GEI Consultants

Geotechnical Design Memorandum

Thornton Source Water Pump Station

Larimer County, Colorado

Submitted to:
Carollo Engineers, Inc
11030 Circle Point Rd, Suite 400
Westminster, Colorado 80020

Submitted by:
GEI Consultants, Inc.
350 Indiana Street, Suite 750
Golden, CO 80401
303.625.9502

October 27, 2025 – Revised April 2026
Project No. 2406846

Sarah Myers, E.I.
Project Manager

COLORADO LICENSED PROFESSIONAL ENGINEER LANCE C. HEYER 10/27/25 49500

Lance Heyer, P.E.
Senior Professional

Table of Contents

List of Tables

Appendices

Executive Summary

The following report presents the results of GEI Consultants geotechnical design recommendations, and construction considerations in support of the Thornton Source Water Pump Station Project (Project) in Larimer County, Colorado. The project consists of a new pump station, surge tanks, intake pipeline and structure, restroom, access road, and LCC bridge box culvert. The proposed pump station discharge will connect into the Segment F pipeline (not included in this contract) for the Thornton Water Project. The purpose of this memorandum is to discuss geotechnical design recommendations and construction considerations for the proposed infrastructure. Subsurface condition data is included in the Geotechnical Data Report (GDR) submitted under a separate cover. The following summarizes our general conclusions, design recommendations, and construction considerations:

1. The encountered subsurface conditions do not preclude the proposed construction based on our current understanding of the project.
2. Drilled shafts are recommended for the proposed pump station, surge tanks and integral structures. Shallow foundations are recommended for the remaining structures with over excavation and replacement.
3. The proposed access road may consist of full depth asphalt concrete or a composite asphalt concrete and aggregate base course.
4. It is our understanding the water pipeline will be designed by others as part of the Thornton Water Project Segment F project. Pipeline recommendations in this report only apply to the inlet pipeline.
5. Bank stabilization near the reservoir shore should have maximum slope of 3:1 (H:V). If a steeper slope is elected, GEI can be retained to provide the necessary design parameters. Bedrock excavation may be required at a few locations during construction. We anticipate bedrock excavation may be completed using standard heavy-duty excavation equipment and will not require specialty equipment or blasting.
6. Groundwater was encountered during the subsurface investigation. Dewatering or watertight shoring will be required for the pump station. Depending on reservoir levels, cofferdams may be required for the intake structures and pipeline.

1. Introduction and Background

Carollo Engineers (Carollo) retained GEI Consultants (GEI) to provide geotechnical and tunnel engineering services for the Thornton Source Water Pump Station Project (Project) in Larimer County, Colorado. The proposed pump station is generally located between Water Supply and Storage Company (WSSC) Reservoir No. 3 and Rocky Ridge Lake Reservoir No. 1 immediately adjacent to the Larimer County Canal and parallel dirt access road. Both reservoirs are non-jurisdictional. Existing infrastructure consists of a concrete culvert diversion structure between the two reservoirs that also acts as the access road bridge. The site is characterized by generally rolling hills and drainages defined by outcrops of local bedrock. The City intends to install a new pump station, surge tanks, restroom, inlet pipe, and outflow pipeline connecting to the WSSC Reservoir No. 3. The proposed pump station will be located within the vicinity of the existing concrete culvert diversion. The pump station will consist of a wet well and bridge crane. Approximately 975 feet of inlet pipe will extend into the WSSC Reservoir No. 3. Additionally, the larger waterline will extend to the southeast as part of TWP Segment F (separate contract).

The City also intends to improve the existing access road leading to the project site, both for construction and for future water operations. The proposed road will extend east from Highway 1 and include a bridge cross of the Larimer County Canal before joining the existing west access road southeast of the pump station.

The purpose of this memorandum is to discuss geotechnical design recommendations and construction considerations for the project. The Geotechnical Data Report (GDR) provides description of the subsurface investigation and its findings.

2. Geotechnical Design Recommendations

The following sections are included primarily for the Engineer designing the Project. Design recommendations include foundation recommendations for the pump station and access road bridge wingwalls, surface and subsurface drainage, bank stabilization, subgrade conditions, lateral earth pressures, frost protection, water soluble sulfates, access road design to support emergency vehicles, and intake pipeline design recommendations. If additional geotechnical design recommendations not included in this report are required, GEI can be contacted to provide the additional information.

