Los Angeles County
Metropolitan Transportation Authority

Westside Purple Line Extension Project, Section 2

Contract C1120

Geotechnical Design Memoranda (GDM)

Amendment 9: February 19, 2016
Amendment 3: November 2, 2015

U.S. Department of Transportation
Federal Transit Administration

Metro

List of Design Memoranda

CENTURY CITY CONSTELLATION STATION

SUMMARY OF REVISIONS TO THE NOVEMBER 2, 2015 GDM

Chapter/Figure/Table	Revisions	Page Nos.
cover page	Listed GDM Amendment dates	
	Revised sentences to state November 2015 GDM is superseded	
Table 2-1	Revised Ko values to present just the field pressuremeter test data	3
	Added footnote stating Ko is based on field pressuremeter test data	4
4.5	Revised dewatering/groundwater control section	12
4.7.2	Revised sentence to state parameters presented in Table 4-1 can be used for numerical modeling	13
Table 4-1	Revised Ko values to be used in preliminary analyses/design	17
	Revised footnote 9 to state the general methodology used to compute Ko; added footnote 9a and 9b explaining change in Ko with depth in San Pedro Formation	18
7.0	Added February 2016 GDR reference to Bibliography	30

MEMO

To	**Ms. Amanda Elioff, P.E.** **Parsons Brinckerhoff (PB)** **777 South Figueroa Street, Suite 1100** **Los Angeles, California 90017**
Date	**Amendment 3: November 2, 2015** **Amendment 9: February 19, 2016**
Project No	**4953-11-1423/19.02** **(PB Task 52.05.010.01.01.01)**
Subject	**Geotechnical Design Memorandum – Century City Constellation Station** Westside Purple Line Extension Los Angeles, California

This Geotechnical Design Memorandum (GDM) containing preliminary geotechnical design data has been prepared for the Century City Constellation Station planned within Section 2 of the proposed Westside Purple Line Extension (WPLE) project for the Los Angeles County Metropolitan Transportation Authority (Metro), as part of the Advanced Preliminary Engineering (Adv. PE) phase study.

The results of the Advanced Conceptual Engineering (ACE), Preliminary Engineering (PE) and Adv. PE phase geotechnical investigations performed for the station were presented in a draft Geotechnical Data Report (GDR), dated September 2015 **and subsequently superseded by February 19, 2016 version**. **This GDM supersedes** November 2, 2015 **GDM.**

This GDM presents preliminary geotechnical recommendations for foundation design, excavation support, station design, dewatering and groundwater control, and for earthwork. In case of any conflict between the interpretation of data or preliminary recommendations presented in this report and the Geotechnical Baseline Report (GBR), the data and preliminary recommendations presented in the GBR will prevail. The Design-Build (D-B) contractor should perform an independent evaluation of the data contained in the GDR and the preliminary recommendations presented herein and provide parameters for final design.

The geotechnical parameters presented in this report reflect the design team's judgment of anticipated subsurface conditions and ground behavior based on the construction means and methods anticipated. The design data presented herein were established considering available geologic and geotechnical data, together with past construction experience and anticipated construction methods in similar ground conditions. Development of the project design recommendations required interpretation of the data obtained from various sources, including: geologic maps; hollow stem auger, rotary, and core borings; other borings conducted for nearby projects; geophysical surveys; and in-situ and laboratory tests, as well as the consideration of information from previous construction projects completed in similar geologic conditions. While actual conditions encountered in the field are expected to be within

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the range of conditions discussed herein, the locations where specific ground and groundwater conditions are encountered may vary from those described in this report. In addition to the specific conditions described herein, the ground behavior will also depend on the construction sequence and methods employed, as well as the Contractor's equipment, experience and workmanship. The project design, therefore, assumes that the construction methods and level of workmanship will be consistent with those that can reasonably be expected from an experienced and qualified contractor.

It is our understanding that this GDM is being prepared for inclusion in the Request for Proposal Package being prepared for a Design-Build Contract for Section 2 of the proposed WPLE project.

It is a pleasure to be of continuing professional service to you. Please call if you have any questions or if we can be of further assistance.

Sincerely,

Amec Foster Wheeler Environment & Infrastructure, Inc.

Hari Ponnaboyina, P.E.
Senior Engineer

Martin B. Hudson, Ph.D., G.E.
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Reviewed by:

Perry A. Maljian, G.E.
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1.0 DESCRIPTION OF STATION

The Century City Constellation station will be constructed using cut and cover methods. An arched roof module is planned for the station design. The majority of the station excavation support will be internally braced with struts. However, tieback systems may be selected by the contractor for portions of the station excavation, and station entrances and appendages. Table 1-1 presents information pertaining to the location of the station and approximate dimensions of the station that were used to develop the geotechnical preliminary recommendations presented in this Geotechnical Design Memorandum (GDM).

Table 1-1: Station Location and Dimensions for Design

Station	Location	Length (feet)	Width (feet)	Depth to Top of Arch Roof, bgs (feet)	Depth to Top of Station Walls bgs (feet)	Depth to Bottom of Station Box bgs (feet)
Century City Constellation[1]	60 feet west of Century Park East to 100 feet west of Solar Way	1,215	60	30 to 40	40 to 50	85 to 90

1. Station entrance is planned in the parking lot located northeast of Constellation Blvd and Avenue of the Stars (10131 Constellation Boulevard) per plans dated September 2015

2.0 PRELIMINARY ENGINEERING PROPERTIES OF PRINCIPAL GEOLOGIC UNITS

The geologic units that will be encountered in the station excavation, from oldest to youngest, are the Pleistocene-age San Pedro Formation, Pleistocene-age Older Alluvium / Lakewood Formation, Holocene-age alluvium, and modern artificial fill. The Pliocene-age sedimentary rock of the Fernando Formation encountered in the eastern portion of the Westside Purple Line Extension (WPLE) is not anticipated in the station excavation.

Preliminary engineering properties were compiled in these principal geologic units and statistical analyses were performed to estimate the lower bound, upper bound and a recommended value for each property. The properties were evaluated by sub-dividing each geologic unit into fine-grained and coarse-grained portions before performing the statistical analysis. The engineering and index properties for the earth materials at the station are listed below:

- SPT Blow Counts
- Moisture Content
- Dry Density
- Total Density
- Fines Content
- Specific Gravity

- Liquid Limit and Plasticity Index
- Expansion/Collapse Potential
- Degree of Saturation
- Void Ratio
- Effective Cohesion and Friction Angle (peak strength values)
- Undrained Cohesion and Friction Angle (peak strength values)
- Elastic Parameters – Young's Modulus and Poisson's Ratio
- Hydraulic Conductivity
- Compression Index (C_c)
- Recompression Index (C_r)
- At-rest Lateral Earth Pressure Coefficient (K_o)
- Soil Abrasion
- Corrosion (Minimum Resistivity, pH, Chloride Content, Sulfate Content)

The estimated range (lower bound and upper bound) and a recommended value of the properties listed above for the Century City Constellation Station are presented in Table 2-1. The laboratory test results are presented in the Geotechnical Data Report (GDR) dated September 2015 for the Century City Constellation Station.

Table 2-1: Preliminary Engineering and Index Properties of Principal Geologic Units for Century City Constellation Station

Parameter	2009 through 2015 Geotechnical and Environmental Investigations Estimated Range of Engineering Parameters															
Geologic Unit	Artificial Fill (Af)				Quaternary Older Alluvium (Qalo)				Lakewood Formation (Qlw)				San Pedro Formation (Qsp)			
	Fine-Grained		Coarse-Grained		Fine-Grained		Coarse-Grained		Fine-Grained		Coarse-Grained		Fine-Grained		Coarse-Grained	
USCS Soil Classification	CL, CL-ML		SM, SC-SM, SC		CL, CH, ML		SM, SC-SM, GC, SC		CL, CH, ML		SW,SW-SM,SP,SP-SM,SM,SC		CL, CH, CL-ML, ML		SW-SM, SP, SP-SM, SM	
Engineering Properties	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]
SPT Blowcounts [2]	1 to 27	11	8 to 14	10	5 to 100	21	14 to 26	19	13 to 68	41	22 to 100	60	21 to 44	36	22 to 100	80
Moisture Content (%)	6 to 27	17	13 to 26	18	10 to 33	20	8 to 15	12	17 to 35	26	1 to 24	13	10 to 38	21	1 to 48	15
Dry Density (pcf)	93 to 118	109	108 to 118	112	87 to 119	107	109 to 120	114	111**	111**	92 to 120	105	82 to 116	104	68 to 119	101
Total Density (pcf)	117 to 134	129	129 to 136	132	115 to 135	128	124 to 134	128	130**	130**	97 to 136	117	113 to 133	125	97 to 136	116
Fines Content (%)	51 to 69	60	43**	43	48 to 97	73	35 to 48	42	51**	51**	9 to 49	24	55 to 94	73	5 to 46	21
Specific Gravity	2.63**	2.63**	*	*	*	*	*	*	*	*	2.69**	2.69	2.72 to 2.77	2.75	2.64 to 2.69	2.66
Liquid Limit (%)	30 to 40	37	37**	37**	27 to 71	45	33**	33**	34 to 94	64	26**	26**	36 to 75	50	*	*
Plasticity Index (%)	18 to 26	23	24**	24**	14 to 53	30	21**	21**	26 to 73	50	11**	11**	17 to 54	30	42**	42**
Expansion (%)	0.10 to 2.40	1.09	0.40**	0.40	2.65 to 7.04	5.29	*	*	*	*	0.00 to 0.65	0.17	0.77 to 2.51	1.64	0.00 to 0.99	0.20
Collapse (%)	*	*	*	*	*	*	*	*	*	*	0.00 to 0.25	0.08	*	*	0.00 to 1.38	0.41
Degree of Saturation (%) [#]	68 to 100	90	91 to 100	95	81 to 100	92	68 to 83	75	90**	90	15 to 89	52	73 to 97	89	16 to 97	64
Void Ratio	0.10 to 0.81	0.51	0.48 to 0.53	0.51	0.39 to 0.93	0.57	0.38 to 0.52	0.46	0.48**	0.48	0.39 to 0.84	0.60	0.40 to 1.07	0.64	0.38 to 1.44	0.67
Effective Friction Angle from DS Test [3] (degrees)	7 to 31	25	27 to 29	28	22 to 32	27	31**	31**	29**	29**	27 to 34	32	34**	34**	25 to 40	32
Effective Cohesion from DS Test [3] (psf)	300 to 1,180	622	500 to 550	525	0 to 1,600	988	270**	270**	600**	600**	100 to 850	324	600**	600**	0 to 1,500	369
Effective Friction Angle from Triaxial Test [4]	23 to 31	28	*	*	22 to 36	29	31**	31**	*	*	*	*	29**	29**	38**	38**
Effective Cohesion from Triaxial Test [4]	400 to 1,180	800	*	*	550 to 2,100	1,075	750**	750**	*	*	*	*	200**	200**	250**	250**
Undrained Friction Angle from Triaxial Test [4]	21 to 25	24	*	*	22 to 35	27	29**	29**	*	*	*	*	21**	21**	39**	39**
Undrained Cohesion from Triaxial Test [4]	220 to 1,100	543	*	*	200 to 1,400	867	100**	100**	*	*	*	*	200**	200**	450**	450**
Unconfined Compressive Strength (psi)	*															
Young's Modulus from Correlations [5] (ksf)	250 to 5,178	1,659	635 to 877	740	502 to 5,071	1,772	1,026 to 1,629	1,301	1,374 to 3,864	2,619	1,613 to 7,331	4,193	2,262 to 11,204	6,356	1,613 to 7,936	5,750
Youngs' Modulus from Triaxial Test [6] (ksf)	1,267 to 2,345	1,684	*	*	1,119 to 2,922	1,889	1,723**	1,723**	*	*	*	*	2,450**	2,450**	3,727**	3,727**
Poisson's Ratio	0.33 to 0.47	0.37	0.34 to 0.35	0.35	0.30 to 0.38	0.35	0.33 to 0.34	0.33	0.34**	0.34**	0.31 to 0.35	0.32	0.31 to 0.39	0.35	0.26 to 0.37	0.32
Hydraulic Conductivity (ft/day) [7]	10^{-7} to 10^{-1} ^	10^{-4} ^	10^{-2} to 10^{1} ^	10^{-1} ^	10^{-7} to 10^{-1} ^	10^{-4} ^	10^{-2} to 10^{1} ^	10^{-1} ^	10^{-7} to 10^{-1} ^	10^{-4} ^	10^{-2} to 10^{2} ^	10^{-1} ^	10^{-7} to 10^{-1} ^	10^{-4} ^	10^{-2} to 10^{2} ^	10^{1} ^
Compression Index (Cc)	0.10 to 0.25	0.17	0.22**	0.22**	0.17 to 0.25	0.20	*	*	*	*	0.05 to 0.17	0.09	0.10 to 0.17	0.14	0.06 to 0.14	0.09
Recompression Index (Cr)	0.01 to 0.04	0.02	0.03**	0.03**	0.04 to 0.06	0.05	*	*	*	*	0.01 to 0.02	0.01	0.00 to 0.03	0.02	0.00 to 0.02	0.01
At-rest Soil Pressure Coefficient., Ko	~~0.83 to 1.02~~*	~~0.90~~*	~~0.94**~~*	~~0.94**~~*	0.~~73**~~**59 to 0.69**	0.~~73**~~**64**	~~*~~**0.91****	~~*~~**0.91****	*	*	~~0.62**~~*	~~0.62**~~*	~~0.50**~~*	~~0.50**~~*	0.4~~7~~**66** to 0.~~60~~**71**	0.~~52~~**68**
Soil Abrasion Test Value	*	*	*	*	*	*	*	*	*	*	44**	44**	*	*	27 to 41	33

Table 2-1: Preliminary Engineering Properties of Principal Geologic Units for Century City Constellation Station (Continued)

Parameter	2009 through 2015 Geotechnical and Environmental Investigations Estimated Range of Engineering Parameters															
Geologic Unit	**Artificial Fill (Af)**				**Quaternary Older Alluvium (Qalo)**				**Lakewood Formation (Qlw)**				**San Pedro Formation (Qsp)**			
	Fine-Grained		**Coarse-Grained**		**Fine-Grained**		**Coarse-Grained**		**Fine-Grained**		**Coarse-Grained**		**Fine-Grained**		**Coarse-Grained**	
USCS Soil Classification	**CL, CL-ML**		**SM, SC-SM, SC**		**CL, CH, ML**		**SM, SC-SM, GC, SC**		**CL, CH, ML**		**SW,SW-SM,SP,SP-SM,SM,SC**		**CL, CH, CL-ML, ML**		**SW-SM, SP, SP-SM, SM**	
Engineering Properties	**Range[1]**	**Recommended[1]**	**Range[1]**	**Recommended[1]**	**Range[1]**	**Recommended[1]**	**Range[1]**	**Recommended[1]**	**Range[1]**	**Recommended[1]**	**Range[1]**	**Recommended[1]**	**Range[1]**	**Recommended[1]**	**Range[1]**	**Recommended[1]**
Corrosivity Results [8]:																
Minimum Resistivity (ohm-cm)	806 to 1,680	806	1,838**	1,838**	840 to 1223	840	*	*	*	*	1,936**	1,936**	640**	640**	774 to 5,020	774
pH	7.4 to 8.2	7.4	7.5**	7.5**	7.2 to 7.9	7.2	*	*	*	*	7.6**	7.6**	7.7**	7.7**	5.8 to 8.2	5.8
Chloride Content (ppm or mg/kg)	16 to 672	672	136**	136**	25 to 188	188	*	*	*	*	128**	128**	968**	968**	39 to 3,823	3,823
Sulfate Content (ppm or mg/kg)	4 to 289	289	71**	71**	21 to 357	357	*	*	*	*	251**	251**	45**	45**	13 to 108	108

* No test data
** Limited data
^ No test data; reported values are based on published data in literature, and/or based on prior experience
# Estimated using assumed specific gravity of 2.65 when a specific gravity test was not performed
pcf = pounds per cubic foot; psf = pounds per square foot; psi = pounds per square inch; cm = centimeter; ppm = parts per million; mg = milligrams; kg = kilograms; DS = direct shear

Notes:
1. Data presented here are based on ACE, PE and Adv. PE phase explorations as well as applicable prior explorations as discussed in the GDR
2. Blow counts from ACE and PE phase environmental hollow-stem-auger borings were not considered
3. Effective cohesion and friction angle are based on peak strength values from slow direct shear tests.
4. Cohesion and friction angle are based on peak shear strength values from Triaxial consolidated-undrained tests. Effective values are based on effective stress and undrained values are based on total stress
5. Based on relationship between elastic modulus and SPT N-value from Stroud (1989) and Duncan Bursey (2007)
6. Based on secant modulus computed at 0.1+0.05% axial strain from Triaxial consolidated-undrained tests
7. Hydraulic conductivity values were based on published data (Department of Water Resources Bulletin 118, California's Groundwater Update, 2003) and site-specific pumping test data
8. For soil corrosivity, the recommended values correspond to minimum resistivity, lowest pH and highest values for chloride and sulfate content
9. Based on field pressuremeter test data

3.0 DYNAMIC SITE CHARACTERISTICS

3.1 Response Spectra

Response spectra were developed for the underground station and the at-grade ancillary structures in accordance with Sections 2.3.1 through 2.3.3 of the Metro Supplemental Seismic Design Criteria of the Metro Rail Design Criteria (MRDC) dated November 2014. Two hazard levels were evaluated as listed below, based on a design life of 100 years for structures per the Metro Seismic Design Criteria of the MRDC.

- Operating Design Earthquake (ODE) – defined as an earthquake event likely to occur only once in the design life, where structures are designed to respond without significant structural damage. The current Metro Code defines ODE as an event with a 50% probability of exceedence in 100 years (corresponding to a return period of 150 years).
- Maximum Design Earthquake (MDE) – defined as an earthquake event with a low probability of occurring in the design life, where structures are designed to respond with repairable damage and to maintain life safety. The current Metro Code defines MDE as an event with a 4% probability of exceedence in 100 years (corresponding to a return period of 2,475 years)

The response spectra for the ODE and MDE events were estimated using the 2008 USGS Interactive Probabilistic Seismic Hazard Analysis (PSHA) Deaggregation tool on the USGS website (USGS, 2011). The USGS deaggregation tool uses the Next Generation Attenuation (NGA) relationships of Boore-Atkinson (2008), Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) for the ground motion prediction equations. Based on the available combinations of exceedence probability and exposure time, ground motions for the MDE event were computed for a probability of exceedence of 2% in 50 years (equivalent to the 4% in 100 year criteria, as stated in the Metro Seismic Design Criteria of the MRDC).

Site-specific seismic shear wave data of the subsurface soils was available from p-and s-wave suspension logging data obtained in soil borings G-413 and G-415 drilled at the station location and from nearby boring T3-B3 performed as part of the prior fault investigation for the WPLE project. In addition, suspension logging data was also available from a prior boring B-2 performed for the proposed expansion of the Century Plaza Hotel. The shear-wave data was compiled from all the borings and an idealized average velocity profile was estimated and then average shear wave velocity within 100 feet or 30 meters (referred to as $V_{s,30}$) of the station bottom and the ground surface for the ancillary structures was computed.

The spectral ordinates of the 5% damped response spectra for both ODE and MDE events, for station and ancillary structures, are presented in Table 3-1. For $V_{s,30}$ of 1,060 and 1,210 feet per second (for the earth materials beneath foundations of both the ancillary structures and station box, respectively), a Site Class D may be used for seismic design.

Table 3-1: Acceleration Response Spectra for Century City Constellation Station and Ancillary Structures

Earthquake Level	Latitude (degrees)	Longitude (degrees)	Geologic Formations	$V_{s,30}$ (feet/second)	Period (sec):	5% Spectral Accelerations (Sa, in g)						
						0.01	0.10	0.20	0.30	0.50	1.00	2.00
ODE	34.0592	-118.4152	Artificial Fill/ Younger/Older Alluvium and San Pedro	1,060 to 1,210	Sa (g):	0.28	0.50	0.64	0.61	0.49	0.29	0.14
MDE					Sa (g):	0.88	1.58	1.97	1.96	1.79	1.15	0.55

3.2 Time Histories and Spectral Matching

In order to evaluate the free-field displacement for racking analysis of the station per Section 3B8.0 of the Metro Supplemental Seismic Design Criteria of the MRDC, it was required to perform a site-specific site response analysis. For this purpose, spectrum-compatible time histories were developed using the 5% damped ODE and MDE response spectra. The target response spectra for the ODE and MDE events were computed for a stiff soil type site condition with an average velocity, ($V_{s,30}$) of about 1,500 feet per second (460 meters per second) assumed to be at a depth of about 150 feet bgs. The target response spectra is presented in Table 3-2 below.

