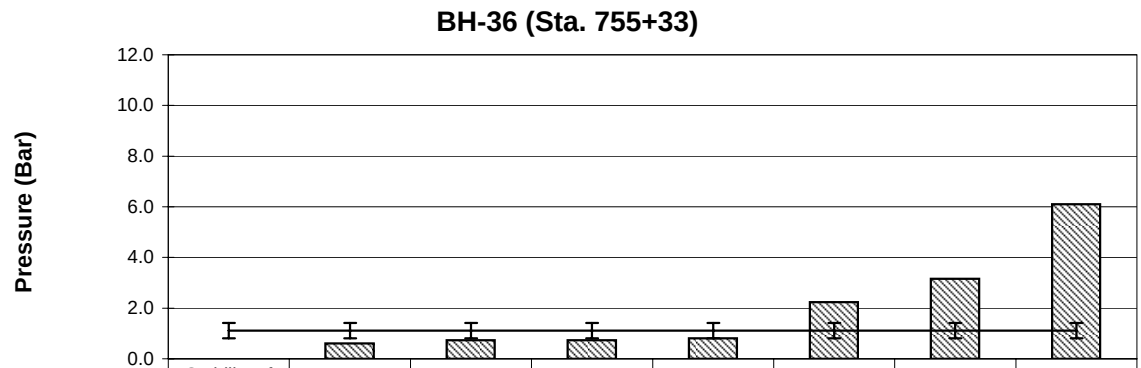
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.5	0.6	0.6	2.0	2.9	5.3
Target Pressure, bar	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9



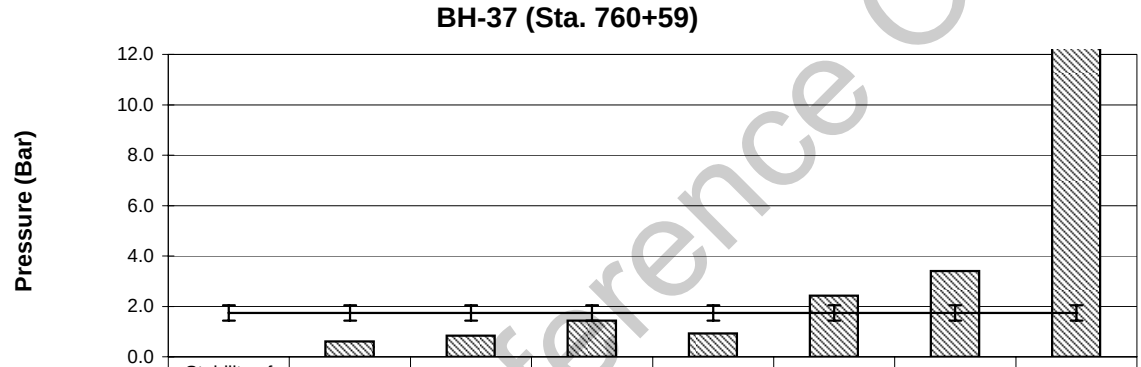
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.6	0.6	0.7	2.0	2.9	5.6
Target Pressure, bar	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

For Reference Only

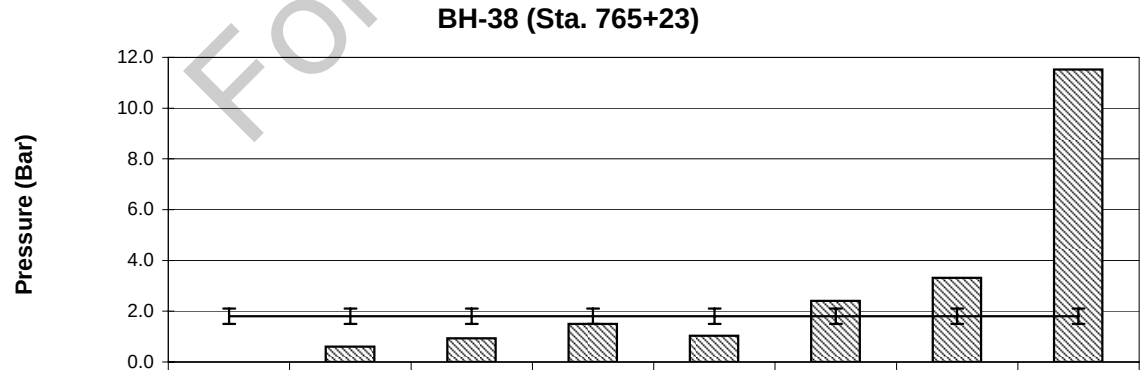
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.7	0.7	0.8	2.2	3.2	6.1
Target Pressure, bar	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	0.8	1.4	0.9	2.4	3.4	12.4
Target Pressure, bar	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7