2.1. Foundation Recommendations

Based on the results of the subsurface investigation:

1. The proposed pump station will be bearing in bedrock (siltstone, shale, claystone and sandstone).
2. The proposed LCC bridge box culvert will be bearing in cohesive soil (clayey sand, lean clay with sand). The LCC Bridge box culvert should be protected from scour or extend below design scour depth (as determined by others).
3. The proposed intake pipeline will be bearing in cohesive soil (lean clay with sand, clayey sand) and bedrock (siltstone, shale, claystone and sandstone). The intake structures will be bearing in cohesive soil within the existing reservoir bottom.
4. The proposed restroom building will be bearing in cohesive soil.
5. The proposed surge tanks will be bearing near the cohesive soil/bedrock interface and separated from the main pump station building.

Table 2-1 Foundation Summary

Proposed Structure	Borings	Recommendation
Pump Station, Restroom Building and Electrical Building	B-6 MW, B-7, B-8, B-1C, B-2C	Due to swelling soil and bedrock, deep foundations (drilled shafts) are recommended. Regarding the electrical and restroom building piers, our recommendation is the minimum competent bedrock penetration start at the bottom of the wet well void form elevation. A greater than minimum competent bedrock penetration can be utilized for individual structures based on relative applied loading conditions which could result in variable structure drilled shaft lengths; the minimum penetration is specific to heaving bedrock and resistance to uplift. We recommend drilled shafts for the restroom building and electrical building based on the structure being integral with the pump station.

Proposed Structure	Borings	Recommendation
LCC Bridge Box Culvert	B-11, B-12	Shallow or mat foundations with a 1.5-ft overexcavation beneath the bedding zone and replacement with site-derived fill. A differential and total movement of 1 and 2 inches, respectively should be considered.
Intake Structure No. 1	B-1, B-2, B-3, B-4, B-5	Shallow foundations with a 5-ft overexcavation and replacement with site-derived fill or with CDOT Class 1 Structure Backfill.
Intake Structure No. 2	B-1, B-2, B-3, B-4, B-5	Shallow foundations with a 15-ft overexcavation and replacement with site-derived fill or with CDOT Class 1 Structure Backfill. Deep foundations (drilled shafts) are also acceptable.
Surge Tanks	B-6 MW, B-7, B-8, B-1C, B-2C	Deep foundations (drilled shafts) are recommended. The piers would provide additional resistance to total and differential movement. Shallow foundations with a 10-ft overexcavation and replacement with site-derived fill or with CDOT Class 1 Structure Backfill is also acceptable. GEI estimates 4 inches of total movement and 2 inches of differential movement using provided maximum allowable bearing pressure (if CLSM is not utilized).

2.1.1. Deep Foundations

We understand L-Pile or a similar modeling program may be used to analyze the drilled shafts. For L-Pile modeling, we recommend the following p-y curves are considered for design: “sand” for non-cohesive soil, “stiff clay w/o free water” for cohesive soil, and “weak rock” for bedrock. If an alternative modeling program is selected to analyze drilled shafts, we recommend a comparable p-y curve is selected for identified subsurface layers.

2.1.1.1. Drilled Shafts

Recommendations for the design of drilled shafts are presented below:

1. Individual shafts should be designed for a maximum allowable end bearing pressure of 50,000 pounds per square foot (psf) and a skin (shaft) friction value of 3,000 psf for the portion of the shaft in bedrock and below the top 5 feet of bedrock penetration. The design values should be used for the dead load plus full live load. The allowable bearing pressure may be increased by 33 percent for short term or infrequent loading such as wind and seismic loads.

2. Shafts should have a minimum penetration into the bedrock of 18 feet. The required penetration should be increased beyond this minimum, if necessary, to account for any dead load deficit. Reference Table 2-1 above for further building clarification. .
3. GEI recommends that a minimum dead load (in kips) of 10 times the diameter of the shaft (in ft) be maintained. Where the minimum dead load pressure cannot be achieved, the deficit should be made up by increasing penetration into the bedrock over the required minimum, using the shaft skin friction value provided above.
4. The drilled shafts should be reinforced for their full length with reinforcing steel sufficient to resist tensile forces due to expansive soils/bedrock. The size and amount of reinforcement should be determined by the structural engineer. Each shaft should be capable of resisting an uplift (in kips) of 30 times the diameter of the shaft (in feet).
5. A small diameter shaft designed to carry load by additional penetration is better suited to resist uplift pressures in comparison to a large diameter shafts with lesser penetration. Considering slenderness and for ease of installation and inspection, a minimum shaft diameter of 24 inches is suggested.
6. All slabs should be structurally supported on the drilled shaft foundations. Slabs-on-grade should be avoided. A void space of 6 inches should be established and maintained below any slabs and grade beams to concentrate the load on the shafts.
7. Movement of drilled shafts is needed to mobilize skin friction. GEI estimates 0.5 inches of vertical movement will be needed to mobilize skin friction and that the differential movement between individual shafts will be equal to or less than 0.5 inches.
8. GEI should be contacted if any shafts, will be spaced closer than 3 shaft diameters (center to center measured spacing). Shaft diameters closer than 3 diameters will require a reduction in recommended geotechnical design parameters.
9. Drilled shafts can also be considered in buoyancy resistance. The uplift loads should be resisted by additional shaft penetration beyond the penetration determined from Parts 2 and 3 above, taking in to account the dead load deficit. An uplift shaft friction value of 3,000 psf may be used.
10. No additional length is required to account for downdrag.
11. Drilled shaft construction should be observed by an experienced Geotechnical Engineer in accordance with the International Building Code. GEI can provide engineering representation during drilled shaft construction to satisfy this requirement.