Table 3-2: Target Acceleration Response Spectra

Earthquake Level	Latitude (degrees)	Longitude (degrees)	Formations	$V_{s,30}$ (feet/second)	Period (sec):	5% Spectral Accelerations (Sa, in g)						
						0.01	0.10	0.20	0.30	0.50	1.00	2.00
ODE	34.0592	-118.4152	San Pedro	1,500	Sa (g):	0.27	0.51	0.63	0.59	0.46	0.26	0.12
MDE					Sa (g):	0.89	1.68	2.09	2.02	1.80	1.02	0.46

Recorded seed time histories from prior earthquakes are generally selected so that the earthquake magnitudes, source-to-site distance, fault mechanism and subsurface site conditions of the recording stations are similar to that of the site conditions and the earthquakes anticipated at the station. Three time histories were obtained from the Pacific Earthquake Engineering Research (PEER) Center, Next Generation Attenuation (NGA) ground motion database for shallow crustal earthquakes in active tectonic regimes. The time histories were selected based on the following factors: geologic and soil characteristics, inclusion of strong directivity or near-source ground motions, and the results of de-aggregation of probabilistic seismic hazard.

Based on the de-aggregation results, the controlling earthquakes for the ODE event have a Magnitude range of 6.6 to 7.7 at a distance of 1.2 to 18.5 miles, and for the MDE event, a Magnitude range of 6.5 to 7.5 at a distance of 1.2 to 3.1 miles. It is noted that the Hollywood (Mw = 6.7), Santa Monica (Mw = 6.6 to 6.8) and Raymond (Mw = 6.8) Faults are relatively close to the station; these faults primarily have strike-slip fault mechanisms (with some component of reverse mechanism). The active trace of the Santa Monica Fault is about 0.3 mile to the northwest of the station. In addition, the northern extension of the Newport Inglewood fault is located about 0.25 mile east-northeast of the station.

The three time histories selected for spectral matching are presented in Table 3-3. Also listed in the table are the site conditions, fault mechanisms and source-to-site distances for each of the recording stations.

Table 3-3: Seed Time Histories

Time History Designation Name	Earthquake	Magnitude (Mw)	Fault Mechanism	Recording Station	Closest Distance to Fault (km)	Recording Component (degrees)*	Recording Station Vs,30 (feet/second)
NGA_752-000	1989 Loma Prieta	6.93	Reverse-Oblique	Capitola	8.65	000	947
NGA_983_292	1994 Northridge	6.69	Reverse	USGS 655 Jensen Filter Plant Generator	5.43	292	1726
NGA_1111_090	1995 Kobe	6.90	Strike-Slip	CUE 99999 Nishi-Akashi	7.08	090	1998
* Indicates orientation of recording station instrument; Reference directions: 000 – North South Direction, 090 – East West Direction, 292 – 292 degrees measured counterclockwise with respect to North							

Spectral matching of the seed time histories was performed in the time domain by adding or subtracting wavelets of limited duration to the original time history using the algorithms from program RSPMATCH (Abrahamson, 1998) incorporated into EZ-FRISK.

3.3 Site Response Analysis and Free-Field Differential Displacement

As stated in Section 3B8.1.3 of the Metro Supplemental Seismic Design Criteria of the MRDC, the racking analyses of the station can be evaluated using any of the following methods depending on the level of assessment required to be used for the particular structural configuration being analyzed.

- Simplified Pseudo-Static Method
- Simplified Dynamic Method
- Two Dimensional Dynamic Method
- Fully Coupled Two Dimensional Finite Element or Finite Difference Method

For the purpose of preliminary design, a simplified pseudo-static method is utilized herein wherein a free-field displacement of the top of the station relative to the station bottom is obtained and a pushover analysis is performed using the displacement profile to determine structural response under seismic loading. Other methods as listed above are also applicable for some types of structural configuration and may be able to be utilized.

In order to obtain the free-field displacement (profile), site response analyses were performed using the one-dimensional equivalent linear program SHAKE91 (Idriss and Sun, 1992) to estimate free-field displacement of the soil column between the top and bottom of the station. Site response analyses were performed using ground motions for ODE and MDE events following the Metro Seismic Design Criteria of the MRDC for design.

Seismic shear wave velocities of the subsurface soils available from suspension logging in drilled boreholes were used in developing the one-dimensional soil profile and dynamic properties of different soil layers. A 150-foot thick soil column was considered in the one-dimensional model. Based on available geologic information, the shear-wave velocity was determined to be about 1,500 feet per second (460 meters per second) for the transmitting base at a depth of 150 feet bgs.

Three appropriate spectrum-matched acceleration time histories, as discussed in Section 3.2, were used as the input outcropping motions in the model. The time histories of shear strains were obtained in each of the layers within the zone of interest (between the top and bottom of the box) for the station racking and then the displacement time histories were computed in these soil layers. The equivalent displacement time history within the zone of interest was estimated by adding displacement time histories in each of these layers. The peak value in the displacement time history was taken as the estimated peak free-field displacement between the top and bottom of the station. The free-field maximum differential displacements computed for the ODE and MDE events in this manner are presented in Table 3-4. The values in the table may be linearly extrapolated for other structure heights.

Table 3-4: Free-Field Displacement (ODE and MDE)

Station Name	Free-Field Displacement (inches)	
	ODE	MDE
Century City Constellation	0.14 inch in 50 feet	1.2 inch in 50 feet

3.3.1 Lateral Spring Stiffness

Lateral and vertical soil spring stiffnesses were estimated for use in two-dimensional and/or three-dimensional structural analysis of the station. The lateral springs can be used to model the stiffness of the soils retained by the below-grade walls of the station, both in the longitudinal and transverse directions. The vertical springs can be used to model the soils supporting the station foundations. The procedure to perform a numerical deformation analysis using the lateral and vertical springs is presented in Section 3B8.1.3 of the Metro Supplemental Seismic Design Criteria of the MRDC.

The lateral stiffnesses were estimated using equations published in FEMA 356/ASCE 41 for foundations in translation in the x- and y-directions. The method uses the shear wave velocity, shear modulus, Poisson's ratio of the surrounding medium and station dimensions (length, width and height) in estimating the spring stiffnesses.

For the ODE and MDE events, the ratio of degraded to small-strain shear wave velocity was estimated as 0.81 and 0.65, respectively, using the guidelines provided in Table 19.2-1 of FEMA P- 750 (2009). For computation of degraded shear modulus, these ratios will be 0.66 and 0.42, for ODE and MDE, respectively using the relationship between the shear wave velocity and shear modulus. Based on the small-strain shear wave velocity of the soils adjacent to the station, a Site Class D was used in computing the degradation factor. Due to the variation of shear wave velocity with depth and along the length of the station, a range (lower-bound and upper-bound) and a best estimate of the shear wave velocity were used in spring stiffness computations. The estimated average lateral spring stiffness for small-strain, ODE, and MDE events are presented in Table 3-5. Depending on the direction of the structural analysis performed (longitudinal or transverse), the spring stiffness in kips per cubic foot (kcf) may be multiplied by the station side area to estimate the spring stiffness in units of kips per foot (kip/ft).

The variable “z” in Table 3-5 is depth below ground surface. Spring stiffness above the top of the station walls should be ignored. The x-axis and y-axis are oriented in the length and width directions of the station, respectively; Kx and Ky represent springs in the x- and y-directions. For the computations presented herein, the length and width of the station presented in Table 1-1 were used. The approximate top of the station wall for the arch roof module is also presented in Table 1-1. It is noted that the spring stiffness presented in the above table will need to be revised if the station dimensions change.

Table 3-5: Lateral Soil Spring Stiffness (Small Strain, ODE, and MDE)

Value	Small Strain	ODE	MDE	Small Strain	ODE	MDE
	Kx (kcf)[a]	Kx (kcf)[a]	Kx (kcf)[a]	Ky (kcf)[b]	Ky (kcf)[b]	Ky (kcf)[a]
Average	15z	11z	3z	1z	0.7z	0.2z
Upper Bound	50z	35z	9z	3.3z	2.3z	0.6z
Lower Bound	6z	4z	1z	0.4z	0.3z	0.1z

[a] Distribute over area of end wall (width * height), unit stiffness increases with depth (z, depth below ground surface in feet)
[b] Distribute over area of side wall (length * height), unit stiffness increases with depth (z, depth below ground surface in feet)

As stated earlier, the ratio of degraded modulus to small strain modulus is 0.66 (about 34% degradation from the small-strain value) is estimated for the ODE event, therefore it is suggested that the spring stiffness values estimated for the ODE event be used for the static condition as well. If the MDE case is analyzed, the respective spring stiffness values presented in Table 3-5 should be used.

The dynamic spring stiffnesses provided herein are based on shear wave data which are not affected by the presence of groundwater, except possibly by pore pressures generated in soil during earthquakes. Dynamic spring stiffnesses provided herein are applicable for both current and design (historically-highest) groundwater conditions. Furthermore, liquefaction hazard at the station location is considered low and therefore, the dynamic lateral spring stiffnesses presented in Table 3-5 are not anticipated to be reduced any further.

3.3.2 Vertical Spring Stiffness

Vertical stiffness was computed using equations published in FEMA 356/ASCE 41 for translation in the z-direction. Soil shear wave velocities below the station bottom were used in estimating the vertical stiffness. Due to the variation of shear wave velocity with depth below the station foundation and along the length of the station, a range (lower-bound and upper-bound) and a best estimate of the shear wave velocity were used in spring stiffness computations.

Table 3-6: Vertical Spring Stiffness (Small Strain, ODE and MDE)

Value	Small Strain kz (kcf)	ODE kz (kcf)	MDE kz (kcf)
Average	235	165	40
Lower Bound	95	70	20
Upper Bound	785	545	140

For the ODE and MDE events, the ratio of degraded to small-strain shear wave velocity was estimated as 0.81 and 0.65, respectively, using the guidelines provided in Table 19.2-1 of FEMA P- 750 (2009). Based on the small-strain shear wave velocity of the soils, a Site Class D was used in computing the degradation factor.

Table 3-6 provides lower bound, upper bound and average vertical stiffnesses for small strain, ODE and MDE events. The spring stiffness in kips per cubic feet (kcf) may be multiplied by the mat foundation area to estimate the spring stiffness in units of kips per foot (kip/ft). For the computations presented herein, the length and width of the station foundation stated in Table 1-1 were used. It is noted that the spring stiffness presented in the above table will be need to revised, if the station dimensions change.

4.0 DESIGN AND CONSTRUCTION

This section provides a summary of the geotechnical evaluation of the subsurface conditions at the station site and their impact on the design and construction of the proposed station and ancillary structures.

4.1 Geotechnical Considerations

Based on the plan and profile dated September 2015, the station will be excavated within artificial fill, Younger alluvium and Quaternary-age-Older alluvium and Lakewood Formation, and the Pleistocene-age San Pedro Formation. The fill placed in the early 1960s extends to depths of up to about 30 feet in the western part of the station excavation. There is a potential for the presence of oversize material in the fill, (i.e., boulders). The natural soils in the formations predominantly consist of medium stiff to stiff clays and silts interlayered with medium dense to dense silty sands and sands. Although local channels with abundant gravels and occasional cobbles may be present, boulders were not encountered in the borings drilled at the station location. Excavation in these soils can be performed using conventional earth-moving equipment. Concretionary deposits and strongly cemented layers in the San Pedro Formation, if encountered, could require the use of jackhammers or hoe-rams to break through. The ground conditions anticipated in the station excavation are quantitatively described in the Geotechnical Baseline Report (GBR).

Although groundwater appears to be at depth, there may be potential for caving of sands in the installation of soldier piles and tieback anchors.

Certain challenges will have to be addressed in conjunction with station excavation, such as the presence of major utility lines crossing the station footprint and the presence of existing tieback anchors from former basement construction that protrude into the planned excavation. Based on the project-specific utility maps, a number of existing utilities such as storm drains, sewers, electrical conduits, and telecommunication lines are located within the upper 10 to 30 feet of ground surface. All utilities will have to be carefully protected in place or relocated where possible. Tieback anchors from adjacent building constructions could extend through the entire width of the station (e.g., tieback anchors from the Century Plaza Theme Towers could extend from the south side to the north side of the station). Some of the tieback anchors used for some of deeper basements have a bell with a larger diameter in the bonded zone of the anchor (near the tip of the anchor).

4.2 Groundwater Levels

Groundwater was not encountered in shallower screened [45 to 80 feet below ground surface (bgs)] PE and Adv. PE phase monitoring wells at the station site. In the deeper screen intervals [80 to 100 feet bgs], groundwater was encountered at depths of about 81 to 84 feet bgs. In addition, water seepage between depths of about 30 to 50 feet bgs was encountered in borings drilled at the station.

4.2.1 Design Groundwater Level

A groundwater-level contour map of the Beverly Hills Quadrangle Seismic Hazard Report (CGS, 1998) shows the historically highest groundwater level is at approximately 30 to 40 feet bgs at the station location. In addition, as stated earlier, water seepage as shallow as 30 feet bgs was encountered in the borings. Therefore, the design groundwater level should be taken as 30 feet bgs for permanent station design.

4.3 Seismic Design Considerations

According to the California Geological Survey (CGS, 1998), the Century City Constellation Station site is not within an area identified as having a potential for liquefaction due to the presence of older alluvial deposits (Pleistocene age) beneath the site. The older alluvium is comprised primarily of stiff to hard, clay rich sediments with lenses and subordinate layers of dense, silty sands. The older alluvium is underlain by dense sands with some interbedded, very stiff to hard clays of the Lakewood and San Pedro Formations. These deposits are not susceptible to liquefaction. Therefore, the potential for liquefaction at the station site is considered to be low.

Although artificial fill soils were encountered in the upper 20 to 35 feet along the western portion of the station, the fill soils were primarily fine grained and not susceptible to liquefaction. Furthermore, these soils are above the historically highest ground water level and are not anticipated to be saturated during the station design life. Therefore, the potential for liquefaction at the station is considered to be low.

The estimated PGA and Peak Ground Velocity (PGV) for ODE and MDE events for the station are presented in Table 4-1. The estimated free-field displacements over the station height for use in racking analysis are presented in Table 3-6.

4.4 Excavation Methods

Excavations as deep as 90 feet will be required for the station construction. Due to proximity of station excavation to existing buildings and limited construction space within the public right-of-way, shoring will be required. Shoring systems such as soldier piles with wood or shotcrete lagging, secant piles, and slurry walls supported by tieback anchors and/or internal bracing with struts and walers may be used. Based on traditional methods used in the area, the station excavation support is assumed to be internally braced with struts.

4.5 Dewatering and Groundwater Control

Groundwater levels were measured at depths of about 80 feet bgs or deeper in the monitoring wells; water seepage at depths of about 29 to 50 feet bgs were also noted in the borings and wells. The proposed excavation will thus extend about 5 to 10 feet below the current water levels.

The primary source of water into the excavation is expected at a depth of about 80 feet bgs and below. In addition, minor water seepage in the upper 50 feet should be anticipated. Amec Foster Wheeler's predecessor firm was with the geotechnical engineer for the approximately 100-foot deep excavation for the Century Plaza Theme Towers located immediately south of the station. During the excavation, water inflows into the excavation were rather small; dewatering wells and/or gravel-filled trenches and sump pumps were not utilized for groundwater control and were not found to be needed during excavation. Figures 5-1 and 5-2 are photographs of the Century Plaza Tower excavation during construction. Therefore, it is anticipated that water inflows in the station excavation will be small relative to station excavations in shallower groundwater conditions.

Based on the prior experience, a ~~peak~~ water inflow rate ~~on the order~~ of **approximately** 50 gallons per minute (gpm) is suggested for use in the preliminary design for sizing of sump pumps and drain pipes. ~~The steady-state water inflow is expected to be lower than 50 gpm.~~ The water inflow rate was estimated assuming the current groundwater level at 80 feet bgs. A qualified dewatering contractor with experience in similar subsurface conditions should be consulted for final design.

4.6 Ground Heave and Basal Stability

The station site should be dewatered to maintain the groundwater level at least 5 feet below the excavation bottom to achieve a factor of safety (defined as the ratio of critical hydraulic gradient to maximum flow exit gradient) of 2.0 against basal stability. The soils at the excavation level are predominantly granular and are not expected to heave significantly upon excavation.

Amec Foster Wheeler's predecessor firms have served as geotechnical engineer of record on projects with some of the deepest excavations in the Los Angeles; most notably for the excavation of Century Plaza Towers located south the proposed station. Within the excavation, ground heave of about 8 inches was measured, although reportedly only in 4% of the slab area resulting from about 100-foot excavation. Refer to Figures 5-1 and 5-2 for photographs of the Century Plaza Tower excavation during construction.

4.7 Excavation Support

4.7.1 Geotechnical Design Parameters

The following sections provide general preliminary recommendations for the design and construction of shoring braced with internal struts/walers for the station and cantilever shoring walls for ancillary structures.

4.7.2 Lateral Earth Pressures

Shoring up to 90 feet deep will be required for support of station excavation. For ancillary structures such as station entrances, shoring as deep as 70 to 80 feet adjacent to the main station to 10 to 20-foot deep for shallower portions of the ancillary structures will also be required.

For design of braced shoring system (internally braced with struts or tiebacks or both), the use of a trapezoidal distribution of earth pressure is recommended. If a slurry wall or other undrained shoring wall system is used, design should be based on at-rest earth pressures recommended for rigid shoring with a trapezoidal earth pressure distribution. In additional to the earth pressure distribution, hydrostatic pressure as described in Section 4.7.3 and surcharge loading as described in Section 4.7.4

should be applied. Consideration of appropriate surcharge pressure is important considering the proximity of the anticipated excavation to adjacent heavily-loaded structures.

For excavations up to 15 feet or less anticipated for ancillary structures, the use of cantilever shoring may be an economical option. For design of cantilever shoring, a triangular distribution of lateral earth pressure may be used.

The geotechnical parameters and earth pressure coefficients required for design of shoring **and numerical modeling** are presented in Table 4-1. Recommended earth pressure distributions for design of shoring are shown in Figure 4-1.

The earth pressure distributions presented in Figure 4-1 assume a level backfill. If the ground surface retained by shoring is sloped, an increase in earth pressure will need to be evaluated on a case-by-case basis.

4.7.3 Hydrostatic Pressures

Permeable shoring systems such as soldier pile and lagging systems need not be designed for hydrostatic pressures. If shotcrete is used instead of wood lagging, weep holes should be placed to provide drainage of the retained soils. If weep holes are provided, hydrostatic pressures need not be considered in the design. If water-tight shoring system, such as secant/tangent piles or a slurry wall shoring system is used, hydrostatic pressures as shown in Figure 4-1, should be added to the lateral earth pressures described above. The design groundwater level should be taken as 30 feet bgs.

4.7.4 Surcharge Pressures

Shoring should be designed to resist a uniform lateral pressure of 100 pounds per square foot due to American Association of State Highway and Transportation Officials (AASHTO) HS20 traffic loading. Applicable surcharge pressures from adjacent buildings and foundations of minor structures should be estimated and added to the earth pressures. Consideration of appropriate surcharge pressure is important considering the proximity of the anticipated excavation to adjacent heavily-loaded structures. Surcharge pressures from heavily loaded construction cranes and other traffic should be added as well. Surcharge pressures should be estimated in accordance with recommendations presented in Metro Rail Standard structural Drawings (Metro, 2014)

4.7.5 Seismic Earth Pressures

The Metro Seismic Criteria does not provide specific recommendation for computing seismic earth pressures for temporary shoring, but Metro standard drawing SS-003 presents a guideline for seismic earth pressure due to retained soil and due to adjoining building(s). Considering that the shoring will be in-place for more than 2 to 3 years, it is recommended that seismic earth pressure due to retained soil be used for the full height of the shoring. Seismic earth pressure due to adjoining buildings should be in accordance with Metro standard drawing SS-003.

The increment of seismic lateral earth pressures due to retained soil should be computed using the seismic earth pressure coefficient increment (K_{AE}) provided in Table 4-1 which was based on PGA for the ODE event. A seismic coefficient (k_h) equal to 1/2 of the computed PGA was used in the computations. The equivalent uniform earth pressure may be computed by taking the same resultant force as computed from the triangular equivalent fluid pressure distribution, as shown in Figure 4-1. Using these design parameters, a seismic earth pressure increment is estimated to be equal to 6H pounds per square foot (uniform), where 'H' is the shoring height.

