PRELIMINARY FOUNDATION REPORT

BATTERY TUNNELS

DOYLE DRIVE REPLACEMENT PROJECT

April 15, 2008

ARUP PB
joint venture

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1.0 INTRODUCTION

This preliminary foundation report provides preliminary foundation recommendations for the Battery Tunnel structures, cut-and-cover tunnels located in San Francisco, California. The proposed structure is part of the Doyle Drive Replacement project which will replace the freeway which stretches from the west end of Lombard Street, through the Presidio of San Francisco, ending at the Golden Gate Bridge toll plaza. The cut-and-cover tunnel will be landscaped above to restore some of the natural beauty of the Presidio, which is now part of the U.S. National Park System.

2.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

2.1 GENERAL SITE GEOLOGY

The bedrock at the site of the Battery Tunnels is generally the Franciscan Formation. This formation consists of a variety of heavily folded and sheared assemblages of Greywacke sandstone, siltstone, chert, serpentinite, and Mélange Matrix (a plastic bedrock unit which is similar to a gravelly clay). The hardness of these bedrock units typically range from very soft to hard and may vary in short distances.

Overburden soils consist of mostly marine deposited sediments. Common sediment sequences found elsewhere along the San Francisco Bay fringes are found at this site. The typical sediments begin with Bay Mud at the surface underlain with marine deposits followed by Colma Formation, which is a dense sand. Beneath the Colma Formation is typically Old Bay Clay, a stiff overconsolidated clay. Another deeper sequence of sand and clay are sometimes encountered.

The footprint of the project extending from the Battery Bluff to Lombard Street was originally tidal marsh. This land was reclaimed in 1912 using hydraulic filling method. Up to 12 feet of fill consisting of sandy clays and silts and clayey sands can be found in this portion of the site.

2.2 SITE SPECIFIC SUBSURFACE CONDITIONS

A site map of the Battery Tunnels area is presented in Figure 1. This map shows the location of the borings, as well as the proposed roadway and tunnel structure. Figure 2 presents a generalized subsurface cross section, showing the soils encountered underneath the southbound tunnel.

The general subsurface condition at the site of the Battery Tunnels consists of 15 to 50 feet of sediments over Franciscan bedrock. The upper layer of the overburden soils is artificial fill. The thickness of the fill ranges between 3 to 10 feet. This unit is typically clean sand, sand with clay and sandy clay/silt and is loose.

Below the fill at the eastern edge of the Battery Tunnels is an upper stiff clay. This unit is about 10 feet thick at the eastern edge of the tunnel and shrinks to zero thickness near the middle of the tunnel. This unit is low plasticity and may contain more silt than clay in some places.

Below the fill along the middle portion of the tunnels is a layer of dune sand. This unit was found to be 17 feet thick at borehole BTSB-R1, it was not found in boring RW6-R1, and was found to be about 3 feet thick in boring BTSB-R2. The dune sand is in a medium dense state and is free of fines.

Below the fill on the western portion of the tunnels and below the dune sand and upper stiff clay is the Colma Formation. The Colma Formation is dense to very dense with typically a trace of fines (about 5 percent) to about 20 percent fines. This unit ranges from 10 to 20 feet in thickness.

Below the Colma Formation under the eastern two-thirds of the tunnel structures is a lower stiff clay. The clay is of medium plasticity. The thickness of this unit ranges from 5 to 10 feet and is not encountered in borehole RW6-R1.

Underlying the overburden soils is the Franciscan Bedrock. The bedrock underneath the western half of the tunnel structure is sandstone. The sandstone is competent with RQD ranging from 0 to about 50 percent. More competent sandstone was also found just east of the eastern edge of the tunnel in boring RW8-R1. In the center and eastern portions of the tunnel, below the overburden soils is soft to very soft bedrock. This material is mainly Mélange Matrix and serpentinite. Mélange Matrix is similar to a gravelly clay. In the serpentinite and Mélange Matrix, cobbles and boulders of sandstone and siltsone are common.

2.3 GROUNDWATER

An initial piezometer reading at RW6-R1 showed groundwater perched at the soil-rock contact (about 18 feet below the existing ground surface). It is expected that the groundwater in the other areas of the site sits on top of the bedrock or the deep stiff clay unit.

3.0 SCOUR

Scour will not be a concern on this portion of the project. However, surface runoff will cause erosion if engineered fill and native soils are not covered with vegetation.

4.0 CORROSIVITY

Aside from the fill deposits, the dune sand and Colma Foundation units are not anticipated to be corrosive. Corrosion samples have been collected from the site during the field investigation and will be tested for their corrosivity properties. Sampling and testing of the site soils and groundwater shall be in conformance with the Corrosion Guidelines for Foundation Investigations (Caltrans, 2003).

5.0 SEISMICITY/LIQUEFACTION

Structural design criteria has been proposed in the report titled *Structural Design Criteria: Cut-and-Cover Tunnels and Nonstandard Retaining Walls, Version 2,* dated January 23, 2008. This document summarizes the seismic demand placed on the structure. In addition, Geomatrix, in conjunction with Dr. Norm Abrahamson, will be retained to perform a probabilistic seismic hazard analysis, select time histories, and develop site specific rock motion spectra for the Battery Tunnels.

Potential liquefaction conditions do not exist in the Battery Tunnels area.

Suspension P-S velocity logging was performed in boreholes BTSB-R2, BTNB-R4, BTNB-R7, and RW6-R1. The velocity results for these four boreholes are presented in Appendix A of this preliminary foundation report.

6.0 FOUNDATION TYPE RECOMMENDATIONS

The Battery Tunnel structures should be founded on a reinforced mat foundation. For preliminary design, an allowable bearing pressure of 5,000 psf can be used for the portion of mat founded on the Colma foundation and 12,000 psf for the portion of the mat bearing directly on the sandstone or a spring constant approach may be used.

Two spring constants will need to be used for the mat foundation. For the eastern portion of the tunnel, the bedrock will not be exposed when excavating for the structure. For this portion, east of Station 82+80, the foundation material will be the Colma Formation unit and a preliminary design spring constant of 450 kip/ft^3 may be used. For the portion of the tunnel west of Station 82+80, the foundation material will be bedrock and a preliminary design spring constant of 1,000 kip/ft^3 may be used.

These spring constants assume that all soft or loose materials are removed prior to the construction of the mat foundation. If soft materials, especially clays, are encountered below the base of the mat, these materials will need to be excavated and replaced with structural backfill.

7.0 EARTH PRESSURES

7.1 VERTICAL EARTH PRESSURES

Landscaping over the top of the tunnel structure will exert a distributed vertical load on the roof of the structure. This dead load pressure from landscaping fill will be equal to 140 pcf times the depth of the fill. For example, if there is 5 feet of fill material placed over the structure, the applied distributed load from this material will be 700 psf.

There will be other vertical loads imposed on the structure from other sources such as traffic, however these loads are not considered in the scope of this preliminary foundation report.

7.2 LATERAL EARTH PRESSURES

Lateral earth pressures will develop on the retaining walls at the ends of the tunnels. Retaining Walls RW-5 and RW-6 are located at the west end of the Battery Tunnels while RW-7 and RW-8 are located at the east end. For preliminary estimation of the earth pressures at Retaining Walls RW-5, RW-6, RW-7, and RW-8, the recommendation presented in the Caltrans memorandum, dated February 5, 2008, should be used. A copy of the memorandum prepared for the Doyle Drive Replacement Retaining Structures is presented in Appendix B.

At-rest lateral earth pressures will develop on the side walls of the tunnels with the placement of the backfill. For preliminary estimation of the at-rest earth pressure at a particular depth, z in feet, the following equation can be used. $p_{at\text{-}rest}$ (lbs/ft^2) = 65 * z.

It is assumed that drainage behind retaining walls and the tunnel sidewalls will be provided to relieve hydrostatic pressure. Surcharges from other loadings should be added where appropriate.

Seismic lateral earth pressures will be developed as part of a detailed seismic analysis and are not included in this preliminary report.

The equations to determine preliminary pressures are given for an individual point. Over the side of the tunnel walls, the earth pressure will be a trapezoidal distributed force. Resultants may be easily determined using standard methods.