2.1.2. Shallow Foundations

GEI recommends the following criteria be utilized in design of the spread footings or mat foundation elements:

1. The foundations should be designed using a maximum allowable bearing pressure of 3,000 psf This value includes appropriate allowable-stress-design (ASD) safety factors.

2. It is acceptable to increase the net allowable bearing capacity by one-third if the load combination utilized considers infrequent loads such as wind or earthquake. Snow loading is not considered an infrequent load condition.
3. Sliding friction between anticipated subgrade and foundation materials should be evaluated using a coefficient of 0.35.
4. The coefficient of subgrade reaction (k) is typically used for flexible foundation analysis and describes the load intensity per unit of displacement. For recompacted onsite fill or structural backfill, a k value of 125 psi/inch is recommended.

2.1.3. Lateral Earth Pressures

Lateral earth pressures presented in Table 2-2 below should be considered during the structural design process for foundation elements and structures extending below grade. Lateral earth pressure values are a function of the properties and the geometry of the retained fill, soil, and/or bedrock and anticipated magnitude of lateral deflection. In addition, the presence of groundwater and saturated materials will increase the total horizontal stress, resulting in higher lateral earth pressures in comparison to retained materials above a groundwater table. Dynamic lateral earth pressure values presented in Table 2-3 were determined in accordance with Mononobe and Okabe seismic coefficient analysis.

Table 2-2 Static Lateral Earth Pressures

Subsurface Material Type	Unit Weight (pcf)	Friction Angle (deg)	Static Earth Pressure Coefficients			Static Equivalent Fluid Pressure (pcf)[1]					
			Active	At-Rest	Passive	Active		At-Rest		Passive[2]	
						Above GWT	Below GWT	Above GWT	Below GWT	Above GWT	Below GWT
Site-Derived Fill	125	25	0.41	0.58	2.46	51	88	72	99	308	217
Imported CDOT Class 1 Structure Backfill	130	34	0.28	0.44	3.54	37	82	57	92	460	302

[1] GWT stands for Groundwater Table
[2] Values shown are half of theoretical calculated value due to large strains required to mobilize passive earth pressure

Table 2-3 Foundation Summary Additional Lateral Earth Pressures due to Seismic Loading

Backfill Material Type	Point of Application of Additional Loading	Seismic Earth Pressure Coefficients			Equivalent Fluid Pressure (pcf)					
		Active	At-Rest	Passive	Active		At-Rest		Passive	
					Above GWT[1]	Below GWT[1]	Above GWT[1]	Below GWT[1]	Above GWT[1]	Below GWT[1]
Imported CDOT Class 1 Structure Backfill or Site-Derived	0.6H up from the base of the wall	0.04	0.06	-0.09	5	5	7	6	-11	-11

Backfill Material Type	Point of Application of Additional Loading	Seismic Earth Pressure Coefficients			Equivalent Fluid Pressure (pcf)					
		Active	At-Rest	Passive	Active		At-Rest		Passive	
					Above GWT[1]	Below GWT[1]	Above GWT[1]	Below GWT[1]	Above GWT[1]	Below GWT[1]
Fill										

[1] GWT stands for Groundwater Table

In order for the lateral earth pressures presented above to remain applicable, backfill material placement adjacent to below-grade walls should be in accordance with procedures outlined in this report. In addition, backfilled material must be placed within a 1 horizontal to 1 vertical (1H:1V) backfill geometry, up and away from the base of the structure. Consideration has not been given to vertical loads applied to the backfill surfaces during or after construction as a result of traffic and/or other surcharge loads. The design high groundwater elevation should be taken as the 100-year flood elevation of Water Supply and Storage Reservoir Number 3, Rocky Ridge Lake Reservoir Number 1 and the Larimer County Canal as necessary.

2.2. Frost Protection

In colder climates, certain soils can freeze and can cause heave resulting in larger than expected differential movement. Typically, clays and clean sands or gravels have negligible frost susceptibility whereas silts, silty sands, and very fine sand have high frost susceptibility. Additionally, as moisture contents increase so does the potential for frost heave. We recommend that perimeter shallow foundations and shallow foundations in non-heated enclosed areas have a minimum cover of 30 inches for frost protection per International Building Code (IBC) and the Structural Design Information guidelines from the Larimer County Building Division (Larimer County, 2025).