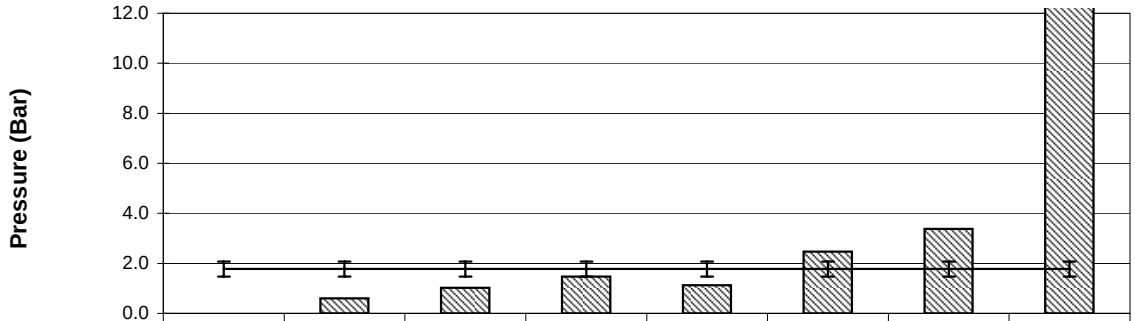


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	0.9	1.5	1.0	2.4	3.3	11.5
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

For Reference Only

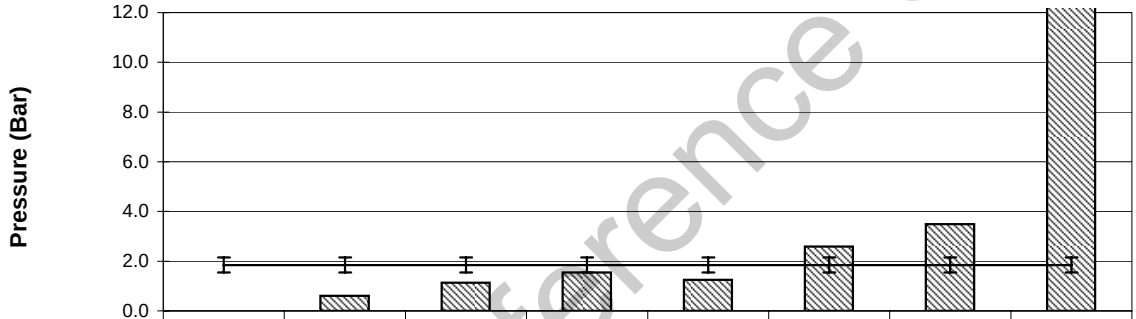
See disclaimer on cover page.

BH-39 (Sta. 768+77)



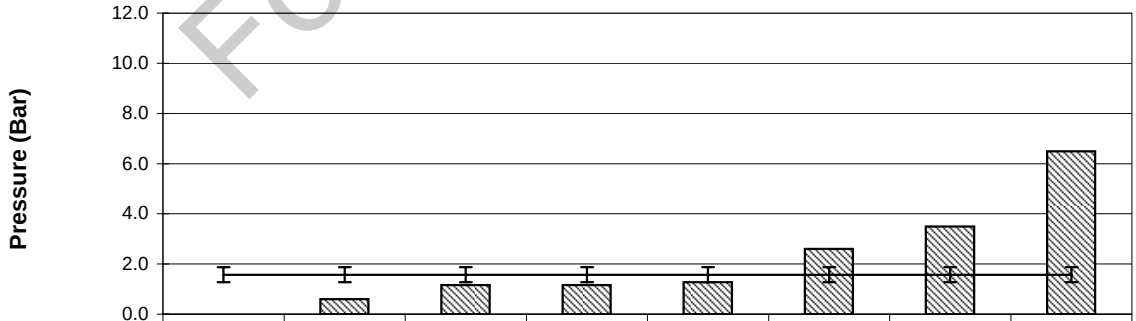
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.0	1.5	1.1	2.5	3.4	14.0
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

BH-95 (Sta. 774+13)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.1	1.5	1.2	2.6	3.5	15.3
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