8.0 CONSTRUCTION CONCERNS

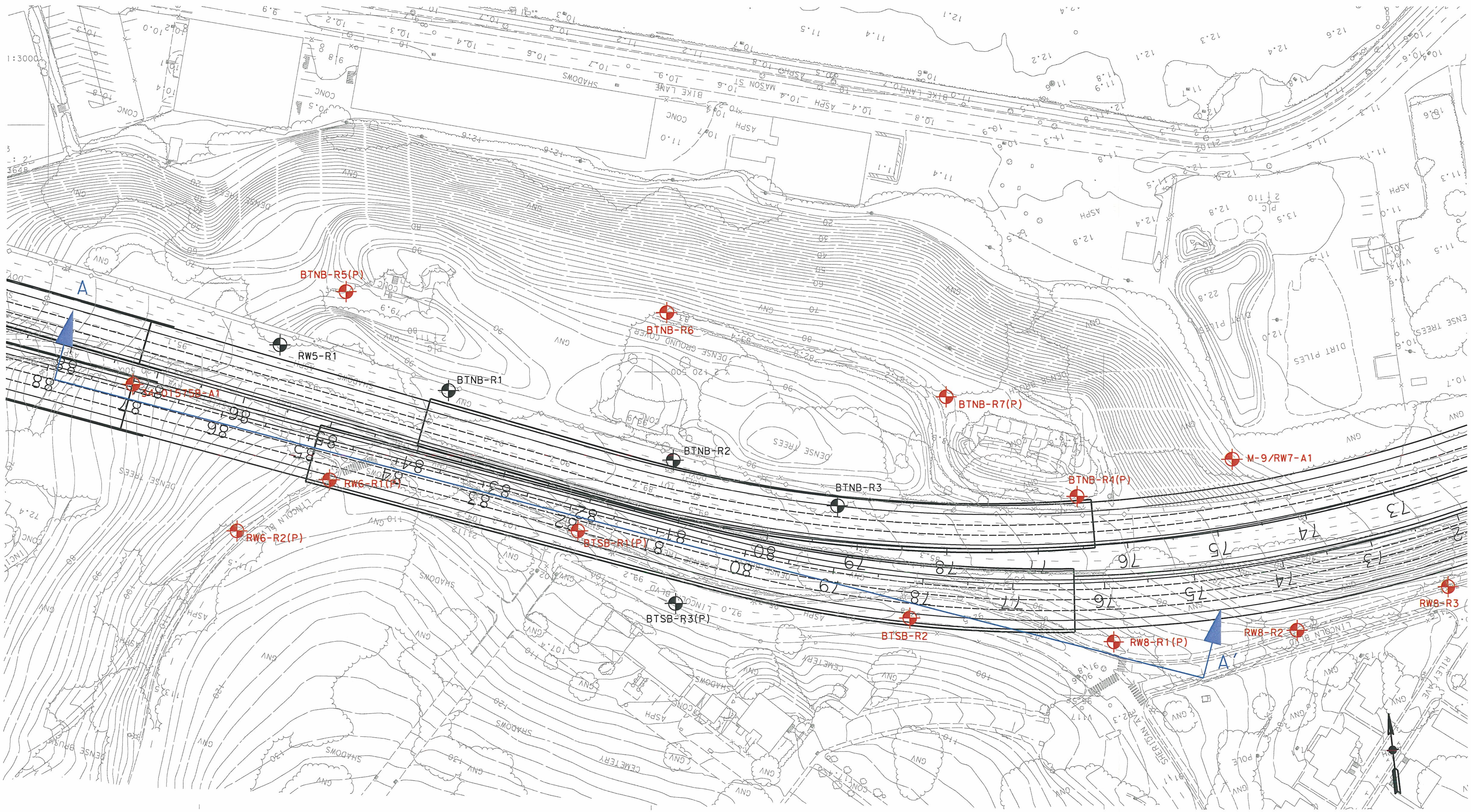
Shoring will need to be constructed for both the northbound and southbound tunnels. For the southbound tunnel, to be constructed first, Lincoln Avenue will need to be closed for installation of a shoring system. In addition, a detailed study of the national cemetery must be conducted to determine if tieback anchors may be required. If tiebacks are not an option for the shoring system, a secant wall with possible internal bracing may be required. For the northern wall on the northbound tunnel, a detailed study of the bunker structures and batteries must be completed to determine if tiebacks are possible for this wall. Again, if tiebacks are not possible an internal bracing system may be required.

Seepage through the Battery bluff is critical in order to preserve the protected wetlands on the northern face of the bluff. The Battery Tunnels must be designed with proper drainage beneath so as not to disrupt this seepage.

9.0 ADDITIONAL FIELD WORK AND LABORATORY TESTING

The laboratory testing program for the 2007/2008 exploration work has not been executed as of the issuing of this preliminary foundation report. The program will consist of laboratory tests that will determine the geotechnical engineering properties of the major soil units.

The recommendations contained in this preliminary foundation report are not final. Any questions regarding the above preliminary recommendations should be directed to the attention of Francis R. Greguras at 415-946-0255.



DOYLE DRIVE PREFERRED ALTERNATIVE

Locations of Geotechnical Explorations

Battery Tunnels

0 50 100

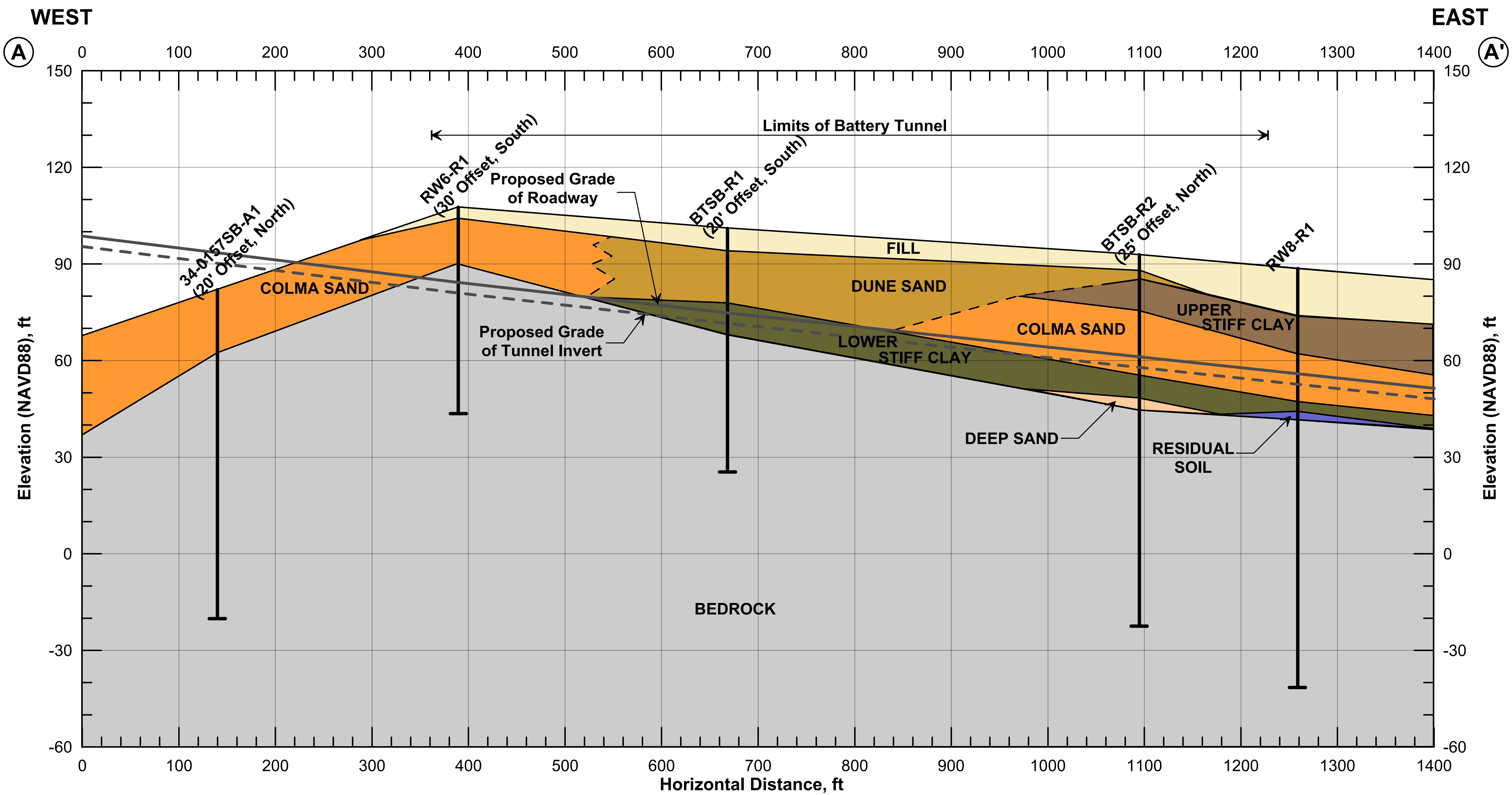
FEET

LEGEND:

Location of Borehole

Future Borehole Location

Figure 1



DOYLE DRIVE PREFERRED ALTERNATIVE

Subsurface Stratigraphy, Section A-A'
Along South Bound Battery Tunnel

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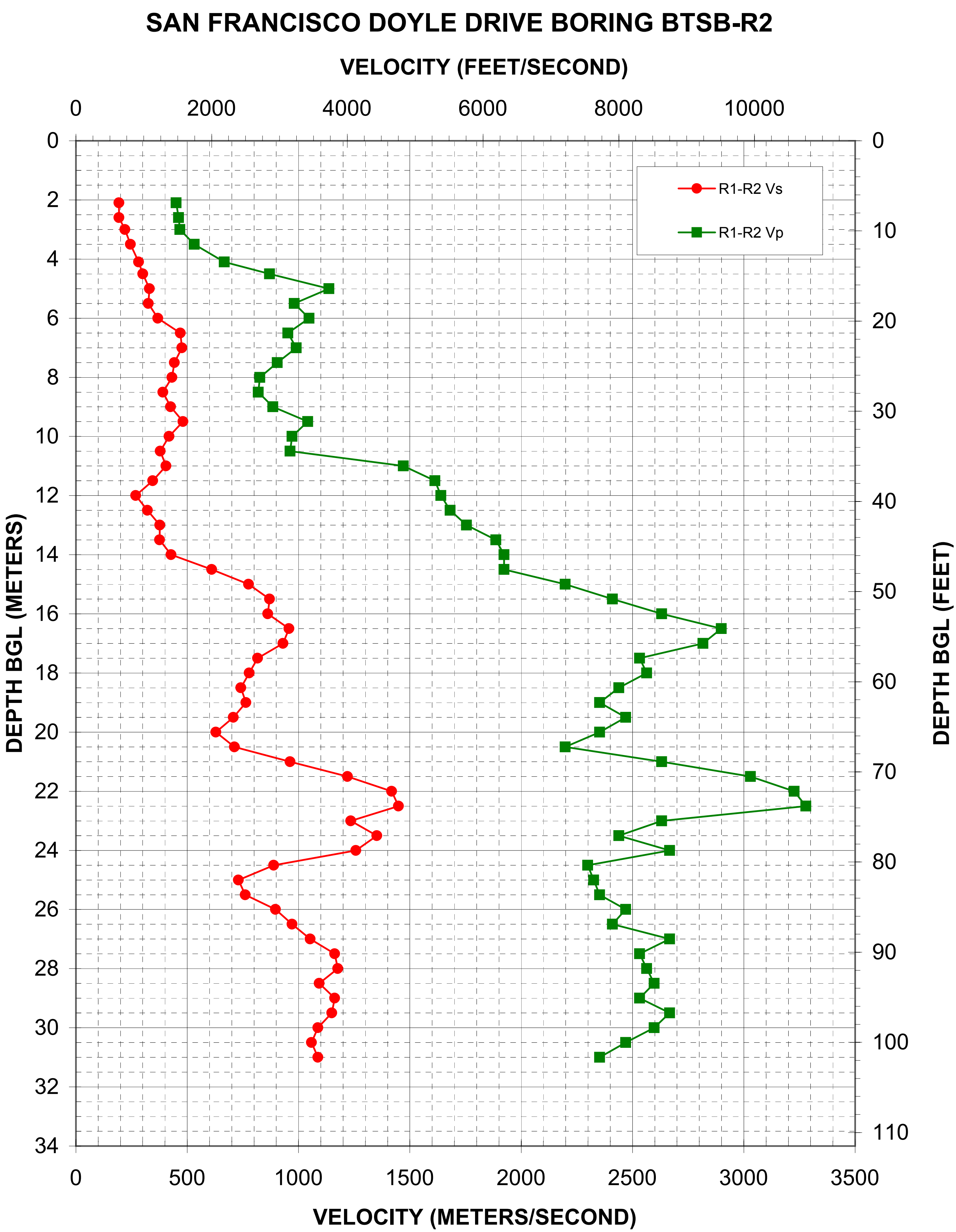
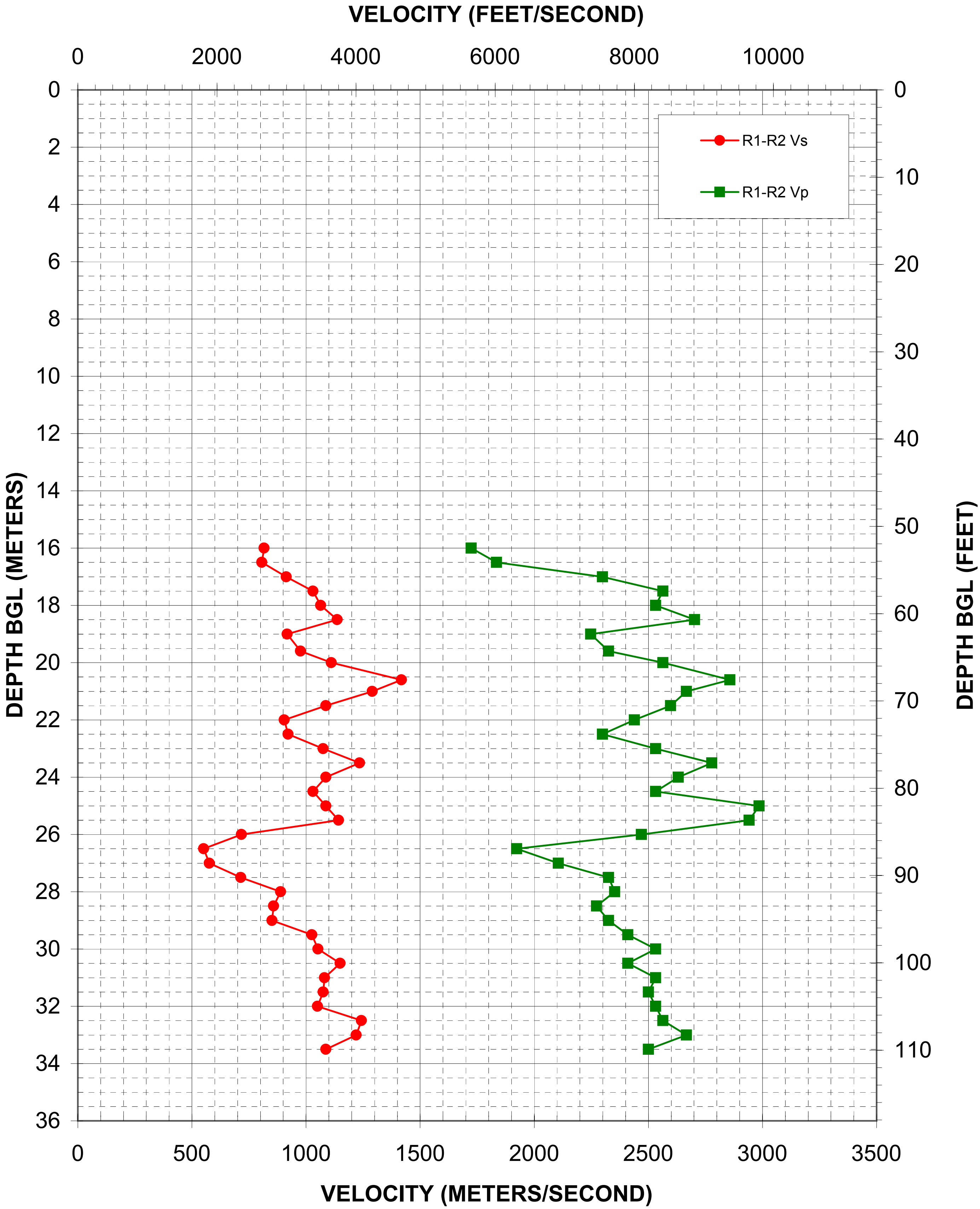
Figure 2