2.3. Buoyancy Control

Structures which extend below the highwater elevation should be evaluated for buoyancy. When evaluating pipe segments for buoyancy, we recommend designing the pipe under the assumption that the pipe is empty during a critical high-water event, such as the 100-year flood, given the proximity of nearby water sources. Where the loading on the structures and pipe segments is not sufficient to counteract buoyancy, additional resistance to uplift can be achieved by using the following methods:

1. For structures, additional resistance to uplift may be achieved by extending the foundation outside the vertical walls and engaging the weight of an additional wedge of soil.
2. For pipes, additional resistance to uplift may be achieved by widening the pipe trench and placing concrete or controlled low-strength material (CLSM) around the pipe and within the wider pipe trench section to engage the weight of an additional soil wedge.

For design purposes, the wedge of soil providing resistance can be defined by including the soil within a 12-degree slope (measured from vertical) up and away from the bottom exterior edge of the extended footing or pipe trench to the ground surface. A saturated unit weight of 125 pcf may be used for the soil wedge if the buoyant force on the structure or pipe segment includes the weight of water displaced by the soil wedge. If the buoyant force on the structure or pipe segment does not include the weight of water displaced by the soil wedge (i.e., only the volume of water that is displaced by the structure itself) then a soil unit weight of 63 pcf should be used.

2.4. Surface and Subsurface Drainage

Construction details should ensure that surface water does not penetrate into utility trenches and migrate beneath the structures. Also, the upper few feet of structure backfill should consist of the onsite clayey soils to reduce the potential for surface water to migrate into foundation wall backfill. Vegetation should not be located within a distance of 5 feet from buildings. Irrigation systems should be arranged so that water under pressure does not spray within 5 feet of foundation walls. Surface finished adjacent to structures including paved surfaces and landscaping should be sloped away from foundations and vertical walls to prevent ponded water.

2.5. Corrosion

Pipe installation trenches are anticipated to be backfilled with properly compacted granular pipe bedding material. Depending on the specific pipe type selected, the corrosion potential of soils and water surrounding trench backfill should be assessed. Based on collected data presented in the GDR, the designer should account for the corrosive soil conditions when selecting metallic pipe materials and associated corrosion protection approaches.

Water soluble sulfate testing was conducted on representative samples. The test results would characterize the concrete risk as “Moderate”. These results should be considered when specifying the allowable cement types and concrete mixes.

2.6. Seismic Site Classification

Based on the International Building Code, the American Society of Civil Engineers Minimum Design Loads for Buildings and other Structures, our subsurface investigation, and our experience and knowledge of seismic conditions in the area, a Site Classification of C is recommended for seismic design.

2.7. Pavement Design Recommendations

The following sections include pavement design recommendations for the proposed access road intended to support emergency vehicles (HS-20 loading) and asphalt pavement around the pump station. Design recommendations provided are in accordance with AASHTO Guide for Design of Pavement Structures (AASHTO, 1993), the City of Thornton Standards and Specifications (City of Thornton, 2012), and the Larimer County Rural Area Road Standards (LCRARS) (Larimer County, 2007). Based on the size and purpose of the access road, design recommendations are based on the minimum requirements for a local roadway in accordance with Section 503 of the City of Thornton Standards and Specifications.

2.7.1. Pavement Design

GEI used the web-based application PaveXpress by the National Asphalt Paving Association to calculate recommended pavement thicknesses of asphalt concrete surfaced roads for the project listed in Table 2-

4. PaveXpress is based on calculation procedures from the AASHTO Guide for Design of Pavement Structures (AASHTO, 1993) for flexible pavements.

Equivalent Single Axle Loads (ESALs) corresponding to a load of 18,000 pounds were the primary input used to calculate pavement thicknesses. The minimum 73,000 ESALs for local roadways as outlined in Table 500-4 of the City of Thornton Standards and Specifications was used to calculate pavement thicknesses.

2.7.2. Subgrade

Subgrade below the access road is anticipated to consist of moist, medium stiff to stiff lean clay with varying sand content based on the boring logs presented in the GDR. Based on the subsurface investigation and laboratory testing, the subgrade is generally classified as AASHTO A-6 soil.