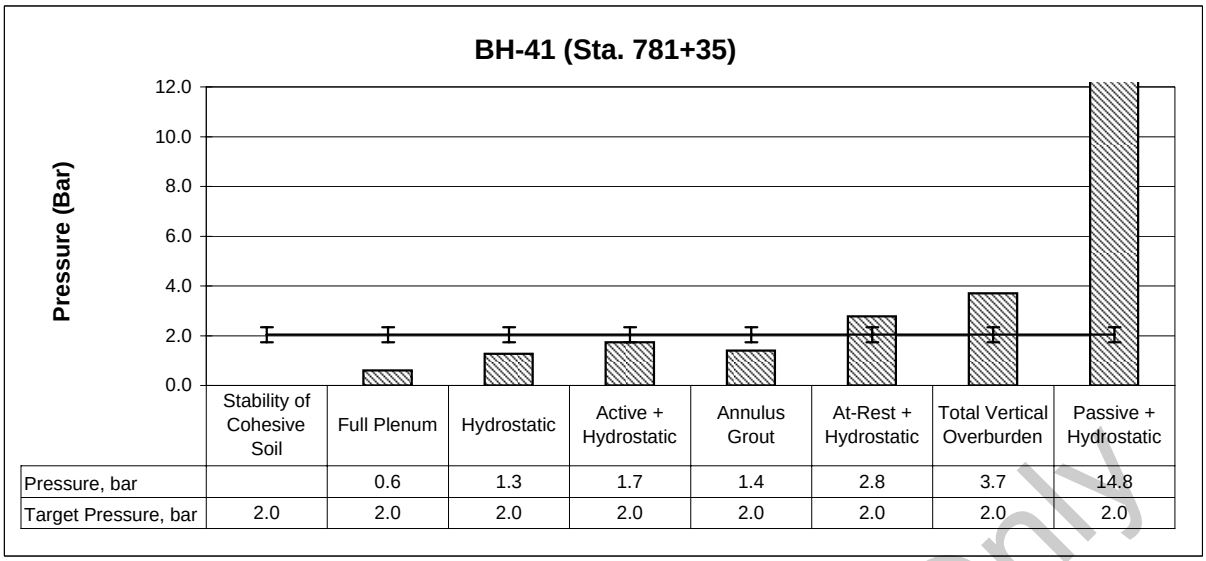
BH-40 (Sta. 775+76)



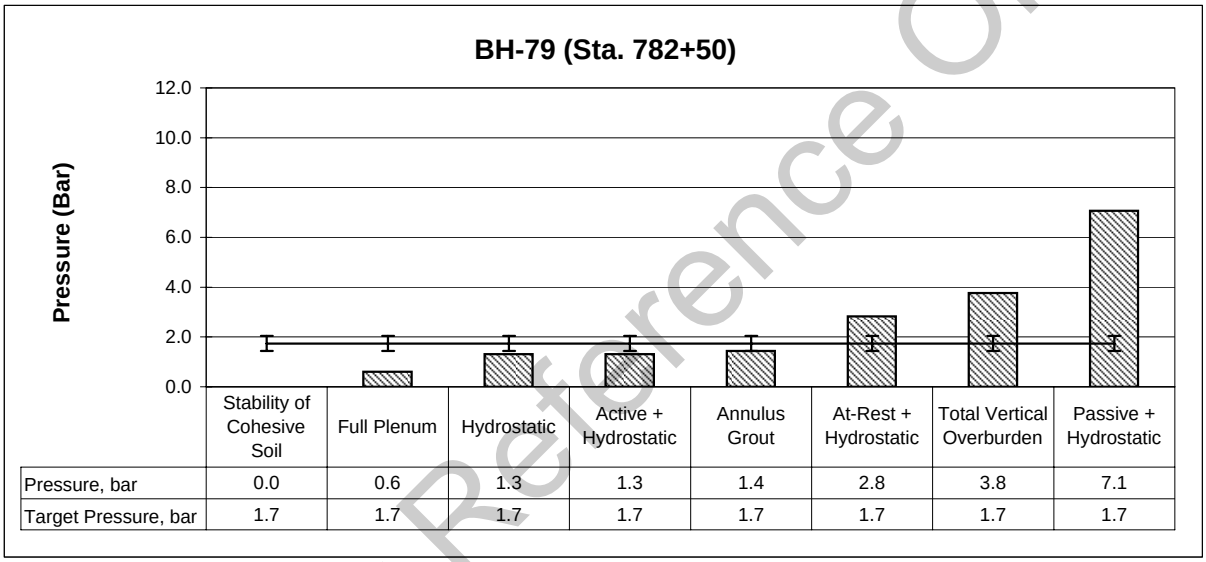
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.6	3.5	6.5
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