For shoring, this seismic earth pressure is an incremental value intended to be added to the static active earth pressure value, and not the design shoring lateral earth pressure (K) or the at-rest earth pressure (K_o). All required load combinations of static and seismic, with appropriate load factors (NCHRP, 2009), are to be utilized in the final design of shoring walls.

4.7.6 Design of Soldier Piles

For the design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils above groundwater level, may be assumed to be 550 pounds per square foot per foot of depth (pcf) below the excavated surface, up to a maximum of 5,500 pounds per square foot. For soils below groundwater level, the allowable lateral value (passive value) of the soils below the level of excavation may be assumed to be 250 pounds per square foot per foot of depth at the excavated surface, up to a maximum of 2,500 pounds per square foot. The passive values include a multiplication factor of 1.5 as recommended by Metro to account for the three-dimensional effects of the passive wedge. A one-third increase in the lateral bearing value may be used when considering seismic and other transient loads for ancillary structures.

To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile, which is below the planned excavated level, should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils.

Provided that the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. For resisting downward loads, the frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken equal to at least 600 pounds per square foot for short embedment of the toe of soldier beams; for longer soldier beam embedment, the friction may be computed using an appropriate pile capacity computation, assuming that the ground is saturated below the base of excavation, and using a surcharge pressure appropriate to the stress conditions at the edge of the station excavation.

4.7.7 Lagging

The soldier piles, struts, and anchors should be designed for the full anticipated lateral pressure. Continuous lagging will be required between the soldier piles. The lagging should be designed in accordance with the drawing for Cut and Cover Underground Structures, titled "Construction Structures Loads and Design Criteria." If shotcrete is used, weep holes should be provided to relieve hydrostatic pressures.

4.7.8 Anchor Design

Installing tieback anchors in the project area will likely require permission from local agencies and owners of adjacent properties and avoidance of underground obstruction such as basements, foundations, and utility lines. Tieback anchor in the public right-of-way will require removal in accordance with local jurisdiction requirements.

Tieback friction anchors may be used to resist lateral loads. For computation purposes, it may be assumed that the unbonded zone adjacent to the shoring is defined by a plane drawn at 30 degrees with the vertical through the bottom of the excavation (as shown on Metro Rail Structural Standard Cut and Cover Underground Structures Drawing No. SS-004). The anchors should extend at least 25 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

The capacities of anchors should be determined by testing of the initial anchors as outlined on Metro Rail Structural Standard Cut & Cover Underground Structures Drawing No. SS-004. Pressure-grouted anchors in the upper 40 feet will develop an average friction value of 1,500 pounds per square foot and a friction value of 2,000 pounds per square foot for anchors below this depth. For anchors 8 inches in diameter, this value corresponds to a bond strength of about 3,000 pounds per lineal foot for anchors in the upper 40 feet and 4,200 pounds per lineal foot for anchors below this depth. Non-pressure-grouted anchors in the upper 40 feet will develop an average friction value of 475 pounds per square foot and a friction value of 650 pounds per square foot for anchors below this depth.

It is noted that the friction values provided above are based on the use of conventional tieback installation equipment. Some reduction of frictional resistance should be anticipated if undue remolding of the soils is caused by contractor’s choice of installation method, particularly in clayey type soils below groundwater.

Existing tieback anchors left (but no longer relied-upon) from construction of the adjacent buildings will protrude into the station excavation. The tieback anchors are likely de-tensioned with the anchor head embedded in the basement wall. Care should be taken to not apply excessive pressure (pull) on the anchor during its abandonment. The tieback anchors should be cut using welding torches, or can be cut using a core barrel (within a soldier pile excavation) within the limits of the station and entrance structure.

4.7.9 Internal Bracing

Internal struts and walers may be used to internally brace the soldier piles. The strut loads should be determined based on the lateral earth pressures for braced condition as shown in Figure 4-1. The vertical spacing between the struts should be designed to reduce ground movements. All struts should be tightly fitted to eliminate any slack and to reduce ground movement. If necessary to reduce shoring deflection, a preload of 25% of the design load may be used.

Procedures to compensate for the effects of temperature changes on the strut loads should be developed and implemented so that proper strut load levels can be monitored and maintained during construction.

4.7.10 Shoring Deflection and Ground Settlement

The amount of horizontal deflection of a shored embankment is dependent on the flexibility of the shored wall, excavation methods, and spacing of support members such as soldier piles, struts, etc. It should be realized, however, that some deflection will occur. Horizontal deflection of braced shoring is highly dependent on strut spacing, strut stiffness, and strut preloading, and soldier pile spacing and stiffness. Other mitigations for shoring deflection that can be considered include the use of grouting across the width of the excavation at selected depth intervals (such as jet grouting performed from one side of the excavation to the other in selected portions of the excavation).

If greater deflection than anticipated occurs during construction, additional bracing may be considered to minimize settlement of the adjacent buildings and utilities in the adjacent streets. If it is desired to reduce the deflection of the shoring, a greater earth pressure could be used in the shoring design, such as a design based on an at-rest pressure condition. In any event, the design for shoring will need to meet the deflection criteria specified for the project by Metro. Refer to the Building Protection Report for the project.

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Table 4-1: Preliminary Geotechnical Design Parameters for Century City Constellation Station

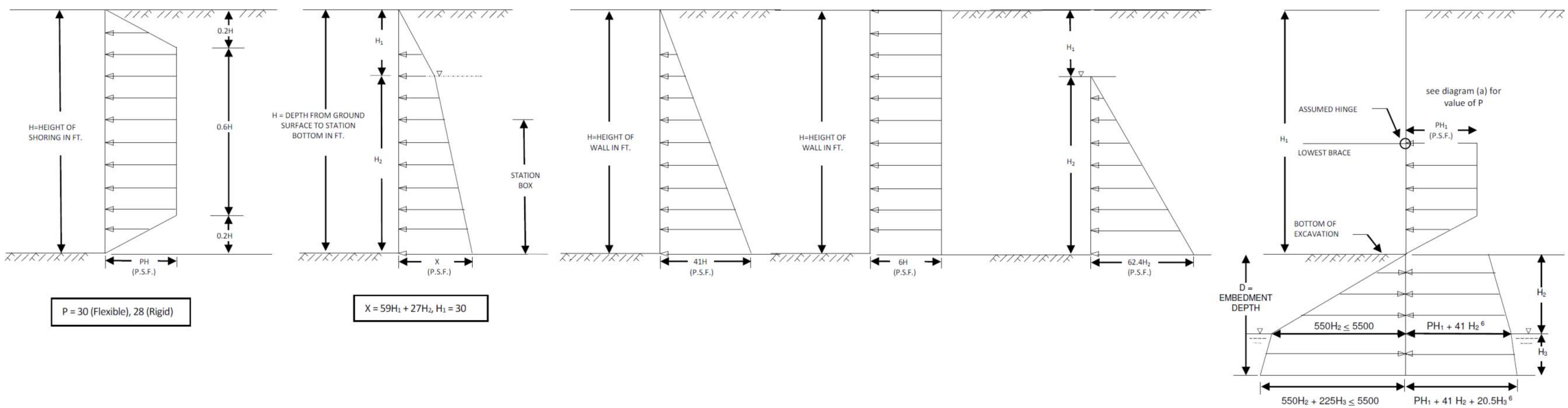
Parameter	Design Value	2009 through 2015 Geotechnical and Environmental Investigations Estimated Range of Engineering Parameters [21]							
Geologic Unit		Artificial Fill (Af)		Quaternary Older Alluvium (Qalo)		Lakewood Formation (Qlw)		San Pedro Formation (Qsp)	
		Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained
Dry Unit Weight of Soil (pcf) [1]	106	93 to 118 (109)	108 to 118 (112)	87 to 119 (107)	109 to 120 (114)	111**	92 to 120 (105)	82 to 116 (104)	68 to 119 (101)
Total Unit Weight of Soil (pcf)	122	117 to 134 (129)	129 to 136 (132)	115 to 135 (128)	124 to 134 (128)	130**	97 to 136 (117)	113 to 133 (125)	97 to 136 (116)
Static Elastic Modulus from SPT Correlation (ksf) [2a]	Varies	250 to 5,178 (1,659)	635 to 877 (740)	502 to 5,071 (1,772)	1,026 to 1,629 (1,301)	1,374 to 3,864 (2,619)	1,613 to 7,331 (4,193)	2,262 to 11,204 (6,356)	1,613 to 7,936 (5,750)
Static Elastic Modulus from Triaxial (ksf) [2b]	Varies	1,267 to 2,345 (1,684)	*	1,119 to 2,922 (1,889)	1,723 to 1,723 (1,723)	*	*	2,450 to 2,450 (2,450)	3,727 – 3,727 (3,727)
Friction Angle (Degrees) [3]									
Field	Varies	*	*	23**	31**	*	*	*	37**
Saturated	Varies	7 to 31 (26)	27 to 29 (28)	22 to 36 (28)	31**	29**	27 to 34 (32)	29 to 34 (32)	25 to 40 (33)
Cohesion (psf) [3]									
Field	Varies	*	*	0 to 1,430 (1,430)	270**	*	*	*	-
Saturated	Varies	300 to 1,180 (800)	500 to 550 (525)	0 to 2,100 (1,032)	270 to 750 (510)	600**	100 to 850 (324)	200 to 600 (400)	0 to 1,500 (362)
Unit Subgrade Modulus (k) (kcf) [4]	Varies							300 (small strain); 240 (ODE); 100 (MDE)	
Allowable Bearing Value (psf)	3,000[5], 8,000[6]							8,000 psf	
Coefficient of Friction (μ) [7]	0.36	0.08 to 0.38 (0.31)	0.32 to 0.35 (0.34)	0.26 to 0.45 (0.34)	0.38**	0.35**	0.32 to 0.42 (0.38)	0.35 to 0.42 (0.38)	0.30 to 0.50 (0.40)
Soil Pressure Coefficient, At-Rest Ko									
Bored Tunnel Section [8]	0.69	*	*	0.59 to 0.63 (0.61)	*	0.61	0.61 to 0.91 (0.76)	0.66 to 0.71 (0.69)	*
Underground Station [9]	**0.48**	0.~~48~~**85 to 0.94 (0.89)**	**0.88 to 0.90 (0.89)**	**0.59 to 0.81 (0.68)**	**0.69 to 0.91 (0.84)**	**0.63****	**0.55 to 0.66 (0.59)**	***[9a] 0.50 to 0.57 (0.54)[9b]**	**0.44 to 0.71 (0.63) [9a] 0.41 to 0.56 (0.51) [9b]**
Soil Pressure Coefficient, Active Ka [10]	0.34	0.32 to 0.78 (0.40)	0.35 to 0.38 (0.36)	0.26 to 0.45 (0.36)	0.32**	0.35**	0.28 to 0.38 (0.31)	0.28 to 0.35 (0.31)	0.22 to 0.41 (0.30)
Soil Pressure Coefficient, shoring K [11]	0.34	0.22 to 0.45							
Soil Pressure Coefficient, Passive Kp [10]	Varies	1.28 to 3.12 (2.64)	2.66 to 2.88 (2.77)	2.20 to 3.85 (2.84)	3.12**	2.88**	2.66 to 3.54 (3.21)	2.88 to 3.54 (3.21)	2.46 to 4.60 (3.37)
Soil Pressure Coefficient, Seismic Ke [12]	0.11	0.11							
Ground surface elevation (ft) [13]	Varies	278 to 294							
Groundwater Elevation (ft) [14]	210	248 to 264 (depth to historically highest groundwater around 30 ft bgs) 186 to 210 (depth to current groundwater 82 to 97 ft bgs)							
Corrosivity Results [15]									
Minimum Resistivity (ohm-cm)	640	806 to 1,680 (1163)	1,838**	840 to 1,223 (1076)	*	*	1,936**	640**	774 to 5,020 (2082)
pH	5.8	7.4 to 8.2 (7.9)	7.5**	7.2 to 7.9 (7.5)	*	*	7.6**	7.7**	5.8 to 8.2 (7.6)
Chloride Content (ppm or mg/kg)	3823	16 to 672 (305)	136**	25 to 188 (101)	*	*	128**	968**	39 to 3,823 (706)
Sulfate Content (ppm or mg/kg)	357	4 to 289 (86)	71**	21 to 357 (151)	*	*	251**	45**	13 to 108 (65)
Poisson's Ratio [16]									
Unsaturated	0.35	*	*	0.38**	0.33**	*	*	*	0.28**
Saturated	0.34	0.33 to 0.47 (0.37)	0.34 to 0.35 (0.35)	0.30 to 0.38 (0.35)	0.34**	0.34**	0.31 to 0.35 (0.32)	0.31 to 0.39 (0.35)	0.26 to 0.37 (0.32)
Liquefaction Potential (Yes or No) [14]									
Above Station bottom	NO								
Below Station bottom	NO								
Dynamic Elastic Modulus (ksf) [17]									
Small Strain (Initial)	13,361	1,149 to 12,528 (5,913)	6,861 to 14,428 (7,999)	5,557 to 21,091 (11,452)	9,498 to 22,164 (12,036)	10,355 to 25,360 (13,631)	8,779 to 28,305 (15,570)	11,958 to 31,826 (18,940)	8,997 to 46,713 (22,191)
ODE	9,299	800 to 8,719 (4,116)	4,776 to 10,042 (5,567)	3,868 to 14,679 (7,970)	6,611 to 15,426 (8,377)	7,207 to 17,651 (9,487)	6,110 to 19,700 (10,837)	8,323 to 22,151 (13,183)	6,262 to 32,512 (15,445)
MDE	2,325	200 to 2,180 (1,029)	1,194 to 2,511 (1,392)	967 to 3,670 (1,993)	1,653 to 3,856 (2,094)	1,802 to 4,413 (2,372)	1,528 to 4,925 (2,709)	2,081 to 5,538 (3,296)	1,566 to 8,128 (3,861)

Table 4-1: Preliminary Geotechnical Design Parameters for Century City Constellation Station (Continued)

Parameter	Design Value	2009 through 2015 Geotechnical and Environmental Investigations Estimated Range of Engineering Parameters [21]							
Geologic Unit		Artificial Fill (Af)		Quaternary Older Alluvium (Qalo)		Lakewood Formation (Qlw)		San Pedro Formation (Qsp)	
		Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained
Shear Wave Velocity (fps) [17]									
Small Strain (Initial)	1,105	340 to 1,206 (709)	780 to 1,594 (843)	710 to 1,806 (1,018)	950 to 2,060 (1,071)	960 to 2,162 (1,125)	940 to 2,319 (1,268)	1,020 to 2,348 (1,276)	960 to 2,770 (1,510)
ODE	920	284 to 1,006 (591)	651 to 1,329 (703)	592 to 1,506 (849)	792 to 1,718 (893)	801 to 1,803 (938)	784 to 1,934 (1,058)	851 to 1,958 (1,064)	801 to 2,310 (1,259)
MDE	460	142 to 503 (296)	325 to 665 (352)	296 to 753 (424)	396 to 859 (447)	400 to 901 (469)	392 to 967 (529)	425 to 979 (532)	400 to 1,155 (630)
Peak Ground Accel. (g)- Horiz. [18]									
ODE	0.29	0.29							
MDE	0.84	0.84							
Peak Ground Velocity (fps)- Horiz [19]									
ODE	1.48	1.48							
MDE	5.78	5.78							
Free-Field Displacement for Station [20]									
ODE	0.14 inch in 50 feet								
MDE	1.2 inch in 50 feet								

* No test data
, ** Limited data
[1]Lab test data from historic, ACE, PE and Adv. PE phase investigations. Use submerged unit weight below design water level
[2a]Based on relationship between elastic modulus and SPT N-value from Stroud (1989) and Duncan and Bursey (2007), CGPR#44, Virginia Tech
[2b]Based on secant modulus computed at 0.1±0.05% axial strain from Triaxial consolidated-undrained tests
[3]Values based on site-specific strength test results
[4]Unit subgrade modulus for design of foundation for service, ODE, and MDE levels
[5]Spread footing supported on undisturbed natural and/or compacted fill (for minor structures). Increase the values by 30% for short-term seismic (ODE and MDE) and wind loads
[6]Mat foundation (or) large spread footings. Bearing value may be increased based on the foundation size, if commensurate settlement is acceptable. Increase the values by 30% for short-term seismic (ODE and MDE) and wind load conditions
[7]Coefficient of friction between mass concrete and subgrade soils
[8]Based on pressuremeter test results
~~[9]Based on site-specific shear strength data~~
[9]Range and average based on consideration of pressuremeter testing, and interpreted strength and OCR (3 for Af, 2 for Qalo and 1.5 for Qlw, 1.25 for Qsp); recommended values based on normally consolidated condition utilizing site specific shear strength data
[9a] For soils above Elevation 205, [9b] For soils below Elevation 205
[10]Active and passive earth pressure coefficients were based on laboratory shear strength data.
[11]Recommended earth pressure coefficient is based on AMEC's prior experience with similar soils along the alignment
[12]Based on Mononobe-Okabe (1926, 1929) procedure and PGA for ODE
[13]Refer to Plate 2-1 of GDR
[14]Estimated based on current groundwater level measurements and historic groundwater level data from Seismic Hazard Zone Report for the Hollywood 7.5-minute Quadrangle, Los Angeles, California (1998)
[15]Design values are based on lowest resistivity, lowest pH, highest chloride and sulfate content test values
[16]Possion's Ratio was computed based on Duncan and Bursey (2007), CGPR#44, Virginia Tech
[17]Small Strain elastic modulus values were based on site-specific shear wave velocity and design Poission's Ratio of 0.35. ODE and MDE values were based on Table 19.2-1 of FEMA P- 750 (2009)
[18]PGA estimated in accordance with Chapter 2, Section 3 of LA Metro Design Code (2010)
[19]Based on PGV-S_1 correlation (equations 13-1 and 13-2) of FHWA-NHI-10-034 (2009)
[20]Based on site-specific SHAKE91 analysis performed during Adv. PE phase.
[21]Average values are shown in parenthesis

Figure 4-1: Lateral Pressure Distribution



Flexible/Rigid Shoring (braced) [1]	**Permanent Braced Walls (Un-dewatered)**	**Cantilever Walls – Dewatered** [3] **(Shoring or permanent)**	**Seismic Increment** [4] **(Shoring only)**	**Hydrostatic (Un-dewatered Condition)** [2]	**Lateral Pressure for determination of Toe Embedments** [5]
(a)	**(b)**	**(c)**	**(d)**	**(e)**	**(f)**

Notes

1. Flexible walls are assumed to be dewatered or free draining; rigid walls are assumed to be un-dewatered and water-tight.
2. Hydrostatic pressures should be used in the design for water-tight rigid shoring systems such as slurry wall, secant-pile, tangent-pile systems and for permanent walls (if not dewatered).
3. For cantilever walls with not dewatered condition, lateral soil earth pressure will be half of the value shown in the figure (i.e., 18.5H); hydrostatic pressures should be added as shown in the figure.
4. For both cantilever and braced shoring and minor walls, the seismic increment per diagram (d) is to be added to the pressure distribution per diagram (a) or to the pressure distribution per diagram (c) with appropriate load factors, but no less than pressures per diagram (b) with appropriate load factors (NCHRP, 2009).
5. The resisting (passive) value includes a multiplication factor of 1.5 as recommended for circular soldier piles per Metro Standard Drawing S-003; the passive pressure should be modified for other passive-resisting elements per S-003 (e.g., a slurry wall system would use the passive values provided divided by 1.5).
6. Below the bottom of excavation, the driving lateral earth pressure need only be applied to the width of the embedded element.

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4.7.11 Monitoring

Some means of monitoring the performance of the shoring system is recommended. The instrumentation could consist of inclinometers, ground settlement monuments as well as load cells and strain gages placed on the struts and soldier piles.

The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops and intermediate points of all the soldier piles as well as installation and regular readings of inclinometers, and regular readings of load cells and/or strain gages. Depending on the proximity of the adjacent structures and utilities, a specific instrumentation and monitoring program should be planned and implemented prior to the commencement of station excavation.

4.8 Permanent Walls Below Grade and Retaining Walls

4.8.1 Lateral Earth Pressures

Section 5.6.4 of the Metro Rail Seismic Design Criteria of the MRDC states that buried permanent station walls should be designed for lateral at-rest earth pressures with a triangular distribution considering the long-term creep of the retained soils. Minor cantilever retaining walls may be designed for active earth pressures with a triangular distribution.

