Fort Ord Regional Trail & Greenway
Canyon Del Rey/SR 218 Segment

Geotechnical Design Recommendations Report

Report Status – Final

March 2023

McMILLEN JACOBS ASSOCIATES

March 24, 2023

Ms. Lindsey Van Parys
GHD
2200 21st Street
Sacramento, CA 95818

Subject: Final Geotechnical Design Recommendations Report
Transportation Agency for Monterey County
Fort Ord Regional Trail & Greenway Canyon Del Rey/SR218 Segment
Del Rey Oaks, California

Dear Lindsey,

We are pleased to submit the attached Final Geotechnical Design Recommendation Report for the Transportation Agency for Monterey County's (TAMC) Canyon Del Rey/SR218 Segment of Fort Ord Regional Trail & Greenway (FORTAG) in Del Rey Oaks, California.

We appreciate the opportunity to serve GHD and TAMC on this project. Please contact us if you have any questions about this report or if we can be of additional assistance.

Sincerely,

McMILLEN JACOBS ASSOCIATES

Su Pyae Sone Soe, PE
Project Engineer

Norman Joyal, PE/GE
Principal Engineer

LICENSED PROFESSIONAL ENGINEER
NORMAN A. JOYAL
C 31821
3-27-2023
CIVIL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
NORMAN A. JOYAL
GE 449
3-27-2023
GEOTECHNICAL
STATE OF CALIFORNIA

Distribution: GHD (PDF) – Email Delivery
Job No.: 6231.0

Table of Contents

List of Tables

List of Figures

(Page Intentially left blank)

1.0 Introduction

This report presents geotechnical design recommendations for the Transportation Agency for Monterey County's (TAMC) Canyon Del Rey/SR 218 Segment of Fort Ord Regional Trail & Greenway (FORTAG) in Del Rey Oaks, California (the Project; see Figure 1). Recommendations for design provided herein are based on interpretation of findings presented in the project's Draft Geotechnical Data Report (the project GDR; MJA, 2021) and Final Geotechnical Design Report for SR 218 Undercrossing Bridge and Retaining Wall No. 1 (MJA 2023a). In addition to this report, the project GDR and the geotechnical design report, we have also provided two other separate and independent foundation reports, one for the planned bridge and one for the planned retaining wall (see MJA 2023b and 2023c).

1.1 Project and Geotechnical Investigation Background

The project includes design and construction of approximately 1.5 miles of planned 12-foot-wide paved recreational trails and greenways, with an undercrossing of State Route (SR) 218. Portions of the alignment will require an 8-foot-high embankment backfill, and portions of the embankment backfill will require approximately 3- to 10-foot-tall retaining walls alongside the trail, and possibly utility relocation. The portion of the alignment where the trail will cross under SR 218 will also require retaining walls up to 15 feet in height. Initially, the trail was planned to cross under SR 218 within a structure to be installed by trenchless or tunneling methods. The geotechnical investigation that was initially completed for the project in 2021 was scoped based on that initial plan. Subsequent to the completion of project borings B-4 and B-5b, it was determined by the design team that conflicts with existing utilities and elevation requirements for the planned undercrossing would prevent the installation of an undercrossing structure by trenchless or tunneling methods, and that therefore, an undercrossing bridge would be necessary.

The scope of the initial geotechnical investigation for the project included 40-foot-deep exploration borings (B-4 and B-5b, as described in Section 2.2) at the planned SR 218 crossing. Boring B-4 was drilled near what is now the southeast corner of the planned bridge and encountered elastic silt and peat below a depth of 34.5 feet. This type of soil would not have had a significant impact on the originally planned design for trenchless/tunnel installation of an SR 218 undercrossing structure; however, it does have an impact on the current design of a deep foundation support for the planned bridge (i.e., driven piles). Consequently, it was necessary to perform additional deeper subsurface explorations to define the ground conditions in the project area below that of the peat layer encountered in boring B-4, for the purpose of designing driven-pile support for the planned bridge.

Four CPTs were planned for the project, one at each planned abutment corner of the bridge, but only two CPTs could be advanced, both on the south side of the bridge at the locations illustrated in Figure 1. The two planned CPTs on the north side of the bridge encountered sand consistent with utility backfill in the hand-auger holes excavated to clear the upper 5 to 6 feet prior to advancing the CPT. For safety, the CPT contractor's policy is to not advance CPTs in material that is consistent with trench backfill materials; consequently, the two planned CPTs on the north side of the bridge were not completed. Project CPT data is provided in the geotechnical design report (MJA, 2023a).

The detail project descriptions for SR 218 undercrossing bridge and Retaining Wall No.1 are provided in the foundation reports (MJA 2023b and 2023c). The recommendations presented herein only focus on the portions of the trail that are not part of SR 218 undercrossing bridge and Retaining Wall No.1 and are based on our interpretation of the geotechnical and geologic conditions in the project area that are presented in the project GDR and summarized in Section 2.0.

2.0 Recommendations

Recommendations provided herein are intended for design and construction of the project in a safe and economic manner, and to ensure the completed project's useful long-term function. Contractors constructing the project are responsible for:

- Reviewing the project GDR
- Supplementing findings of the project GDR with their own investigations
- Interpreting findings from the project GDR and their own investigations
- Selecting and implementing appropriate construction means, methods, and monitoring

Contractors should be required to successfully construct the project design in a manner so that no one gets injured, and no existing structure, improvement, or utility becomes damaged during or as a result of the work required to construct the project.

2.1 Project Elements

Based on the project GDR and our review of the preliminary project drawings (GHD, 2023), the following project elements will require geotechnical-related design recommendations:

- Dewatering, especially at planned retaining wall cuts and bridge crossing
- Temporary excavations for the construction of retaining walls and other improvements, and their corresponding shoring support
- Ground conditions with adverse behavior, including that which may warrant ground improvement before excavation
- Site preparation and earthwork for planned cuts and fills
- Possible roadway embankment modifications to facilitate construction of the bridge crossing
- Retaining walls along the uphill and/or downhill sides of the trail
- Utility relocates to facilitate bridge construction
- Backfill requirements and potential backfill settlement
- Vertical loading on relocated utilities

2.2 Subsurface Exploration

The location of borings and CPTs completed for the project are mapped in Figure 1. Boring log legends, the logs of the borings are provided in the appendices of the project GDR and the CPT results are provided in the geotechnical design report (MJA, 2023a). A partial summary of information from the project boring logs and CPTs is provided in Table 1 and Table 2, respectively.

Table 1. Partial Summary of Information from Project Borings

Boring[1]	Top Elev (ft)[2]	BGS Depth[3] (ft) Total	to GW (Seep) / Level	Interval (ft)	Bedrock or USCS[4] Group Symbol	SPT[5] (N)	Qu[5] (ksf)	Notes[6]
B-1	28	10	5.0	0-5	SM	6	-	fill
				5-10	SP	3	-	
B-2	63	17.5	NE	0-16	SP-SM	49, 40	0.1	mica
				16-17.5	Bedrock	50/3”		Monterey Formation (?)
B-3	74	10	NE	0.0-5	SM	-	-	fill in upper 2.5’
				5-10	CH	5	1.1	Wc = 40, ɣd = 79 pcf
B-4	84.5	40	(10)/31.0	0–16	SM/SC	5, 7	-	fill in upper 5’
				16–17.5	ML	4	-	Wc = 43, ɣd = 74 pcf
				17.5–23.5	SM/SC	9	-	
				23.5–25.5	CL/CH	-	-	
				25.5–34.5	SP-SM	12, 20	-	
				34.5–37	MH	-	-	diatomite/bentonite (?)
				37–40	MH & PT	4	-	LL =112, PI = 42
B-5a	95.5	2	NE	0–2	SP	-	-	refusal in fill on concrete & metal
B-5b	95.5	40	26.0	0–3	SM	-	-	fill
				3–12	SP-SM/SP-SC	4	-	concrete in fill from 10.5 to 12 feet that was eventually bypassed
				12–32	SM/SC	3, 4, 4, 5	-	Wc = 78, ɣd = 52 pcf (tuffaceous)
				32–40	MH & Bedrock	26, 27	0.4	Monterey Formation (?)
B-6	134	20	NE	0-4.5	SM	-	-	
				4.5-9.5	SP-SM/SP-SC	14	-	
				9.5-20	SM	51, 43	-	

(1) Drilled in August 2021. See Figure 1 for mapped boring locations. See logs and lab test results in Appendices B and C of the project GDR (MJA, 2021).

(2) Approximate ground surface elevation from Whitson (2020) and GHD (2023).

(3) BGS = below ground surface. GW = groundwater. NE = not encountered. Groundwater seepage depth during drilling and groundwater level depth measured in boring at time of backfilling, not necessarily the static groundwater level depth.

(4) Unified Soil Classification System (USCS) and group symbol defined in Appendix A.

(5) N = ASTM D1586 Standard Penetration Test Blow Count for interval. Qu = unconfined compressive strength.

(6) Wc = moisture content. ɣd = dry density.

Table 2. Partial Summary of Information from Project CPTs

CPT [1]	Top Elevation (ft) [2]	BGS Depth [3] (ft) Total	to GW	Soil Behavior Type (SBT) [4]	Notes[5]
CPT-1A	90.0	2.38	NE	sands, sand mixtures	CPT rig shifted when refusal was encountered on suspected concrete and had to be abandoned
CPT-1B	91.5	63.4	17.6	sand mixtures, silt mixtures, sands, clays	Refusal on concrete at ~11 feet, punched through concrete and advanced to refusal N60 value ranging from 1 to 70 bpf. Sensitive fine grained layers about 2 to 6 feet thick, prominent layer 35 to 50 feet bgs.

CPT (1)	Top Elevation (ft) (2)	BGS Depth (3) (ft)		Soil Behavior Type (SBT) (4)	Notes(5)
		Total	to GW		
CPT-2	88.5	66.5	14.6	sand mixtures, sands, silt mixtures, clays	N60 value ranging from 1 to 66 bpf. Sensitive fine grained layers about 2 to 6 feet thick, prominent layer 35 to 50 feet bgs.

(1) Performed in February 2023. See Figure 1 for mapped CPT locations. See CPT results in Appendix D of geotechnical design report (MJA, 2023a).
(2) Ground surface elevation from Whitson (2020).
(3) BGS = Below ground surface. GW = Groundwater. NE = not encountered.
(4) SBT scatter plots provided in Appendix D of geotechnical design report (MJA, 2023a).
(5) N60 = SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone using Robertson and Wride (1998). bpf = blow per foot.

2.3 Design Groundwater Level

The depth to groundwater level for project construction will vary relative to changes in seasons (i.e., rainfall), elevation, topography, and the proximity of drainageways, water bodies, and dewatering activities (e.g., wells). The depth to groundwater typically shallows during the rainy season and collects in areas of low elevation and basinal topography (e.g., near Laguna Del Rey, Frog Pond, and mean sea level) and near drainageways (e.g., the drainageway along the alignment south of SR218 and crossing SR218 near the planned tunnel). Areas of shallow perched groundwater (i.e., groundwater located above the elevation of static groundwater levels) may exist in the project area, including trapped within porous and permeable import materials (e.g., drain rock), where used to backfill existing parallel or crossing utility excavations and roadways. The depth of groundwater encountered in project exploration borings ranges from 5 to 31 feet below ground surface as indicated in the logs provided in Appendix B and summarized in Table 1 of the project GDR. The depth to the seasonal high water table in mapped project area soils is as shallow as the ground surface in the Rindge Muck (see Figure 4 Table of the project GDR).

2.3.1 Dewatering

All project construction should be performed in dry excavations. Dewatering for excavations to depths below groundwater will not only be necessary but will be critical components to successful construction of the project. The use of sump pumps and drainage rock blankets in project excavations may provide reasonably dry and stable bottoms in typically dry summertime periods. However, more extensive levels of dewatering (e.g., external dewatering wells) may be required if project excavations occur in or are to last through periods of rainy weather.

The contractor should be responsible for the design, implementation, monitoring, removal, and effects of construction dewatering. The contractor should submit dewatering plans for Agency review prior to start of work. The dewatering plans should be coordinated with the contractor’s shoring plan and ground improvement plan. The design of dewatering should be based on the actual rate of groundwater inflow into excavations and the type of shoring to be used.

Dewatering methods will need to vary along the planned alignment to account for dewatering method limitations (Figure 2) and variations of the following:

- Rainfall
- Groundwater level
- Proximity to creeks

- Excavation depth
- Duration of excavation
- Subsurface ground types and conditions

The rate of groundwater flow is a function of (1) the unbalanced head across which the flow is occurring, and (2) the permeability of the ground. Permeability typically ranges from greater than 1 cm/sec (very high) in granular deposits and backfill (e.g., sands and gravels) to less than 10^{-7} cm/sec (very low) in clayey deposits. Grain size distributions for some of the ground to be dewatered as encountered in the project borings are plotted in laboratory test results provided in Appendix C of the project GDR

There is a potential for a high rate of groundwater flow through native ground that is granular (i.e., sands and gravels). There is also a potential for a high rate of groundwater flow through trench backfill (including embedment and foundation materials) of nearby parallel or crossing utilities, particularly where it consists of granular materials. Large volumes of groundwater within highly permeable granular backfill can be stored over long distances (e.g., along the length of the historic drainageway, and the length of utilities and intersecting utilities). However, given the predominant ground type encountered in the borings is sand with varying gradation, the expectation is that water conveyed through pervious utility trench backfill, would naturally seep, and dissipate in the naturally porous sand until a horizontal barrier is encountered that allows water to accumulate.