For Reference Only

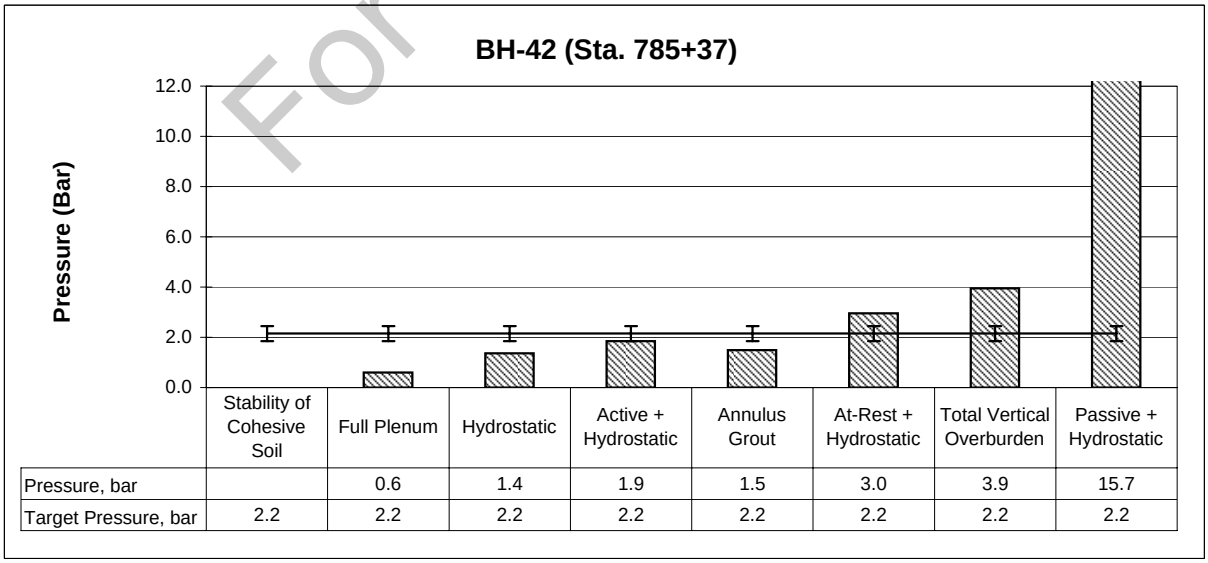
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.3	1.7	1.4	2.8	3.7	14.8
Target Pressure, bar	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0



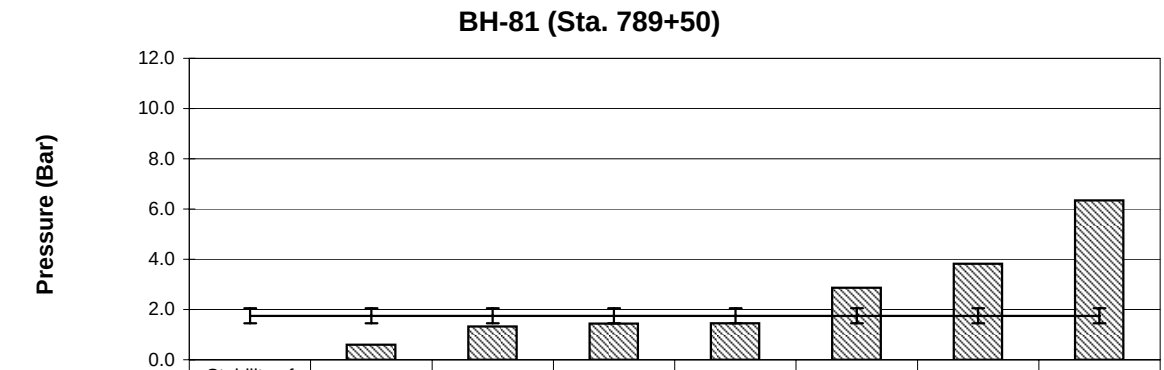
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.3	1.3	1.4	2.8	3.8	7.1
Target Pressure, bar	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7