The geotechnical parameters and earth pressure coefficients required for design permanent station walls at the station are presented in Table 4-1. Recommended earth pressure distributions for design of permanent walls are shown in Figure 4-1.

The earth pressure distributions presented in Figure 4-1 assume a level backfill. If the ground surface retained by retaining wall is sloped, the increase in earth pressure will need to be evaluated on a case-by-case basis.

4.8.2 Hydrostatic Pressures

Permanent dewatering systems are not to be used for the station. Therefore, portions of station walls below design groundwater level and the station mat foundation should be designed for hydrostatic pressures. Furthermore, station walls below grade are to be waterproofed to avoid intrusion of water and gas into the station.

If minor retaining walls are located below the design water level, the wall is to be designed for hydrostatic pressure or be provided with weep holes or drainage behind the wall.

The design groundwater water level of 30 feet bgs should be used in computing hydrostatic pressures for design of secant/tangent pile or slurry wall shoring systems which would act as permanent structures, if used.

4.8.3 Surcharge Pressures

Applicable lateral and vertical surcharge pressures from adjacent buildings, foundations of minor structures and vehicular loading (AASHTO HS20 vehicular loading) should be estimated and added to the earth pressures stated above. Surcharge pressures from heavily loaded construction cranes and other traffic should be added to the above pressures. Such pressures will be estimated when loading details of the construction traffic and building loads are available. Surcharge pressures should be estimated in accordance with recommendations presented in Metro Rail Standard structural Drawings (Metro, 2010)

The station roof also should be designed to resist the weight of the overburden soil. The weight of the compacted soil may be estimated using a total unit weight of 125 pounds per cubic foot.

4.8.4 Seismic Earth Pressures

The Metro Seismic Criteria does not provide specific recommendation for computing seismic earth pressures for permanent walls, but Metro standard drawing SS-003 presents a guideline for seismic earth pressure due to retained soil and due to adjoining building. Seismic earth pressure due to adjoining buildings should be in accordance with Metro standard drawing SS-003 for retaining wall design.

The increment of seismic lateral earth pressures due to retained soil should be computed using the seismic earth pressure coefficient increment (K_{AE}) provided in Table 4-1 which was based on PGA for the ODE event. An effective PGA equivalent to half of the computed PGA was used in the computations. The equivalent uniform earth pressure may be computed by taking the same resultant as the triangular equivalent fluid pressure distribution, as shown in Figure 4-1. Using these design parameters, a seismic earth pressure increment is estimated to be 6H pounds per square foot (uniform), where ‘H’ is the unbalanced permanent wall height, depending on the type of excavation support. The seismic earth pressure is an incremental value intended to be added to the static active earth pressure value. All required load combinations of static and seismic, with appropriate load factors (NCHRP, 2009), are to be utilized in the final design of permanent walls.

4.9 Gassy Condition Design Considerations

The station is located within the City of Los Angeles Department of Building and Safety designated “methane buffer zone” zone. Since the station will be constructed below design groundwater level, the station structure will have to be thoroughly waterproofed. The impermeable waterproofing membrane should be selected such that it can be used as both water and gas intrusion barrier

The waterproofing/gas barrier should be used around the entire station structure as well as for the minor structures such as ramps, stairways, and other ancillary structures that connect to the station.

In addition, consideration should be made for construction activities in a gassy environment, with an appropriate plan being prepared for explosion, inhalation, and other safety items, as well as consideration for monitoring and/or mitigation of gases that might be released during excavation, for the purpose of the general public outside the construction zone.

4.10 Foundations (Permanent Station Structure and Station Entrance)

The station bottom is currently planned at a depth of about 85 to 90 feet below ground surface. The soils at the station depths are sufficiently firm to allow support of the station on mat foundation.

Minor structures, such as ramps, stairways, and other structures ancillary to the station, can be supported on conventional spread footings bearing in properly compacted fill and/or undisturbed natural soils.

The geotechnical parameters for foundation design of permanent station and at-grade ancillary structures are presented in Table 4-1.

4.10.1 Bearing Value

The mat foundation for the station, established in stiff and/or dense soils, may be designed to impose a net dead-plus-live load pressure of 8,000 pounds per square foot.

Spread footings for minor structures near the existing ground surface that are at least 2 feet wide and at least 2 feet below the lowest adjacent grade or floor level, and are established in properly compacted fill and/or undisturbed natural soils, may be designed to impose a net dead-plus-live load pressure of 3,000 pounds per square foot. The excavations should be deepened as necessary to extend into satisfactory soils.

A one-third increase may be used in the above bearing values for wind or seismic loads. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot; the weight of soil backfill can be neglected when determining the downward loads.

Since the station foundation excavation extends below or near current groundwater depth, the bottom of the excavation may be easily disturbed by construction equipment. Therefore, if needed, a layer of crushed rock or a waste slab (slurry slab) may be considered to stabilize the subgrade after excavation. Furthermore, a layer of BX1200 geogrid or equivalent could be used in conjunction with the gravel layers to provide additional protection against unstable subgrade conditions, particularly for heavy construction equipment.

4.10.2 Settlement

The average bearing pressure on the station mat foundation is anticipated to be less than the overburden removed by the excavation. Therefore, settlement of the station supported on mat foundation in the manner recommended above is expected to be negligible. Some differential settlement within the mat foundation should be anticipated due to variable soil conditions. Differential settlements are expected to be less than ½ inch in 50 feet.

Settlements of the minor structures supported at-grade cannot be estimated as the structural loads and foundation details are not available at this time. However, differential settlement should be anticipated at the interface between station and station entrances.

4.10.3 Lateral Resistance

Main Station and Station Entrance Structures

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 may be used between the base of the foundations and the supporting soils. The passive resistance of undisturbed natural soils against station walls may be assumed to be equal to a uniform pressure of 3,000 pounds per square foot. The structural elements (station walls and the floor decks) should be designed to transfer the required passive resistance load from one side of the box to the other; for example, surcharge pressures from major existing structures adjacent to one side of the excavation will need to be transferred through the station structure and resisted by means of friction along the bottom of the station and passive resistance on the other side of the station.

The passive resistance of undisturbed natural soils and/or properly compacted fill soils against footings for minor structures may be assumed to be equal to the pressure developed by a fluid with a density of 350 pounds per cubic foot, if above the design groundwater level. For soils below the design groundwater level, an equivalent fluid pressure of 175 pounds per cubic foot should be used. The passive resistance of existing fill soils may be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot. When existing buildings or deep excavations are located adjacent to the station, a full passive wedge against foundations may not be developed; in this case, the above recommended passive (fluid) pressure should be limited to a maximum passive value computed as the product of the applicable fluid pressure and lateral distance between the new foundations and existing building basement walls/excavation face.

When existing buildings or deep excavations are located adjacent to the station, a full passive wedge against foundations may not be developed; in this case, the above recommended passive (fluid) pressure should be limited to a maximum passive value computed as the product of the applicable fluid pressure and lateral distance between the new foundations and existing building basement walls/excavation face.

A one-third increase in the passive value may be used for wind or seismic loads.

The frictional resistance and the passive resistance of the soils may be combined without reduction in determining the total lateral resistance.

4.10.4 Soil Springs

For the design of mat foundation and soil-structure interaction (SSI) analysis, soil springs were developed for static and dynamic conditions. The coefficient of subgrade reaction for mat design is presented in Table 4-1. The recommended dynamic lateral and vertical soil springs for the SSI analysis of the station are presented in Table 3-5. If the dimensions change, the soil springs presented in these tables will need to be revised.

4.11 Earthwork

Based on the subsurface conditions encountered in the prior and current investigations, the excavation of the station can be accomplished using conventional earth moving equipment. Earth and site preparation activities for the station construction are expected to consist of the following:

- Excavations for shoring elements
- Excavation for station
- Subgrade preparation for station mat foundation and near-surface footings for ancillary structures
- Excavations for utility trenches, backfill over station, footings and utility trenches

Excavation for the station and entrance structures will need temporary shoring. All work should be in compliance with applicable City (Los Angeles), State (California) and federal (Occupational Safety and Health Act) requirements.

The on-site soils excavated from station excavation may be re-used as backfill material over the station, if not contaminated. The on-site soils are predominantly clayey and will be difficult to compact in confined spaces such as in utility or wall backfill. Import granular material may be required to backfill the excavation.

Any required soil backfill should be placed in loose lifts not more than 8-inches-thick and compacted. The fill should be compacted to at least 95% of the maximum density obtainable by the ASTM Designation D1557 method of compaction. The backfill should be sufficiently impermeable when compacted to restrict the inflow of surface water. Some settlement of deep backfill should be allowed for in planning utility connections and overlying concrete hardscape.

4.12 Corrosion Potential

To evaluate the potential for deleterious effects of the on-site soils on structural concrete and steel and on metal piping, chemical testing was performed on selected soil samples. Based on the corrosion test results, the on-site soils are severely corrosive to ferrous metals, aggressive to copper, and sulfate attack on concrete is considered to be moderate to severe. A corrosion mitigation report prepared by HDR/Schiff Associates based on the PE Phase test results is included in Appendix D of the Station GDR for reference.

5.0 PRIOR EXPERIENCE WITH SIMILAR PROJECTS

AMEC's predecessor firms LeRoy Crandall and Associates performed numerous geotechnical investigations near the Century City Constellation Station for existing high rise buildings. The building addresses and the excavation and shoring details are presented below.

- **Watt Plaza (1875 Century Park East)**

  The twin towers have 24 levels above grade and one subterranean level for parking. The parking structure located to the north of the towers has 7 levels above ground and three subterranean levels. Excavation for basement beneath the tower extended to a depth of about 15 feet bgs; excavation beneath the parking structure extended to about 70 feet bgs. Conventional soldier pile with lagging shoring and tieback anchors was used for the excavation support. Drilled tieback anchors were designed for a frictional resistance of 700 pounds per square foot.

- **Century Plaza Towers (2029 Century Park East and 2000 Avenue of the Stars)**

  The twin triangular towers have 42 levels above grade and up to 6 subterranean levels for parking. Excavation for basement extended to depths of about 70 to 80 feet bgs at Century Park East to about 90 to 100 feet bgs at Avenue of the Stars. Conventional soldier pile with lagging shoring and tieback anchors was used for the excavation support. Some of the pictures from the basement excavation are presented on Figures 5-1 and 5-2.

- **Sun America Building (1999 Avenue of the Stars)**

  The building has 38 levels above grade with 3 subterranean levels extending to a depth of 40 feet. Conventional soldier pile with lagging shoring and tieback anchors was used for the excavation support. Drilled tieback anchors were designed for a frictional resistance of 600 pounds per square foot.

- **Hyatt Regency Century Plaza Hotel (2025 Avenue of the Stars)**

  The existing building has 16 levels above grade with up to four subterranean level extending to a depth of 60 feet bgs. The excavation was made by sloping the excavation side walls at inclination of 1H:1V (horizontal: vertical) within the existing fill and ¾H:1V within natural soils.

  In addition, two new 40-story towers are proposed west of the existing hotel with up to seven subterranean levels extending to up to about 80 feet bgs.

Figure 5-1: Century Plaza Theme Towers (Photo 1)

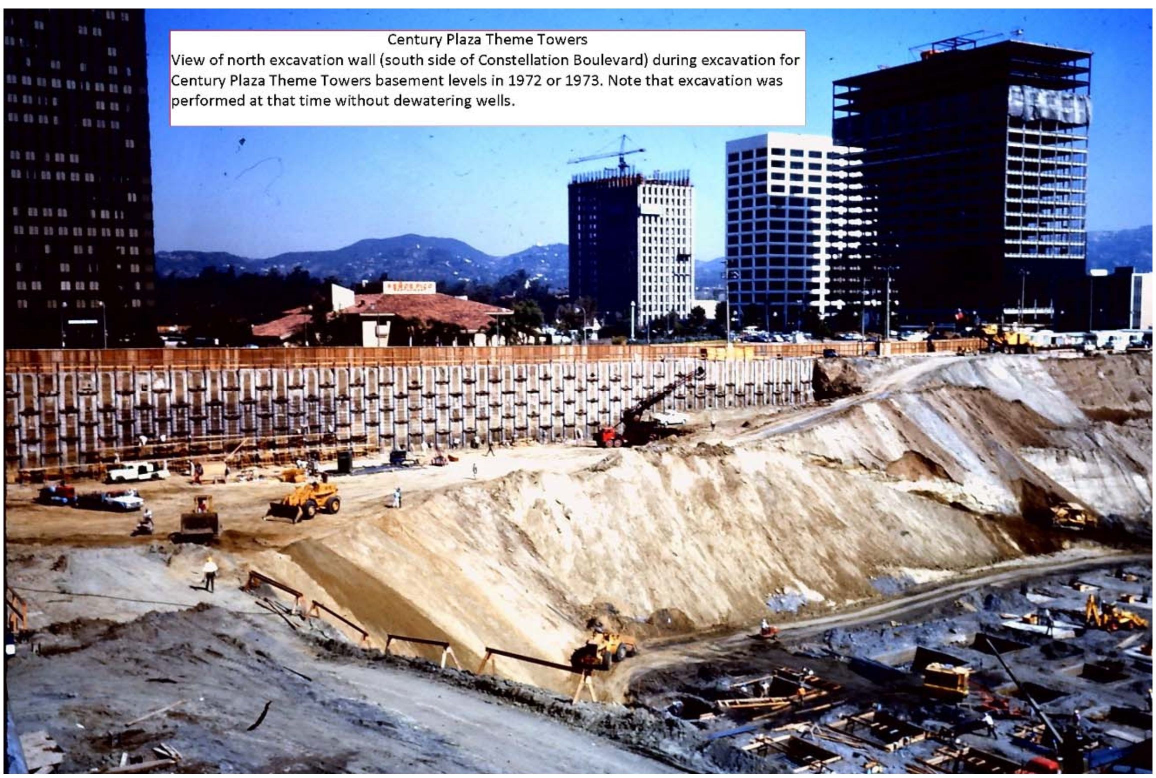
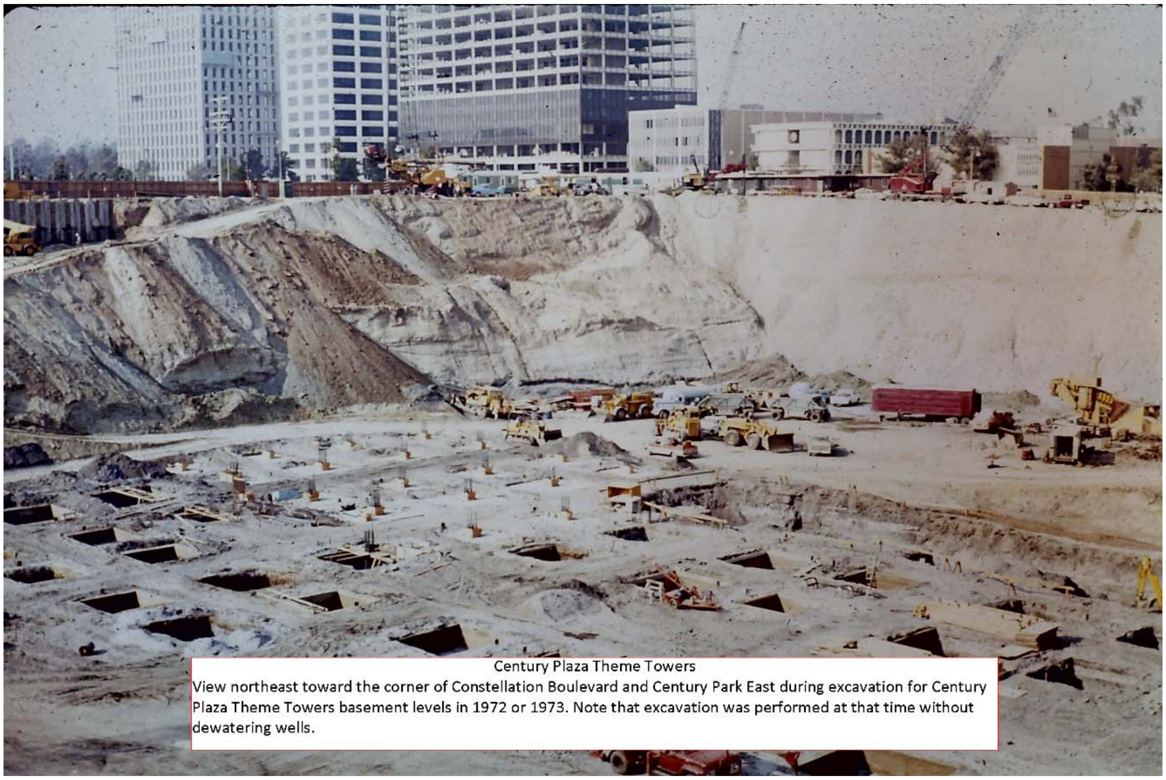


Figure 5-2: Century Plaza Theme Towers (Photo 2)



6.0 LIMITATIONS AND BASIS FOR PRELIMINARY RECOMMENDATIONS

The professional services have been performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this Technical Memorandum. This GDM has been prepared for the Los Angeles County Metropolitan Transportation Authority (Metro) and its design consultants and contractors to be used solely for the evaluation for the Century City Constellation Station planned as part of the proposed WPLE project. The memorandum has not been prepared for use by other parties, and may not contain sufficient information for purpose of other parties or other uses.

In developing this memorandum, Amec Foster Wheeler (PB team member) relied on subsurface information obtained during Adv. PE phase and by its predecessor companies AMEC and MACTEC in the AA, ACE, and PE phase studies and its other predecessor companies, Law/Crandall and LeRoy Crandall and Associates, as well as subsurface information obtained by other firms. Subsurface conditions are, by their nature, uncertain and may vary from those encountered at the locations where visual inspections, borings, surveys, or other explorations were made.

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WILSHIRE/RODEO STATION

SUMMARY OF REVISIONS TO THE NOVEMBER 2, 2015 GDM (Amd 9)

Chapter/Figure/Table	Revisions	Page Nos.
Cover page	Presented GDM Amendment dates	
	Added a note to state that September 2015 and November 2015 GDRs and GDMs are superseded by February 2016 (Amendment 9) versions	
	Added a note to state separate GDM is submitted for Constellation Station	
	Deleted sentence that stated that a supplement GDM will be submitted	
1.0	Corrected depths of station bottom based on December 2015 profile	1
2.0	Changed “station” to “Wilshire/Rodeo Station”; Added a sentence to state the depth of San Pedro below station bottom	1
	Changed GDR reference from September 2015 to February 2016	2
Table 2-1	Revised table to include Adv. PE phase data; revised footnotes	3, 4
3.1	Revised section to include shear wave data from Adv. PE phase and revised site class based on new estimated Vs,30 values; deleted sentence that stated response spectra and site will be evaluated when additional suspension logging data is available	5
Table 3-1	Revised ODE/MDE response spectra for station based on new Vs,30; replace footnote 2 for clarification	6
Table 3-2	Revised ODE/MDE target response spectra for station based on new Vs,30	6
3.2	Revised Vs at a depth of 150 ft bgs	6
	Replaced Next Generation Attenuation with NGA	6
	Revised controlling earthquakes based on new deaggregation results	6
	Revised distance and directionality from site to Santa Monica and Newport Inglewood Faults	7
3.3	Replace Vs estimated from blow counts with suspension logging results from Adv. PE phase	8
	Revised Vs at a depth of 150 ft bgs	8
Table 3-4	Revised estimated free-field displacements based on revised SHAKE analysis	8
3.3	Delete sentence that stated that additional explorations are planned in Adv. PE phase	8
3.3.1	Revised section heading	8
3.3.1.1	Added subsection heading for lateral spring stiffness	9
	Revised sentence that stated spring stiffness will need to be revised if station dimensions change	9
Table 3-5	Revised lateral spring stiffness based on new station depths and shear-wave data from Adv. PE phase	9
3.3.1.1	Corrected modulus degradation values for ODE event	9

Chapter/Figure/Table	Revisions	Page Nos.
3.3.1.2	Added subsection heading for vertical spring stiffness; Revised sentence that stated spring stiffness will need to be revised if station dimensions change	10
Table 3-6	Revised vertical spring stiffness based on new station depths and shear-wave data from Adv. PE phase	10
4.1	Changed station profile reference from September 2015 to February 2016	10
4.2	Revised groundwater levels based on Adv. PE phase data	11
4.2.1	Provided clarification that groundwater depth for construction should be taken as 60 feet bgs	11
4.4	Corrected estimation excavation depth	11
4.5	Revised dewatering and groundwater control section	12
4.5.1	Revised to present estimated peak and steady-state water inflow rate based on drawdown model analyses	12
	Revised section to state depths of San Pedro below station bottom	13
	Deleted sentence that stated pumping test is planned in the Adv. PE phase	13
4.5.2	Added section to present dewatering induced settlement	13
4.7.2	Revised depths of station and station ancillary structure shoring heights; revisions to text to improve sentence structure	13
	Revised sentence to state parameters presented in Table 4-1 can be used for numerical modeling	14
	Changed “additional” to “addition”	14
4.7.4	Deleted reference to building protection report	14
4.7.6	Revised frictional resistance for embedded portion of soldier pile	16
4.7.8	Changed “entrance structures” to “station entrance structures”	17
4.7.10	Deleted reference to building protection report	17
Table 4-1	Revised recommended parameters based on Adv. PE phase data; revised footnotes	19, 20, 21
Figure 4-1	Revised figure of lateral earth pressure distribution	22, 23
4.9	Revised sentence to state low concentrations of methane was found in gas wells	26
4.10	Renamed “station entrances” as “station entrance structures”; Revised depths of station bottom	26
4.10.1	Added sentence to state a subgrade protection plan should be developed	27
4.10.2	Renamed “station entrances” as “station entrance structures”	27
4.10.3	Change “main station” to “station”	27
4.11	Renamed “station entrances” as “station entrance structures”; Added City of Beverly Hills as the applicable city requirements	28
	Clarified sentence to state onsite soils can be used if granular and that clayey soils will be difficult to compact to 95%	28
4.12	Added a sentence to state prior PE phase corrosion report is still applicable	29
6.0	Added references to 2016 GDR, EDR and pump testing report	31

MEMO

To	**Ms. Amanda Elioff, P.E.** **Parsons Brinckerhoff (PB)** **777 South Figueroa Street, Suite 1100** **Los Angeles, California 90017**
Date	**Amendment 3: November 2, 2015** **Amendment 9: February 19, 2016**
Project No	**4953-11-1423** **(PB Task 52.05.010.01.01.02)**
Subject	**Geotechnical Design Memorandum – Wilshire/Rodeo Station** Westside Purple Line Extension Beverly Hills, California

This Geotechnical Design Memorandum (GDM) containing preliminary geotechnical design data has been prepared for the Wilshire/Rodeo Station planned within Section 2 of the proposed Westside Purple Line Extension (WPLE) project for the Los Angeles County Metropolitan Transportation Authority (Metro), as part of the Advanced Preliminary Engineering (Adv. PE) phase study.