The rate of groundwater flow into project excavations will vary along the project alignment due to (1) the difference in elevation between the base of the excavation and depth to groundwater (i.e., unbalanced groundwater head), and (2) the lateral and vertical variability of composition and consistency (and hence permeability) of the subsurface native ground and fill materials.

Collectively, the contractor’s project dewatering design, together with their shoring and, if any, ground improvement designs for the retaining walls and bridge are to preserve the undisturbed bearing capacity of the subgrade soils at the bottom of excavations and the strength integrity where vertical excavations are made.

The contractor’s dewatering system should achieve the following minimum performance requirements:

- A reasonably dry base of excavation
- Stable excavation walls and bottom
- Draw down the groundwater level to a minimum of 3 feet below and beyond the excavation bottom and sidewalls to establish a reasonably dry base of excavation and to prevent ground piping and groundwater boiling through the excavation bottom where shoring is not designed to resist hydrostatic pressures
- Filter native ground and prevent loss of ground from dispersion and erosion
- Prevent damaging settlement to nearby structures, utilities and/or pipelines
- Prevent the migration of contaminated groundwater plumes, if any
- Be installed and removed in accordance with governing jurisdictional requirements
- Not cause subsidence and/or settlement damage
- Allow for controlled release of groundwater to its static level in a manner that prevents ground disturbance and prevents flotation or movement of structure or pipelines

The contractor should be required to submit alternate dewatering, ground improvement and shoring designs, and the contractor should be prepared to implement the alternative designs should the initial designs not achieve the minimum performance requirements. Uncontrolled seepage of groundwater through excavation sidewalls or bottom may cause the excavations to be unstable and unsuitable for support. Consequently, the contractor should be prepared to locally dewater or modify (e.g., by ground improvement) construction excavations, if and where needed, to provide stable and relatively dry excavations.

If the shoring is not designed to resist hydrostatic pressures and the dewatering system is designed to draw the groundwater level down a minimum of 3 feet below the excavation bottom, then groundwater level monitoring wells should be required adjacent to the excavation to monitor groundwater levels prior to and while the excavation is open. Monitoring is to confirm that the groundwater level is adequately lowered prior to and throughout the duration of the excavation.

Prolonged dewatering causes an increase in effective stress on underlying soils. There is also a risk of soil dispersion and erosion into improperly filtered wells. Therefore, settlement monitoring points should be set between and on nearby critical structures, utilities and regularly monitored during active dewatering. Modifications to contractor shoring, dewatering and/or ground improvement should be required if settlements are measured or if damaging settlements are likely to occur given measured trends, the anticipated duration of dewatering, and the location of settlement monitoring points relative to existing critical structures, utilities and pipelines.

Installation and removal of wells is to be in accordance with governing (e.g., county and state) requirements.

2.4 Temporary Excavations

2.4.1 Excavatability

Contractors should be made solely responsible for choosing their excavation means and methods based on their evaluation of the excavatability of the subsurface materials to be encountered in project excavations. The contractor should be required to submit excavation plans for the Agency's review prior to mobilization. The contractor's evaluation should include a review of the project GDR coupled with its own subsurface investigation and interpretation.

Project excavations through native soils like that encountered in project borings as described in the project GDR and summarized in Table 1 and Table 2 can be made with appropriately sized conventional excavation equipment. However, additional excavation effort or special excavation equipment (e.g., hoe rams, jack hammers, ripper teeth) and methods may be required if concrete obstructions as encountered in project exploration borings and CPTs (B-4, B-5a, B-5b, CPT-1a, and CPT-1b), and Monterey Formation bedrock are encountered (see boring logs in Appendix B of the project GDR). The unconfined compressive strength of ground sampled and tested from project exploration borings ranged from 100 psf to 1,100 psf (see laboratory test results in Appendix C of the project GDR).

2.4.2 Cal/OSHA Soil Classification

Cal/OSHA soil or ground classification for temporary excavations includes the following types:

Type A: Excludes materials that are part of a sloped or layered system dipping into the excavation at a slope ≥4H:1V, but includes cohesive ground with an unconfined compressive strength of ≥1.5 tsf that is:

- Not fissured
- Not subject to vibration from heavy traffic, pile driving, or similar effects
- Not been previously disturbed

Type B: Excludes material that is part of a sloped or layered system dipping into the excavation at a slope ≥4H:1V, but includes the following:

- Cohesive ground with an unconfined compressive strength between 0.5 and 1.5 tsf
- Angular gravel and silt
- Previously disturbed ground, except soil otherwise classified as Type C
- Ground fissured or subject to vibration and not otherwise Type C ground
- Dry rock that is not stable

Type C: Includes the following:

- Material that is part of a sloped or layered system dipping into the excavation at a slope ≥4H:1V
- Cohesive or disturbed ground with unconfined compressive strength ≤0.5 tsf
- Sand and nonangular gravel
- Submerged ground or ground from which water is freely seeping
- Submerged rock that is not stable

A determination of the Cal/OSHA classification of ground types encountered in project excavations should be made the responsibility of the contractor’s Cal/OSHA approved and qualified “competent person.” Cal/OSHA ground types can vary over short lateral and vertical distances. Therefore, project excavations should be continually monitored and documented by the contractor’s competent person, and the contractor should be prepared to make changes and modifications to shoring requirements for excavation safety in response to these changes, consistent with governing regulations, in the field and at the time of excavation.

Majority of the ground encountered in project test borings are classified as Cal/OSHA Type C with a few exceptions which are classified as Type B, especially the ground at the deeper part of the project test borings.

2.4.3 Shoring

The contractor should select, design, implement, monitor, remove, and be responsible for all effects of project excavations and shoring. A professional civil engineer licensed in the State of California should design, sign, and stamp the contractor’s shoring plans and calculations. The contractor should submit shoring plans and calculations together with all other interdependent submittals (e.g., dewatering and ground improvement) for the Agency's review before start of work.

Project shoring must be consistent with Cal/OSHA Construction Safety Orders, Article 6; be coordinated with project dewatering (Section 2.3.1) and ground improvement (Section 2.5), if required; and achieve the following minimum performance requirements:

- Protect personnel that enter the excavation
- Be compatible with the ground conditions encountered, including foundation, embedment, and backfill material of existing pipeline and historic drainageways (contractor potholing should evaluate these materials for purposes of shoring design)
- Resist lateral earth pressures including those from hydrostatic pressures and lateral loads from existing structures, vehicular traffic, construction equipment, and spoils
- Sequence and perform excavations and install shoring in a manner that protects and prevents damage to nearby utilities, improvements, and structures
- Provide stable excavation walls and bottom including preventing raveling, running, and flowing ground from excavation walls and associated loss of adjacent ground even when they are subjected to vibrations from construction
- Provide a dry base of excavation
- Prevent ground vibrations, settlement, and heave that could damage utilities, structures, and improvements
- Remove or abandon shoring in a manner and sequence that do not damage structures, pavements, utilities, and improvements. This includes being in step with the backfilling sequence so that shoring is not removed ahead of backfilling; and not causing disturbance and loosening of subsurface material.
- Completely fill any void space created by shoring removal with CLSM or equivalent.

Contractor shoring submittals should contain contingent systems that the contractor will implement should any segments of shoring not meet the minimum performance requirements. Preliminary braced earth shoring diagrams and pressures are provided in Figure 3, and minimum surcharge pressures and diagram are provided in Figure 4. These diagrams and pressures represent typical ground conditions mapped and encountered in project test borings assuming a completely dewatered excavation.

The final earth shoring pressures and surcharge pressure diagrams used in calculations by the contractor's shoring designer must be based on the following at the time of construction:

- Ground and groundwater condition
- Shoring type, design, and installation method
- Dewatering and ground improvement
- Traffic loads, stockpiling, and any other surcharge load adjacent to the excavation

Cal/OSHA Type B soils will generally have a sufficiently long stand-up time to allow for short reaches of full-depth excavation prior to installation of shoring such as trench boxes or speed shores with intermittent backing.

Solid sheeting is required by Cal/OSHA in Type C materials. Excavations in Cal/OSHA Type C materials will tend to flow, run, or fast ravel, and will have little to no stand-up time. Where exposed in excavations, unimproved granular utility backfill will flow, run, or fast ravel back to the cut line of the original excavation for the utility. We are not aware of the details of original backfill material of historical drainageways and existing utilities, or the trench geometry of excavations used in construction of existing nearby parallel and crossing utilities (that is, whether they are vertical, or side sloped). Backfill of historical drainageways and nearby parallel and crossing utility excavations should be evaluated by contractors for purposes of their final shoring and ground improvement designs, if required.

Dry (i.e., above groundwater), granular, noncohesive soils (e.g., typical utility backfills, or poorly graded sand encountered in project test borings) are also Cal/OSHA Type C soils and will have little to no stand-up time and will tend to run or fast ravel when exposed in unshored vertical excavations. Similarly, dry fine-grained soils with low cohesion will have little stand-up time and will tend to fast ravel when exposed in vertical excavations. Running or fast-raveling, granular, noncohesive materials and fast raveling, fine-grained soils with low cohesion (particularly those subject to construction vibrations) will have insufficient strength and stand-up time to safely maintain full-depth vertical excavations long enough for complete trench box or solid-sheet speed shore installation (solid sheeting is required by Cal/OSHA in Type C soil). The use of trench boxes in these conditions will require (1) careful interior (within the trench box) excavation, with backer plates pushed below the bottom of the trench box ahead of the excavation; and/or (2) prior ground improvement (e.g., external dewatering and/or permeation grouting of existing granular utility backfill).

The use of solid-sheet speed shores in running and fast-raveling ground conditions (i.e., Cal/OSHA Type C material) that have not been improved (e.g., by permeation grouting or dewatering) will result in excavation wall loss and related undermining of adjacent pavements, utilities, and structures where the stand-up time of the material exposed in the vertical excavation is less than that required for placement of the solid-sheet speed shores.

Shoring systems that do not provide continuous positive support of excavation walls (i.e., passive systems like trench boxes that allow for minor movement of the trench wall toward the excavation) may also cause surface settlement and related damage to nearby utilities, structures, and improvements. A summary of the potential surface settlement of passively shored excavations is provided in Table 3.

Table 3. Potential Surface Settlement of Passively Shored Excavations

Soil Type	Surface Settlement (% of Excavation Depth) [1]	Lateral Zone of Disturbance (Multiples of Excavation Depth) [1]
Sand	0.5%H	H
Soft to Medium Stiff Clay	1–2%H	3–4H
Stiff Clay	<1%H	2H

[1] From Suprenant and Basham (1993).

2.4.4 Temporary Excavations and Slopes

Temporary excavations and slopes are to be protected from erosion and surface water runoff. The maximum temporary excavation and slope inclination (horizontal:vertical) that is allowed by Cal/OSHA without supporting design by a Professional Engineer for Type B soil and Type C soil is 1H:1V and 1.5H:1V, respectively. Cal/OSHA also requires that temporary excavations and slopes greater than 20 feet in vertical height be designed by a Professional Engineer. Cal/OSHA defines the maximum allowable temporary slope inclination as the steepest incline of an excavation face that is acceptable for the most favorable site conditions (i.e., assuming no adjacent soil stockpile or heavy equipment) as protection against cave-ins.

Contractors and their temporary excavation slope and shoring designer are to acknowledge (1) Cal/OSHA requirements, and (2) that development of their own assessment of safe temporary excavation and slope inclinations is a field decision to be made at the time of construction by the Contractor’s competent person.

2.5 Ground Improvement

Ground with running, flowing, and fast-raveling behavior that is likely to be exposed (e.g., typical utility backfills, or poorly graded sand encountered in project test borings) will have little to no stand-up time in unshored vertical excavations. This ground type could produce large groundwater inflows if it happened to be in communication with the Frog Pond. The failure surface of an inadequately shored and unstable excavation in running granular noncohesive materials could extend to the angle of repose of the material, which is typically on the order of 30 degrees for sand and gravel. The failure surface of an inadequately stabilized excavation in flowing noncohesive materials would be flatter. Consequently, inadequately stabilized excavations will result in existing nearby utilities, structures, and roadways being damaged by loss of support, undermining, or vibration-induced settlement.

Therefore, excavation backfill of existing parallel and crossing utilities should be evaluated through potholing by the contractor for its final shoring and ground improvement designs. Excavations that occur in running, flowing, or fast-raveling materials should be stabilized by ground improvement, such as dewatering or grout stabilization, where not completely and continuously shored (i.e., at utility penetrations through shoring) to avoid related damage to existing utilities, pipelines, structures, and roadways.

The selection, design, implementation, and monitoring of ground improvement for the project should be made the sole responsibility of the contractor, and the design should be submitted by the contractor for the Agency’s review before the start of work.