2.7.3. R-Value and Resilient Modulus of Subgrade Materials

The resilient modulus is a measure of subgrade stiffness used in pavement design. Resilient modulus is typically calculated empirically using the soil R-value. The R-value laboratory test measures the resistance to lateral deformation under vertical loading of a soil. Two laboratory R-Value tests were performed on two representative bulk samples consisting of a soil composite generated from borings B-9, B-10, B-14, and B-15 which resulted in a minimum R-value of less than 5. Using the laboratory R-value, PaveXpress calculated a resilient modulus of 3,775 pounds per square inch (psi).

2.7.4. Pavement Alternative Recommendations

Recommended pavement section alternatives and thicknesses are presented in Table 2-5 below. The parameters used to generate these values and the associated PaveXpress outputs for each alternative are included in Appendix A. Pavement designs for full-depth Aggregate Base Course (ABC) are not required per Section 5.2 of the LCRARS (Larimer County, 2007); recommendations for full-depth ABC are based on our interpretation of the available geotechnical information and our experience in similar conditions. Recommendations for pavement materials are provided in Section 3.2.5.

Table 2-4 Recommended Pavement Alternatives

Pavement Alternative	Material Thicknesses (inches)	
	Asphalt Concrete (AC)	Aggregate Base Course (ABC)
Full Depth Asphalt Concrete (AC)	6.5	0
Composite Asphalt Concrete (AC) and Aggregate Base Course (ABC)	4	8.5
Full Depth Aggregate Base Course (ABC)	0	12
Full Depth Recycled Asphalt	9	0
Composite Recycled Asphalt	4	10

2.8. Pipeline-Specific Recommendations

It is our understanding the water pipeline will be designed by others as part of the Thornton Water Project Segment F project. This section is for the inlet pipeline since conventional cut and cover has been selected.

2.8.1. Modulus of Soil Reaction (E')

The modulus of soil reaction (E') is an empirically derived coefficient which is related to the stiffness of the native soils and the nature and degree of compaction of the pipe bedding and backfill material. Various design guidelines specific to pipe material provide recommended values for E'. For full embedment of pipe in compacted granular pipe bedding, where the trench is excavated in soils that are loose/medium stiff or better (stiffer or denser, typically with blow counts greater than 4 blows per foot), and where the trench width measured at the pipe elevation (Bd) is at least 2 times the pipe diameter (D), modulus of soil reaction values as shown in Table 2-6 below can be used. For portions of the pipeline that have blow counts of 4 blows per foot or fewer, a lower E' value should be used.

Table 2-5 Modulus of Soil Reaction (E')

Pipe Type	E' (psi)	
	0- to 5-foot Depth of Cover	5- to 10-foot Depth of Cover
Steel[1]	1,000	1,400

[1] Per AWAA M11 (American Water Works Association, 2017)

2.8.2. Differential Movement

Pipelines excavated through various soils and bedrock will be susceptible to differential movements up to 2 inches. Pipe penetrations into structures should be designed to tolerate a total of 2 inches of upward or downward relative movement. Existing and proposed structures are expected to function as relatively rigid structures; therefore, flexible joints and/or connections between lines and structures are recommended. Allowance for differential movement should be expected and planned for where pipelines penetrate structures. We recommend CLSM backfill be used in pipe trenches within 10 feet of structures, in lieu of traditional pipe bedding and pipe trench backfill to mitigate the migration of groundwater which could lead to soil movement. GEI recommends pipe couplings at structure penetrations are capable of tolerating in-service dynamic movements.

2.8.3. Thrust Restraint

Buried, pressurized pipes experience thrust forces at various pipe configurations including tees, reducers, dead ends, valves, bends, and wyes. In order to balance internal transient pressures, thrust blocks or restrained joints are recommended for pipe restraint. For thrust block design, an allowable lateral bearing pressure of 1,500 psf is recommended. A minimum of 1 foot of cover over thrust blocks is recommended and thrust blocks should be oriented such that the passive pressure influence zones do not overlap for adjacent blocks and do not impact other subsurface utilities. If restrained joints are selected, various coefficients of friction for potential pipe materials against anticipated trench backfill material are presented in Table 2-6 below.

Table 2-6. Coefficient of Friction for Buried Pipe

Pipe Material	Coefficient of Friction
Steel	0.30

2.8.4. Seepage Control

Free-draining pipe bedding creates a conduit for the migration of groundwater along the trench. The migration of groundwater can alter local hydrogeologic regimes and can contribute to potential volume changes in moisture-sensitive (swelling or collapsible) soils. To reduce groundwater migration in the trench, GEI recommends that trench plugs be installed along the pipeline at a spacing not to exceed 1,000 feet and at structure locations.