The results of the Advanced Conceptual Engineering (ACE) and Preliminary Engineering (PE) phase geotechnical investigations performed for the station were presented in a draft Geotechnical Data Report (GDR), dated September 2015. **Subsequently, additional borings were drilled during the Adv. PE phase and the additional data was incorporated in a GDR (Amendment 9) dated February 19, 2016. Accordingly, the prior GDM was revised to include the additional data obtained in the Adv. PE phase. This GDM supersedes the November 2, 2015 GDM.**

This GDM presents preliminary geotechnical recommendations for foundation design, excavation support, station design, dewatering and groundwater control, and for earthwork. In case of any conflict between the interpretation of data or preliminary recommendations presented in this report and the Geotechnical Baseline Report (GBR), the data and preliminary recommendations presented in the GBR will prevail. ~~A supplemental GDM is planned to incorporate additional data when obtained. The recommendations and conclusions presented in this GDM may change when additional data are evaluated.~~ The Design-Build (D-B) contractor should perform an independent evaluation of the data contained in the GDR and the preliminary recommendations presented herein and provide parameters for final design.

The geotechnical parameters presented in this report reflect the design team's judgment of anticipated subsurface conditions and ground behavior based on the construction means and methods anticipated. The design data presented herein were established by considering available geologic and geotechnical data, together with past construction experience and anticipated construction methods in similar ground conditions. Development of the project design recommendations required interpretation of the data obtained from various sources, including: geologic maps; samples from hollow stem auger, rotary, and core borings; other borings previously conducted for nearby projects; geophysical surveys; and in-situ and laboratory tests, as well as the consideration of information from previous construction projects

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completed in similar geologic conditions. While actual conditions encountered in the field are expected to be within the range of conditions discussed herein, the locations where specific ground and groundwater conditions are encountered may vary from those described in this report. In addition to the specific conditions described herein, the ground behavior will also depend on the construction sequence and methods employed, as well as the Contractor's equipment, experience and workmanship. The project design, therefore, assumes that the construction methods and level of workmanship will be consistent with those that can reasonably be expected from an experienced and qualified contractor.

It is our understanding that this GDM is being prepared for inclusion in the Request for Proposal Package being prepared for a Design-Build Contract for Section 2 of the proposed WPLE project.

It is a pleasure to be of continuing professional service to you. Please call if you have any questions or if we can be of further assistance.

Sincerely,

Amec Foster Wheeler Environment & Infrastructure, Inc.

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1.0 DESCRIPTION OF STATION

The Wilshire/Rodeo station will be constructed using cut and cover methods. An arch roof module is planned for the station. The majority of the station excavation support will be internally braced with struts. However, tieback systems may be selected by the contractor for portions of the station excavation, and station entrances and appendages. Table 1-1 presents information pertaining to the location of the station and approximate dimensions of the station that were used to develop the preliminary geotechnical recommendations presented in this Geotechnical Design Memorandum (GDM).

Table 1-1: Station Location and Dimensions for Design

Station	Location	Length (feet)	Width (feet)	Depth to Top of Arch Roof, bgs (feet)	Depth to Top of Station Walls bgs (feet)	Depth to Bottom of Station Box bgs (feet)
Wilshire/Rodeo[1]	105 feet east of the east edge of South Canon Drive to 105 feet west of the west edge of South Beverly Drive	1,000	60	15 to 30	25 to 40	~~70~~**75** to ~~85~~**90**

1. Station entrance is planned in the southwest corner of Wilshire Boulevard and South Reeves Drive (9430 Wilshire Blvd) per plans dated ~~September~~**December,** 2015

2.0 PRELIMINARY ENGINEERING PROPERTIES OF PRINCIPAL GEOLOGIC UNITS

The geologic units that will be encountered in the station excavation, from oldest to youngest, are the Pleistocene-age older alluvium, Holocene-age alluvium, and modern artificial fill. The Pleistocene-age San Pedro Formation and Pliocene-age sedimentary rock of the Fernando Formation encountered in the eastern portion of the Westside Purple Line Extension (WPLE) is not anticipated in the **Wilshire/Rodeo** station excavation. **The depth to top of San Pedro Formation is about 15 to 55 feet below the station bottom.**

Preliminary engineering properties were compiled in these principal geologic units and statistical analyses were performed to estimate the lower bound, upper bound and recommended value for each property. The properties were evaluated by sub-dividing each geologic unit into fine-grained and coarse-grained portions before performing statistical analysis. The engineering and index properties for the earth materials at the station are listed below:

- SPT Blow Counts
- Moisture Content
- Dry Density
- Total Density
- Fines Content

- Specific Gravity
- Liquid Limit and Plasticity Index
- Expansion/Collapse Potential
- Degree of Saturation
- Void Ratio
- Effective Cohesion and Friction Angle
- Undrained Cohesion and Friction Angle
- Elastic Parameters – Young's Modulus and Poisson's Ratio
- Hydraulic Conductivity
- Compression Index (C_c)
- Recompression Index (C_r)
- At-rest Lateral Earth Pressure Coefficient (K_o) (note this is the in-situ K_0 value, which may or may not be represented of the final design K_0 value used for the station itself, depending on excavation and associated stress changes in the ground)
- Soil Abrasion
- Corrosion (Minimum Resistivity, pH, Chloride Content, Sulfate Content)

The estimated range (lower bound and upper bound) and a recommended value of the properties listed above for the Wilshire/Rodeo Station are presented in Table 2-1. The laboratory test results and plots are presented in the Geotechnical Data Report (GDR) dated ~~September 2015~~**February, 2016** for **the** Wilshire/Rodeo Station.

Table 2-1: Preliminary Engineering Properties of Principal Geologic Units for Wilshire/Rodeo Station

Parameter	2009 through 2015 Geotechnical and Environmental Investigations Estimated Range of Engineering Parameters															
Geologic Unit	Artificial Fill (Af)				Quaternary Younger Alluvium (Qal)				Quaternary Older Alluvium (Qalo)				San Pedro Formation (Qsp)			
	Fine-Grained		Coarse-Grained		Fine-Grained		Coarse-Grained		Fine-Grained		Coarse-Grained		Fine-Grained		Coarse-Grained	
USCS Soil Classification	CL, CL-ML		SM, SC-SM, SC		CL, CH, ML		SM, SC-SM, GC, SC		CL, CH, ML		SW,SW-SM,SP,SP-SM,SM,SC		CL, CH, CL-ML, ML		SW-SM, SP, SP-SM, SM	
Engineering Properties	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]
SPT Blowcounts [2]	*	*	*	*	7 to 23	14	8**	8**	4 to 100	30	4 to 100	50	26 to 73	39	51 to 100	81
Moisture Content (%)	*	*	*	*	9 to 34	20	9 to 21	15	5 to 39	18	1 to 27	12	4 to 24	17	8 to 21	14
Dry Density (pcf)	*	*	*	*	85 to 118	102	99 to 122	112	90 to 130	110	98 to 129	113	109 to 122	114	101 to 126	114
Total Density (pcf)	*	*	*	*	101 to 134	121	120 to 137	129	107 to 147	130	104 to 140	126	116 to 138	131	122 to 136	129
Fines Content (%)	*	*	*	*	51 to 84	65	49**	49**	50 to 89	69	10 to 49	28	53 to 78	66	33 to 50	41
Specific Gravity	*	*	*	*	*	*	*	*	2.49 to 2.82	2.72	2.56 to 2.86	2.74	*	*	*	*
Liquid Limit (%)	*	*	*	*	31 to 52	43	35**	35**	26 to 51	36	26 to 39	30	36 to 40	38	*	*
Plasticity Index (%)	*	*	*	*	18 to 34	26	22**	22**	8 to 34	21	9 to 16	12	22 to 26	23	*	*
Expansion (%)	*	*	*	*	*	*	*	*	0.03 to 4.42	0.94	0.02 to 0.09	0.06	0.82**	0.82**	*	*
Collapse (%)	*	*	*	*	*	*	*	*	*	*	*	*	-0.07**	-0.07**	*	*
Degree of Saturation (%) [#]	*	*	*	*	44 to 90	74	80 to 99	90	42 to 100	88	17 to 100	67	19 to 100	83	57 to 84	74
Void Ratio	*	*	*	*	0.49 to 0.78	0.63	0.37 to 0.70	0.52	0.32 to 0.79	0.55	0.32 to 0.70	0.49	0.38 to 0.55	0.48	0.32 to 0.65	0.48
Effective Friction Angle from DS Test [3] (degrees)	*	*	*	*	18 to 34	26	*	*	18 to 30	27	27 to 40	33	24**	24**	12**	12**
Effective Cohesion from DS Test [3] (psf)	*	*	*	*	500 to 2,700	1,275	*	*	50 to 2,200	600	150 to 1,850	600	1,250**	1,250**	950**	950**
Effective Friction Angle from Triaxial Test [4]	*	*	*	*	29**	29**	31**	31**	22 to 43	31	31 to 38	36	*	*	*	*
Effective Cohesion from Triaxial Test [4]	*	*	*	*	350**	350**	370**	370**	250 to 1,600	916	250 to 1,300	690	*	*	*	*
Undrained Friction Angle from Triaxial Test [4]	*	*	*	*	23**	23**	36**	36**	17 to 45	27	28 to 43	35	*	*	*	*
Undrained Cohesion from Triaxial Test [4]	*	*	*	*	400**	400**	650**	650**	50 to 1,750	486	100 to 2,000	950	*	*	*	*
Young's Modulus from Correlations [5] (ksf)	*	*	*	*	350 to 1,150	728	586**	586**	300 to 8,775	2,424	251 to 7331	3,445	2,369 to 6,935	3,431	3,195 to 7,936	5,486
Youngs' Modulus from Triaxial Test [6] (ksf)	*	*	*	*	1,295**	1295**	1,431**	1,431**	1810 to 5,170	3,060	2363 to 6170	4,179	*	*	*	*
Poisson's Ratio	*	*	*	*	0.31 to 0.41	0.37	0.29**	0.29**	0.23 to 0.41	0.35	0.24 to 0.36	0.31	0.37**	0.37**	0.44**	0.44**
Hydraulic Conductivity (ft/day) [7]	10^{-7} to 10^{-1} ^	10^{-4} ^	10^{-2} to 10^{1} ^	10^{-1} ^	10^{-7} to 10^{-1} ^	10^{-4} ^	10^{-2} to 10^{1} ^	10^{-1} ^	10^{-7} to 10^{-1} ^	10^{-4} ^	10^{-2} to 10^{2} ^	10^{1} ^	10^{-7} to 10^{-1} ^	10^{-4} ^	10^{-2} to 10^{2} ^	10^{1} ^
Modified Compression Index (Cce)	*	*	*	*	*	*	*	*	0.06 to 0.12	0.09	0.10**	0.10**	0.11**	0.11**	*	*
Modified Recompression Index (Cre)	*	*	*	*	*	*	*	*	0.01 to 0.03	0.02	0.01**	0.01**	0.02**	0.02**	*	*
At-rest Soil Pressure Coeff[9]., Ko	*	*	*	*	*	*	*	*	0.45 to 0.69	0.60	0.49**	0.49**	*	*	*	*
Soil Abrasion Test Value	*	*	*	*	*	*	*	*	2 to 4	3	12**	12**	*	*	*	*
Corrosivity Results:																
Minimum Resistivity (ohm-cm)	*	*	*	*	960 to 1,480	960	*	*	811 to 3,560	811	2,440 to 4,040	2,440	1,000**	1,000**	*	*
pH	*	*	*	*	6.7 to 7.7	6.7	*	*	4.0 to 8.2	4.0	7.2 to 7.8	7.2	8.3**	8.3**	*	*
Chloride Content (ppm or mg/kg)	*	*	*	*	2 to 54	54	*	*	3 to 180	180	9 to 15	15	10**	10**	*	*
Sulfate Content (ppm or mg/kg)	*	*	*	*	33 to 103	103	*	*	12 to 204	204	46 to 104	104	122**	122**	*	*

Table 2-1 (Continued): Preliminary Engineering Properties of Principal Geologic Units for Wilshire/Rodeo Station

Parameter	2009 through 2015 Geotechnical and Environmental Investigations Estimated Range of Engineering Parameters															
Geologic Unit	Artificial Fill (Af)				Quaternary Younger Alluvium (Qal)				Quaternary Older Alluvium (Qalo)				San Pedro Formation (Qsp)			
	Fine-Grained		Coarse-Grained		Fine-Grained		Coarse-Grained		Fine-Grained		Coarse-Grained				Coarse-Grained	
USCS Soil Classification	CL, CL-ML		SM, SC-SM, SC		CL, CH, ML		SM, SC-SM, GC, SC		CL, CH, ML		SW,SW-SM,SP,SP-SM,SM,SC		CL, CH, CL-ML, ML		SW-SM, SP, SP-SM, SM	
Engineering Properties	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]	Range[1]	Recommended[1]

* No test data
** Limited data
^ No test data; reported values are based on published data in literature, and/or based on prior experience
"NP" indicates non-plastic material
# Estimated using assumed specific gravity of 2.65 when a specific gravity test was not performed
pcf = pounds per cubic foot; psf = pounds per square foot; psi = pounds per square inch; cm = centimeter; ppm = parts per million; mg = milligrams; kg = kilograms; DS = direct shear

Notes:
1. Data presented here are based on ACE and PE phase explorations as well as applicable prior explorations as discussed in the GDR
2. Blow counts from ACE and PE phase environmental hollow-stem-auger borings were not considered
3. Effective cohesion and friction angle are based on peak strength values from slow direct shear tests.
4. Cohesion and friction angle are based on peak shear strength values from Triaxial consolidated-undrained tests. Effective values are based on effective stress and undrained values are based on total stress
5. Based on relationship between elastic modulus and SPT N-value from Stroud (1989) and Duncan Bursey (2007)
6. Based on secant modulus computed at 0.1+0.05% axial strain from Triaxial consolidated-undrained tests
7. Hydraulic conductivity values were based on published data (Department of Water Resources Bulletin 118, California's Groundwater Update, 2003) and based on site-specific pumping test results.
8. For soil corrosivity, the recommended values correspond to minimum resistivity, lowest pH and highest values for chloride and sulfate content
9. Based on field pressuremeter test data

3.0 DYNAMIC SITE CHARACTERISTICS

3.1 Response Spectra

Response spectra were developed for the underground station and the at-grade ancillary structures in accordance with Sections 2.3.1 through 2.3.3 of the Metro Supplemental Seismic Design Criteria of the Metro Rail Design Criteria (MRDC) dated November 2014. Two hazard levels were evaluated as listed below, based on a design life of 100 years for structures per the Metro Seismic Design Criteria of the MRDC.

- Operating Design Earthquake (ODE) – defined as an earthquake event likely to occur only once in the design life, where structures are designed to respond without significant structural damage. The current Metro Code defines ODE as an event with a 50% probability of exceedence in 100 years (corresponding to a return period of 150 years).
- Maximum Design Earthquake (MDE) – defined as an earthquake event with a low probability of occurring in the design life, where structures are designed to respond with repairable damage and to maintain life safety. The current Metro Code defines MDE as an event with a 4% probability of exceedence in 100 years (corresponding to a return period of 2,475 years)

The response spectra for the ODE and MDE events were estimated using the 2008 USGS Interactive Probabilistic Seismic Hazard Analysis (PSHA) Deaggregation tool on the USGS website (USGS, 2011). The USGS deaggregation tool uses the Next Generation Attenuation (NGA) relationships of Boore-Atkinson (2008), Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) for the ground motion prediction equations. Based on the available combinations of exceedence probability and exposure time, ground motions for the MDE event were computed for a probability of exceedence of 2% in 50 years (equivalent to the 4% in 100 year criteria, as stated in the Metro Seismic Design Criteria of the MRDC).

Site~~-~~-specific seismic shear wave data **of the subsurface soils** was ~~not~~**available from p- and s-wave suspension logging data** obtained **in soil boring G-408 drilled** at the station ~~during the PE phase, however, primary and secondary wave (p- and s-~~**location. The shear-**wave~~) suspension logging is proposed in the Adv. PE phase. For the PE phase,~~ **data was used to develop an estimated idealized average velocity profile, then** the average shear wave velocity within ~~the upper~~ 100 feet or 30 meters (referred to as $V_{s,30}$~~), measured from below the bottom of station, was estimated using empirical correlations~~**)** of the ~~blow counts with the shear wave velocity.~~**station bottom (for station design) and within 100 feet of the ground surface (for design of ancillary structures) was computed.** A $V_{s,30}$ of ~~920~~**1,500 feet per second and 1,120** feet per second was estimated for the earth materials beneath ~~foundations of both~~ the **station foundation and for at-grade** ancillary structures ~~and station~~**, respectively**.

The spectral ordinates of the 5% damped response spectra for both ODE and MDE events, for station and ancillary structures, are presented in Table 3-1. ~~For~~**Based on the estimated** $V_{s,30}$ ~~of 920 feet per second (~~for the ~~earth materials beneath foundations of both the~~**station and at-grade** ancillary structures ~~and station),~~**,** a Site Class ~~D~~**"C"** may be used for seismic design **of the station and Site Class "D" may be used for seismic design of the ancillary structures**.