APPENDIX A

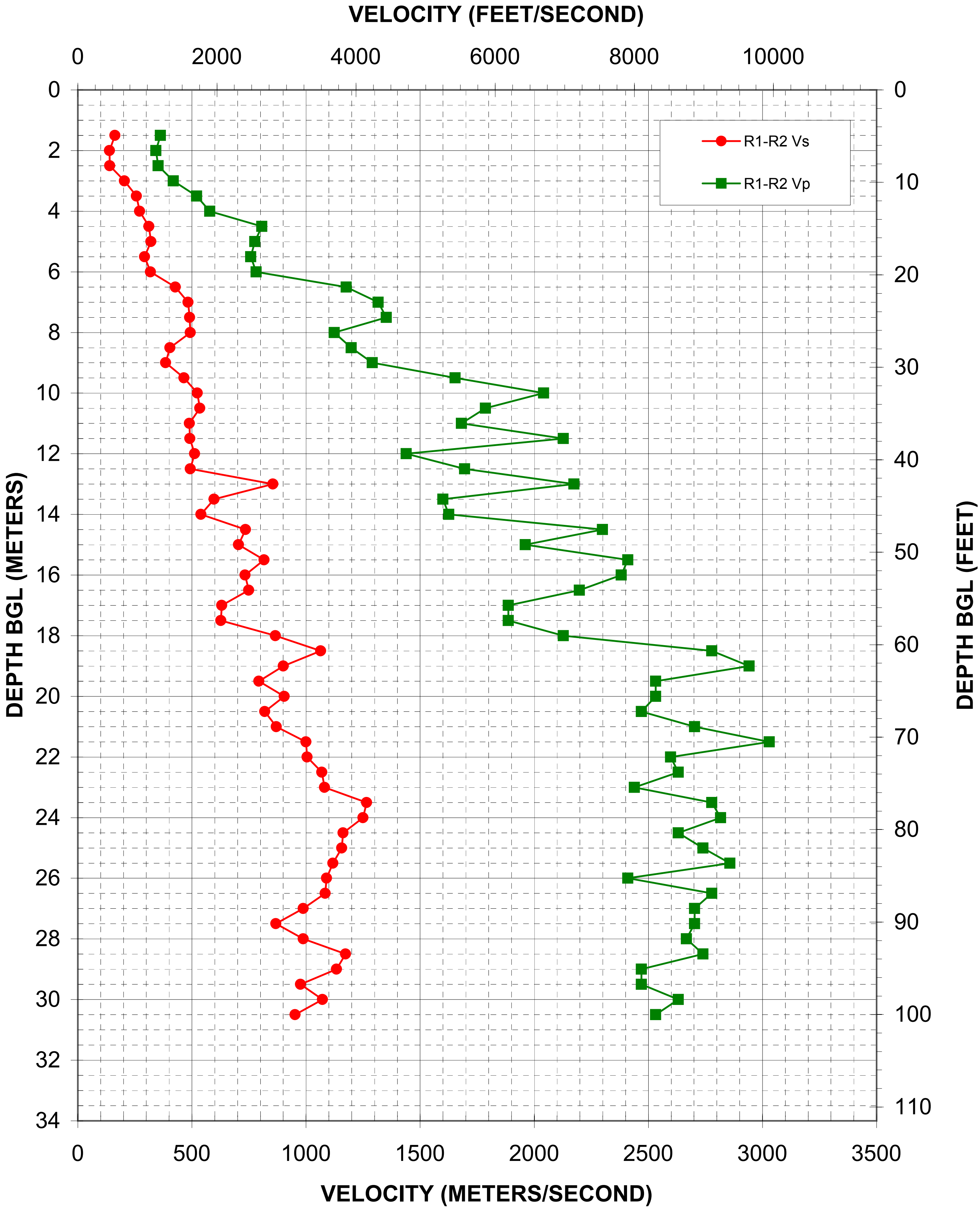
Shearwave Velocity Results
Boreholes BTSB-R2, BTNB-R4, BTNB-R7, and RW6-R1

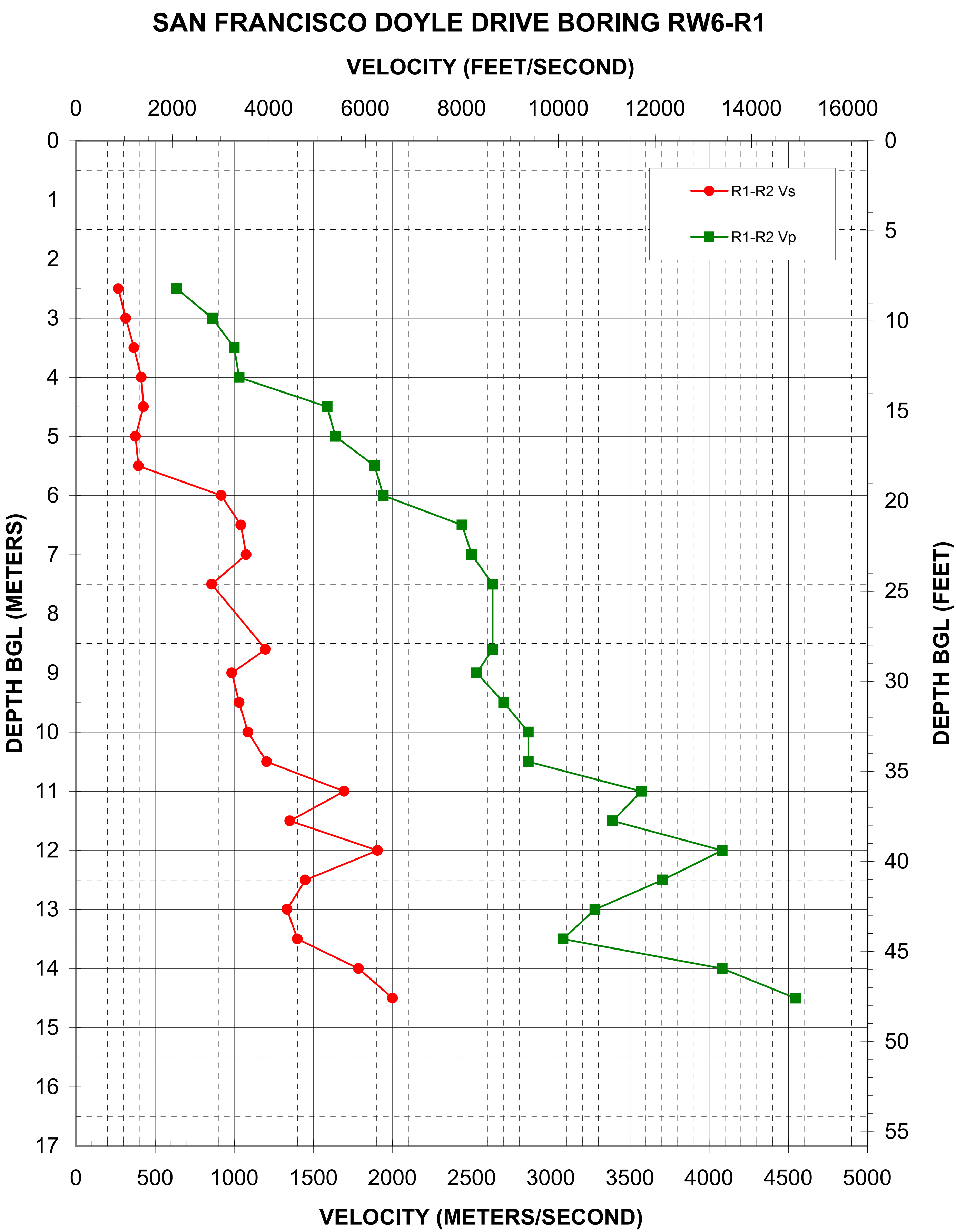

SAN FRANCISCO DOYLE DRIVE BORING BTSB-R2
VELOCITY (FEET/SECOND)
0
2000
4000
6000
8000
10000
DEPTH BGL (METERS)
DEPTH BGL (FEET)
VELOCITY (METERS/SECOND)
0
500
1000
1500
2000
2500
3000
3500
R1-R2 Vs
R1-R2 Vp

SAN FRANCISCO DOYLE DRIVE BORING BTNB-R4



SAN FRANCISCO DOYLE DRIVE BORING BTNB-R7





SAN FRANCISCO DOYLE DRIVE BORING RW6-R1
VELOCITY (FEET/SECOND)
0
2000
4000
6000
8000
10000
12000
14000
16000
R1-R2 Vs
R1-R2 Vp
DEPTH BGL (METERS)
DEPTH BGL (FEET)
VELOCITY (METERS/SECOND)
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000

APPENDIX B

Copy of Caltrans Memorandum
Dated February 5, 2008
Doyle Drive Replacement Retaining Structures Preliminary Foundation Report

Memorandum

Flex your power!
Be energy efficient!

To: MR. ABBAS TOURZANI
Senior Bridge Engineer
Office of Bridge Design-West
Division of Engineering Services

Date: February 05, 2008

Attention: Phil Lutz

File: 04-SF-101-KP 13.4/15.1
(PM 8.3/9.4)
04-163701
Doyle Drive Replacement
Retaining Structures
Structure # 34-0155L/R,
34-0162M, 34-0166 & Other
Retaining Walls

From: TUNG NGUYEN
Transportation Engineer
Office of Geotechnical Design – West
Geotechnical Services
Division of Engineering Services

WAJAHAT NYAZ
Chief, Branch C
Office of Geotechnical Design-West
Geotechnical Services
Division of Engineering Services

Subject: Preliminary Foundation Report (PFR) for Hook Under-Crossing (SB and NB) - Structure No. 34-0155L/R, Lincoln – Retaining Wall Structure 34-0162M, Girard Road Depressed – Retaining Wall Structure No. 34-0166 and Other Retaining Walls

PROJECT LOCATION

This Preliminary Foundation Report (PFR) addresses all of the retaining walls for the Doyle Drive Replacement project (Advance Planning Study (APS) Alternative 5). The report is based on the APS plans and the most recent project plans, dated January 19, 2008 provided by the Office of Structure Design. The control line stationing and some structure names and numbering have been revised in the most recent project plans. For clarity, this report addresses the proposed structure using both the new and old structure names and numbers where appropriate. The old structure names and numbers are in shown in parenthesis. The stations referenced in this report are based on the most recent project plans.