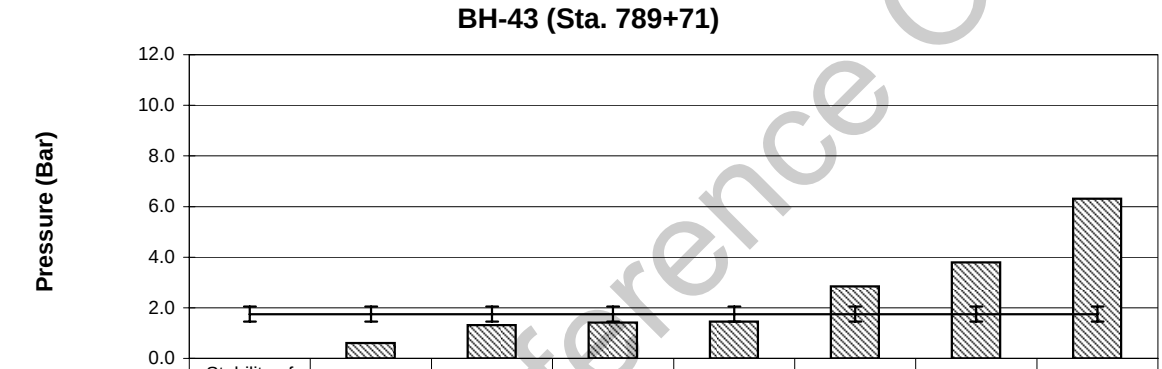
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.4	1.9	1.5	3.0	3.9	15.7
Target Pressure, bar	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2

For Reference Only

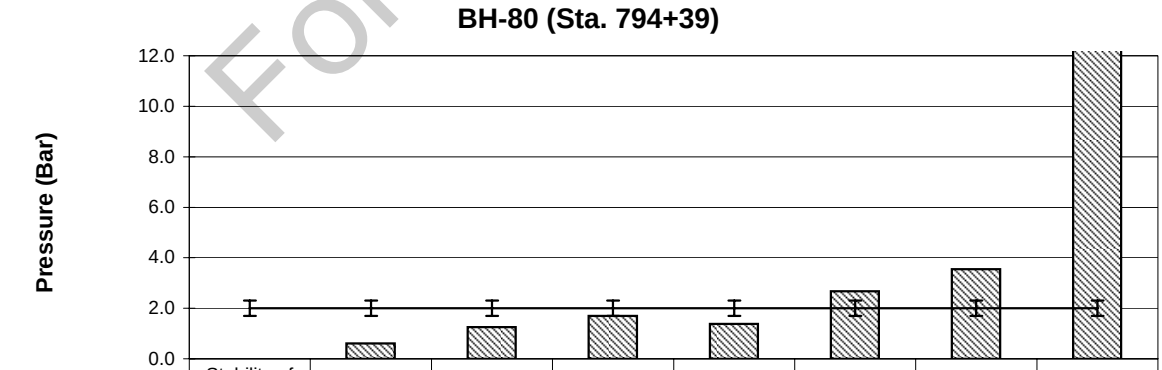
See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.3	1.4	1.5	2.9	3.8	6.3
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.3	1.4	1.5	2.8	3.8	6.3
Target Pressure, bar	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8

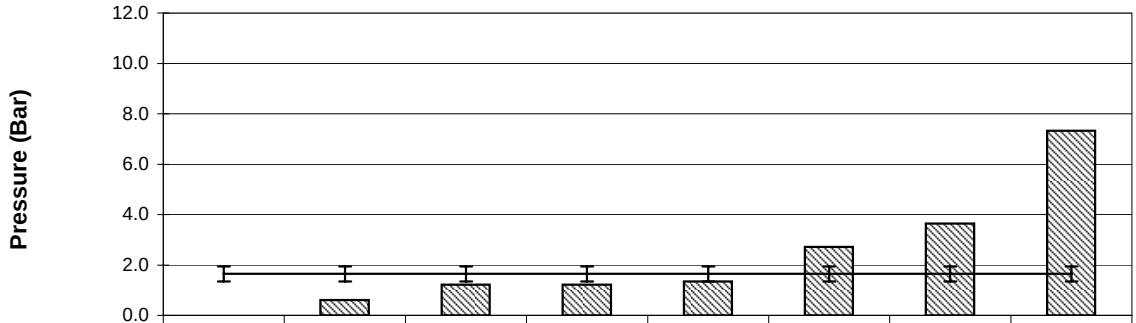


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.3	1.7	1.4	2.7	3.6	13.8
Target Pressure, bar	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