2.5.1 Permeation Grouting

Permeation grouting typically consists of injecting a chemical grout or a fluid mixture of cement and water with fluidizer additives into porous and permeable ground. Permeation grouting for the project could include constructing bulkheads in permeable utility backfill to facilitate filling all voids within existing coarse granular backfill (backfill also refers to related embedment and foundation material) with grout for one or more of the following purposes:

- To stabilize materials that would be unstable when exposed in vertical excavations.

- To stabilize materials that would be subject to vibration densification.
- To reduce the rate and volume of perched groundwater transmission within porous and permeable materials.

Permeation grouting should completely bind the granular material into a single coherent grouted mass (grouted prism). Such grouting is to be done prior to shoring installation and prior to any dewatering. For permeation grouting, the grout mix should be designed with fluidizers so that its set time allows for permeating grout flow to completely fill all voids in the existing granular material being grouted between pregrouted bulkhead ends.

Prior to grouting, the contractor should notify Underground Service Alert to mark subscribing subsurface utilities in the vicinity of the planned excavations and grouting. The contractor should use the project drawings and make a detailed site inspection to locate existing subsurface utilities prior to the start of project grouting and excavations. Proposed grout injection sites within 5 feet of an existing utility should be probed for utility clearance prior to grout pipe installation and relocated where necessary.

Grouting submittals should also include the following:

- A layout plan and subsurface profile detailing the location and identification of grout injection casings and vertical grout injection stages within these casings including proposed method of casing installation and sequence of grouting. This would include fast-setting grout bulkheads.
- Materials including grout mix design, unconfined compressive strengths, and set times.
- Equipment including drill rigs, mixers, pumps, grout injection casings, and gauges.
- Methods and procedures of grouting execution.
- Anticipated grout injection volumes (i.e., intake volumes based on void ratio/porosity of material to be grouted, and dimensions of grouted prism required to resist earth pressures and hydraulic head).
- Methods of monitoring and evaluating quality assurance.
- Cleanup and restoration.

The grout mix must not be corrosive to existing utilities or contain environmentally hazardous materials. It should be designed with fluidizers to have a set time to allow for the permeating grout flow to completely fill all voids in the existing granular material being grouted. The grout mix should be designed and injected at pressures that will not damage nearby utilities, pipelines, and structures and will not cause heave of the ground surface. The ground surface and utilities should be monitored for heave, and all existing utilities, pipelines, and structures should be protected from damage during the grouting work. Temporary elevation benchmarks should be installed in the grouting area and at suitable distances outside of the grouting area for reference checks. Elevations should be monitored during all grouting operations, and a daily log of cumulative changes in the benchmark elevations should be maintained.

Grouting through grout injection casings should continue until grout returns are noted in the next adjacent predrilled/driven grout injection casing. Success of this system will require a careful balance between grout injection pressures (always to be less than the pressures that could damage the existing utilities), grout fluidity/set times, and grout injection casing layout dimensions. A daily log of grouting operations should be maintained including grout injection casing number, location, grouting pressure and rate, stage depth, and grout quantity and batch used. Grout batch records should include time of mix and amount and type of components used (e.g., water, cement, additives), and anticipated set time. A sample should be taken from each batch mixed and properly identified, stored, and tested for compressive strength. During and upon completion of grouting, the work area should be cleaned and restored to its original condition including adequate disposal of all generated waste and wastewater. The top of each grout injection hole should be repaired to match the existing pavement grade in accordance with jurisdictional regulations.

2.6 Site Preparation and Earthwork

We recommend that project construction occur during the dry summer season. Construction during the rainy season may result in the following types of consequences, the mitigation and/or repair of which will add to project construction time and cost:

- Perched groundwater, including in bedding and/or backfill materials of existing utility trenches;
- Erosion and sloughing of exposed excavations; and
- Surface instability, such as pumping ground or rutting from equipment and traffic.

All areas where the engineered fill or retaining wall foundation is to be placed should be stripped of vegetation, surface soils containing roots or other organic materials, surface obstructions (e.g., pavements, aggregate base), and any other artificial materials. If areas or pockets of soft, loose, or saturated soils are encountered after stripping, they should be overexcavated to competent native material and replaced with moisture-conditioned and uniformly compacted on-site soil or aggregate fill to finished grade, per recommendations provided in Sections 2.7.1, 2.7.2 or 2.7.3. If a firm and stable base is not encountered within 2 feet of overexcavation, it can be established by the placement of foundation materials as described in Section 2.9.1.

The native soils exposed at the base of the areas to receive engineered fill and retaining wall foundations that are primarily in sand should be scarified to a minimum depth of 8 inches, moisture conditioned to be at or near optimum moisture content, and recompacted to a minimum relative compaction of 90% of maximum dry unit weight as determined by ASTM D1557. Should clay soils be encountered at the foundation level, it should be moisture conditioned to be around 2 to 5% over optimum prior to compacting to 90%. Exposed subgrade soils and/or engineered fill materials should be kept moist and not be allowed to dry out prior to construction of foundations. If surface shrinkage cracks are present, the depth of scarifying and moisture conditioning should extend to the full depth of cracking.

Prior to placement and compaction of engineered fill, and during recommended scarification, the exposed subgrade should be evaluated as to its cleanliness, engineering properties, and suitability as foundation soils by a qualified geotechnical engineer. Soils determined to be unsuitable (e.g., organic material, oversize material, highly expansive clay) should be completely removed.

2.7 Roadway Embankment

Site preparation recommendations prior to placement of engineered fill is provided in Section 2.6. The embankment fill should be composed of (1) on-site soil subject to limitations described in Section 2.7.1, (2) Caltrans Class 2 Aggregate Base as described in Section 2.7.2, and (3) Caltrans Class 3 Aggregate

Base as described in Section 2.7.3. Fill materials should be placed in maximum 8-inch-thick horizontal loose lifts and compacted to a minimum relative compaction of 90 percent of maximum dry unit weight at or near optimum moisture content for sands or at a moisture content of 2 to 5 percent over optimum for clays, as determined by ASTM D1557. The embankment must be built up as evenly as possible, limiting the height differential of placed material to about 18 inches. All site preparation and fill placement related to roadway embankment construction must be observed and tested by a qualified geotechnical engineer. Keyway embedded a minimum 12 inches into competent native material should be installed at the toe of the existing slope as illustrated in Figure 5 to key the embankment fill materials into the existing ground.

Where the embankment is to be constructed on an existing slope of 3H:1V or steeper, the existing ground surface should be benched to key the new embankment fill into the existing slope as illustrated in Figure 5. The bench should have a minimum width of 3 feet and the maximum height of 2 feet. When the native material along the existing slope has lower permeability than the embankment fill materials, water can build up between the existing slope and the embankment fill and destabilize the embankment side slope. To prevent this, subdrains should be installed inside the keyway (and possibly at bench cuts) to collect water seeping through the embankment fill. Additionally, drainage should also be installed at the top of the embankment along the trail to direct surface water off the embankment slope and prevent erosion due to surface water runoff. Lastly, we recommend that the slope face be overbuilt and then cut back to final grade to reveal compacted ground in the face.

The embankment side slopes should be designed at a maximum inclination of 3H:1V. Embankment side slopes are susceptible to erosion and scour due to surface water flow over the slope face, especially when the embankment is constructed using cohesionless materials. Therefore, it is recommended that embankment side slopes be armored and/or well vegetated to prevent side slope erosion. The most critical time for erosion control is immediately after embankment construction when the soil on the surface of the side slope is exposed to rainfall, and vegetation has not yet taken hold, i.e., if vegetation is used as the primary erosion control measure.

Settlement due to embankment fill placement will vary along the alignment, however, the settlement will occur immediately and incrementally as the embankment is backfilled. Therefore, the contractor can adjust the final grade accordingly after the settlement.

2.7.1 On-Site Soil

On-site soil must be nonexpansive to very low expansive soil that is free of contamination, vegetation, and other deleterious materials and contain no material greater than 18 inches in size, including earth clods. It must meet the testing specification provided in Table 4. Moisture-conditioning of backfill material (drying wet and saturated soils and wetting dry soils) will most likely be required to achieve proper backfill compaction for on-site soils.

Table 4. On-Site Fill Requirements

Test (reference)	Requirement
Plasticity Index (Cal 204)	NP–12
Liquid Limit (Cal 204)	< 30
Expansive Index (UBC 18-2)	< 20

2.7.2 Caltrans Class 2 Aggregate Base (Class 2AB)

Caltrans Class 2AB uniformly graded to the requirements in Table 5 may be used as engineered fill.

Table 5. Class 2AB

Sieve Size	Percent Passing	
1 in.	100	
3/4 in.	90–100	
No. 4	35–60	
No. 30	10–30	
No. 200	2–9	
Requirement	**Limit**	
Plasticity Index	< 12	
Liquid Limit	< 30	
Test	**California Method No.**	**Requirement**
Resistance (R-Value)	301	78 min.
Sand Equivalent	217	22 min.

2.7.3 Caltrans Class 3 Aggregate Base (Class 3AB)

Caltrans Class 3AB uniformly graded to the requirements in Table 6 may be used as engineered fill.

Table 6. Class 3AB

Sieve Size	Percent Passing	
1 in.	100	
3/4 in.	90–100	
No. 4	40–70	
No. 30	12–40	
No. 200	3–15	
Requirement	**Limit**	
Plasticity Index	< 12	
Liquid Limit	< 30	
Test	**California Method No.**	**Requirement**
Resistance (R-Value)	301	50 min.
Sand Equivalent	217	18 min.

2.8 Retaining Walls

Retaining walls will be required at locations along the trail where embankment backfill or cutting of the natural ground is required for the trail construction. According to project exploration borings B-1 to B-5 provided in the project GDR, the subsurface conditions at the locations explored consisted predominantly of silts and sands. According to the site drawings (GHD, 2023), the height of the retaining walls is estimated to be between 3 to 5 feet along the trail and up to 15 feet high for Retaining Wall No. 1. Recommendations for reinforced concrete cantilever walls with foundation options for both spread footings and drilled piers are provided in the following sections, but they should not be co-mingled. Retaining wall recommendations provided herein are exclusively for the retaining walls along the trail that are not part of Retaining Wall No. 1. Recommendations for Retaining Wall No. 1 are provided in Foundation Report for Retaining Wall (MJA, 2023c).

2.8.1 Spread Footings

Spread footings need to be designed using the foundation parameters provided in Table 7. These parameters are not subject to a 1/3 increase for transient loading such as wind and seismic forces except for the allowable bearing pressure. The spread footings need to have a minimum embedment depth of 24 inches into a competent native material. A keyway embedded a minimum of 12 inches into a competent native material can be installed to provide additional passive resistance, if needed, in lieu of having the whole footing embedded into a competent native material. A qualified geotechnical engineer should be present during the foundation excavations to verify that the footings are in competent native material. The thickness of the structural members and details of the reinforcement shall be designed by a licensed structural or civil engineer.

Table 7 Retaining Wall Foundation Design Parameters

Allowable Bearing Pressure (psf)	Poisson's Ratio	Young's Modulus (tsf)	Coefficient of Sliding Friction
1,500	0.3	200	0.25

2.8.2 Drilled Piers

Drilled piers to support vertical loads of the retaining walls and to resist the uplift force from the lateral loads including earth load, wind load and seismic load should be designed to derive their support by skin friction within medium stiff to stiff or medium dense native soils and/or Monterey Formation. Drilled cast-in-place straight shaft piers must be a minimum of 18 inches in diameter and designed using an allowable skin friction provided in Table 8. These skin friction values may be increased by one-third for transitory loading such as wind or seismicity. Any skin friction resistance within the upper 2 feet of the ground surface must be ignored. Lateral loads acting on the retaining wall due to seismic load and/or movement of soil behind the wall will be resisted by passive pressures acting against the sides of the piers. Drilled piers can be designed for lateral loading using the passive earth pressures provided in Table 9. This passive pressure can be assumed to be acting against 2 times the diameter of the individual pier. Where the ground descends immediately below the toe of the pier-supported structure, apply passive pressure on the downgradient side of the structure starting 12 inches below pier and structure interface.

Table 8. Allowable Skin Friction

Conditions	Allowable Skin Friction (psf)
Skin Friction to resist vertical loads	500
Resistance to uplift force due to lateral loads	350

2.8.3 Lateral Earth Pressures

Retaining walls whose tops are not free to deflect should be designed for at-rest conditions. The following design criteria apply to retaining walls which are not free to deflect, that are a maximum of 12 feet in height with horizontal backfill and have a drainage system consisting of drain rock (see Section 2.9.1) with perforated drainpipes or weep holes to prevent hydrostatic pressures that might be caused by water trapped behind the retaining wall. The contractor can select appropriate Geo-composite material as an alternative drainage system, and it should be placed as per the manufacturer's guidelines. Retaining walls meeting the criteria described above can be designed for the active and passive earth pressures provided in Table 9. Where the ground descends immediately below the toe of the structure, apply passive pressure on the downgradient side of the structure starting at 12 inches below the ground surface at the toe.

Table 9. Lateral Earth Pressures

Ultimate Static Lateral Earth Pressures[1] Expressed as Equivalent Fluid Density (psf/ft in a triangular distribution)	
At-rest Pressure	60 pcf
Active Pressure	35 pcf
Passive Pressure[(2)]	250 pcf

[(1)] Safety factors should be applied. Assumes structures are less than 15 feet deep. See report text for additional application pressures.