The project proposes to replace existing Doyle Drive from Marina Boulevard and Lombard Streets to the southern approach of the Golden Gate Bridge on State Route (SR) 101 from Post Mile (PM) 8.3 to PM 9.4 (or KP 13.4 to KP 15.1).

The Advance Planning Study (APS) plans prepared by consulting firm Parsons Brinckerhoff and the post recent project plans, dated January 19, 2008, show that Nineteen (19) new retaining walls (RW 1 to RW 19) are proposed in this project. RW 1 to RW 4, are located in the vicinity of the Presidio Parkway (SR 101/SR 1) Interchange. RW 9 to RW 19, are located in the proposed Tennessee Hollow and Girard Road depression area. RW 5 and RW 6 are located on the outside shoulders of northbound and southbound Doyle Drive (SR 101) immediately east of the proposed Battery Tunnel. RW 7 and Lincoln Retaining Wall (RW 8) – Structure No 34-0162M are located on the outside shoulders of northbound and southbound Doyle Drive (SR 101) between the Main Post and Battery Tunnels. Retaining wall RW 8 is the largest retaining wall in this project and it spans the entire distance between the Main Post and Battery Tunnels.

Retaining walls RW 11 to RW 19 are located at the proposed Girard Road Depressed Section and they are all part of a below ground reinforced concrete slab "boat" section structure. These retaining walls are part of one single structure that will consist of a depressed section and related on-off ramps. These walls are located at:

- Southbound Doyle Drive off-ramp to Girard Road
- Girard Road on-ramp to northbound Doyle Drive
- Girard Road
- Gorgas Road

In this report RW 11 to RW 19 will be treated together as Girard Road Depressed – Retaining Wall Structure No 34-0166. Similarly, portion of RW 1 and RW 2 are part of the northbound and southbound Hook Under-Crossings – Structures No 34-0155 L/R (PPI UC), which consists of a cast-in-place box structures. These walls will be treated as part of the northbound and southbound Hook Under-Crossing (PPI UC) structures.

Since the retaining walls plans have not been developed at this stage of the project design, we have estimated the retaining wall heights from the typical sections, roadway profile and project topographic map. These wall heights are not final and are meant only for the preliminary foundation recommendations. For retaining walls locations (layout), refer to the most recent project plans dated January 19, 2008.

SITE GEOLOGY AND SITE SUBSURFACE CONDITIONS

Site Geology

Shallow bedrock of the Franciscan Formation generally underlies the more elevated western areas of the project site. This Formation consists of heavily folded and sheared assemblage of Greywacke Shale Sandstone, Chert, and Serpentinite. The overburden soil consist of artificial fill, slope debris, ravine fill and/or the Colma Formation, which is an unconsolidated fine to medium-grained sand unit with clay beds. The lower elevations on the eastern side of the project site reflect an estuarine deposition environment where the bedrock is located at a significant depth. The surficial soil in the eastern side of the project site consists of dune and beach sands and soft clayey silt layers. These soils are generally underlain by Colma Formation, which overlies the Franciscan bedrock. The bedrock is exclusively serpentinite west of Station 90+00±, and Sandstone/Shale is found east of Stations 90+00±.

In the past, the project site consisted of an extensive tidal marsh separated from the Bay by a beach and dunes. The area that extends from the Crissy Field in a southerly direction towards Lombard Street and underlies Doyle Drive east of the Post Commissary Building was filled with hydraulic fill in 1912. The hydraulic fill consists of loose sands with variable amount of silt and clay. The specific subsurface conditions at each structure (retaining wall) location are discussed in the following section.

Site Specific Subsurface Conditions

The subsurface conditions described herein are based on the preliminary subsurface investigations performed for this project by Taber Consultants and review of pertinent available reports, and boring logs of projects performed at, or in the vicinity of, the project site. For simplicity, the soil/rock and groundwater conditions described in this report have been subdivided based on structure (retaining wall) locations and site geologic similarity.

Northbound and Southbound Hook Under-Crossings (PPI UC) - Retaining Walls RW 1 & RW 2 and Retaining Walls RW 3 & RW 4

The subsurface condition at these locations is characterized by the information obtained from nearby exploration borings No. 22, 29, and 30 conducted for the original Presidio

Road construction in 1933. These borings were drilled to the depths of 19 ft, 15 ft, and 15 ft, respectively. The boring locations map and boring logs are included in the Doyle Drive Preliminary Geotechnical Report (PGR).

The existing boring logs show that the site is underlain by a maximum of 10 ft of Quaternary clayey and sandy deposits, which are classified as slope debris and ravine fill. The slope debris and ravine fill are underlain by serpentine bedrock. Groundwater information at this location could not be determined from the available information.

Please note that the subsurface information in the above mentioned boring logs is limited. The boring logs lack quantitative and qualitative material descriptions and they have limited or no soil consistency, density, strength, and rock hardness information. These boring logs also lack groundwater information.

Retaining Walls RW 5 and RW 6

Borings No. 17 and 18 drilled for the Presidio Road construction in 1933 and boring GB-6 drilled, in January 2001, for this project are located in the vicinity of RW 5 and RW 6. Borings No. 17, 18, and GB-6 were drilled to depths of 26 ft, 15 ft, and 17 ft, respectively. The boring locations map and boring logs are included in the Doyle Drive Preliminary Geotechnical Report (PGR).

Borings No. 17 and 18 show that the site is underlain by 11 to 20 ft of surficial soils consisting of sandy loam, sandy, silty clay, and clay with fine to coarse sand and gravel. The clay materials are soft to very hard with low plasticity and unconfined compressive strength ranging from 3 ksf to 20 ksf. Published geologic map show that the dominant surficial soils at this location are dune sand in contact with Colma Formation. Dune sand are generally poorly sorted, well rounded, medium to coarse sand. Colma Formation is described as Pleistocene light brown to orange, fine to medium grained sand with minor clay material. Sandstone (bedrock) was encountered in the borings below the surficial soils. The sandstone encountered is generally very intensely weathered and fractured with Rock Quality Designation (RQD) of zero. Results of point load tests performed on samples from depths of 13 ft and 16 ft show that the uniaxial compressive strength of sandstone rock varies from 4.20 ksi to 5.35 ksi.

We anticipate that the cut for the RW 5 will be entirely in surficial soils and cut for the RW 6 may encounter weathered and fractured sandstone (rock) at lower elevations.

Groundwater condition was not described in Borings No. 17 and 18 and groundwater was not encountered in Boring GB-6. At this time, it is our assessment that groundwater will not be encountered in the cuts for RW 5 and RW 6.

Retaining Wall RW 7

The most relevant information for this wall can be obtained from the 1998 PC Exploration/Woodward-Clyde boring, B-1, drilled for a slope (storm damage) repair project (at PM 9.0). This boring was drilled on a slope, to the depth of about 37 ft, immediately north of the existing Doyle Drive (SR 101) and behind the proposed RW 7. The slope at this location failed and it was subsequently repaired.

The boring B-1 encountered loose sand, silty sand, sandy silt, and clayey sand to a depth of 7.2 ft. Below these deposits, are loose to medium dense sand and sandy clay/clayey sand that extend to a depth of 33.5 ft and overlay 2-ft thick very stiff gravelly clay. Under this layer to the maximum depth of exploration, is dark green to black, hard and highly sheared Serpentine. Results of consolidated undrained triaxial tests on samples taken at depths 10 ft and 16 ft show that sand deposits has an effective friction angle of 35 degree. The Standard Penetration Test (SPT) blow count (N) in this material ranges from 8 to 15. Since the slope has been repaired, the proposed cut for RW 7 will likely be in compacted fill and surficial soils described above and bedrock will not be encountered.

According to the PC Exploration/Woodward-Clyde Report, groundwater was encountered at the depth of 15 ft in boring B-1 at the time of drilling. However, the groundwater elevation was not shown in the boring log of B-1.

Lincoln Retaining Wall (RW 8)

Borings GB-5 and GB-4 and Cone Penetration Test (CPT) sounding CPT-8, performed during the preliminary investigation to a depth of 50 ft in January 2001, describe the material likely to be encountered at RW 8 location. Based on the borings and the CPT, the cut in the western portion of RW 8 is likely to encounter 0 to 12 ft of medium to very dense (N of 16 to 28) silty fine sand underlain by very dense ($N > 50$) clayey sand. The middle portion of the cut is likely to encounter very stiff clay with fine sand and very dense ($N > 50$) clayey fine sand. The grain size analyses showed that the percentage of fines (clay materials) in the sandy deposits is about 14 %. The cut in the eastern portion of RW 8 is likely to be in stiff to very stiff clay, silty clay, and sandy clay. The bedrock was encountered in borings GB-5 at the depth of 59 ft (elevation 32 ft) and bedrock is not likely to be encountered in the proposed cut for retaining wall RW8.