For Reference Only

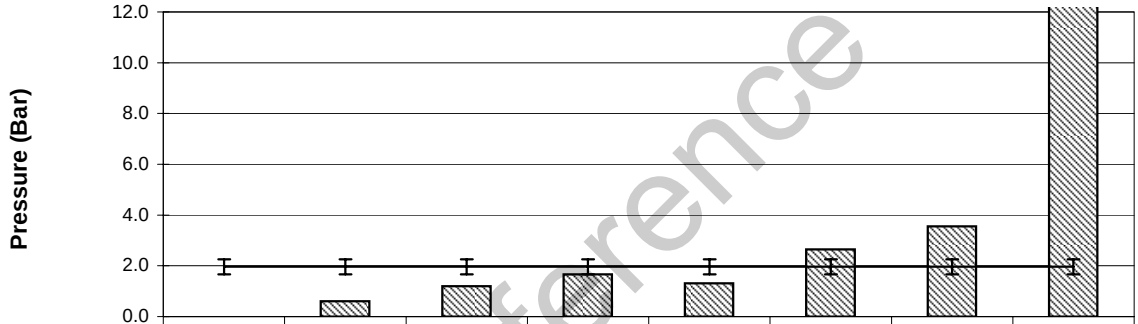
See disclaimer on cover page.

BH-102 (Sta. 796+48)



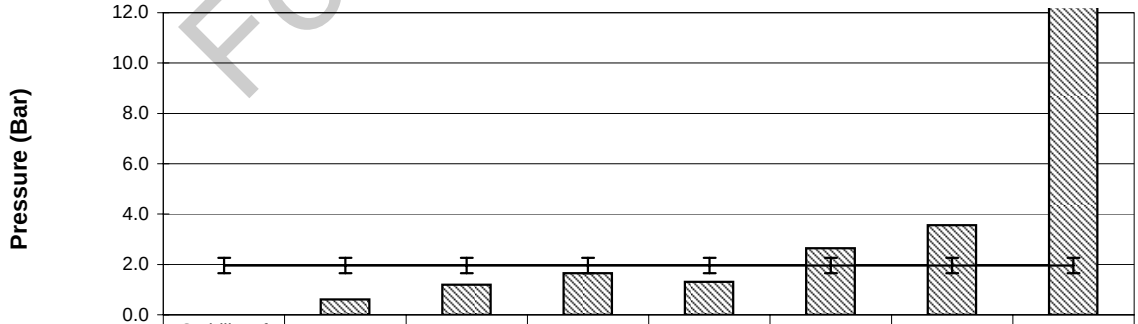
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.2	1.2	1.3	2.7	3.6	7.3
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

BH-103 (Sta. 798+16)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.2	1.7	1.3	2.7	3.6	13.8
Target Pressure, bar	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

BH-44 (Sta. 798+27)

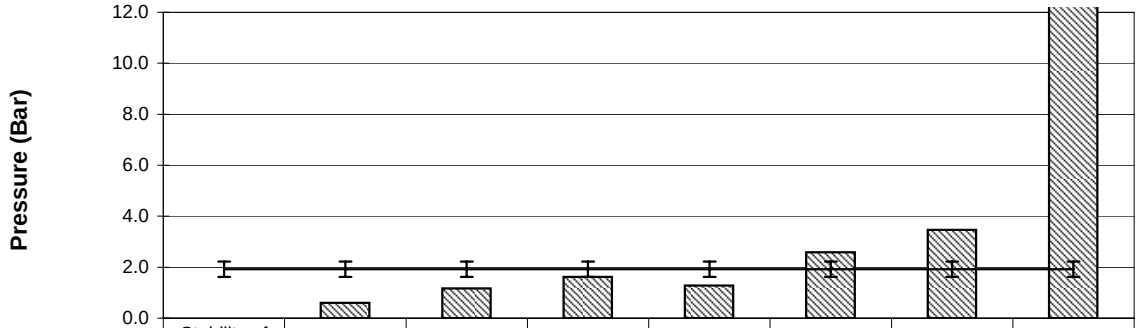


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.2	1.7	1.3	2.7	3.6	13.9
Target Pressure, bar	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