~~The response spectra and Site Class will need to be re-evaluated when additional p and s-wave suspension logging data is available.~~

Table 3-1: Acceleration Response Spectra for Wilshire/Rodeo Station and Ancillary Structures (Ground Surface)

Earthquake Level	Latitude (degrees)	Longitude (degrees)	Structures Structure	Geologic Formations[2]	$V_{s,30}$ [1] (feet/second)	Period (sec):	5% Spectral Accelerations (Sa, in g)						
							0.01	0.10	0.20	0.30	0.50	1.00	2.00
ODE	34.067	-118.399	**At-grade ancillary structures**	Older Alluvium~~/San Pedro/~~	~~920~~[2]**1,120**	Sa (g):	0.~~30~~**31**	0.~~51~~**56**	0.~~64~~**69**	0.~~64~~**67**	0.~~54~~**56**	0.~~34~~**33**	0.~~17~~**16**
MDE						Sa (g):	0.~~83~~**94**	1.~~40~~**54**	1.~~77~~**93**	1.~~82~~**95**	1.~~71~~**78**	1.~~20~~**16**	0.~~64~~**57**
ODE			**Station**	**Older Alluvium/ San Pedro**	**1,500**	**Sa (g):**	**0.28**	**0.52**	**0.64**	**0.59**	**0.46**	**0.26**	**0.12**
MDE						**Sa (g):**	**0.89**	**1.69**	**2.09**	**2.05**	**1.79**	**1.02**	**0.46**

[1] $V_{s,30}$ = shear wave velocity within the 30 meters of earth material below the station bottom or within the 30 meters of material below ground surface for ancillary structures.
~~[2] Seismic shear wave velocities were computed using empirical correlation of shear wave velocity with SPT N1 value (PEER, 2010).~~
[2] The excavations for station and ancillary structures will be within Younger and Older Alluvium, however San Pedro Formation is at a depth of less than 100 feet below the station bottom.

3.2 Time Histories and Spectral Matching

In order to evaluate the free-field displacement for racking analysis of the station per Section 3B8.0 of the Metro Supplemental Seismic Design Criteria of the MRDC, it was required to perform a site-specific site response analysis. For this purpose, spectrum-compatible time histories were developed using the 5% damped ODE and MDE response spectra. The target response spectra for the ODE and MDE events were computed for a stiff soil/soft rock type site condition with an average velocity of 1,~~837~~**500** feet per second (~~560~~**460** meters per second) assumed to be at a depth of about 150 feet bgs. The target response spectra is presented in Table 3-2.

Table 3-2: Target Acceleration Response Spectra (150 Feet bgs)

Earthquake Level	Latitude (degrees)	Longitude (degrees)	Formations	V_s (feet/second)	Period (sec):	5% Spectral Accelerations (Sa, in g)						
						0.01	0.10	0.20	0.30	0.50	1.00	2.00
ODE	34.067	-118.398	San Pedro	1,~~837~~**500**	Sa (g):	0.~~39~~**28**	0.~~50~~**52**	0.~~61~~**64**	0.~~55~~**59**	0.~~41~~**46**	0.~~22~~**26**	0.~~10~~**12**
MDE					Sa (g):	0.~~88~~**89**	1.~~73~~**69**	2.~~15~~**09**	2.~~07~~**05**	1.~~63~~**79**	~~0.89~~**1.02**	0.~~39~~**46**

Recorded seed time histories from prior earthquakes are generally selected so that the earthquake magnitudes, source-to-site distance, fault mechanism and subsurface site conditions of the recording stations are similar to that of the site conditions and the earthquakes anticipated at the station. Three time histories were obtained from the Pacific Earthquake Engineering Research (PEER) Center, ~~Next Generation Attenuation (NGA)~~**NGA** ground motion database for shallow crustal earthquakes in active tectonic regimes. The time histories were selected based on the following factors: geologic and soil characteristics, inclusion of strong directivity or near-source ground motions, and the results of de-aggregation of probabilistic seismic hazard.

Based on the de-aggregation results, the controlling earthquakes for the ODE event have a Magnitude range of 6.~~6~~**5** to 7.~~7~~**5** at a distance of 1.~~2~~**0** to ~~18.5~~**12.0** miles, and for the MDE event, a Magnitude range of 6.5 to 7.5 at a distance of 1.~~2~~**0** to 3.~~1~~**0** miles. It is noted that the Hollywood (Mw = 6.7), Santa Monica

(Mw = 6.6 to 6.8) and Raymond (Mw = 6.8) Faults are relatively close to the station; these faults primarily have strike-slip fault mechanisms (with some component of reverse mechanism). ~~The~~ **Based on the 2008 USGS Fault Model, the** active trace of the Santa Monica Fault is about ~~0.3~~**one** mile to the ~~northwest~~**west** of the station. In addition, the northern extension of the Newport Inglewood fault is located about 0.~~25~~**5** mile ~~east-northeast~~**west-northwest** of the station.

The three time histories selected for spectral matching are presented in Table 3-3**.** Also listed in the table are the site conditions, fault mechanisms and source-to-site distances for each of the recording stations.

Table 3-3: Seed Time Histories

Time History Designation Name	Earthquake	Magnitude (Mw)	Fault Mechanism	Recording Station	Closest Distance to Fault (km)	Recording Component (degrees)*	Recording Station Vs,30 (feet/second)
NGA_752-000	1989 Loma Prieta	6.93	Reverse-Oblique	Capitola	8.65	000	947
NGA_983_292	1994 Northridge	6.69	Reverse	USGS 655 Jensen Filter Plant Generator	5.43	292	1726
NGA_1111_090	1995 Kobe	6.90	Strike-Slip	CUE 99999 Nishi-Akashi	7.08	090	1998

* Indicates orientation of recording station instrument; Reference directions: 000 – North South Direction, 090 – East West Direction, 292 – 292 degrees counterclockwise with respect to reference North

Spectral matching of the seed time histories was performed in the time domain by adding or subtracting wavelets of limited duration to the original time history using the algorithms from program RSPMATCH (Abrahamson, 1998) incorporated into EZ-FRISK.

3.3 Site Response Analysis and Free-Field Differential Displacement

As stated in Section 3B8.1.3 of the Metro Supplemental Seismic Design Criteria of the MRDC, the racking analyses of the station can be evaluated using any of the following methods depending on the level of assessment required to be used for the particular structural configuration being analyzed.

- Simplified Pseudo-Static Method
- Simplified Dynamic Method
- Two Dimensional Dynamic Method
- Fully Coupled Two Dimensional Finite Element or Finite Difference Method

For the purpose of preliminary design, a simplified pseudo-static method is utilized herein wherein a free-field displacement of the top of the station relative to the station bottom is obtained and a pushover analysis is performed using the displacement profile to determine structural response under seismic loading. Other methods as listed above are also applicable for some types of structural configuration and may be able to be utilized, or may be required to be used.

In order to obtain the free-field displacement (profile), site response analyses were performed using the one-dimensional equivalent linear program SHAKE91 (Idriss and Sun, 1992) to estimate free-field displacement of the soil column between the top and bottom of the station. Site response analyses were performed using ground motions for ODE and MDE events following the Metro Seismic Design Criteria of the MRDC for design.

~~As stated earlier, shear-wave measurements were not obtained in any of the explorations performed at the Wilshire/Rodeo station in the ACE or PE phases.~~ Shear-wave velocities of the materials were estimated using ~~empirical correlations of shear-wave velocity with Standard Penetration Test (SPT) blow counts obtained in several borings and using correlations proposed in PEER (2010).~~**suspension logging data obtained in boring G-408/P-306.** A generalized shear-wave velocity profile was then derived and used in site response analysis. A 150-foot thick soil column was considered in the one-dimensional model. Based on available geologic information, the shear-wave velocity was determined to be 1,~~837~~**500** feet per second (~~560~~**460** meters per second) for the transmitting base at a depth of 150 feet bgs.

Three appropriate spectrum-matched acceleration time histories, as discussed in Section 3.2, were used as the input outcropping motions in the model. The time histories of shear strains were obtained in each of the layers within the zone of interest (between the top and bottom of the box) for the station racking and then the displacement time histories were computed in these soil layers. The equivalent displacement time history within the zone of interest was estimated by adding displacement time histories in each of these layers. The peak value in the displacement time history was taken as the estimated peak free-field displacement between the top and bottom of the station. The free-field maximum differential displacements computed for the ODE and MDE events in this manner are presented in Table 3-4. The values in the table may be linearly extrapolated for other structure heights.

Table 3-4: Free-Field Displacement (ODE and MDE)

Station Name	Free-Field Displacement (inches)	
	ODE	MDE
Wilshire/Rodeo	0.~~40~~**25** inch in vertical 55 feet	~~2.50~~**1.4** inch in 55 vertical feet

~~Additional explorations consisting of rotary-wash boring with primary and shear-wave (p and s-wave) suspension logging are planned to be performed in the Adv. PE phase. The free-field displacement values provided above should be considered as preliminary and will need to be re-evaluated when shear-wave velocities are available from the planned borings.~~

3.3.1 ~~Lateral~~ Spring Stiffness

The Lateral and vertical soil spring stiffnesses were estimated for use in two-dimensional and/or three-dimensional structural analysis of the station. The lateral springs can be used to model the stiffness of the soils retained by the below-grade walls of the station, both in the longitudinal and transverse directions. The vertical springs can be used to model the soils supporting the station foundations. The procedure to perform a numerical deformation analysis using the lateral and vertical springs is presented in Section 3B8.1.3 of the Metro Supplemental Seismic Design Criteria of the MRDC.

3.3.1.1 Lateral Spring Stiffness

The lateral stiffnesses were estimated using equations published in FEMA 356/ASCE 41 for foundations in translation in the x- and y-directions. The method uses the shear wave velocity, shear modulus, Poisson's ratio of the surrounding medium and station dimensions (length, width and height) in estimating the spring stiffnesses.

For the ODE and MDE events, the ratio of degraded to small-strain shear wave velocity was estimated as 0.92 and 0.78, respectively, using the guidelines provided in Table 19.2-1 of FEMA P- 750 (2009). For computation of degraded shear modulus, these ratios will be 0.85 and 0.61, for ODE and MDE, respectively using the relationship between the shear wave velocity and shear modulus. Based on the small-strain shear wave velocity of the soils adjacent to the station, a Site Class D was used in computing the degradation factor. Due to the variation of shear wave velocity with depth and along the length of the station, a range (lower-bound and upper-bound) and a best estimate of the shear wave velocity were used in spring stiffness computations. The estimated average lateral spring stiffness for small-strain, ODE, and MDE events are presented in Table 3-5. Depending on the direction of the structural analysis performed (longitudinal or transverse), the spring stiffness in kips per cubic foot (kcf) may be multiplied by the station side area to estimate the spring stiffness in units of kips per foot (kip/ft).

Table 3-5: Lateral Soil Spring Stiffness (Small Strain, ODE, and MDE)

Value	Small Strain	ODE	MDE	Small Strain	ODE	MDE
	Kx (kcf)[a]	Kx (kcf)[a]	Kx (kcf)[a]	Ky (kcf)[b]	Ky (kcf)[b]	Ky (kcf)[a]
Average	~~18~~**13.3** z	~~15~~**11**.2 z	~~10.~~8**.1** z	1.~~5~~**2** z	1.~~2~~**0** z	0.~~9~~**7** z
Upper Bound	~~21.3~~**19.2** z	~~18.0~~**16.2**	~~12.8~~**11.7** z	1.7 z	1.4**5** z	1.~~0~~**1** z
Lower Bound	~~14~~**9**.5 z	~~12.2~~**8.0** z	**5**.8~~.7~~ z	~~1.2~~**0.9** z	~~1.~~0**.7** z	0.~~7~~**5** z

[a] Distribute over area of end wall (width * height), unit stiffness increases with depth (z, depth below ground surface in feet)
[b] Distribute over area of side wall (length * height), unit stiffness increases with depth (z, depth below ground surface in feet)

The variable "z" in Table 3-5 is depth below ground surface. Spring stiffness above the top of the station walls should be ignored. The x-axis and y-axis are oriented in the length and width directions of the station, respectively; Kx and Ky represent springs in the x- and y-directions. For the computations presented herein, the length and width of the station presented in Table 1-1 were used. The approximate top of the station wall for the arch roof module is also presented in Table 1-1. ~~It is noted that the~~**The** spring stiffness presented in ~~the above table~~**Table 3-5** will need to be revised if the station dimensions ~~change~~**are changed**.

As stated earlier, the ratio of degraded modulus to small strain modulus is 0.~~61~~**85** (about ~~94~~**25**% degradation from the small-strain value) is estimated for the ODE event, therefore it is suggested that the spring stiffness values estimated for the ODE event be used for the static condition as well. If the MDE case is analyzed, the respective spring stiffness values presented in Table 3-5 should be used.

The dynamic spring stiffnesses provided herein are based on shear wave data which are not affected by the presence of groundwater, except possibly by pore pressures generated in soil during earthquakes. Dynamic spring stiffnesses provided herein are applicable for both current and design (historically-highest) groundwater conditions. Furthermore, liquefaction hazard at the station location is considered low and therefore, the dynamic lateral spring stiffnesses presented in Table 3-5 are not anticipated to be reduced any further.

~~3.3.1.1~~3.3.1.2 Vertical Spring Stiffness

Vertical stiffness was computed using equations published in FEMA 356/ASCE 41 for translation in the z-direction. Soil shear wave velocities below the station bottom were used in estimating the vertical stiffness. Due to the variation of shear wave velocity with depth below the station foundation and along the length of the station, a range (lower-bound and upper-bound) and a best estimate of the shear wave velocity were used in spring stiffness computations.

For the ODE and MDE events, the ratio of degraded to small-strain shear wave velocity was estimated as 0.92 and 0.78, respectively, using the guidelines provided in Table 19.2-1 of FEMA P- 750 (2009). Based on the small-strain shear wave velocity of the soils, a Site Class ~~D~~**C** was used in computing the degradation factor.

Table 3-6 provides lower bound, upper bound and average vertical stiffnesses for small strain, ODE and MDE events. The spring stiffness in kips per cubic feet (kcf) may be multiplied by the mat foundation area to estimate the spring stiffness in units of kips per foot (kip/ft). For the computations presented herein, the length and width of the station foundation stated in Table 1-1 were used. ~~It is noted that the~~**The** spring stiffness presented in ~~the above table~~**Table 3-6** will be need to revised~~,~~ if the station dimensions ~~change~~**are changed**.

Table 3-6: Vertical Spring Stiffness (Small Strain, ODE and MDE)

Value	Small Strain kz (kcf)	ODE kz (kcf)	MDE kz (kcf)
Average	~~210~~**255**	~~180~~**216**	~~130~~**156**
Lower Bound	~~250~~**505**	~~210~~**426**	~~150~~**309**
Upper Bound	~~170~~**183**	~~145~~**154**	~~100~~**112**

4.0 DESIGN AND CONSTRUCTION

This section provides a summary of the geotechnical evaluation of the subsurface conditions at the station site and their impact on the design and construction of the proposed station and ancillary structures.

4.1 Geotechnical Considerations

Based on the plan and profile dated ~~September~~**December,** 2015, the Wilshire/Rodeo Station will be excavated primarily within Holocene-age Younger Alluvium and Quaternary-age-Older Alluvium. The soils in this formation predominantly consists of medium stiff to stiff clays and silts interlayered with medium dense to dense silty sands and sands. Excavation in these soils can be performed using conventional earth-moving equipment. Although boulders were not encountered in the borings, cobbles may be present and if encountered they can be excavated by conventional earth moving equipment. Caving of granular materials should be anticipated during installation of soldier piles and tieback anchors, particularly below groundwater depth. The ground conditions anticipated in the station excavation are quantitatively described in the Geotechnical Baseline Report (GBR).

Certain challenges will have to be addressed in conjunction with station excavation, such as the presence of major utility lines crossing the station footprint and the presence of existing tieback anchors from former basement constructions that protrude into the planned excavation. Based on the project-specific utility maps, a number of existing utilities such as storm drains, sewers, electrical conduits, and telecommunication lines are located within the upper 10 to 15 feet of the ground surface. All utilities will have to be carefully protected in place or relocated where possible.

4.2 Groundwater Levels

Groundwater was encountered in prior borings and **in** borings ~~drilled for WPLE project~~**and monitoring wells installed** at the Wilshire/Rodeo Station. In the prior **historic** borings at and near the station, groundwater was encountered at depths of about 40 to 64 feet below ground surface (bgs). In the monitoring ~~well~~**wells** installed for WPLE project **at the station**, groundwater seepage ~~was encountered at~~ **(or)** a ~~depth~~**perched groundwater condition was encountered at depths** of about 32 **to 57** feet in the shallower ~~screen~~**screens** [27.5 to ~~32.5~~**60** feet bgs]. In the deeper screen ~~interval [55~~**intervals [61** to ~~60~~**118** feet bgs], groundwater was encountered at depths of about ~~53.5~~**56** to ~~57~~**64** feet bgs.

It is believed that the groundwater levels encountered in the ~~prior~~ borings and monitoring wells indicate that a perched or semi-perched groundwater condition exists at the site~~. The primary water bearing zone is believed to be the San Pedro Formation which is located at least 30 to 40 feet below the planned station bottom.~~ **within the upper 60 feet. The depth to the true (non-perched) groundwater level at the station site is estimated to be at a depth of about 60 to 70 feet bgs.**

4.2.1 Design Groundwater Level

A groundwater-level contour map of the Beverly Hills Quadrangle Seismic Hazard Report (CGS, 1998) shows the historically highest groundwater level is at approximately 30 to 40 feet bgs at the station location. In addition, as stated earlier, water seepage as shallow as 32 feet bgs was encountered in the borings. Therefore, the design groundwater level should be taken as 30 feet bgs for station **permanent** design. Note that the elevation of the design groundwater level varies in parallel to the variations in the ground surface elevation. **The groundwater depth for the temporary (construction) condition should be taken as 60 feet bgs across the station length.**

4.3 Seismic Design Considerations

According to the California Geological Survey (CDMG, 1998), the Wilshire/Rodeo Station site is not within an area identified as having a potential for liquefaction due to the presence of older alluvial deposits (Pleistocene age) beneath the site. Furthermore, the soils below the station foundation consist of relatively dense sands and stiff silts and clays. Therefore, the potential for liquefaction in the soils below the station bottom is considered to be low.

Although artificial fill soils were encountered in the upper 5 to 10 feet, the fill soils were primarily fine grained and not susceptible to liquefaction. Furthermore, these soils are above the historically highest ground water level and are not anticipated to be saturated during the station design life. Therefore, the potential for liquefaction at the station entrance structures is considered to be low.

The estimated PGA and Peak Ground Velocity (PGV) for ODE and MDE events for the station are presented in Table 4-1. The estimated free-field displacements over the station height for use in racking analysis are presented in Table 3-6.

4.4 Excavation Methods

Excavations as deep as ~~85~~**90** feet will be required for the station construction. Due to proximity of station excavation to existing buildings and limited construction space within the public right-of-way, shoring will be required. Shoring systems such as soldier piles with wood or shotcrete lagging, secant piles, and slurry walls supported by tieback anchors and/or internal bracing with struts and walers may be used. Based on traditional methods used in the area, the station excavation support is assumed to be internally braced with struts.

Based on the subsurface data obtained to date, a conventional soldier pile and lagging shoring system is considered feasible. A secant/tangent pile or slurry wall system could also **be** considered for the purpose of creating a relatively water-tight shoring system and to reduce the water inflows into the excavation and to eliminate significant dewatering and dewatering-induced settlement. Note that if it is desired to create a relatively water-tight shoring system, then an evaluation of groundwater inflows should also be made for the bottom of excavation. There are indications that a fine-grained layer (aquitard) may be present at a depth which would reduce groundwater inflows through the bottom of the excavation; the presence of such a layer would need to be carefully evaluated before relying on the existence of such a layer.

4.5 Dewatering and Groundwater Control

~~The groundwater levels were measured at depths of about 40 to 64 feet in the prior borings and monitoring wells; water seepage at a depth of about 32 feet bgs was also noted in the one monitoring well installed at the station. The proposed excavation will extend about 30 to 45 feet below the current water levels. As stated earlier, the groundwater levels encountered in the prior borings and monitoring wells indicate that a perched or semi-perched groundwater condition exists at the site. The primary water bearing zone is believed to be the San Pedro Formation which is located at least 30 to 40 feet below the station bottom. The pumping test proposed in the Adv. PE phase is still pending; based~~**The proposed excavation will extend about 20 to 30 feet below the true (non-perched) water level. Based** on the subsurface stratigraphy and groundwater levels observed in the pumping test borings and prior borings, it appears that groundwater inflows into station excavation can be controlled with strategically located ~~deep~~ dewatering wells supplemented ~~by~~**with** gravel filled trenches and sump pumps~~. However, a more definitive recommendation can be made once the pumping test results are evaluated~~.

4.5.1 Estimated Groundwater Inflows

The subsurface geology at the station site is heterogeneous and is comprised of variable interbedded clays, silts and sand units. Groundwater inflows into the station excavation will result from **saturated material of Older Alluvium below a depth of 60 feet bgs; in addition some water inflows should be expected from** the perched or semi-perched granular zones~~. As stated earlier,~~ **in** the **upper 60 feet.**

Considering the complexities of the stratigraphy and hydrogeology at the planned station location, and the fact that only one pump test has been performed a reasonable range in K_h values was estimated and used in simplified drawdown model analyses to provide preliminary dewatering estimates (see Pumping Test Report dated February, 2016) Based on the analyses, a maximum short-term (first few months of pumping) dewatering flow rate of approximately 250 to 400 gallons per minute (gpm) and long-term dewatering flow rate of approximately 150 to 300 gpm are estimated. Depending on the assumptions used for estimating hydraulic conductivity, dewatering values could be

estimated to be different than these. For planning and/or design purposes, we suggest that the upper limits in both short-term and long-term cases be used.