Trench plugs should be constructed from low-permeability material and should be keyed into the in-situ soils along the trench bottom and sidewalls at least 12 inches and extending at least 12 inches above the top of the pipe bedding elevation. Trench plugs may be constructed from compacted clay soils with a USCS classification of CL or CH with at least 60 percent fines (passing the #200 sieve) and a Plasticity Index of 15 or greater. Trench plugs constructed from clay soils should have a minimum thickness of 2 feet as measured parallel to the pipe. Trench plugs may also be constructed from reinforced concrete, lean concrete, soil cement (flowable fill), or CLSM.

3. Construction Considerations

The following sections are intended for the Engineer producing specifications and the Contractor constructing the proposed project. Construction considerations include temporary excavations, backfill materials, fill placement and compaction, bedrock or oversize material excavation, drilled shaft, and construction dewatering.

3.1. Temporary Excavations

General site safety including temporary excavations are the sole responsibility of the contractor performing construction. GEI is providing temporary excavation information strictly as an informational benefit to the project team, specifically the general contractor. An Occupational Safety and Health Administration (OSHA) defined competent person should be identified by the contractor to be in charge of temporary excavations. In general, the contractor's competent person should have experience or training in determining soil types, benching and shoring, and have the ability to detect potential temporary slope stability and protective system issues.

OHSA defines an excavation as a man-made cut, trench, or depression formed by the removal of earth. A trench is a specific type of narrow excavation with a geometry including a greater depth than width and a width of 15 feet or less. Trenches 5 feet deep or greater should be sloped, retained with shoring, or shielded appropriately. Shielding most commonly includes trench boxes. In general, shoring can include inclined, horizontal, or vertical systems depending on the excavation geometry and availability of retention alternatives. Sloping and benching should be in accordance with OSHA guidelines (Ocupational Safety and Hazard Administration, 2025). Benching should include a maximum 4-foot vertical face for each bench and overall excavation geometry less than or equal to the OSHA defined slope.

Deep excavations are anticipated. A registered professional engineer should approve the contractor's approach for trenches greater than or equal to 20 feet in depth. In addition, shoring for trench excavations should be designed by a professional engineer or be based on tabulated data prepared or approved by a registered professional engineer in accordance with OSHA 1926.652(b) and (c) (Ocupational Safety and Hazard Administration, 2025).

GEI has evaluated observed soil conditions likely to be penetrated by the proposed construction. Based on the OSHA determined soil Types, Table 3-1 below presents maximum recommended temporary excavation slopes to be utilized during construction.

Table 3-1 OSHA Temporary Excavation Slopes

Backfill Material Type	OSHA Classification	Maximum Recommended Slope (H:V)
Non-Cohesive Soils	Type C	1½:1
Cohesive Soils	Type B	1:1
Bedrock	Type A	0.75:1

[1] H:V is an abbreviation for Horizonal:Vertical
[2] Valid for trench excavations less than 20 feet in depth

During construction, heavy equipment or excavated material stockpiles should be kept away from excavation edges to the extent possible. It is recommended that heavy equipment or excavated material stockpiles be kept at least 10 feet from the edges of excavations. Underground utilities should be fully understood and documented prior to initiating excavations. Finally, the contractor's competent person should inspect trenches and excavations routinely for signs of instability including sliding, toppling, subsidence and bulging, heaving or squeezing, boiling, and/or other visual concerns.

3.1.1. Bedrock Excavation

Based on the subsurface investigation, bedrock consisting of siltstone, shale, sandstone and claystone of the Pierre Shale will be encountered during excavation for the project. The use of heavy-duty excavation equipment is anticipated to be sufficient to excavate the bedrock present at the project site. Heavy-duty excavation equipment may consist of excavators using a bucket equipped with appropriate teeth such as tiger. In some cases, in the sandstone an excavator mounted hydraulic breaker or hammer will be necessary to supplement excavation. Unconfined compressive strength testing values within bedrock are as high as approximately 4 ksi which should be considered in excavation equipment selection.

3.1.2. Permanent Slopes/Bank Stabilization

GEI recommends a maximum slope of 3:1 (H:V) for permanent cuts and fills. Erosion control may be required for slope stabilization. There are several options regarding erosion control including erosion control blankets, fiber rolls, reinforcement mats and vegetation and other measures. If a slope greater than 3:1 (H:V) is required for permanent cut and fill slopes, then GEI can provide retaining wall design.

3.1.3. Shored Temporary Excavations

The contractor may elect to use a temporary shoring system to limit excavation extents for the project. For excavations greater than 20 feet deep (intake structure, wet well and portions of the inlet pipeline), the contractor will be required to submit a design prepared by a professional engineer registered in the State of Colorado to demonstrate that the shoring system maintains both local and global slope stability for the duration of the open excavation.