For Reference Only

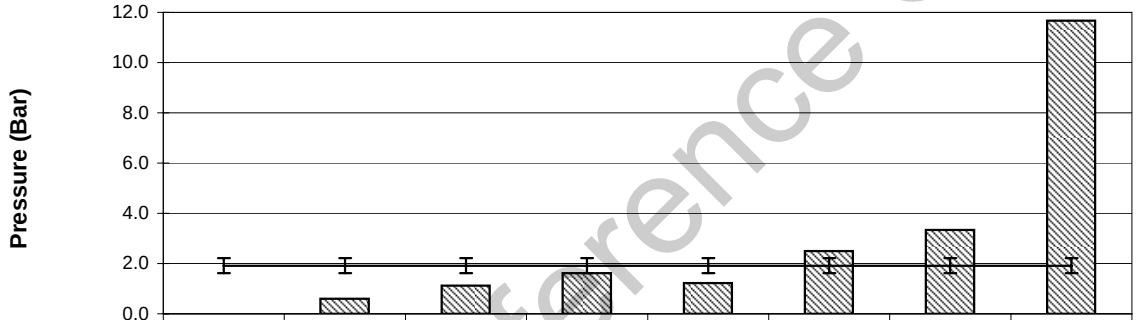
See disclaimer on cover page.

BH-106 (Sta. 800+20)



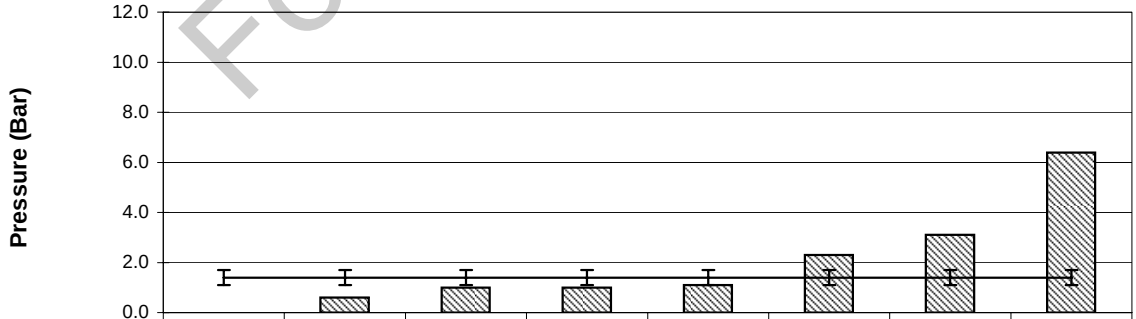
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.2	1.6	1.3	2.6	3.5	13.5
Target Pressure, bar	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9

BH-45 (Sta. 802+44)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	1.1	1.6	1.2	2.5	3.3	11.7
Target Pressure, bar	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9

BH-46 (Sta. 809+36)

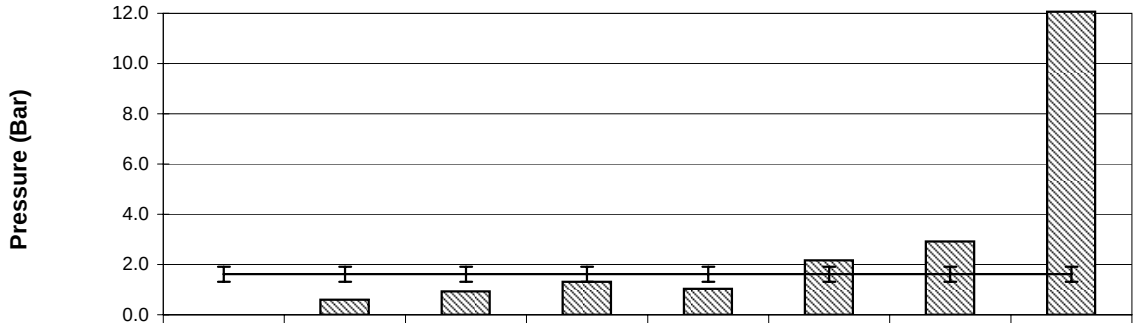


	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	1.0	1.0	1.1	2.3	3.1	6.4
Target Pressure, bar	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4

For Reference Only

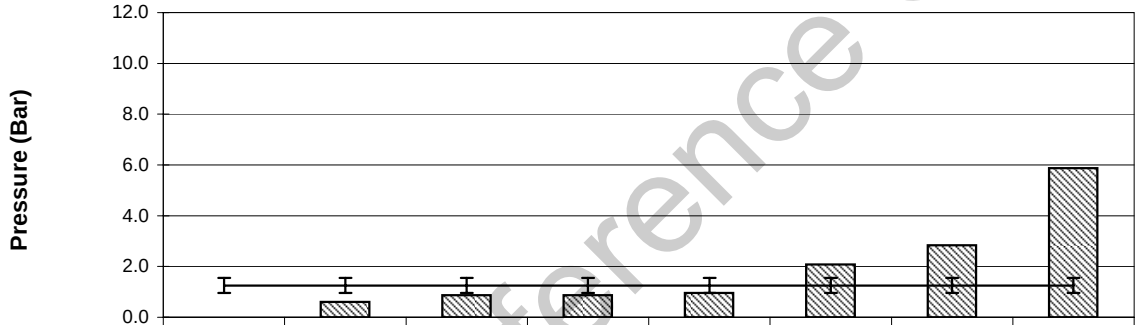
See disclaimer on cover page.