[(2)] Neglect the fill materials. For passive pressures, a safety factor of at least 2.0 should be applied to avoid the lateral movement of the structure, which would be necessary to reach full ultimate passive soil strength mobilization.

The following modifications to design lateral earth pressures should be made to both at-rest and active pressures provided in Table 9 where applicable:

- Lateral surcharge from adjacent structures where an imaginary 1.5H:1V plane projected downward from an existing or planned new structure projects above or intersects the side of the planned new adjacent structure.
- Dynamic pressures (P_e) from seismic shaking. A dynamic earth pressure of $P_e = 30 \times H$, expressed as pounds per square foot, should be applied as a triangular distribution over a depth of H (where H = depth of wall embedment below grade in feet). The resultant should be applied at a distance of 0.3H from the bottom of the structure.
- Lateral surcharge from equipment and vehicles (Figure 4), where it exceeds the dynamic earth pressure.

In addition to passive earth pressures, the sliding friction at the base of concrete structures can be used to resist lateral loads. Table 7 summarizes the coefficient of friction for the base of concrete foundations on the native soil.

2.9 Pipe Bedding and Backfill

The contractor should be made solely responsible for protecting project utilities from damage during placement and compaction of excavation backfill and pipe embedment materials where utility relocates may be required. Excavation backfilling and pipe embedment should be performed in accordance with the following requirements where not exceeded by any governing agency and/or pipe manufacturer requirements. See Figure 6 for a typical excavation backfill diagram.

2.9.1 Foundation Material

Foundation material, if and where required as described herein, should consist of a minimum thickness of either 6 inches of CLSM (Section 2.9.4) or 12 inches of clean, durable, natural, 1.5-inch crushed (i.e., angular) drain rock (Table 10) wrapped with geotextile fabric (Table 11). The geotextile fabric is used to separate the open-graded drain rock from surrounding finer grained native soils and pipe bedding material, and should be overlapped a minimum of 12 inches. Foundation material is required for any excavations in which the trench or excavation bottom is unstable (e.g., pumping subgrade under foot load, boiling), yielding, or disturbed by construction activity, or where overexcavation occurs.

Table 10. Foundation Material

Sieve Size	Percent Passing
2 in.	100
1-1/2 in.	90–100
3/4 in.	5–30
3/8 in.	5–20
No. 200	0–4

The geotextile fabric should be a nonwoven material consisting of polyester, nylon, and polypropylene filaments formed into a stable network. The fabric shall be permeable, not act as a wicking agent, be inert to commonly encountered chemicals, be rot proof, and be resistant to ultraviolet light. The geotextile fabric shall also conform to the physical properties in Table 11. The fabric should be overlapped a minimum of 12 inches.

Table 11. Geotextile Fabric

Property	Test Value	ASTM Test Method
Weight	5.4 oz/yd^2 (min.)	D5261
Grab tensile strength	150 lb (min.)	D4632
Elongation at break	50% (max.)	D4632
Puncture strength	80 lb (min.)	D6241
Burst strength	300 psi (min.)	D3786
Apparent opening size	#70 (max.)	D4751
Permittivity	1.0 sec^{-1} (min.)	D4491
UV resistance	70% (min.)	D4355

The decision on use and extent of foundation material should be made by the construction manager at the time of construction. The thickness of the foundation material that should be used is to be based on the depth to a firm unyielding stable base or until a firm unyielding stable base can be created by the placement of the foundation material.

2.9.2 Pipe Embedment Material

As long as placement and compaction does not damage the pipe, and where approved by the pipe manufacturer, the pipe embedment material should consist of the following:

- Controlled low strength material (CLSM) described in Section 2.9.4
- Class 2 aggregate base (Class 2AB) uniformly graded within the requirements given in Table 5

Pipe embedment (i.e., material type, compaction, and thickness surrounding the pipe) should be sufficient to provide the necessary lateral support for flexible pipe material to prevent pipe deformation. Pipe embedment material should extend a minimum distance of 6 inches below the pipe to 12 inches above the pipe.

2.9.3 Excavation Backfill Material

In unpaved/unimproved areas, excavations can be backfilled with clean, inorganic on-site native soils derived from project excavations meeting the on-site fill requirements in Table 4. The on-site soil used for backfilling should be free of contamination, vegetation, and other deleterious materials and contain no material greater than 4 inches in size, including earth clods. Moisture-conditioning of backfill material (drying wet and saturated soils and wetting dry soils) will most likely be required to achieve proper backfill compaction.

In paved areas or areas that are to receive improvements, excavations should be backfilled to the finished subgrade with Class 2AB material. CLSM may be used in lieu of Class 2AB material where Class 2AB material cannot be properly compacted (e.g., below existing utilities or where trench sidewalls yield because of trench backfill compaction effort).

2.9.4 Controlled Low Strength Material (CLSM)

Where excavation backfill cannot be properly compacted, then CLSM should be used, including as backfill below existing crossing utilities. Requirements for CLSM include the following:

- Be a self-compacting, hand-excavatable mixture of cement, pozzolan, coarse and fine aggregate, and water that has been mixed in accordance with ASTM C94 and is in a flowable state during placement
- Have a minimum 28-day compressive strength of no less than 50 psi and a maximum 28-day compressive strength of no more than 150 psi
- Be noncorrosive (physiochemical properties that do not damage the pipe)
- Be placed in appropriate lifts or with methods to prevent movement of the pipe, including by flotation
- Be installed with approved anchor blocks or deadman concrete collars, as needed, to secure the pipe in place

Placement of backfill, pavement section, or concrete on top of CLSM should not be allowed until the CLSM passes the ball drop test of ASTM D6024.

2.9.5 Compaction

The following recommendations assume that the pipeline can support mechanical compaction as recommended herein. Where this is not the case, then the pipe embedment material and excavation backfill material should consist of CLSM. Relative compaction referred to herein is in accordance with ASTM D1557, unless stated otherwise.

All water that accumulates in the bottom of excavations must be removed so that the work can be done under relatively dry conditions. Where excavation bottoms are unstable, foundation material (CLSM) or densified (crushed rock wrapped in fabric) should be placed to provide a stable trench bottom capable of supporting compaction of the pipe embedment material. Three passes with a vibra-plate compactor are typically adequate to sufficiently densify crushed rock to a firm, unyielding state.

Pipe embedment material consisting of Class 2AB should be compacted to a minimum of 90% relative compaction at a moisture content at or near optimum. The pipe embedment material at the bottom of the pipe (i.e., pipe subgrade) should be compacted to a smooth, uniform plane to match the desired pipe slope. Where applicable, flange or bell holes should be excavated out at each pipe joint to ensure uniform pipe support and proper line and grade over the full length of each pipe segment. After the pipe is laid in the excavation, embedment material should be uniformly placed in maximum 8-inch-thick lifts on each side of the pipe and hand-shovel sliced around the haunches to support the sides of the pipe and to prevent pipe displacement, and then compacted to 90% relative compaction at or above optimum moisture condition. Compacting and testing Class 2AB below the spring line of the pipe will be dependent on the excavation width selected for installation of the pipeline and on the shoring and dewatering systems. It may not be practical to test compaction below the spring line with less than 12 inches of side clearance between the pipe and trench/shoring wall. Above the spring line, the pipe embedment material should be placed in maximum 8-inch-thick loose lifts and compacted to a minimum of 90% relative compaction at or above optimum moisture content.

Excavation backfill should be placed in maximum loose lifts of 8 inches above the pipe embedment material. In unpaved areas where no future surface improvements are planned, excavation backfill should be compacted to a minimum of 85% relative compaction at or near optimum moisture content. In paved areas or in areas where future surface improvements are planned, excavation backfill should be compacted to a minimum of 90% relative compaction to within 3 feet of the pavement subgrade and to a minimum of 95% relative compaction within the upper 3 feet of backfill. Inadequate compaction of excavation backfill (i.e., less than that recommended herein) may cause excessive settlements resulting in damage to the pavement and other surface improvements.

At all times during the placement of the pipeline, and placement/compaction of the pipeline embedment material, it is the contractor's responsibility to protect the pipeline from damage (e.g., overstressing the pipeline with heavy equipment). For flexible pipelines, proper compaction of the pipeline embedment material is critical, especially in the area around the pipe haunches (i.e., under and around the pipe spring line). If the pipeline embedment material is not compacted as recommended above, it may result in excessive deflection or failure of the pipeline. The contractor's performance in achieving compaction requirements for pipeline embedment material should be closely monitored and tested. This includes ensuring that removal of trench shoring is completed in step with placement and compaction of embedment material. Removal of shoring must not cause loosening of the compacted pipeline embedment material (i.e., either by gaps formed upon shoring removal or by vibration). Where shoring removal is permitted, the shoring should be removed in vertical stages, with final placement and final compaction of lifts of pipeline embedment material completed only within the unshored portion of the excavation.

2.10 Backfill Settlement

The amount of backfill settlement will depend mostly on the condition of the excavation bottom and the contractor's performance in achieving the minimum recommendations provided herein with the backfill methodology selected (compaction and/or cementitious backfill). It is imperative that stable excavation bottoms are maintained at all times and that loose, disturbed, or otherwise softened soils are not allowed. Backfill loading on such soils can produce random, localized and abrupt settlements over short sections of the utility that can exceed 1 inch.

2.10.1 Compression

Where excavations are located beneath paved surfaces, the finished pavement may reflect backfill compression settlement. Backfill placed within excavations will compress (settle) by self-weight even when well compacted. We estimate settlement of foundation, pipeline embedment, and excavation backfill materials compacted as recommended in this report to be less than 0.2 to 0.4% of their thickness. CLSM that passes the ASTM D6024 ball drop test will not compress by self-weight. Settlement will be greater than these estimates where native soils are used as excavation backfill. Where native soil backfill is used, long-term settlement by self-weight could be on the order of 1.0 to 2.0% of the native soil backfill thickness.

2.10.2 Recompression

Excavations for replacement pipeline will be backfilled to their original grade, and the compacted backfill will exert no significant additional loads on the underlying undisturbed native ground. Only elastic recompression of the native ground induced by backfill placement is anticipated. Elastic recompression will occur quickly upon load application. The maximum recompression of undisturbed trench excavation bottoms for this project is estimated to be less than 0.5 inch and should occur upon backfilling. The maximum differential recompression between differing undisturbed ground types along the pipeline should be less than 1/4 inch.

2.11 Vertical Pipeline Loads

The vertical loads will consist primarily of dead loads imposed by trench backfill and are described below. In addition to dead loads, intermittent live loads may be imposed on the pipelines by vehicle traffic. Design criteria for live loads on the pipeline from vehicular traffic (H20 loading) are provided in Figure 7. The unit weight of CLSM or compacted Class 2AB materials, used for pipe embedment material and excavation backfill material (as recommended in this report), may be taken as 150 pcf, and the unit weight of compacted native soil may be taken as 125 pcf.

2.11.1 Rigid Pipe

Design criteria for dead loads on rigid pipe under trench conditions are presented in Figure 8. Design criteria for dead loads on rigid pipe under embankment conditions are presented in Figure 9.

The following Marston formula (Moser, 2008) may be used to estimate the vertical soil loads on rigid pipes placed in backfilled trenches. The vertical load is dependent on the width of the trench (B) in feet:

$$W = C \gamma B^2$$

where:

W = Vertical soil load on rigid pipe due to trench backfill (lb/ft);

γ = Unit weight of compacted backfill/overlying materials:
- 125 pcf for compacted native soil overburden;
- 150 pcf for Class 2 aggregate base or CLSM backfill.

C = Marston's coefficients for trench (t) conditions, presented graphically in Figure 8 for different trench depth (H) to width (B) ratios (i.e., H/B).

2.11.2 Flexible Pipe

Dead loads due to backfill soil overburden on a flexible pipeline assuming trench conditions can be estimated using the following Prism Method based formula (Moser, 2008):

$$W = D \gamma H$$

where:

W = Vertical soil load on a flexible pipeline due to trench backfill/overlying soil (pounds/linear foot);
D = Pipe outside diameter (feet);
γ = Unit weight of trench backfill (pcf); and
H = Height of trench backfill above the pipeline (feet).

Special attention is required where excavation widths are larger than common trenches (i.e., more than a few feet wider than the pipeline) since in such cases the loading on the pipe would be based more on embankment conditions. Pipe loading under embankment conditions is considerably greater than under trench conditions. Specific evaluation of pipe loading in excavations should be made based on the specific geometry of the excavation and the pipeline placement.

2.12 Composite Modulus of Soil Reaction

Vertical loads on flexible pipe cause the pipe to decrease in vertical diameter and increase in horizontal diameter. The horizontal movement develops a passive resistance which helps to support the pipe. The composite modulus of soil reaction (E'_c) is useful for estimating the passive soil resistance that will develop in a trench for flexible pipes. E'_c is a function of the soil modulus of the pipe zone material (E'_{pz}), the soil modulus of the trench wall material (E'_{tw}), trench width, depth of cover, and pipeline diameter (Figure 10). E'_{pz} and E'_{tw} are in turn a function of the strength of each material.