The Contractor will develop a construction sequencing plan that meets stakeholder requirements, manages cost and construction risk and mitigates public impacts. In general, the construction sequence will progress as follows: temporary shoring (predrilled sheet piles or secant piles), material excavation then drilled shaft construction. However, depending on the selected support, material excavation may occur prior to the temporary shoring. The method of installation, quality of workmanship, and the sequence of excavation can influence the overall area of influence and the ground's response. We recommend temporary shoring is offset from permanent foundation members by a minimum of 10 feet to prevent impact of disturbed ground on performance of the permanent foundation system. Further, we recommend these limits be identified on the Construction Drawings.

Typically trench boxes are used to support lateral loads during excavation and construction in soils that have sufficient standup time to excavate the trench, install the trench box, complete installation, and remove the trench box without full contact of the soils. Tight shoring typically involves full contact of the soils with the shoring system. Typical tight shoring systems include predrilled sheet pile walls, ring beam and liner plates, soldier pile and lagging walls, secant pile walls, soil nail walls and slide rail systems. There is the option to leave the sheet piles or secant piles in place or only remove the top several feet. Best practice will be to provide support as soon as practical after excavation in stages or concurrent with excavation. All methods require expertise and experience by a competent shoring contractor.

3.2. Site Grading and Earthwork

Appropriate site preparation, material placement and compaction, and backfill selection can reduce the risk of post-construction settlement and potential issues related to lateral earth pressures. General site grading and earthwork considerations are presented in the following sections.

3.2.1. Site Preparations

Areas supporting backfill and pipe bedding should be properly prepared. Once the rough grade has been established in excavated areas, and prior to placement of backfill, the exposed subgrade should be carefully inspected via either proof rolling or probing and testing, as determined necessary by the geotechnical engineer and as outlined in this report. Proof rolling is required in addition to the compaction tests for the box culvert and roadway. Should the subgrade be considered unstable by the geotechnical engineer or should the contractor desire to improve workability conditions in the trench or excavation, the procedures outlined in Section 3.4 should be followed. Frozen, wet, soft, or loose soil, as well as any other undesirable material should be removed. Once suitable soil conditions are achieved, the subgrade should be scarified and compacted prior to fill placement. The suitable exposed soil materials should be scarified and moistened or dried, as necessary, to a minimum depth of 8 inches below the proposed construction. Scarified material should be compacted to the minimum specifications defined in the following sections.

A minimum over excavation of 1.5 feet is required beneath the LCC Bridge box culvert bedding zone. This over excavation should extend a minimum of 2 feet outside of the area of the box culvert in all directions at the depth of over excavation. The zone should also continue up and away from the base of excavation at a 1H:1V slope.

3.2.2. Structure and Trench Backfill Material

Backfill is anticipated adjacent to structure walls, and within pipeline trenches. Pipeline trenches and foundation walls may be backfilled with site-derived fill or with CDOT Class 1 Structure Backfill. If another material is considered for any of the presented applications, GEI should be contacted to review submitted particle size distribution and plasticity testing results.

For CDOT Class 1 Structure Backfill, the maximum PI is 6 and maximum LL is 35; gradation is as shown in the Table 3-2 below:

Table 3-2 CDOT Class 1 Structure Backfill

Sieve Size	Percentage Passing
2 inch	100
#4	30-100
#50	10-60
#200	5-20

CDOT Class 1 Structural Backfill should be used as fill beneath proposed foundations and backfill around proposed structure walls providing the appropriate lateral earth pressures are used in design.

Over excavation replacement material may consist of lean clay or CLSM for the backfill of the over excavation zone of foundation excavations as described in Section 2. We recommend that lean clay be non-expansive consisting of a minimum fines content of 50 percent and a minimum plasticity index of 8. We anticipate that onsite soils will be suitable for use; however, processed existing bedrock is not recommended for over excavation backfill.

Piping is anticipated to be within cohesive or non-cohesive soil or bedrock. Material meeting the requirements of the City of Thornton Standards and Specifications shall be squeegee, non-fractured, rounded, and shall conform to the following limits when tested by means of laboratory sieves shown in Table 3-3 below. This material should be of a minimum of 6 inches and maximum of 12 inches of bedding material placed on the trench bottom and the bedding shall then be installed to a minimum of 6 inches and a maximum of 12 inches above the top of the pipe is recommended as backfill for the pipe bedding zone. Due to pipelines extending through expansive soils and bedrock, an allowance for differential movement up to 2 inches where pipelines enter rigid structures such as the pump station or intake structure should be accommodated with the use of flexible connections.

Table 3-3 City of Thornton Pipe Bedding Material

Sieve Size	Total % Passing by Weight
3/8-inch	100
No. 200	0-5

A minimum 6 inches of Class 6 base coarse material or better should be incorporated as the bedding zone directly beneath the LCC Bridge box culvert.