BH-47 (Sta. 813+52)



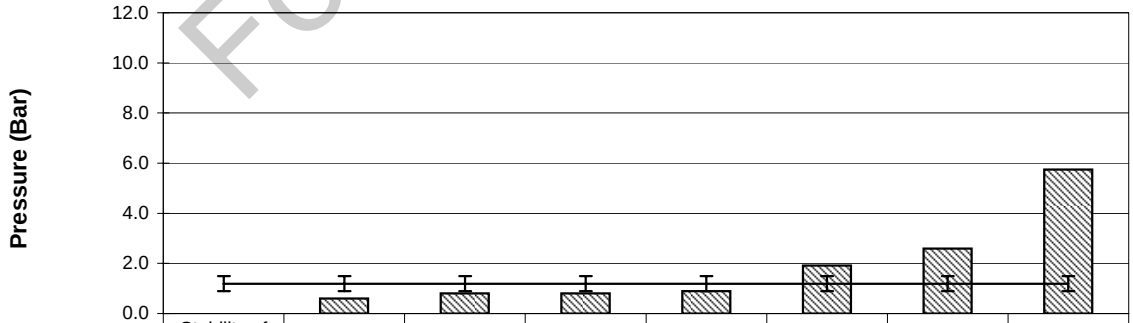
	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar		0.6	0.9	1.3	1.0	2.2	2.9	12.1
Target Pressure, bar	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6

BH-48 (Sta. 818+34)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.9	0.9	1.0	2.1	2.8	5.9
Target Pressure, bar	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3

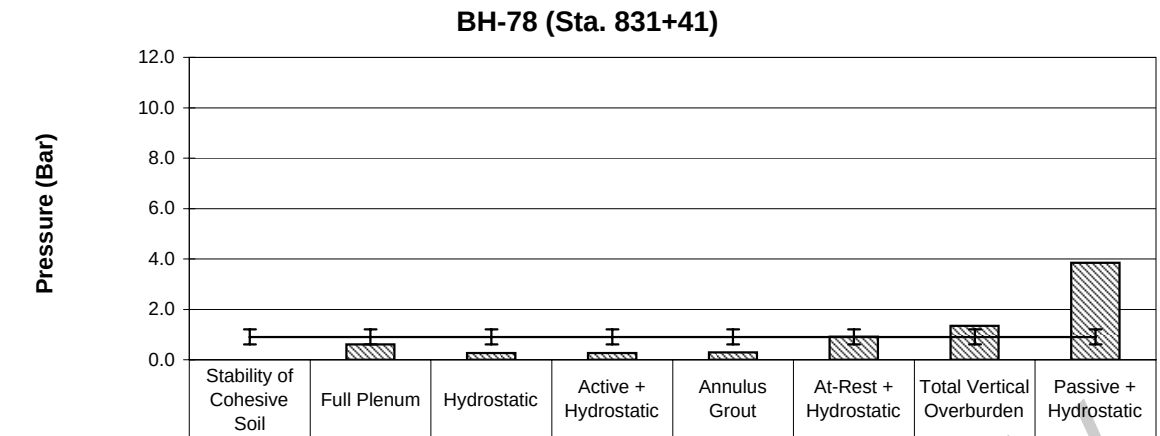
BH-49 (Sta. 824+28)



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.8	0.8	0.9	1.9	2.6	5.7
Target Pressure, bar	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2

For Reference Only

See disclaimer on cover page.



	Stability of Cohesive Soil	Full Plenum	Hydrostatic	Active + Hydrostatic	Annulus Grout	At-Rest + Hydrostatic	Total Vertical Overburden	Passive + Hydrostatic
Pressure, bar	0.0	0.6	0.3	0.3	0.3	0.9	1.3	3.9
Target Pressure, bar	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

For Reference Only

See disclaimer on cover page.