For new pipeline installed by open-cut trenching and bedded according to the materials and compaction recommendations provided in this report, an E'_{pz} will be constant at 2,000 psi. In order to maintain these minimum values, it is imperative that properly compacted pipe zone material not be disturbed or loosened by shoring removal.

E'_{tw} varies along the project alignment in proportion to the consistency and/or density of the soil forming the excavation walls at the depth of the pipeline. The excavation wall materials encountered in project test borings typically have an E'_{tw} ranging from 250 psi to 2,000 psi. E'_{tw}:E'_{pz} ratios for these soil types are provided in Table 12.

Using the chart provided in Figure 10, and based on an appropriate E'_{tw}:E'_{pz} ratio, the soil support combining factor S_c can be determined based on the trench width to pipeline diameter ratio used in design, and then used to calculate E'_c based on the formula $E'_c = S_c E'_{pz}$ from Jeyapalan (2001).

Table 12. E′c Parameters

E'_{pz} (psi) for CLSM or Compacted CL2AB [1]	E'_{tw}		$E'_{tw}:E'_{pz}$ Ratio
2,000 psi	Medium stiff or loose soil (3<N<8) [2]	250 psi	0.13
	Stiff or loose to medium dense soil (8<N<15)	500 psi	0.25
	Very stiff or medium dense soil (15<N<30)	1,000 psi	0.50
	Hard ground (N>30)	2,000 psi	1.00

[1] Pipeline embedment material specified and compacted as recommended in this report.
[2] N = ASTM D1586 Standard Penetration Blow Count.

2.13 Pavement Design

Prior to placement and compaction of Class 2AB along the road, the exposed subgrade should be stripped of unsuitable materials. Unsuitable materials include—but may not be limited to—dry, loose, soft, wet, expansive, organic, and compressible natural soils. The native soils exposed at the base of the areas to receive Class 2AB should be scarified to a minimum depth of 12 inches, moisture conditioned at or above optimum, and recompacted to a minimum relative compaction of 95% of maximum dry unit weight as determined by ASTM D1557. Exposed subgrade soils should be kept moist and not be allowed to dry out prior to the placement of Class 2AB. If surface shrinkage cracks are present in clay soils, the depth of scarifying and moisture conditioning should extend to the full depth of cracking. The Class II aggregate base should be compacted in lifts no greater than 8 inches in loose thickness to at least 95% of maximum dry density. The soil moisture content of the aggregate base should be near optimum soil moisture content. The maximum dry density and optimum moisture content should be determined by ASTM D1557 test methods.

The planned traffic loads for asphalt concrete trail will consist of occasional slow-moving maintenance trucks. For pavement design analysis, we have assumed flexible pavement with a basement soil R-value of 25 (silty clayey sand and poorly graded sand) and traffic index (TI) provided by GHD as presented in Table 13. The results of the pavement section are provided in Table 13.

Table 13. Asphalt Concrete Pavement Section

Traffic Index	Asphalt Concrete (in.)	Class 2AB (in.) (1)
4	4	5
5	5	5

(1) Class 2AB as described in Section 2.7.2

Portland cement concrete (PCC) pavement sections were calculated for traffic indices of 5 and 10 (as provided by GHD) for 10-foot-wide multi use trail and concrete truck apron at Fremont and SR 218 intersection, respectively. Pavement sections were designed per AASHTO guidelines for rigid pavement design (AASHTO, 1993) for concrete compressive strength of 5,000 psi and effective modulus of subgrade reaction of 100 pci. The results of the pavement section are provided in Table 14. PCC pavement sections should be reinforced with welded wire fabric or mild reinforcing steel placed within the middle 1/3 of the pavement section to help control concrete cracking.

Table 14. Portland Cement Concrete Pavement Section

Traffic Index	PCC (in.)	Class 2AB (in.) [(1)]
5	6	6
10	8	6

[(1)] Class 2AB as described in Section 2.7.2

3.0 Limitations

This report has been prepared for the exclusive use of GHD and TAMC for portions of the Canyon Del Rey/SR218 Segment of Fort Ord Regional Trail & Greenway (FORTAG) project in the city of Del Rey Oaks California, as described herein. Project details referred to herein are from information provided in the preliminary drawings prepared by GHD (2023).

Subsurface conditions between locations of the borings may vary from that logged in project borings as described and provided in the project GDR (MJA, 2021) and Geotechnical Design Report (MJA, 2023a). If the ground conditions that are exposed during construction differ from those indicated in project boring logs provided in the project GDR and the Geotechnical Design Report, then McMillen Jacobs Associates must be retained to evaluate the exposed ground conditions and to provide written confirmation or modifications to the recommendations provided in this report relative to the exposed ground conditions. The project budget and schedule should contain a contingency to allow for such evaluation and reporting.

The geotechnical recommendations provided in this report have been formulated in a manner consistent with the level of care and skill ordinarily exercised by members of the geotechnical profession currently practicing in the area under similar project constraints for this type of project.

4.0 References

American Society of Civil Engineers (ASCE). 2018. Minimum design loads for buildings and other structures. ASCE 7-16.

American Association of State Highway and Transportation Officials (AASHTO). 1993. AASHTO Guide for Design of Pavement Structures.

ASTM International:

ASTM C94 – Standard Specification for Ready-Mixed Concrete.

ASTM D1557 – Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3)).

ASTM D1586 – Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils.

ASTM D3786 – Standard Test Method for Bursting Strength of Textile Fabrics—Diaphragm Bursting Strength Tester Method.

ASTM D4355 – Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc-Type Apparatus.

ASTM D4491 – Standard Test Methods for Water Permeability of Geotextiles by Permittivity.

ASTM D4632 – Standard Test Method for Grab Breaking Load and Elongation of Geotextiles.

ASTM D4751 – Standard Test Methods for Determining Apparent Opening Size of a Geotextile.

ASTM D5261 – Standard Test Method for Measuring Mass per Unit Area of Geotextiles.

ASTM D6024 – Standard Test Method for Ball Drop on Controlled Low Strength Material (CLSM) to Determine Suitability for Load Application.

ASTM D6241 – Standard Test Method for the Static Puncture Strength of Geotextiles and Geotextile-Related Products Using a 50-mm Probe.

California Code of Regulations. 2019 California Building Standards Code. Title 24.

California Occupational Safety and Health Administration (Cal/OSHA). California Code of Regulations. Title 8 Program Regulations, Subchapter 4, Construction Safety Orders, §§1539–1547, Article 6, Excavation.

GHD. 2023. 100% submittal drawings for FORTAG Trail Project, Transportation Agency for Monterey County.

Jeyapalan, J.K. 2001. Modulus of Soil Reaction (E') Values for Pipeline Design. In *Proceedings of the ASCE Pipeline 2001 Conference, Advances in Pipelines Engineering and Constructio*n, July 15–18, San Diego, CA.

McMillen Jacobs Associates (MJA). 2021. Draft Geotechnical Data Report, Fort Ord Regional Trail & Greenway Canyon Del Rey/SR 218 Segment.

McMillen Jacobs Associates (MJA). 2023a. Fort Ord Regional Trail & Greenway SR 218 Undercrossing Bridge and Retaining Wall No. 1 – Final Geotechnical Design Report. Prepared for Transportation Agency of Monterey County.

McMillen Jacobs Associates (MJA). 2023b. Fort Ord Regional Trail & Greenway SR 218 Undercrossing Bridge – Final Foundation Report for Bridge. Prepared for Transportation Agency for Monterey County.

McMillen Jacobs Associates (MJA). 2023c. Fort Ord Regional Trail & Greenway Retaining Wall No. 1 – Final Foundation Report for Retaining Wall. Prepared for Transportation Agency for Monterey County.

Moser, A.P. 2008. *Buried Pipe Design,* 3rd Edition, New York, NY: McGraw-Hill, Inc.

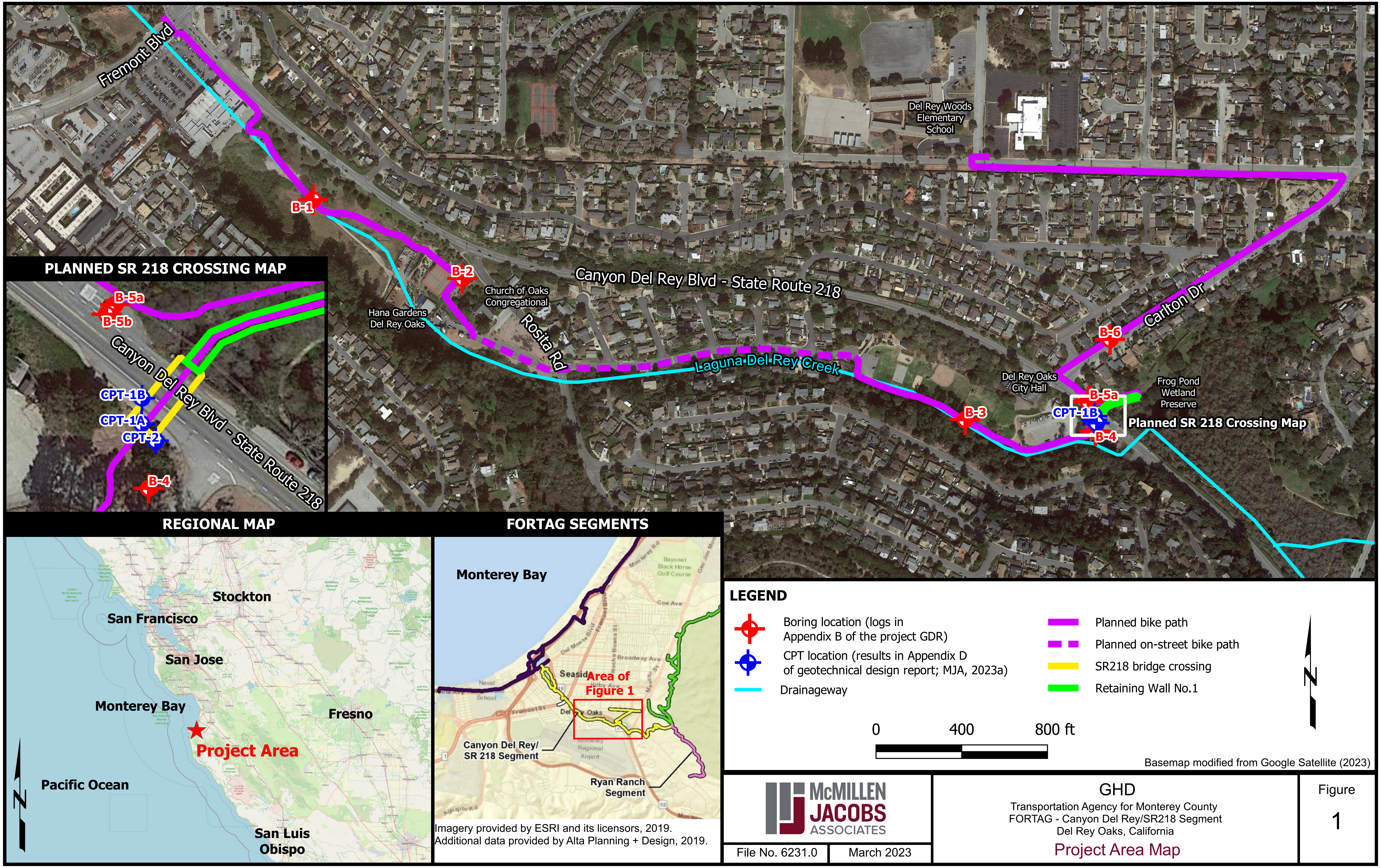
Naval Facilities Engineering Command. 1986. Chapter 3, Figure 12. *Design Manual 7.02 – Foundations and Earth Structures*.

Robertson, P.K. and C.E. Wride. 1998. Evaluating cyclic liquefaction potential using the cone penetration test. *Canadian Geotechnical Journal*, 35: 442–459.