Groundwater was encountered in GB-5 at elevation 64 ft in January of 2001. We anticipate that groundwater will be encountered in the western portion of the proposed wall cut. Groundwater was not measured in boring GB-4 and CPT-8 during the investigation. We also anticipate that a perched groundwater table develops at this site during the rainy season.

Girard Street Depressed Section (Location of Retaining Walls 9 to 19)

The Girard Street location has been studied extensively in the past. Many borings and CPT soundings have been performed and several monitoring wells have been installed at the site. However, most of the available information was generated for environmental and hydrogeology studies and it is of limited use for this study. We used a variety of existing information to characterize the subsurface and groundwater condition at the Girard Depressed Section site. The previous investigations used for site characterization include:

- The 1997 International Technology Corporation site investigation in the vicinity of Buildings 207 and 231 on the south sides of the Highway 101 ramps. Four CPT soundings 231-HP15, 231-HP16, 231-HP18, and 231-HP31, performed for the (1997) investigation were used in this study.

- The 2003 hydrological investigation by Taber Consultants for this (Doyle Drive) project. Two CPT soundings, CPT-3 and CPT-4, performed to the depths of 35 and 49 ft, respectively and 3 boring, HGB-1, HGB-2 and HGB-3, drilled to the depths of about 90 ft were used for this study. Borings HGB-1, HGB-2, and HGB-3 were converted to piezometers (monitoring wells) to monitor groundwater level at different depths. The historical groundwater monitoring data of these piezometers are not available.

- The 2007 MACTEC Correction Action Plan (CAP) investigations for Buildings 207/231 and 1065. This investigation was performed for the environmental remediation of Buildings 207/231 and 1065 sites. Numerous borings were drilled for this investigation. This CAP investigation reports included geologic cross-sections for a portion of the Girard Depressed Section site. Several of the borings were converted to piezometers (monitoring wells) to monitor groundwater level at different depths. The historical groundwater monitoring data of these piezometers are included in the CAP reports.

Based on the results of the investigations mentioned above, the subsurface and groundwater condition at the Girard Depressed Section can be characterized as follows:

The Girard Depressed Section site consists of zero to 10 ft (maximum) thick artificial fill, which generally consists of loose (N of 7) silty sand, sandy silt, and very soft to soft sandy clay with gravel. The fill is underlain by a thin layer of very soft to soft (N of 0 to 4) dark gray clay with peat, silty clay, and clayey silt. The thickness of this layer varies considerable and ranges from 2 ft to 10 ft. These clayey deposits have low to medium plasticity index and undrained shear strength of 400 psf or higher. The soft clayey deposits are underlain by a 5 to 15 ft thick layer of loose to very dense (N of 7 to 65) silty/clayey fine sand with about 22% fines. The loose to dense silty/clayey fine sands are underlain by a layer of very soft to stiff silty clay (N of 0 to 16). All of these deposits mentioned above overlay medium to very dense silty and clayey sand with thickness up to 44 ft (N of 16 to refusal). These sandy deposits have 5 to 41% fines. Under the sand layer, to the maximum depth of exploration, is a layer of very stiff to hard (N of 16 to refusal) sandy clay, silty clay, and clay with sand and very dense silty sand. The clayey materials have medium to high plasticity index and undrained shear strength of 3800 psf or higher.

Groundwater was encountered in the borings at the depth of about 5 ft below the ground surface. The project final Hydrology Report and the 2007 CAP reports indicate that the site of the Girard Depressed Section has a shallow unconfined aquifer and deeper confined aquifer. The Hydrology Report states that the potentiometric head was found to be above the ground surface. However, this report does not provide the quantitative potentiometric head or include the groundwater monitoring data. The CAP Report (for the area of Building1065/1027) also show that the groundwater head in the lower aquifer is higher than the water level of the upper aquifer in the northeastern portion of the site and at some locations it is higher than the existing ground surface. At this time, we do not have quantitative information to determine the actual potentiometric head. The upper and lower aquifers are generally separated by an aquitard consisting 2 to 3 feet thick layer of stiff silty clay. The data obtained from the above referenced investigation indicate that the permeability of the deeper confined aquifer is low to medium.

SCOUR EVALUATION

Scour evaluation will be addressed in the final hydraulic report. This report is being prepared by consulting firm, Parsons Brinkhoff.

CORROSION EVALUATION

The site is anticipated to be corrosive due to its proximity to the San Francisco Bay and the past existence of a tidal marsh. Soil/rock and groundwater samples will be collected from this site during the geotechnical investigation and tested for corrosion potential. Sampling and testing of site soils and groundwater shall be in conformance with the Corrosion Guidelines for Foundation Investigations (Caltrans, 2003).

PRELIMINARY SEISMIC STUDY

Hossain Salimi of the Office of Geotechnical Design-West will provide preliminary seismic recommendations including preliminary liquefaction analysis for this project.

PRELIMINARY FOUNDATION RECOMMENDATIONS

Northbound and Southbound Hook Under-Crossing (PPI) - Retaining Walls RW 1 & 2

A portion of RW 1 and RW 2 will be the earth retaining sidewalls of the box structures that will serve as under-crossings and the remaining portions of these walls will be cantilever walls that will retain cut leading up to the under-crossings. The maximum retained height of these walls is anticipated to be ±14 ft. We recommend that the box structure and cantilever potions of retaining walls RW 1 and RW 2 be treated as separate and independent structures.

<u>Box Structure</u>

We do not anticipate settlement or bearing capacity issues at this location and therefore, shallow foundation will be adequate. The sidewalls of the box structure should be designed as restrained retaining structures using at-rest earth pressure. The preliminary loading and allowable bearing pressure for the box structure are provided in Table 1.

Table 1: Earth Pressure for the Northbound and Southbound PPI Box Structures

Equivalent At-Rest Earth Pressure (pcf)	Equivalent Passive Earth Pressure (pcf)	Seismic Earth Pressure (psf)	Allowable Bearing Pressure (psf)
55	442	19H	5000

H is the design height

The loading assumes that the drainage of box structure sidewalls (to relief hydrostatic pressure) will be provided. If drainage cannot be provided, assume a hydrostatic pressure for the 2/3 of the wall design height. Traffic loading should be added where appropriate.

Cantilever Walls

Since we do not anticipate settlement or bearing capacity issues at this location and the retaining wall heights are anticipated to be less than 14 ft, Standard Retaining Wall (Type I) on spread footing will be appropriate at this location. A joint should be provided between the cantilever walls and the box structure to accommodate relative movement.

Retaining Walls RW 3 & RW 4

Based on the available information, it is our assessment that retaining wall RW 3 will retain fill and cut and retaining wall RW 4 will retain only fill. RW 3 and RW 4 are generally medium to small size walls with maximum heights of ±12 ft and ±21 ft, respectively. Based on the site and subsurface conditions, it is our preliminary assessment that Standard Retaining Wall (Type I) on spread footing will be appropriate for these walls.

Retaining Walls RW 5 and RW 6

Retaining walls RW 5 and RW 6 are predominantly cut walls with maximum heights of 15 ft and 26 ft, respectively. These walls are located immediately west of the proposed cut-and-cover Battery Tunnel and will conform to the tunnel sidewalls. The cut portion of the walls should be constructed in conjunction with the tunnel. According to the APS, the temporary shoring system for the tunnel excavation will consist of soldier pile earth retaining system.

CIDH pile wall with tiebacks, secant/tangent pile wall, stressed anchor wall or conventional gravity retaining wall are possible alternatives for these walls. However, it is our understanding that the issue of temporary/permanent easement behind the wall is still unresolved. Availability of easement will play a big part on the selection of the retaining system at this location. Depending on the easement situation, we recommend three alternatives for these walls. The preliminary design parameters for these alternatives are listed in Table 2.

Option 1- permanent easement is available. In this scenario, we recommend a stresses anchor wall with whaler beams. We anticipate a maximum of two rows of anchors inclined at 10 to 15 degrees to the horizontal. These anchors will be about 50 ft (maximum) long. Requiring a permanent easement of at least 50 ft at all locations where the wall height exceeds 12 ft. We do not recommend soil-nail wall at this time because soil nails anchors will require drilling hole at 5 ft on-center (or less), both horizontally and vertically. Because of the sandy nature of the subsurface material, there is a high potential that the drilled holes will not stay open and casing will be required making installation of a large number of soil nails very time consuming and expensive. Caltrans do not have an approved self-drilled anchor system for permanent applications. We anticipate instability of the cut face during construction. Temporary measures such as installation of vertical nails (micropiles) will likely be needed to temporarily stabilize the cut face.

Alternatively, CIDH pile wall or Secant Pile wall with tie-backs can be used. Tiebacks will be needed at locations where the wall height exceeds 12 ft. The number of tiebacks rows and requirement for permanent easement will be the same as mentioned above.