The primary water bearing zone **beneath the station site** is San Pedro Formation which is at ~~depth~~**a depth of 15 to 55 feet below the station bottom**; station excavations or shoring soldier piles ~~will~~**are** not **anticipated to** extend ~~in to~~**into** this formation. Groundwater inflow will also occur through the bottom of the excavation, but this inflow may be significantly reduced considering the potential fine-grained aquitard layer(s) below the planned bottom of excavation; if the groundwater inflow is assumed to be reduced due to the presence of one or more aquitard layers, then a careful evaluation of that (those) layer(s), perhaps including additional explorations, will need to be performed. ~~.~~

~~A pumping test is currently planned in the Adv. PE phase investigation and results will be evaluated, when available to estimate water inflow rate. A qualified dewatering contractor with experience in similar subsurface conditions should also be consulted.~~

4.5.2 Estimated Dewatering Induced Ground Settlement

The dewatering for the station will result in settlement as the effective stress in the materials below the current groundwater level is increased. The dewatering will only have a significant reduction in pore pressure in the coarser-grained layers within the station excavation; there are aquitard layers of clay at or just below the station bottom which will prevent pore water pressure decrease and hence will prevent dewatering-induced compression of layers below the bottom of excavation. Based on the laboratory consolidation testing of the fine-grained soils in the soils from the ground surface to the level of the bottom of the station at the station, and elastic compression of the coarse-grained materials based on in-situ testing at the station, we have estimated the dewatering-induced settlement to be about 0.2 inch immediately adjacent to the station, decreasing away from the station, depending on the dewatering surface drawdown contours. A further evaluation of potential dewatering-induced settlement will need to be made once details of the dewatering system and timing of the excavation/dewatering are determined.

4.6 Ground Heave and Basal Stability

The station site should be dewatered to maintain the groundwater level at least 5 feet below the excavation bottom to achieve a factor of safety (defined as the ratio of critical hydraulic gradient to maximum flow exit gradient) of 2.0 against basal stability. The soils at the excavation level are not expected to heave significantly upon excavation. However, an evaluation will need to be made of the groundwater inflow and hydrostatic pressure on the bottom.

4.7 Excavation Support

4.7.1 Geotechnical Design Parameters

The following sections provide general preliminary recommendations for the design and construction of shoring braced with internal struts/walers for the station and cantilever shoring walls for ancillary structures.

4.7.2 Lateral Earth Pressures

Shoring up to ~~85~~**90** feet deep will be required for support of station excavation. For **station entrance** structures~~ such as station entrances~~, shoring as deep as ~~70~~**75** to ~~80~~**85** feet near the station to 10 to 20-foot deep shoring at the entry point will also be required.

For design of braced shoring system (internally braced with struts or tiebacks or both), the use of a trapezoidal distribution of earth pressure is recommended. If a slurry wall or undrained wall system is used, design should be based on at-rest earth pressures recommended for rigid shoring with a trapezoidal earth pressure distribution. In ~~additional~~**addition** to the earth pressure distribution, hydrostatic pressure as described in Section 4.7.3 and surcharge loading as described in Section 4.7.4 should be applied. Consideration of appropriate surcharge pressure is important considering the proximity of the anticipated excavation to adjacent heavily-loaded structures.

For excavations up to 15 feet or less anticipated for ancillary structures, the use of cantilever shoring may be an economical option. For design of cantilever shoring, a triangular distribution of lateral earth pressure may be used.

The geotechnical parameters and earth pressure coefficients required for design of shoring **and numerical modeling** are presented in Table 4-1. Recommended earth pressure distributions for design of shoring are shown in Figure 4-1.

The earth pressure distributions presented in Figure 4-1 assume a level backfill. If the ground surface retained by shoring is sloped, the increase in earth pressure will need to be evaluated on a case-by-case basis.

4.7.3 Hydrostatic Pressures

Permeable shoring systems such as soldier pile and lagging system need not be designed for hydrostatic pressures. If shotcrete is used instead of wood lagging, weep holes should be placed to provide drainage of the retained soils. If weep holes are provided, hydrostatic pressures need not be considered in the design. If water-tight shoring systems, such as secant/tangent piles or a slurry wall shoring system is used, hydrostatic pressures as shown in Figure 4-1, should be added to the lateral earth pressures described above. The design groundwater level should be taken as 30 feet bgs.

4.7.4 Surcharge Pressures

In addition to the lateral earth pressures presented in the previous sections, shoring should be designed to resist a uniform lateral pressure of 100 pounds per square foot due to American Association of State Highway and Transportation Officials (AASHTO) HS20 traffic loading. Applicable surcharge pressures from adjacent buildings and foundations of minor structures should be estimated and added to the earth pressures. Consideration of appropriate surcharge pressure is important considering the proximity of the anticipated excavation to adjacent heavily-loaded structures. Surcharge pressures from heavily loaded construction cranes and other traffic should be added as well. Surcharge pressures should be estimated in accordance with recommendations presented in Metro Rail Standard Structural Drawings (Metro, 2014). ~~Also, refer to the Building Protection Report.~~

4.7.5 Seismic Earth Pressures

The Metro Seismic Criteria does not provide specific recommendation for computing seismic earth pressures for temporary shoring, but Metro standard drawing SS-003 presents a guideline for seismic earth pressure due to retained soil and due to adjoining building(s). Considering that the shoring will be in-place for more than 2 to 3 years, it is recommended that seismic earth pressure due to retained soil be used for the full height of the shoring. Seismic earth pressure due to adjoining buildings should be in accordance with Metro standard drawing SS-003 for shoring design.

The increment of seismic lateral earth pressures due to retained soil should be computed using the seismic earth pressure coefficient increment (K_{AE}) provided in Table 4-1, which was based on PGA for the ODE event. A seismic coefficient equal to ½ of the computed PGA was used in the computations. The equivalent uniform earth pressure may be computed by taking the same resultant force as computed from the triangular equivalent fluid pressure distribution, as shown in Figure 4-1. Using these design parameters, a seismic earth pressure increment is estimated to be equal to 6H pounds per square foot (uniform), where 'H' is the shoring height.

For shoring, this seismic earth pressure is an incremental value intended to be added to the static active earth pressure value, and not the design shoring lateral earth pressure (K) or the at-rest earth pressure (K_o). All required load combinations of static and seismic, with appropriate load factors (NCHRP, 2009), should be utilized in the final design of shoring walls.

4.7.6 Design of Soldier Piles

For the design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils above groundwater level, may be assumed to be 600 pounds per square foot per foot of depth (pcf) below the excavated surface, up to a maximum of 6,000 pounds per square foot. For soils below groundwater level, the allowable lateral value (passive value) of the soils below the level of excavation may be assumed to be 300 pounds per square foot per foot of depth at the excavated surface, up to a maximum of 3,000 pounds per square foot. The passive values include a multiplication factor of 1.5 as recommended by Metro to account for the three-dimensional effects of the passive wedge. A one-third increase in the lateral bearing value may be used when considering seismic and other transient loads for ancillary structures.

To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile, which is below the planned excavated level, should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils.

Provided that the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. For resisting downward loads, the frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken equal to at least ~~250~~**500** pounds per square foot for short embedment of the toe of soldier beams; for longer soldier beam embedment, the friction may be computed using an appropriate pile capacity computation, assuming that the ground is saturated below the base of excavation, and using a surcharge pressure appropriate to the stress conditions at the edge of the station excavation.

4.7.7 Lagging

The soldier piles, struts, and anchors should be designed for the full anticipated lateral pressure. Continuous lagging will be required between the soldier piles. The lagging should be designed in accordance with the drawing for Cut and Cover Underground Structures, titled "Construction Structures Loads and Design Criteria." If shotcrete is used, weep holes should be provided to relieve hydrostatic pressures.

4.7.8 Anchor Design

Installing tieback anchors in the project area will likely require permission from local agencies and owners of adjacent properties and avoidance of underground obstruction such as basements, foundations, and utility lines. Tieback anchor in the public right-of-way will require removal in accordance with local jurisdiction requirements.

Tieback friction anchors may be used to resist lateral loads. For computation purposes, it may be assumed that the unbonded zone adjacent to the shoring is defined by a plane drawn at 30 degrees with the vertical through the bottom of the excavation (as shown on Metro Rail Structural Standard Cut & Cover Underground Structures Drawing No. SS-004). The anchors should extend at least 25 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

The capacities of anchors should be determined by testing of the initial anchors as outlined on Metro Rail Structural Standard Cut & Cover Underground Structures Drawing No. SS-004. Pressure-grouted anchors in the upper 40 feet will develop an average friction value of 1,500 pounds per square foot and a friction value of 2,000 pounds per square foot for anchors below this depth. For anchors 8 inches in diameter, this value corresponds to a bond strength of about 3,000 pounds per lineal foot for anchors in the upper 40 feet and 4,200 pounds per lineal foot for anchors below this depth. Non-pressure-grouted anchors in the upper 40 feet will develop an average friction value of 500 pounds per square foot and a friction value of 650 pounds per square foot for anchors below this depth. For anchors 12 inches in diameter, this value corresponds to a bond strength of about 1,500 pounds per lineal foot for anchors in the upper 40 feet and 2,000 pounds per lineal foot for anchors below this depth.

It is noted that the friction values provided above are based on the use of conventional tieback installation equipment. Some reduction of frictional resistance should be anticipated if undue remolding of the soils is caused by contractor's choice of installation method, particularly in clayey type soils below groundwater.

Existing tieback anchors left (but no longer relied-upon) from construction of the adjacent buildings will protrude into the station excavation. The tieback anchors are likely de-tensioned with the anchor head embedded in the basement wall. Care should be taken to not apply excessive pressure (pull) on the anchor during its abandonment. The tieback anchors should be cut using welding torches, or can be cut using a core barrel (within a soldier pile excavation) within the limits of the station and **station** entrance ~~structure~~**structures**.

4.7.9 Internal Bracing

Internal struts and walers may be used to internally brace the soldier piles. The strut loads should be determined based on the lateral earth pressures for braced condition as shown in Figure 4-1. The vertical spacing between the struts should be designed to reduce ground movements. All struts should be tightly fitted to eliminate any slack and to reduce ground movement. If necessary to reduce shoring deflection, a preload of 25% of the design load may be used.

Procedures to compensate for the effects of temperature changes on the strut loads should be developed and implemented so that proper strut load levels can be monitored and maintained during construction.

4.7.10 Shoring Deflection and Ground Settlement

The amount of deflection of a shored embankment is dependent on the flexibility of the shored wall, excavation methods, and spacing of support members such as soldier piles, struts, etc. It should be realized, however, that some deflection will occur. Deflection of braced shoring is highly dependent on strut spacing, strut stiffness, strut preloading, and soldier pile spacing and stiffness. Other mitigations for shoring deflection that can be considered include the increased use of tie-back anchors.

If greater deflection than anticipated occurs during construction, additional bracing may be considered to minimize settlement of the adjacent buildings and utilities in the adjacent streets. If it is desired to reduce the deflection of the shoring, a greater earth pressure could be used in the shoring design, such as a design based on an at-rest pressure condition. In any event, the design for shoring will need to meet the deflection criteria specified for the project by Metro~~. Refer to the Building Protection Report.~~

4.7.11 Monitoring

Some means of monitoring the performance of the shoring system is recommended. The instrumentation could consist of inclinometers, ground settlement monuments as well as load cells and strain gages placed on the struts and soldier piles.

The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops and intermediate points of all the soldier piles as well as installation and regular readings of inclinometers and regular readings of load cells and/or strain gages. Depending on the proximity of the adjacent structures and utilities, a specific instrumentation and monitoring program should be planned and implemented prior to the commencement of station excavation.

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Table 4-1: Preliminary Geotechnical Design Parameters for Wilshire/Rodeo Station

Parameter	Value	2009 through 2015 Geotechnical and Environmental Investigations Estimated Range of Engineering							
Geologic Unit		Artificial Fill (Af)		Quaternary Younger Alluvium (Qal)		Quaternary Older Alluvium (Qalo)		San Pedro Formation (Qsp)	
		Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained
Dry Unit Weight of Soil (pcf) [1]	~~109~~108	*	*	~~89~~85 to 118 (~~103~~102)	99 to 122 (112)	~~96~~90 to 130 (110)	98 to 129 (113)	~~112~~109 to ~~118 (115~~122 (114)	101 to 126 (~~113~~114)
Total Unit Weight of Soil (pcf)	~~127~~126	*	*	~~106~~101 to 134 (~~122~~121)	120 to 137 (129)	~~120~~107 to ~~141~~147 (130)	104 to 140 (~~127~~126)	116 to ~~137 (126~~138 (131)	122 to 136 (~~127~~129)
Static Elastic Modulus from SPT Correlation (ksf) [2a]	Varies	*	*	~~82~~350 to ~~2,744 (930~~1,150 (728)	-586	~~1211~~300 to ~~6,184 (3,019~~8,775 (2,424)	251 to ~~6,266 (3341~~7,331 (3,445)	~~3,888~~2,369 to ~~14,940 (7,158~~6,935 (3,431)	3,195 to ~~6,266 (4,981~~7,936 (5,486)
Static Elastic Modulus from Triaxial (ksf) [2b]	Varies	*	*	~~1,295~~1295	1431~~1,431~~	~~1810~~1,810 to 5,170 (3,~~506 (2,738~~060)	~~*~~2,363 to 6,170 (4,179)	*	*
Friction Angle (Degrees) [3]									
Field	25**	*	*	18 to 34 (25)	*	*	*	*	*
Saturated	30	*	*	18 to 34 (26)	31**	18 to 43 (~~30~~28)	27 to 40 (~~32~~33)	24**	12**
Cohesion (psf) [3]									
Field	~~1533~~1,533**	*	*	~~0~~900 to 2,700 (~~1533~~1,533)	*	*	*	*	*
Saturated	~~540~~761	*	*	~~350~~150 to 2,700 (~~1238~~1,020)	370**	50 to ~~2,200 (739~~2200 (791)	150 to ~~1,850 (645~~1850 (600)	1250**	950**
Unit Subgrade Modulus (k) (kcf) [4]						~~210~~200 (small strain); ~~175~~180 (ODE); ~~125~~150 (MDE)			
Allowable Bearing Value (psf)	2,000[5], 8,000[6]-					~~8,000 psf~~			
Coefficient of Friction (µ) [7]	0.~~36~~37	*	*	0.21 to 0.41 (0.31)	0.38~~ to 0.38 (0.38)~~**	0.21 to 0.55 (0.~~36~~34)	0.32 to 0.49 (0.~~40~~41)	0.29	0.14
Soil Pressure Coefficient, At-Rest Ko									
Bored Tunnel Section [8]	0.~~57~~60	*	*	*	*	0.~~45~~49 to 0.~~68~~69 (0.~~57~~60)	*	*	*
Underground Station [9]	0.48	*	*	0.96 to 1.05 (1.00)	0.99**	0.59 to 0.88 (0.71)[9a] 0.45 to 0.80 (0.58)[9b]~~0.48~~	0.~~47~~68 to 0.~~64~~83 (~~0.48)~~74) [9a] 0.49 to 0.73 (0.66) [9b]	0.65**	*
Soil Pressure Coefficient, Active Ka [10]	0.~~32~~33	*	*	0.29 to 0.44 (0.36)	0.32**	0.19 to 0.~~41~~44 (0.~~33~~35)	0.22 to 0.38 (0.~~31~~30)	0.42**	*
Soil Pressure Coefficient, shoring K [11]	0.~~37~~38	*	*	0.33 to 0.51 (0.42)	0.37**	0.22 to 0.~~48~~51 (0.~~38~~41)	0.26 to 0.44 (0.~~36~~34)	0.49**	*
Soil Pressure Coefficient, Passive Kp [10]	3.~~09~~21	*	*	~~1.89~~2.28 to 3.46 (2.~~63~~88)	3.12**	~~1.89~~2.28 to 5.29 (~~3.07~~2.94)	2.61 to 4.50 (3.~~36~~47)	2.37**	*
Soil Pressure Coefficient, Seismic Ke [12]	0.11								
Ground surface elevation (ft) [13]	Varies	218.9 to 230.3							
Groundwater Elevation (ft) [14]	Varies	~~160.0 to 199.0~~							
Corrosivity Results [15]									
Minimum Resistivity (ohm-cm)	~~900~~811	*	*	960 to ~~960 (960~~1,480 (1,225)	*	~~900~~811 to ~~2,400 (1454~~3,560 (1,673)	2,440 to 4,040 (2,960)	1,000**	*
pH	~~4~~6.7	*	*	6.7~~.6~~ to 7.6~~ (7.6~~ (7.4)	*	4. to 8.2 (~~7.8 (7.3~~6)	7.2 to 7.8 (7.6)	8.3**	*
Chloride Content (ppm or mg/kg)	~~120~~180	*	*	~~332~~ to ~~33 (33~~54 (29)	*	~~123~~ to ~~120 (67~~180 (43)	469 to ~~104 (64~~15 (12)	~~122~~10**	*
Sulfate Content (ppm or mg/kg)	~~21~~204	*	*	~~233~~ to ~~2 (2~~103 (58)	*	312 to ~~21 (13~~204 (86)	946 to ~~15 (12~~104 (64)	~~10~~122**	*
Poisson's Ratio [16]									
Unsaturated	0.37	*	*	0.31 to 0.41 (0.37)	*	*	*	*	*
Saturated	0.34	*	*	0.38**	0.29**	0.23 to 0.41 (0.35)	0.~~27~~24 to 0.36 (0.~~32~~31)	0.37**	0.44**
Liquefaction Potential (Yes or No) [14]									
Above Station bottom	NO								
Below Station bottom	NO								
Dynamic Elastic Modulus (ksf) [17]									
Small Strain (Initial)	~~17,128~~16,198	*	*	~~13,773~~5,627	*	~~*~~6,520 to 34,554 (19,104)	~~15,954~~13,185 to ~~33,872 (17,753~~50,121 (23,073)	~~13,996~~22,201 to ~~33,192 (18,581~~53,187 (29,633)	~~14,690~~26,369 to ~~30,906 (16,053~~63,873 (35,405)
ODE	~~11,804~~13,671	*	*	~~9,492~~4,749	*	~~*~~5,503 to 29,164 (16,124)	~~10,995~~11,128 to ~~23,344 (12,235~~42,302 (19,474)	~~9,646~~18,738 to ~~22,875 (12,805~~44,890 (25,011)	~~10,124~~22,255 to ~~21,299 (11,063~~53,908 (29,882)
MDE	~~1,713~~12,667	*	*	~~1,377~~4,400	*	~~*~~5,099 to 27,021 (14,939)	~~1,595~~10,311 to ~~3,387~~	~~1,400~~17,361 to ~~3,319~~	~~1,469~~20,620 to ~~3,091~~

Parameter	Value	2009 through 2015 Geotechnical and Environmental Investigations Estimated Range of Engineering							
Geologic Unit		Artificial Fill (Af)		Quaternary Younger Alluvium (Qal)		Quaternary Older Alluvium (Qalo)		San Pedro Formation (Qsp)	
		Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained
							~~(1,775~~39,194 (18,043)	~~(1,858~~41,592 (23,173)	~~(1,605~~49,948 (27,687)
Shear Wave Velocity (fps) [17]									
Small Strain (Initial)	1,~~274~~222	*	*	~~1,146~~730	*	~~*~~810 to 2,382 (1,299)	1,~~234~~100 to 2,~~541~~561 (1,~~300~~419)	1,~~156~~390 to ~~2,508~~3,049 (1,~~325~~607)	1,~~184~~520 to ~~2,427~~3,347 (1,~~237~~763)
ODE	1,~~058~~122	*	*	~~952~~671	*	~~*~~744 to 2,188 (1,193)	1,~~024~~011 to 2,~~110~~353 (1,~~079~~304)	~~959~~1,277 to 2,~~082~~801 (1,~~100~~477)	~~983~~1,396 to ~~2,015~~3,074 (1,~~027~~620)
MDE	~~403~~1,080	*	*	~~362~~646	*	~~*~~716 to 2,106 (1,149)	~~390~~973 to ~~804 (411~~2,265 (1,255)	~~365~~1,229 to ~~793 (419~~2,696 (1,421)	~~374~~1,344 to ~~768 (391~~2,959 (1,559)