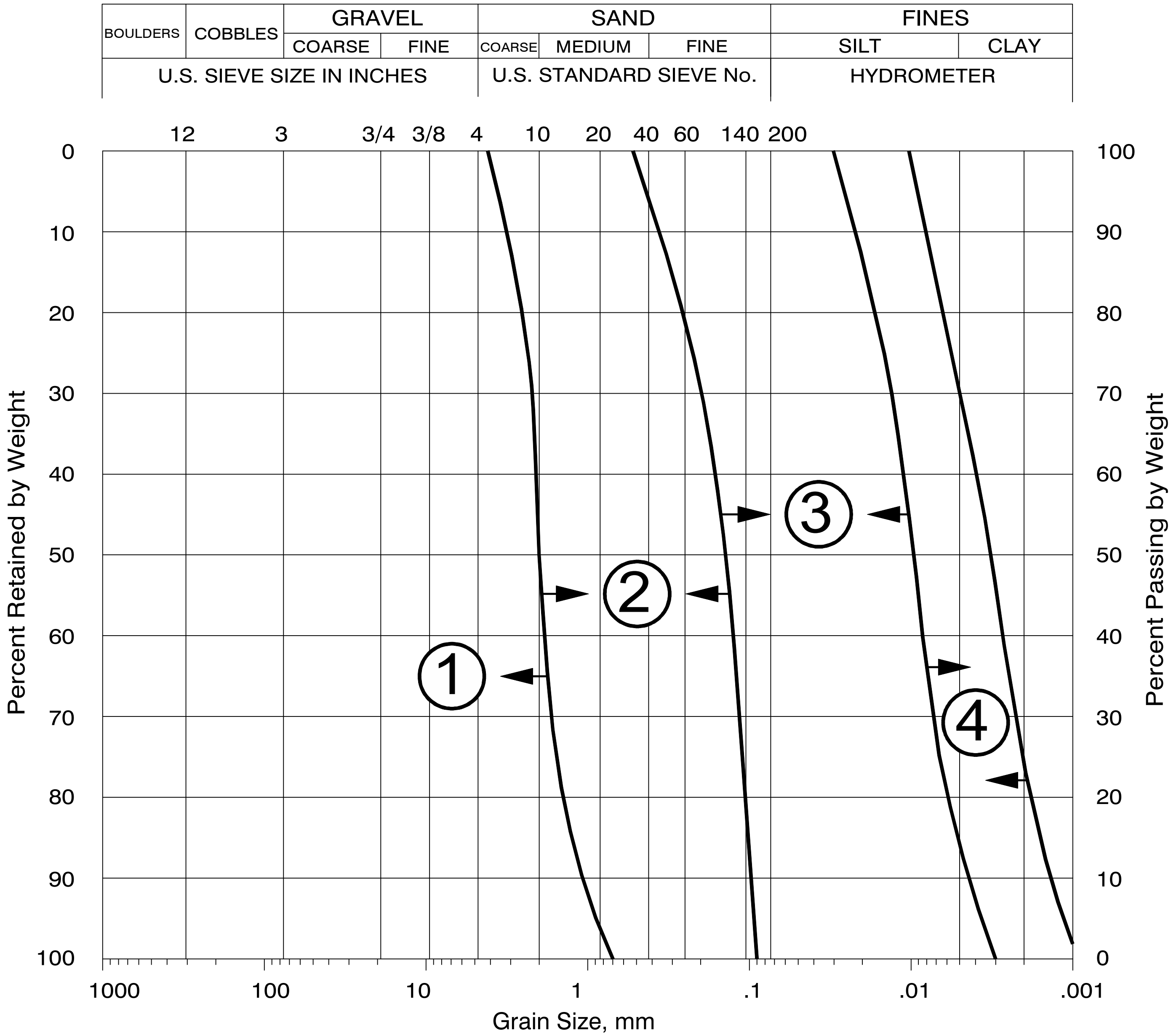
Suprenant, B.A., and K.D. Basham. 1993. *Excavation Safety, Understanding and Complying with OSHA Standards.* Addison, Illinois: The Aberdeen Group.

Whitson Engineers (Whitson). 2020. Topographic Survey, State Route 218, Transportation Agency of Monterey County, Dated: January 2020.

Figures



Fremont Blvd
B-1
B-2
Church of Oaks Congregational
Hana Gardens Del Rey Oaks
Rosita Rd
Canyon Del Rey Blvd - State Route 218
Laguna Del Rey Creek
Del Rey Woods Elementary School
Carlton Dr
B-6
B-3
Del Rey Oaks City Hall
B-5a
CPT-1B
B-4
Frog Pond Wetland Preserve
Planned SR 218 Crossing Map
PLANNED SR 218 CROSSING MAP
B-5a
B-5b
Canyon Del Rey Blvd - State Route 218
CPT-1B
CPT-1A
CPT-2
B-4
REGIONAL MAP
Stockton
San Francisco
San Jose
Monterey Bay
Fresno
Project Area
Pacific Ocean
San Luis Obispo
N
FORTAG SEGMENTS
Monterey Bay
Seaside
Area of Figure 1
Canyon Del Rey/ SR 218 Segment
Ryan Ranch Segment
Imagery provided by ESRI and its licensors, 2019.
Additional data provided by Alta Planning + Design, 2019.
LEGEND
Boring location (logs in Appendix B of the project GDR)
CPT location (results in Appendix D of geotechnical design report; MJA, 2023a)
Drainageway
Planned bike path
Planned on-street bike path
SR218 bridge crossing
Retaining Wall No.1
0
400
800 ft
N
Basemap modified from Google Satellite (2023)
McMILLEN JACOBS ASSOCIATES
File No. 6231.0
March 2023
GHD
Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oaks, California
Project Area Map
Figure
1



NOTES:

1. Subaqueous excavations or cutoff wall required.
2. Limits for gravity systems including sumps, well points, and deep wells.
3. Limits for well point vacuum methods.
4. Electro osmosis possible.

REFERENCE: Naval Facilities Engineering Command, 1986, Design Manual 7.02 Foundations and Earth Structures, Figure 14.

McMILLEN JACOBS ASSOCIATES

File No. 6231.0	March 2023

GHD

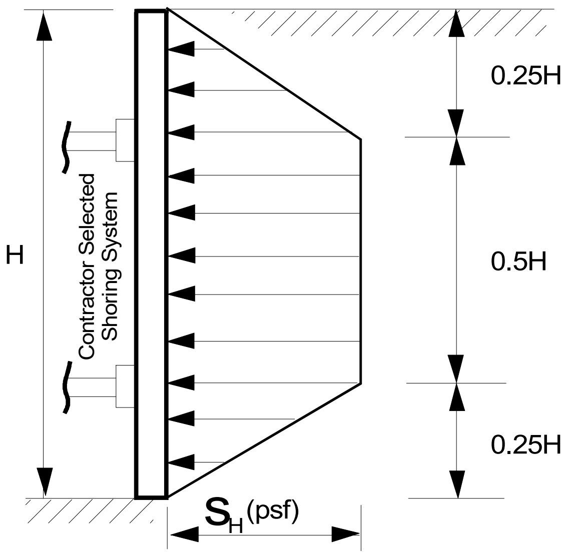
Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oak, California

Dewatering Limits vs. Grain Size

Figure

2

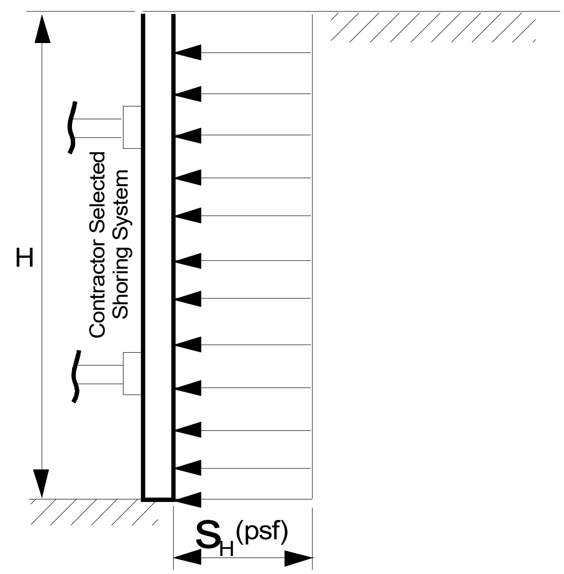
FINE-GRAINED SOIL PRESSURE DIAGRAM



S_H = 40H psf (medium stiff to stiff conditions)

S_H = 35H psf (stiff to hard condition)

COARSE-GRAINED SOIL PRESSURE DIAGRAM



S_H = 45H psf (loose to medium dense conditions)

① These diagrams are for excavations less than 30 feet in unsaturated ground as indicated.

② These preliminary pressure diagrams do not take into account hydrostatic pressures nor the effects of stockpiling of trench spoils, excavation equipment, etc. The effects of these conditions must be added to these pressure diagrams where applicable.

③ Excavation base stability should be analyzed after base width has been selected.

④ Final shoring pressure diagrams will need to be developed by the contractor's shoring designer based on the contractor's shoring system and the actual ground conditions encountered during construction.

McMILLEN JACOBS ASSOCIATES

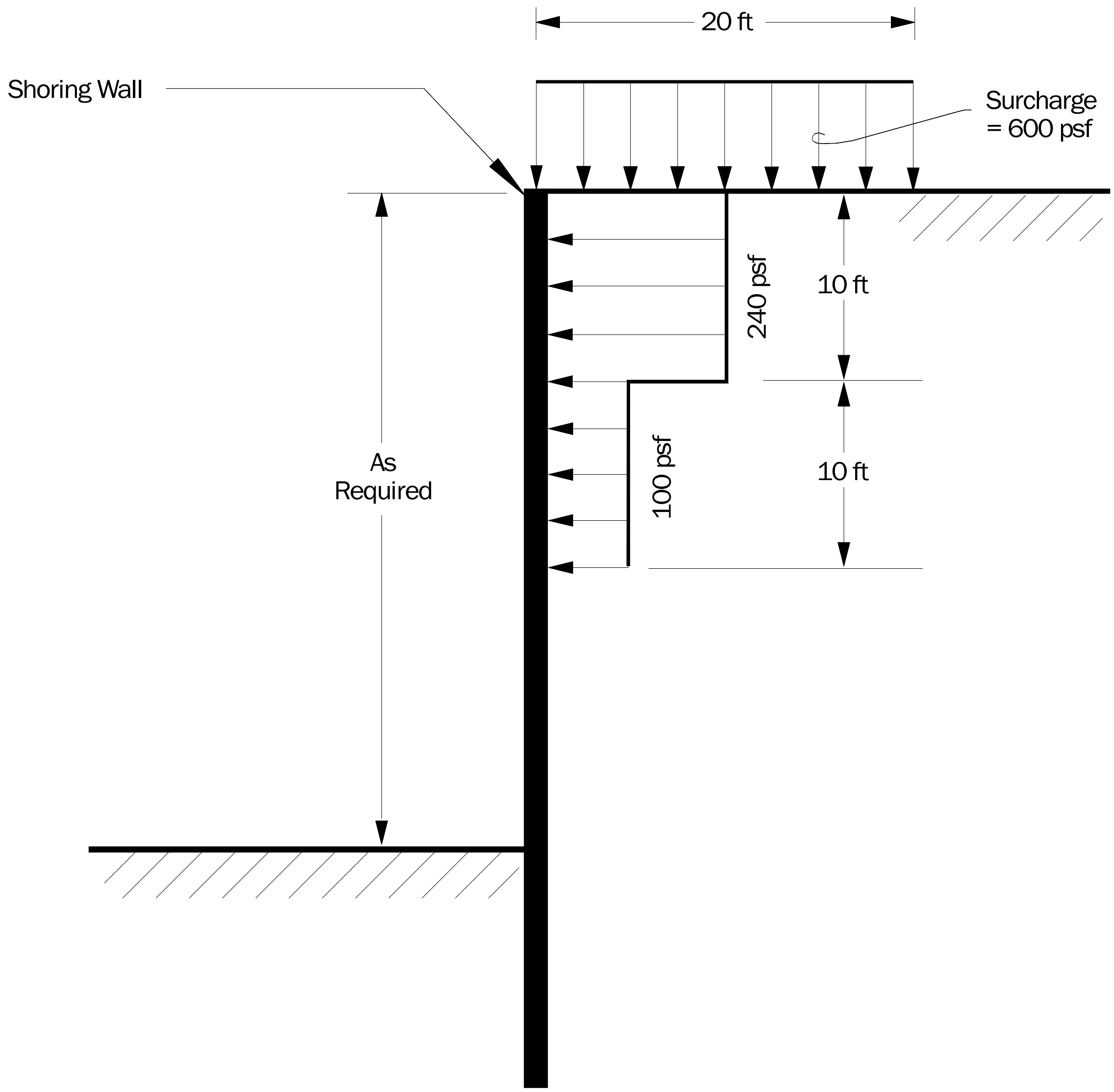
File No. 6231.0 | March 2023

GHD

Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oak, California

Preliminary Braced Shoring Pressure Diagrams

Figure 3



NOTES:

These are minimum shoring pressures to be used for traffic and equipment surcharges. Shoring pressures from construction activities or equipment that produce larger or different surcharge loading patterns than that shown should be determined by the shoring designer using geotechnical computational methods.