Option 2- easement is not available – In this scenario, we recommend a tangent/secant pile wall. The tangent/secant pile wall will consist of 2 to 3 ft diameter overlapping or adjoining drilled piles with "I" beam in every (or every other) pile. At locations where the wall height exceeds 15 ft, large diameter (3 ft) CIDH piles with large size double "I" beams will be needed to provide the needed stiffness and to limit deflection to an acceptable level. Because of the sandy nature of the subsurface soils and presence of groundwater, the installation of CIDH piles is likely to be difficult. We anticipate that that this alternative will be significantly more expensive than other options discussed in this report.

<u>Option 3 - only temporary easement is available</u>. In this scenario, a temporary soil-nail can be constructed at a distance of 0.9 x H (where H is the wall design height) behind the proposed RW 6 face to make room for construction of a gravity retaining wall. Once the gravity retaining walls is constructed, the space between the temporary soil-nail wall and the permanent gravity retaining wall can be backfilled. The permanent gravity wall can be Standard Retaining Wall (Type 1) on spread footing.

To address the constructability concerns associated with temporary soil-nail installation mentioned above, we recommend using self-drilled anchor for the temporary soil-nail wall. The self-drilled anchors can be abandoned in place.

Even though the existing borings do not show presence of groundwater, it is our opinion that groundwater should be anticipated, at the lower elevation when the cut is made at this locations. The walls must be designed with a drainage system to prevent build-up of hydrostatic pressure behind the wall. Some dewatering may be needed, if conventional gravity wall option is used.

Please note that RW 5 and RW 6 are essentially extension of the proposed Battery Tunnel, and their construction operations are likely to be contagious with the tunnel. The design of these walls should be performed in coordination with the tunnel temporary shoring design. Using similar system for both walls and tunnel shoring is likely to be more economical.

Retaining Wall RW 7

Retaining wall RW 7 is a cut wall with maximum height of 36 ft. This wall is located immediately east of the proposed cut-and-cover Battery Tunnel (on the north side of proposed Doyle Drive) and like RW 5 and RW 6, it will conform to the tunnel sidewalls. The existing slope at this location has had instability (storm damage) in the past and a repair was performed at this location in 2000. The slope failure was entirely in the surficial soil above the bedrock. A back-analysis of the failed slope at this location was performed in 1998. The back-analysis indicated that the surficial material of this slope has an overall effective friction angle of 35 degrees and effective cohesion of zero.

The alternatives recommended for RW 5 and RW 6 are also applicable to RW 7 except that the Option 2 cannot be fully implemented. It is our preliminary opinion that it will not be possible to construct the secant/tangent pile wall without anchors for wall heights in excess of 25 ft at this location and therefore, some temporary or permanent easement will be needed. We anticipate permanent easement of at least 50 ft or a temporary easement as discussed in RW 5 and RW 6 (Option 3) section. The preliminary design parameters for these alternatives are listed in Table 2.

Lincoln Retaining Wall (RW 8)

Lincoln Retaining Wall, RW 8, is a cut wall with maximum height of about 30 ft and a 10 ft high slope on top of the wall. The APS plans show that this wall is battered at an inclination of 4:1 (vertical to horizontal). RW 8 wall is located on south side of the proposed Doyle Drive and it spans the entire distance between the two cut-and-cover tunnels and it will conform to the tunnel sidewalls. We anticipate that this wall will be constructed in conjunction with the cut for the tunnels. A portion of the RW 8 will be located immediately (within a distance of few feet) in front of Building 106. It is our understanding that Building 106 will be under pinned.

CIDH pile wall with tie-backs, secant/tangent pile wall, stressed anchor wall or gravity conventional retaining wall are possible alternatives for this wall. However, it is our understanding that the issue of temporary/permanent easement behind the wall is still unresolved. Availability of easement will play a big part on the selection of the retaining system at this location. Depending on the easement situation, we recommend three alternatives for this wall. The preliminary design parameters for the design of these alternatives are listed in Table 2.

Option 1 - permanent easement is available. In this scenario, we recommend a stresses anchor wall with whaler beams. We anticipate a maximum of two rows of anchors inclined at 10 to 15 degrees to the horizontal. These anchors will be about 50 ft (maximum) long. Requiring a permanent easement of at least 50 ft at all locations where the wall height exceeds 12 ft. We do not recommend soil-nail wall at this time because soil nails anchors will require drilling a large number of holes (at 5 ft on-center or less), both horizontally and vertically. There is a high potential that the drilled hole will not stay open and casing will be required making installation of the anchors (nails) very time consuming and expensive. Caltrans do not have approved self-drilled anchor system for permanent applications. This option should be able to accommodate the proposed wall batter.

Alternatively, CIDH pile wall or tangent/secant pile wall with tie-backs can be used. Tiebacks will be needed at locations where the wall height exceeds 12 ft. The requirement for permanent easement should be the same as mentioned above. It will be very difficult to accommodate the proposed wall batter if CIDH pile wall or tangent/secant pile wall with tie-backs system is used. We anticipate that if any of these systems are selected, the wall batter will have to be eliminated.

Option 2 - easement is not available – It is our preliminary opinion that it will not be possible to construct this wall as planned without permanent and or temporary easement. Our analysis indicate that, for all practical purpose, secant/tangent pile wall system cannot be constructed without anchors for heights in excess of 25 ft and at locations around Building 106 where wall deflection has to be minimal. At these locations, the wall will require anchors and consequently permanent easement will be needed. Based on the APS plans, these locations are from Stations ±76+15 to ± 76+45 and ±65+70 to ±66+60.

At all other locations, secant/tangent pile retaining can be constructed. The tangent/secant pile wall will consist of 2 to 3 ft diameter overlapping or adjoining drilled piles with "I" beam in every (or every other) pile. At location where wall height exceeds 15 ft, large diameter (3 ft) CIDH piles with large size double "I" beams will be needed to provide the needed stiffness and to limit deflection to an acceptable level. It will be very difficult to batter the wall at 4:1 (vertical to horizontal) using this system. If this option is selected, the wall batter should be eliminated. Because of the sandy nature of the subsurface soils at some locations and presence of groundwater, the installation of the CIDH piles is likely to be difficult and casing may be needed. We anticipate that construction of secant/tangent pile retaining system is likely to be significantly more expensive than other options discussed in this report.

At location where anchors are needed, CIDH pile wall with tie-backs can be viable alternatives for this wall. The anchor spacing and easement requirement will be similar to Option 1 above. At this time, we do not recommend soil-nail wall or stressed anchor wall with a whaler.

Option 3 - only temporary easement is available. Again, the portion of proposed wall in the vicinity of Building 106 (±65+70 to ±66+60) cannot be constructed as planned without permanent easement. The rest of the wall can be constructed as described below.

Construct a temporary soil-nail at a distance of 0.9 x H (where H is the wall design height) behind the proposed RW 8 face to make room for construction of a gravity retaining wall. Once the gravity retaining walls is constructed, the space between the temporary soil-nail wall and the permanent gravity retaining wall can be backfilled. The permanent gravity wall can be modified form of Standard Retaining Wall (Type 1 or Type 3). This option should be able to accommodate the proposed wall batter.

To address the constructability concerns associated with soil-nail installation mentioned earlier, we recommend using self-drilled anchor for the temporary soil-nail wall. The self-drilled anchors can be abandoned in place.

It is not possible to construct the temporary wall between Stations ±65+70 and ±66+60 due to the proximity of Building 106. This section of RW 8 will require tiebacks anchors as described earlier.

At this time, we do not have the as-built foundation plans of Building 106 and the need and scope of the underpinning work cannot be estimated. The underpinning and impact of excavation on Building 106 will be evaluated in detail in the final Foundation Report.

Table 2: Preliminary Design Soil Parameters and Pressures for Retaining Walls RW 5 to RW 8

Wall #	Internal Friction Angle ϕ (degrees)	Internal Cohesion c (psf)	Unit Weight (pcf)	Seismic Pressure (psf)	Hydrostatic Pressure
5	35	0	125	19H	
6	35	0	125	19H	
7	32	0	125	19H	Below Elev. 33 ft
8	35	0	125	19H	Lower 1/3of design height

Where H is the design height

Girard Street Depressed Section (Retaining Walls 11 to 15 and 18 to 19)

The Girard Street Depressed Section will require cuts of up to ±22 ft in height. The sides of the proposed depression will be retained by a series of retaining walls and the bottom slab will be on pile foundation. Most of the Girard Depression area is underlain by a pressurized confined aquifer. The deepest portion of the proposed cut will extend into the pressurized confined aquifer. Effective dewatering, stabilization of the excavation bottom, and controlling settlement of the nearby structures/utilities due to dewatering are major concerns in the design and construction Girard Street Depressed Section. Furthermore, the site has high potential for liquefaction.