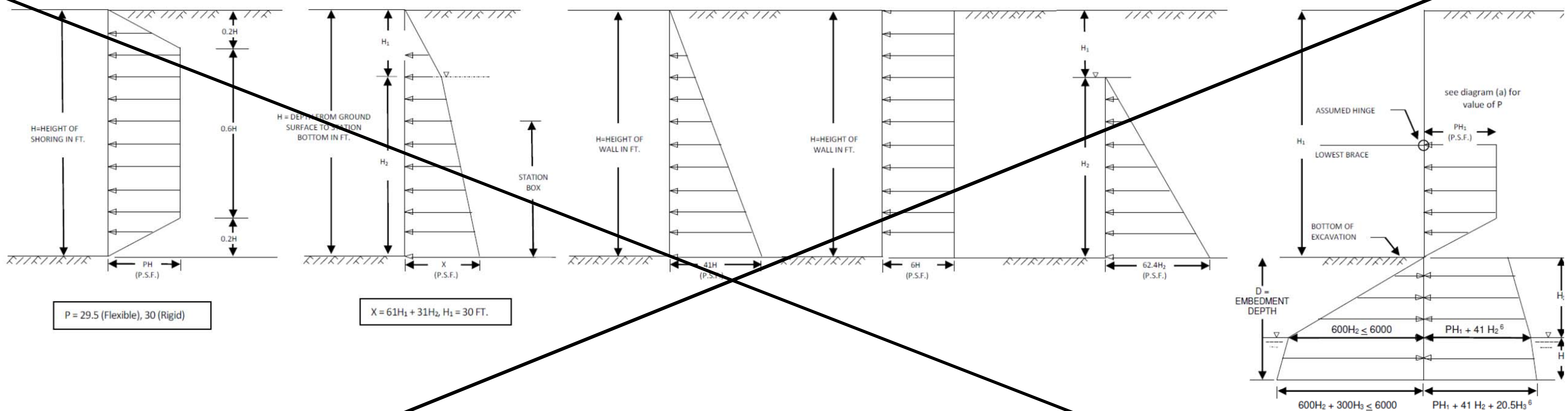
Table 4-1 (Continued): Preliminary Geotechnical Design Parameters for Wilshire/Rodeo Station

Parameter	Recommended Value	2009 through 2015 Geotechnical and Environmental Investigations Estimated Range of Engineering Parameters							
Geologic Unit		Artificial Fill (Af)		Quaternary Younger Alluvium (Qal)		Quaternary Older Alluvium (Qalo)		San Pedro Formation (Qsp)	
		Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained	Fine-Grained	Coarse-Grained
Peak Ground Accel. (g)- Horiz. [18]									
ODE	0.~~28~~**29**								
MDE	0.~~89~~**87**								
Peak Ground Velocity (fps)- Horiz. [19]									
ODE	1.~~23~~**55**								
MDE	~~4.82~~**6.02**								
Free-Field Displacement for Station [20]									
ODE	~~0.4 inch in 55 vertical feet~~**0.25**								
MDE	~~2½ inches in 55 vertical feet~~**1.40**								

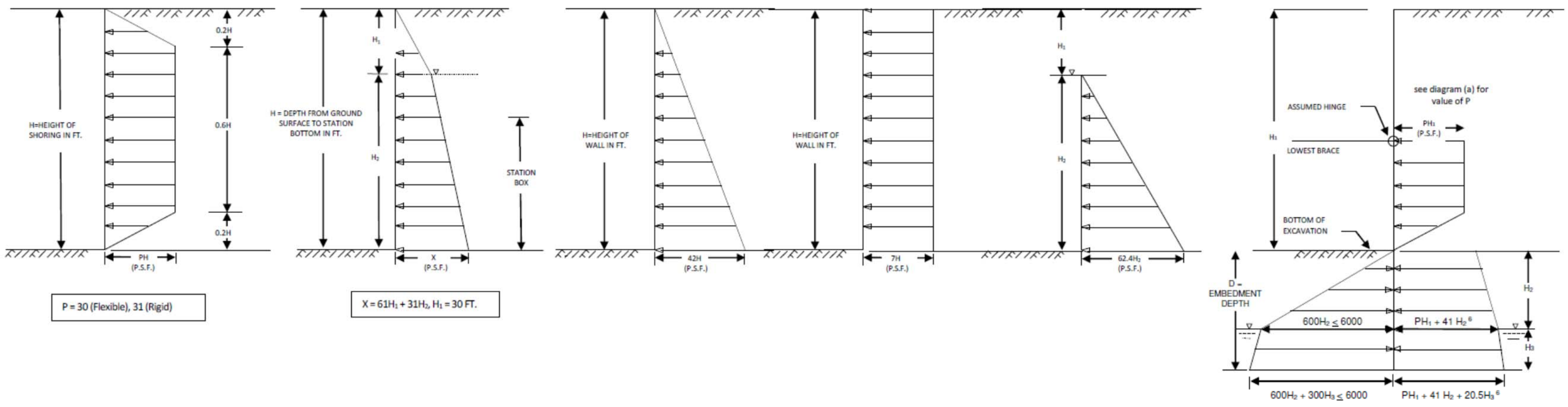
* No test data
** Limited data

[1]Lab test data from historic, ACE, PE and Adv. PE phase investigations. Use submerged unit weight below design water level
[2a]Based on relationship between elastic modulus and SPT N-value from Stroud (1989) and Duncan and Bursey (2007), CGPR#44, Virginia Tech
[2b]Based on secant modulus computed at 0.1±0.05% axial strain from Triaxial consolidated-undrained tests
[3]Values based on site-specific strength test results
[4]Unit subgrade modulus for design of foundation for service, ODE, and MDE levels
[5]Spread footing supported on undisturbed natural and/or compacted fill (for minor structures). Increase the values by 30% for short-term seismic (ODE and MDE) and wind loads
[6]Mat foundation (or) large spread footings. Bearing value may be increased based on the foundation size, if commensurate settlement is acceptable. Increase the values by 30% for short-term seismic (ODE and MDE) and wind load conditions
[7]Coefficient of friction between mass concrete and subgrade soils
[8]Based on pressuremeter test results
~~[9]Based on site-specific shear strength data~~
[9] Range and average based on consideration of pressuremeter testing, and interpreted strength and OCR (4 for Qal, 2 to 2.5 for Qalo and Qsp); recommended values based on normally consolidated condition utilizing site specific shear strength data
[9a] For soils above Elevation 180, [9b] For soils below Elevation 180
[10]Active and passive earth pressure coefficients were based on laboratory shear strength data.
[11]Recommended earth pressure coefficient is based on AMEC's prior experience with similar soils along the alignment
[12]Based on Mononobe-Okabe (1926, 1929) procedure and PGA for ODE
[13]Refer to Plate 2-1 of GDR
[14]Estimated based on current groundwater level measurements and historic groundwater level data from Seismic Hazard Zone Report for the Beverly Hills 7.5-minute Quadrangle, Los Angeles, California (1998)
[15]Design values are based on lowest resistivity, lowest pH, highest chloride and sulfate content test values
[16]Possion's Ratio was computed based on Duncan and Bursey (2007), CGPR#44, Virginia Tech
[17]Small Strain elastic modulus values were based on site-specific shear wave velocity and design Poisson's Ratio of 0.35. ODE and MDE values were based on Table 19.2-1 of FEMA P- 750 (2009)
[18]PGA estimated in accordance with Chapter 2, Section 3 of LA Metro Design Code (2013)
[19]Based on PGV-S_1 correlation (equations 13-1 and 13-2) of FHWA-NHI-10-034 (2009)
[20]Based on site-specific SHAKE91 analysis performed during Adv. PE phase.
[21]Average values are shown in parenthesis

Figure 4-1: Lateral Pressure Distribution



Flexible/Rigid Shoring (braced) [1]	**Permanent Braced Walls (Un-dewatered)**	**Cantilever Walls – Dewatered** [3] **(Shoring or permanent)**	**Seismic Increment** [4] **(Shoring only)**	**Hydrostatic (Un-dewatered Condition)** [2]	**Lateral Pressure for determination of Toe Embedments** [5]
(a)	(b)	(c)	(d)	(e)	(f)



Flexible/Rigid Shoring (braced) [1]

(a)

Permanent Braced Walls (Un-dewatered)

(b)

Cantilever Walls – Dewatered [3] **(Shoring or permanent)**

(c)

Seismic Increment [4] **(Shoring only)**

(d)

Hydrostatic (Un-dewatered Condition) [2]

(e)

Lateral Pressure for determination of Toe Embedments [5]

(f)

Notes

1. Flexible walls are assumed to be dewatered or free draining; rigid walls are assumed to be un-dewatered and water-tight.
2. Hydrostatic pressures should be used in the design for water-tight rigid shoring systems such as slurry wall, secant-pile, tangent-pile systems and for permanent walls (if not dewatered).
3. For cantilever walls with not dewatered condition, lateral soil earth pressure will be half of the value shown in the figure (i.e., 20.5H); hydrostatic pressures should be added as shown in the figure.
4. For both cantilever and braced shoring and minor walls, the seismic increment per diagram (d) is to be added to the pressure distribution per diagram (a) or to the pressure distribution per diagram (c) with appropriate load factors, but no less than pressures per diagram (b) with appropriate load factors.
5. The resisting (passive) value includes a multiplication factor of 1.5 as recommended for circular soldier piles per Metro Standard Drawing S-003; the passive pressure should be modified for other passive-resisting elements per S-003 (e.g., a slurry wall system would use the passive values provided divided by 1.5).
6. Below the bottom of excavation, the driving lateral earth pressure need only be applied to the width of the embedded element.

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4.8 Permanent Walls Below Grade and Retaining Walls

4.8.1 Lateral Earth Pressures

Section 5.6.4 of the Metro Rail Seismic Design Criteria of the MRDC states that buried permanent station walls should be designed for lateral at-rest earth pressures with a triangular distribution considering the long-term creep of the retained soils. Minor cantilever retaining walls may be designed for active earth pressures with a triangular distribution.

The geotechnical parameters and earth pressure coefficients required for design of permanent station walls at the station are presented in Table 4-1. Recommended earth pressure distributions for design of permanent walls are shown in Figure 4-1.

The earth pressure distributions presented in Figure 4-1 assume a level backfill. If the ground surface retained by retaining wall is sloped, the increase in earth pressure will need to be evaluated on a case-by-case basis.

4.8.2 Hydrostatic/Uplift Pressures

Permanent dewatering systems are not to be used for the station. Therefore, portions of station walls below design groundwater level and the station mat foundation should be designed for hydrostatic pressures. Furthermore, station walls below grade are to be waterproofed to avoid intrusion of water and gas into the station.

If minor retaining walls are located below the design water level, the wall should be designed for hydrostatic pressure or be provided with weep holes or drainage behind the wall.

The design groundwater water level of 30 feet bgs should be used in computing hydrostatic pressures for design of secant/tangent pile or slurry wall shoring systems which would act as permanent structures, if used.

4.8.3 Surcharge Pressures

Applicable lateral and vertical surcharge pressures from adjacent buildings, foundations of minor structures and vehicular loading (AASHTO HS20 vehicular loading) should be estimated and added to the earth pressures stated above. Surcharge pressures from heavily loaded construction cranes and other traffic should be added to the above pressures. Such pressures will be estimated when loading details of the construction traffic and building loads are available. Surcharge pressures should be estimated in accordance with recommendations presented in Metro Rail Standard structural Drawings (Metro, 2010)

The station roof also should be designed to resist the weight of the overburden soil. The weight of the compacted soil may be estimated using a total unit weight of 125 pounds per cubic foot.

4.8.4 Seismic Earth Pressures

The Metro Seismic Criteria does not provide specific recommendation for computing seismic earth pressures for permanent walls, but Metro standard drawing SS-003 presents a guideline for seismic earth pressure due to retained soil and due to adjoining building. Seismic earth pressure due to adjoining buildings should be in accordance with Metro standard drawing SS-003 for retaining wall design.

The increment of seismic lateral earth pressures due to retained soil should be computed using the seismic earth pressure coefficient increment (K_{AE}) provided in Table 4-1 which was based on PGA for the ODE event. An effective PGA equivalent to half of the computed PGA was used in the computations. The equivalent uniform earth pressure may be computed by taking the same resultant as the triangular equivalent fluid pressure distribution, as shown in Figure 4-1. Using these design parameters, a seismic earth pressure increment is estimated to be 6H pounds per square foot (uniform), where 'H' is the unbalanced permanent wall height. The seismic earth pressure is an incremental value intended to be added to the static active earth pressure value.

4.9 Gassy Condition Design Considerations

The planned Wilshire/Rodeo station is underlain at depth by the San Vicente Oil Field according to Map Sheet 117 of the State of California Department of Conservation, Division of Oil and Gas Geothermal Resources (DOGGR, 2006). ~~Methane should be expected at the station but the gas concentrations in this area may not be as high as the Fairfax District and the Century City area.~~**Low concentrations of methane was found in the gas probes/standpipes installed at the station. .** Since the station will be constructed below design groundwater level, the station structure will have to be thoroughly waterproofed. The impermeable waterproofing membrane should be selected such that it can be used as both water and gas intrusion barrier

The waterproofing/gas barrier should be used around the entire station structure as well as for the minor structures such as ramps, stairways, and other ancillary structures that connect to the station.

In addition, consideration should be made for construction activities in a gassy environment, with an appropriate plan being prepared for explosion, inhalation, and other safety items, as well as consideration for monitoring and/or mitigation of gases that might be released during excavation, for the purpose of the general public outside the construction zone.

4.10 Foundations (Permanent Station Structure and Station Entrance Structures)

The station bottom is currently planned at a depth of about ~~70~~**75** to ~~85~~**90** feet ~~below ground surface~~**bgs**. The soils at the station depths are sufficiently firm to allow support of the station on mat foundation.

Minor structures, such as ramps, stairways, and other structures ancillary to the station, can be supported on conventional spread footings bearing in properly compacted fill and/or undisturbed natural soils.

The geotechnical parameters for foundation design of permanent station and at-grade ancillary structures are presented in Table 4-1.

4.10.1 Bearing Value

Mat foundation for the station, established in stiff and/or dense soils, may be designed to impose a net dead-plus-live load pressure of 8,000 pounds per square foot.

Spread footings for minor structures near the existing ground surface that are at least 2 feet wide and at least 2 feet below the lowest adjacent grade or floor level, and are established in properly compacted fill and/or undisturbed natural soils, may be designed to impose a net dead-plus-live load pressure of 3,000 pounds per square foot. The excavations should be deepened as necessary to extend into satisfactory soils.

A one-third increase may be used in the above bearing values for wind or seismic loads. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot; the weight of soil backfill can be neglected when determining the downward loads.

Since the station foundation excavation extends below current groundwater depth, the bottom of the excavation may be easily disturbed from construction equipment. Therefore, if needed, a layer of crushed rock or a waste slab (slurry slab) may be considered to stabilize the subgrade after excavation. Furthermore, a layer of Tensar BX1200 geogrid or equivalent could be used in conjunction with the gravel layers to provide additional protection against unstable subgrade conditions, particularly for heavy construction equipment. **A subgrade protection plan as specified in the contract documents should be developed and should address underslab drainage and active/passive gas venting system for station permanent design.**

4.10.2 Settlement

The average bearing pressure on the station mat foundation is anticipated to be less than the overburden removed by the excavation. The settlement of the station supported on mat foundation in the manner recommended above is expected to be on the order of few inches. All of the settlement will result from recompression of the soils and therefore is anticipated to occur shortly after construction of the station structure and during the backfilling. Some differential settlement within the mat foundation should be anticipated due to variable soil conditions. Differential settlements are expected to be less than ½ inch in 50 feet.

Settlements of the minor structures supported at-grade cannot be estimated as the structural loads and foundation details are not available at this time. However, differential settlement should be anticipated at the interface between station and station ~~entrances~~**entrance structures**.

4.10.3 Lateral Resistance

~~Main~~ Station and Station Entrance Structures

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.3 may be used between the base of the foundations and the supporting soils. The passive resistance of undisturbed natural soils against station walls may be assumed to be equal to a uniform pressure of 3,000 pounds per square foot. The structural elements (station walls and the floor decks) should be designed to transfer the required passive resistance load from one side of the box to the other; for example, surcharge pressures from major existing structures adjacent to one side of the excavation will need to be transferred through the station structure and resisted by means of friction along the bottom of the station and passive resistance on the other side of the station.

The passive resistance of undisturbed natural soils and/or properly compacted fill soils against footings for minor structures may be assumed to be equal to the pressure developed by a fluid with a density of 400 pounds per cubic foot, if above the design groundwater level. For soils below the design groundwater level, an equivalent fluid pressure of 200 pounds per cubic foot should be used.

When existing buildings or deep excavations are located adjacent to the station, a full passive wedge against foundations may not be developed; in this case, the above recommended passive (fluid) pressure should be limited to a maximum passive value computed as the product of the applicable fluid pressure and lateral distance between the new foundations and existing building basement walls/excavation face.

A one-third increase in the passive value may be used for wind or seismic loads.

The frictional resistance and the passive resistance of the soils may be combined without reduction in determining the total lateral resistance.

4.10.4 Soil Springs

For the design of mat foundation and soil-structure interaction (SSI) analysis, soil springs were developed for static and dynamic conditions. The coefficient of subgrade reaction for mat design is presented in Table 4-1. The recommended dynamic lateral and vertical soil springs for the SSI analysis of the station are presented in Tables 3-5 and 3-6. If the dimensions change, the soil springs presented in these tables will need to be revised.

4.11 Earthwork

Based on the subsurface conditions encountered in the prior and current investigations, the excavation of the station can be accomplished using conventional earth equipment. Earth and site preparation activities for the station construction are expected to consist of the following:

- Excavations for shoring elements
- Excavation for station
- Subgrade preparation for station mat foundation and near-surface footings for ancillary structures
- Excavations for utility trenches, backfill over station, footings and utility trenches

Excavation for station and **station** entrance structures will need temporary shoring. All work should be in compliance with applicable City (Los Angeles **and Beverly Hills**), State (California) and federal (Occupational Safety and Health Act) requirements.

The on-site soils excavated from station excavation may be re-used as backfill material over the station, if not contaminated~~.~~ **and granular.** The on-site soils are predominantly clayey and will be difficult to compact in confined spaces such as in utility or wall backfill~~.~~ **and to 95% of the maximum density.** Import granular material may be required to backfill the excavation.

Any required soil backfill should be placed in loose lifts not more than 8-inches-thick and compacted. The fill should be compacted to at least 95% of the maximum density obtainable by the ASTM Designation D1557 method of compaction. The backfill should be sufficiently impermeable when compacted to restrict the inflow of surface water. Some settlement of deep backfill should be allowed for in planning utility connections and overlying concrete hardscape.

4.12 Corrosion Potential

To evaluate the potential for deleterious effects of the on-site soils on structural concrete and steel and on metal piping, chemical testing was performed on selected soil samples. Based on the corrosion test results, the on-site soils are severely corrosive to ferrous metals, aggressive to copper, and sulfate attack on concrete is considered to be severe. A corrosion mitigation report prepared by HDR/Schiff Associates based on the PE Phase test results is included in Appendix D of the Station GDR for reference. **The recommendations provided in the PE phase corrosion mitigation report were reviewed in light of the additional data obtained in the Adv. PE phase. Since the additional data was not outside of the previous range of data, the PE phase corrosion mitigation report was still considered applicable.**

5.0 LIMITATIONS AND BASIS FOR PRELIMINARY RECOMMENDATIONS

The professional services have been performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this Technical Memorandum. This GDM has been prepared for the Los Angeles County Metropolitan Transportation Authority (Metro) and its design consultants and contractors to be used solely for the evaluation for the Wilshire/Rodeo Station planned as part of the proposed WPLE project. The memorandum has not been prepared for use by other parties, and may not contain sufficient information for purpose of other parties or other uses.

In developing this memorandum, Amec Foster Wheeler (PB team member) relied on subsurface information obtained during Adv. PE phase and by its predecessor companies AMEC and MACTEC in the AA, ACE, and PE phase studies and its other predecessor companies, Law/Crandall and LeRoy Crandall and Associates, as well as subsurface information obtained by other firms. Subsurface conditions are, by their nature, uncertain and may vary from those encountered at the locations where visual inspections, borings, surveys, or other explorations were made.

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