McMILLEN JACOBS ASSOCIATES

File No. 6231.0 | March 2023

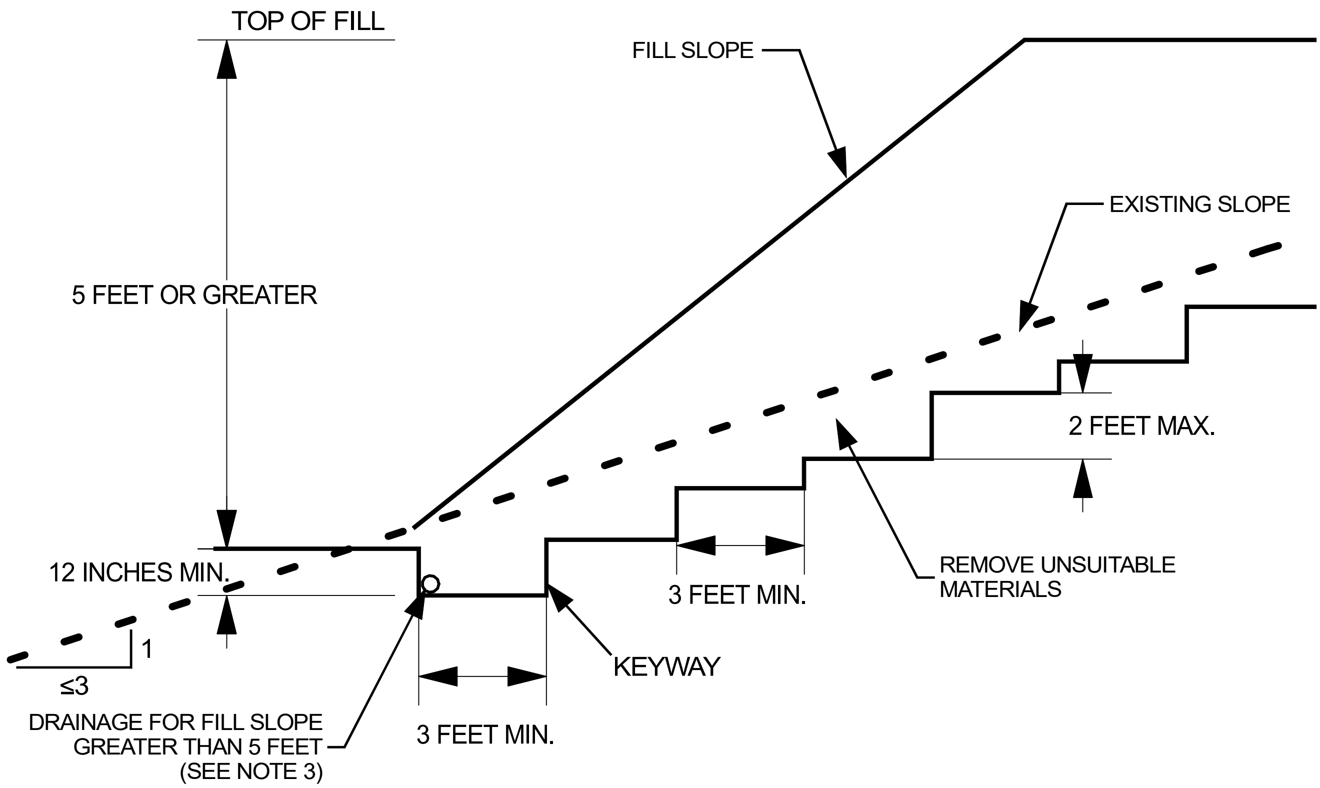
GHD

Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oak, California

Minimum Shoring Pressure for Traffic and Equipment Surcharge

Figure

4



NOTES:

1. Not to scale.

2. Keyway is required for embankment construction on both relative flat ground surface and sloping ground with an existing slope of 3H:1V or steeper. However, benching the existing slope is only required when the existing slope is 3H:1V or steeper.

3. Drainage is only required when native material along the existing slope has lower permeability than the embankment fill materials to prevent water building up between the existing slope and the embankment fill and potentially destabilizing the embankment side slope

McMILLEN JACOBS ASSOCIATES

File No. 6231.0 | March 2023

GHD

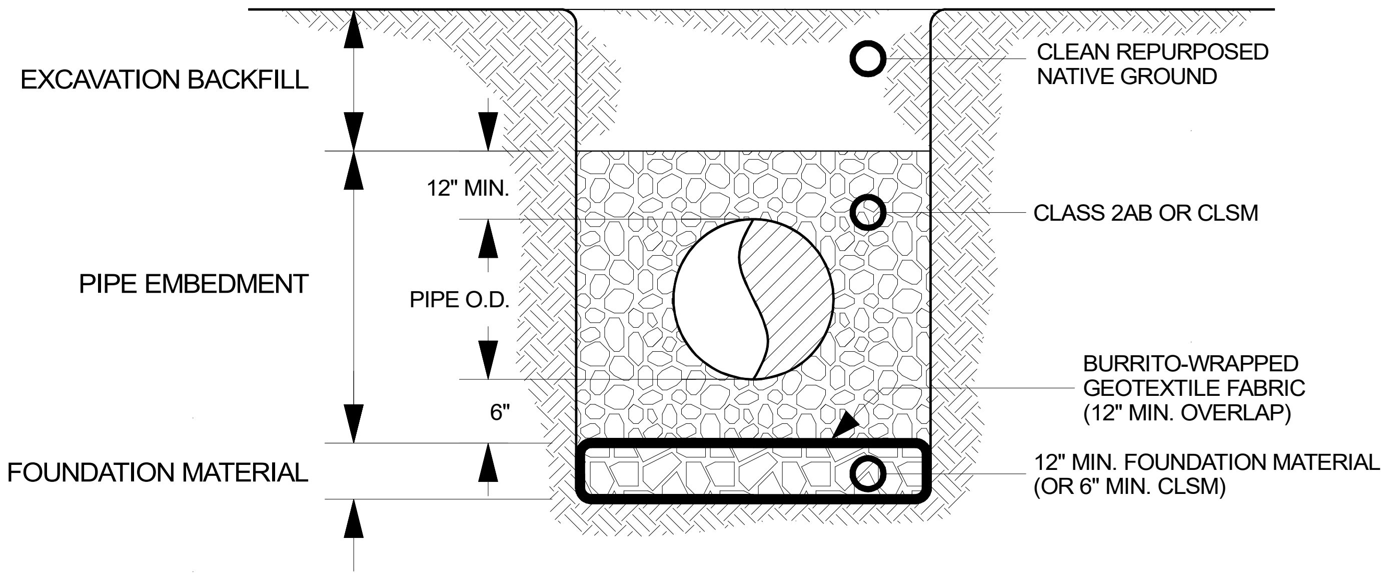
Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oak, California

Roadway Embankment

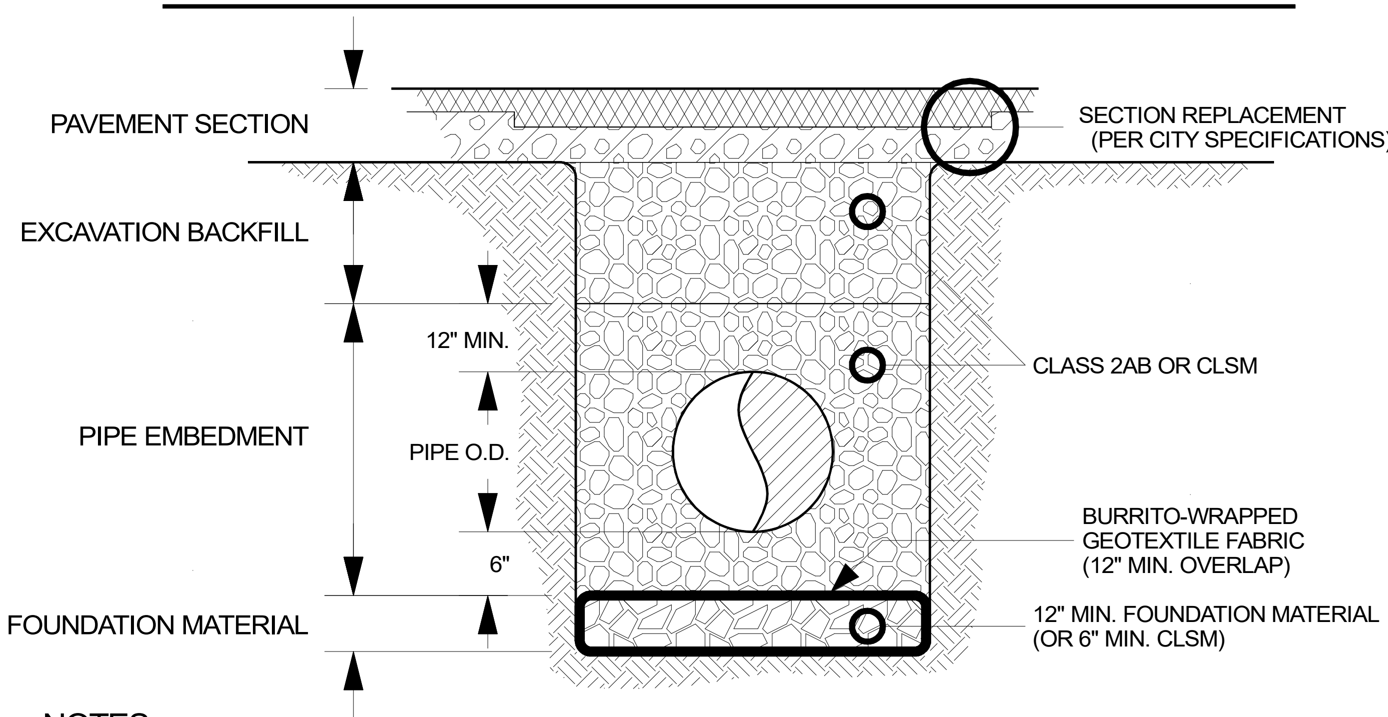
Figure

5

EXCAVATION BACKFILL BELOW UNPAVED AREAS



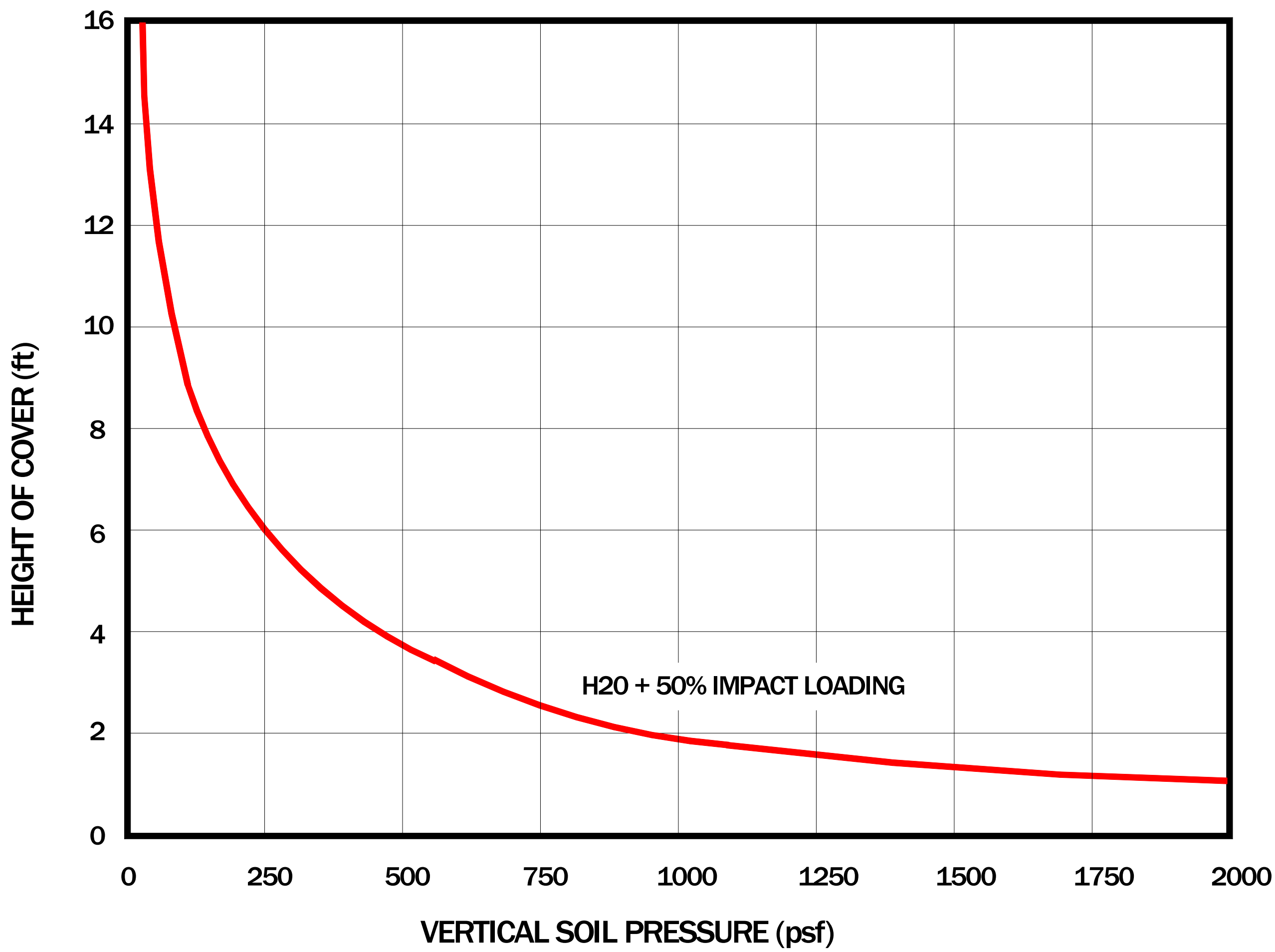
EXCAVATION BACKFILL BELOW PAVED AREAS



NOTES:

1. Not to scale.
2. See report text for material recommendations and compaction requirements.

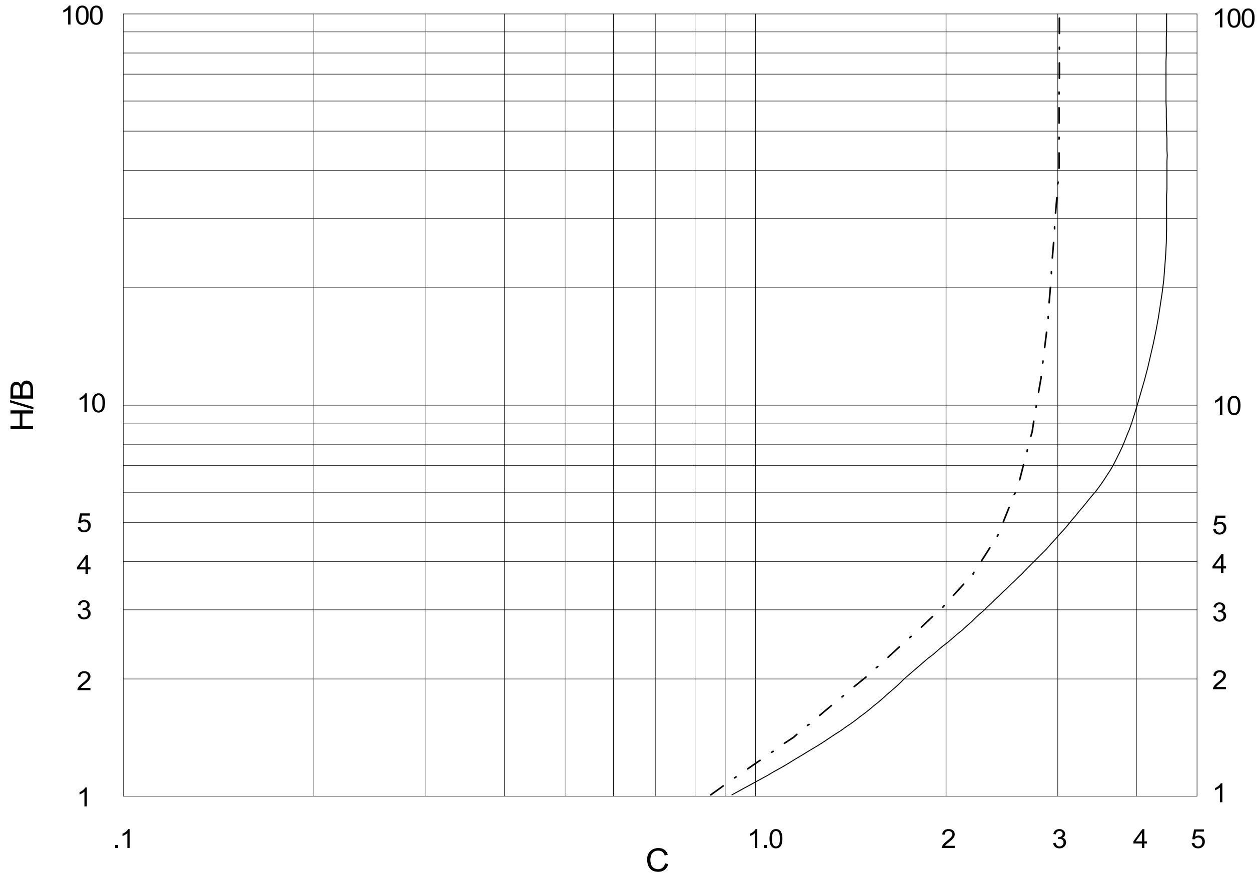
McMILLEN JACOBS ASSOCIATES	GHD Transportation Agency for Monterey County FORTAG - Canyon Del Rey/SR218 Segment Del Rey Oak, California Excavation Backfill	Figure 6
File No. 6231.0 — March 2023		



NOTES:

1. Apply vertical soil pressure to diameter of pipeline (horizontal projection) to calculate vertical pipe load.
2. H20 + 50% IMPACT LOADING: Simulates a highway load of a 20-ton truck with a 50% impact factor to account for the dynamic effects of traffic.
3. Modified from "Buried Pipe Design," Moser, A.P. and Folkman, S. Mcgraw Hill, New York, 2008.

McMILLEN JACOBS ASSOCIATES	GHD Transportation Agency for Monterey County FORTAG - Canyon Del Rey/SR218 Segment Del Rey Oak, California	Figure 7
File No. 6231.0 / March 2023	Vertical Soil Pressure Due to Live Loads	



LEGEND:

— · — · — · — = Compacted Granular Backfill (Class 2 Aggregate Base)

——————— = Excavated Fill Soil

$$W = C \gamma B^2$$

where: W = Vertical soil load on rigid pipe due to trench backfill (pounds/foot)

γ = Unit weight of trench backfill or overlying soil (pounds/cubic foot)

H = Depth of backfill (feet)

B = Trench width (feet)

NOTE: Marston's load coefficients are used to calculate vertical soil loads on rigid pipes installed by open-cut trenching. Refer to report text for soil loads on flexible pipes and pipes under embankment conditions (Moser, 2001).

McMILLEN JACOBS ASSOCIATES

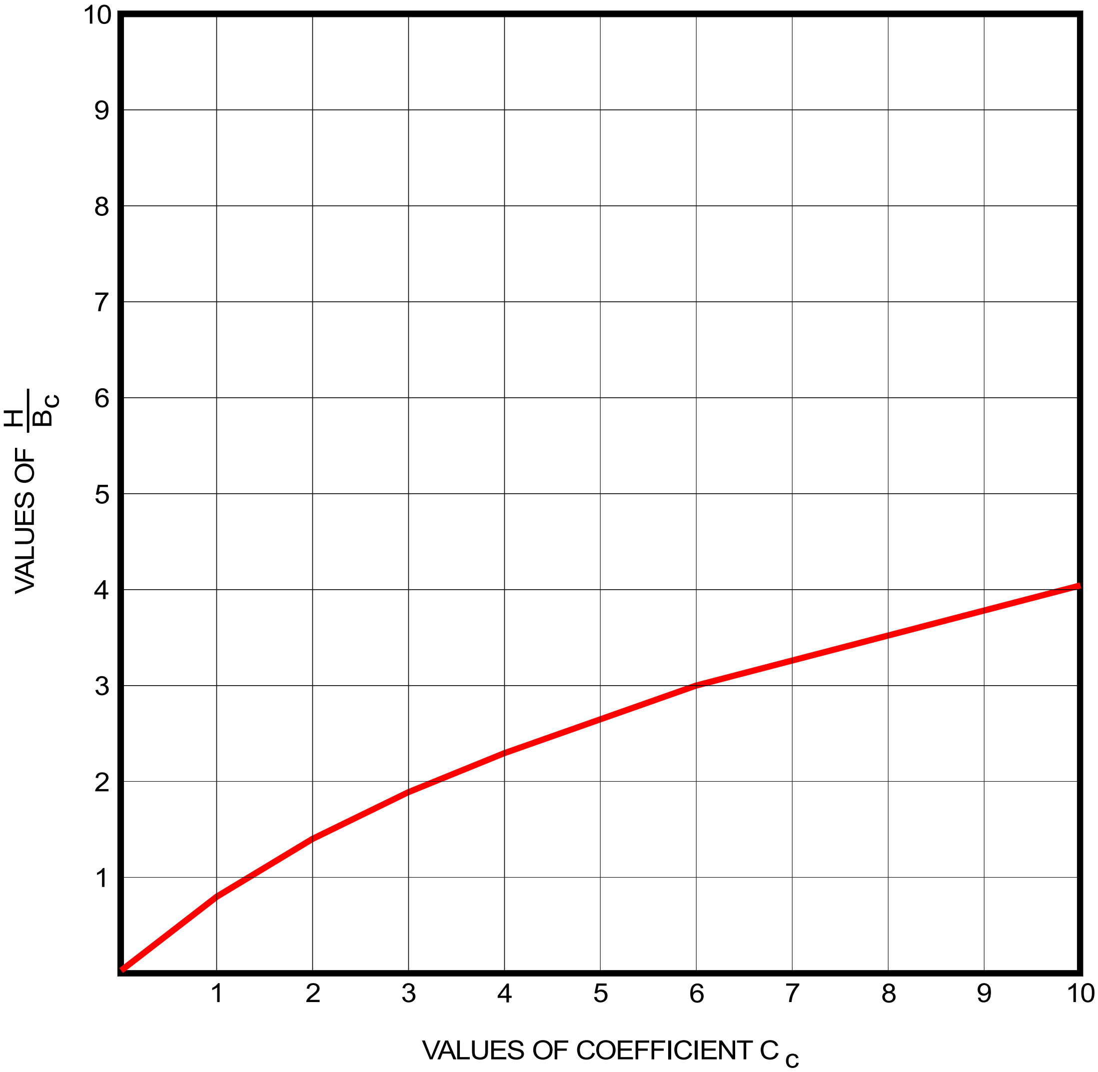
GHD

Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oak, California

Marston's Load Coefficients for Trench Conditions

Figure 8

File No. 6231.0 | March 2023



$$W_c = C_c \gamma B_c^2$$

where: W_c = Vertical soil load on rigid pipe due to embankment fill (ppf)

γ = Unit weight of embankment fill (pcf)

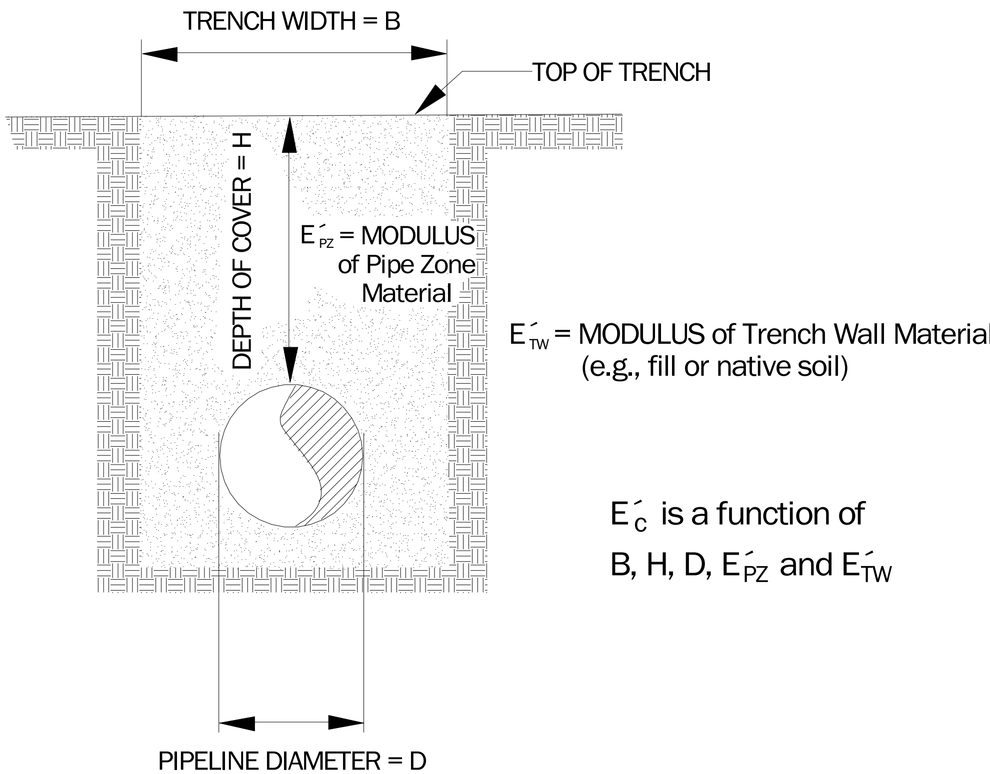
H = Depth of embankment fill (feet)

B_c = Outer pipe diameter (feet)

NOTE:
Marston's Load Coefficient, C_c is used to calculate vertical soil load on a rigid pipe under embankment loading (full positive projection).

Modified from Moser et al.,(2008)

McMILLEN JACOBS ASSOCIATES		GHD Transportation Agency for Monterey County FORTAG - Canyon Del Rey/SR218 Segment Del Rey Oak, California Marston's Load Coefficients for Embankment Loading	Figure 9
File No. 6231.0	March 2023		



E'_c is a function of B, H, D, E'_{PZ} and E'_{TW}

$\frac{E'_{TW}}{E'_{PZ}}$	S_c for various B:D ratios					
	1.5	**2.0**	**2.5**	**3.0**	**4.0**	**5.0**
0.1	0.15	0.30	0.60	0.80	0.90	1.00
0.2	0.30	0.45	0.70	0.85	0.92	1.00
0.4	0.50	0.60	0.80	0.90	0.95	1.00
0.6	0.70	0.80	0.90	0.95	1.00	1.00
0.8	0.85	0.90	0.95	0.98	1.00	1.00
1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.5	1.30	1.15	1.10	1.05	1.00	1.00
2.0	1.50	1.30	1.15	1.10	1.05	1.00
3.0	1.75	1.45	1.30	1.20	1.08	1.00
≥ 5.0	2.00	1.60	1.40	1.25	1.10	1.00

Modified from Jeyapalan (2001)

$$E'_c = (S_c)(E'_{pz})$$

McMILLEN JACOBS ASSOCIATES

File No. 6231.0 | March 2023

GHD

Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oak, California

Composite Modulus of Soil Reaction - E'

Figure 10