Based on the information available we recommend the following:

- Minimize the area (footprint) of the excavation. The plans call for a 122 ft (max) wide excavation of which approximately 50 ft will be backfilled with soil to create landscaped slope. We recommend that the excavation width be reduced to the amount needed to accommodate the roadway and the landscape is eliminated. The sidewalls can be stepped to allow for planting.

- Create a cut-off system (walls) to isolate the depressed area. The cut-off system must be watertight and it should extend deep into the confined aquifer to minimize the effect of dewatering (settlement) on the nearby structures. Alternatively, the cut off wall can be restricted to the shallow aquifer if a continuous aquitard can be constructed to isolate the confined aquifer.

- The cut off wall can be either temporary structure or part of the permanent structure. Please note that the cut-off system is temporary and the permanent section should be designed as a watertight section.

  Interlocking sheet piles, soil-cement mix columns and jet grout columns are possible options for the cut-off system. If the cut-off system will be a temporary structure and noise/vibration generated due to the sheet pile installation is not an issue, sheet pile option will be the most appropriate option. If the cut-off system is designed as a part of the permanent structure or there are noise restrictions to preclude pile driving, a soldier beam and soil-cement/jet grout column will be appropriate. At location where deflection of the cut-off system has to be controlled to minimize impact to the adjacent structure(s), tie-back anchors may be necessary. For preliminary design purpose, we recommend the following:

- If the cut-off system is a temporary structure, create a cofferdam of interlocking sheet piles to isolate the excavation area. The water retaining capability of the sheet pile can be increased by using a sealant system. The tip of the sheet pile is estimated at elevation –52 ft.

- If the cut-off system is a part of the permanent structure, install a soldier beam and soil-cement/jet grout columns wall. For preliminary purpose, we recommend 27 inch primary grout piles with W24 I-beam insert at 4 ft on-center and grout in between the piles to form a water tight barrier. The pile grout tip should be at elevation –52 ft. The W24 I-beam need not extend to grout tip and can be terminated at elevation –35 ft. The construction of the cut-off wall should be the first order of work at this site.

The choice of the appropriate cut-off wall will depend on many factors and cost analysis and this issue will be discussed in detail at a later stage when the detailed subsurface investigation is completed.

- There is a high potential that the excavation bottom will blow out due to artesian pressure as the excavation progresses. Our calculations show that a blow-out should be anticipated due to the excavation of the overburden soils below elevation 1.65 ft. This blow-out will create unstable condition for slab and pile installation. To prevent the blow-out, create a 5 ft thick water tight grout certain (aquitard) at the top of the confined aquifer in the entire depressed area within the confines of the cutoff walls. The top elevation of the curtain (aquitard) should be at elevation +1 ft. The curtain should be created after the cut-off walls are installed but prior to any excavation at the site. The grout curtain by itself may not be sufficient to prevent blowout at all locations and additional measure may be needed. It is our opinion that in addition to measures recommended above, it will be necessary to install bleeder wells through the natural and artificial aquitards (to relief pressure) and dewatering the site. The spacing and diameter of the bleeder wells will depend on the permeability and water head in the deeper aquifer. Dewatering of the shallow aquifer should be performed prior to installation of bleeder wells.

The bottom slab must be designed to resist hydrostatic pressure from water head in excess 24 ft above the bottom of the slab. It is our opinion that tie-downs or pile foundation will be needed to prevent uplift of the bottom slab. Please note that bleeder wells are affective only as long as dewatering is ongoing. In the long-term, the bleeder wells will become ineffective. The piles should derive uplift resistance from soil below the liquefiable zone.

The site is underlain by loose sandy deposits that can liquefy during a seismic event. The anticipated zone of liquefiable soil is located between elevation –10 ft and – 20 ft. Refer to the Preliminary Seismic Report for more detailed discussion on liquefaction. We also anticipate poor soil/pile cap contact due to the artesian pressure. Based on the site conditions, the best option will be to install pipe piles at this location. If pipe piles are no feasible due to noise/vibration restrictions, CIDH pile can be used. However difficult CIDH pile installation should be anticipated due to the presence of pressurized aquifer, and loose sandy soil. Temporary and/or permanent casing will be needed and the piles have to be installed in wet condition with slurry head that can equalize the artesian water pressure. It is possible that permanent casing will be needed in the upper zone of the piles. Another option will be install soil-cement columns and insert H beams in lieu of CIDH piles. In this scenario, we recommend that uplift capacity be computed based only of H-pile section since it will be difficult to verify the soil-cement pile quality. Alternatively, a pile test can be performed to verify the uplift capacity.

The preliminary pile tension (uplift) will be derived from soil below elevation –23 ft. For preliminary purpose, assume unit soil friction along the pile shaft is shown in Table 3. The earth retaining sidewalls of the Girard Depressed Section should be designed using earth pressure loading provided in Table 3. The depressed section will likely be required a mechanical pump system to remove surface water.

Table 3: Preliminary Earth Pressure and Bearing Pressure for Girard Depressed Section Boat Section

Equivalent active Earth Pressure (pcf)	Passive Earth Pressure (pcf)	Seismic Loading (psf)	Pile Unit Friction Resistance (Tension) (psf)
80	250	19H	45H* (900 max)

** Triangular distribution from Elev. –23 ft to –43; Below Elev. –43 ft, use a constant value of 900 psf*

Retaining Walls RW 9, RW 10, RW 16 and RW 17

Retaining walls RW 9 and RW 10 are cut walls, retaining walls RW 16 and RW 17 are fill walls which will retain the approach fill to the Girard Road under-crossing. Based on our preliminary assessment, these walls can be standard retaining wall (Type 1) on pile foundation. Again, because of the anticipated noise vibration restrictions, the piles for these walls will have to be CIDH piles. The piles design criteria should be the same as that of the Girard Depressed Section, except that the ultimate pile vertical compression capacity should be twice the tension capacity stated in Table 3. Please note that the artesian conditions exist at these wall locations.

Construction Concerns

- Difficult CIDH installation should be anticipated. The anticipated difficulties have been addressed at each wall locations in the Recommendation Section of this report. Strict noise and vibration restriction are anticipated, this will likely preclude driven piles/sheet-piles at the site.

- Soil-mixing or grouting to construct the aquitard and/or cut-off walls can problematic under artesian pressure head that is above the ground surface. There is a possibility of continuous leakage of water at the grout/soil interface, which can result in voids due the migration of fines. To prevent this scenario, the head of the pressurized aquifer has to be reduced by at least 7 ft. A pumping test should be conducted (during the geotechnical investigation) to determine what how the artesian head can be reduce temporarily to facilitate the construction of the artificial aquitard and cut off walls. An additionally significant amount of water will likely be generated during the pumping test and dewatering confined aquifer to lower the artesian head during construction. Proper estimate of the water to be removed is needed.

- Historical building that are located close to the proposed retaining walls (especially Building 106) and those building that are located in the vicinity of Girard Depressed Section will require monitoring. This will include establishment of baseline for measurement prior to construction and periodic monitoring during construction. Specific items can include video survey, monitoring (survey) of control points for vertical and horizontal deformations, and crack monitors.

- Because of the granular (sandy) nature of the material at some locations, we anticipate zones of instability in the cut face for retaining wall during construction. Vertical micropiles or similar system may be needed to temporarily stabilize the cut face.

- Soft and unstable sub-grade conditions will likely be encountered and may require stabilization to operate construction equipment.

- Regulation pertaining to cross contamination of the aquifers during drilling operation has to be considered when constructing CIDH piles. Use of casing to isolate zones within the drilled hole may be required. Control of groundwater flow due to the artesian pressure will also be required.

Additional Field Work and Laboratory Testing

Our Office in cooperation with ARUP developed a program of subsurface investigation including boring and CPT soundings. Our additional recommendations for this program are:

1. Extend boring RW7-A1 to the depth of 82.5 ft (25 m) or at least 10 ft into the good bedrock which ever is lesser.
2. Extend boring RW11-R1, RW18-A1 to 50 ft (15 m).
3. Extend boring RW14-R1, RW17-A1, and RW15-R1 to 100 ft (30 m).
4. Convert borings RW14-A1, RW17-A1, ENB-R1, and RW15-R1 into multistage piezometers to monitor groundwater levels of different aquifers.

At Girard depression area, the upper part of 35 ft shall be continuously sampling samples for laboratory testing including grain size analysis and plasticity index. Consolidation and triaxial tests shall be conducted for retaining wall design. Moreover, pumping tests should also be conducted in this area to characterize hydrogeologic conditions for effective dewatering the site during construction of the Girard Depressed Section, cut and cover tunnels and bride footings.

The recommendations contained in this report are not final. A request for final recommendations should be made during final project design phase, and sent to the Office of Geotechnical Design – West. Final report can only be prepared after the general plans are furnished for each structure. Any questions regarding the above recommendations should be directed to the attention of Tung Nguyen, 510-622-1775 or Wajahat Nyaz, 510-622-1777 at the Office of Geotechnical Design-West, Branch C.

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