3.2.3. Reuse Potential of Excavated Soils

GEI reviewed the reuse potential of the expansive soils for backfill in order for the excavated material to be used around the project site. In general, we do not recommend utilizing excavated expansive soils or bedrock as fill for any excavation or adjacent to subsurface structural walls due to the risk of further expansion and associated structure performance and heave. However, if reuse of excavated expansive soils is considered, separation and/or processing of the onsite excavated materials will be required. A sufficient laydown area will be needed to break up (commonly via disking), blend, and moisture condition excavated materials. Disc blades should be notched and equipped with self cleaning scrapers to prevent the accumulation of materials between and on the discs. Processing onsite materials should generally include breaking down clods/chunks into a soil like consistency with maximum particle sizes to not exceed 0.5 inches and moisture conditioning to greater than optimum as presented in Table 3-4. Due to the risk of expansive soil backfill, GEI recommends continuous observation of the processing, moisture conditioning, and backfill process by a qualified Geotechnical Engineer during construction. GEI can be retained in this role at the direction of Carollo or the City.

From a risk management perspective, GEI recommends removal and replacement of the expansive soils. Segregating materials utilizing higher quality import materials at structure locations and reprocessed expansive soils in general fill areas that can accommodate heave risk can also be considered.

3.2.4. Fill Placement and Compaction

We recommend fill placement occurs in maximum 8-inch loose lifts for fill under and backfill adjacent to structures. Lift thickness for pipe bedding should not exceed 6 inches. Minimum recommended compaction specifications are outlined in Table 3-4 below.

Table 3-4 Soil Compaction Recommendations

Backfill Material Type	Moisture and Compaction Specifications		Minimum Relative Density
	Moisture Content[1]	Dry Density[2]	
Pipe Bedding			≥ 65%
Over Excavation Backfill	-1% to +3%	≥ 95%	
Site-Derived Trench Backfill and Backfill Against Exterior Walls	-1% to +3%	≥ 95%	
CDOT Class 1 Structure Backfill and Non-Frost Susceptible Soil	-2% to +2%	≥ 95%	
CDOT Class 2 (Reprocessed On-Site Expansive Soils)	%1% to +3%	95% to 98%	

[1] Moisture content relative to the optimum moisture content as determined by Standard proctor compaction testing (ASTM D698)
[2] Dry density relative to the maximum dry density as determined by Standard proctor compaction testing (ASTM D698)

Mechanical compaction is required for all materials placed as backfill during construction. Compaction of cohesive soil is best accomplished with equipment such as a jumping jack or padfoot roller. Non-cohesive soil is best compacted with a vibratory plate or vibratory smooth-drum roller. Compaction utilizing any flooding type technique should not be allowed. Care should be taken when compacting backfill adjacent to structures. Generally, we recommend operating only light-weight compaction equipment such as jumping jacks and vibratory plates immediately adjacent to structures.

Quality assurance of backfill material and backfill placement will be necessary to reduce potential for long term differential settlements. Inspection of subgrade materials prior to placing or forming and casting structural elements is also critical to project success. GEI recommends a qualified testing agency is retained to provide quality assurance services during the backfill process.

3.2.5. Pavement Materials

The following sections provide recommendations for pavement materials to be used in the construction of the access road and road around the pump station including Asphalt Concrete (AC) and Aggregate Base Course (ABC). The native subgrade should be scarified, moisture conditioned, and compacted prior to placement of pavement materials as described in Section 3.2.2.

3.2.5.1. Hot Mix Asphalt Pavement Materials

AC pavement should consist of Hot Mix Asphalt (HMA) composed of aggregates with an asphalt binder and anti-stripping additives. The HMA pavement should consist of a plant mix composed of a mixture of high-quality aggregate and bituminous material, which meets the requirements of a job-specific mix formula established by the HMA supplier's engineer. The asphalt material should be based on a SuperPave Gyratory Design Revolution of 75 with an asphalt mix Grading SX and an asphalt cement binder grade PG 64-22. The asphalt material may be placed in a single 3-inch lift.

3.2.5.2. Aggregate Base Course

If applicable to the chosen pavement design, Aggregate Base Course (ABC) shall consist of either CDOT Class 5 or CDOT Class 6 base course with a maximum liquid limit (LL) of 30 and a grain size distribution meeting the gradation requirements outlined in Table 3-5 below. ABC should be properly moisture conditioned and compacted prior to placement of AC as outlined in Section 0. For full-depth ABC, a geotextile fabric such as Tensar BX1300, Tensar TX140, Mirafi BXG120, or approved equivalent should be placed between the prepared subgrade and the compacted ABC.

Table 3-5. CDOT Class 1 Aggregate Base Coarse

Sieve Size	Percentage Passing	
	Class 5	Class 6
1 1/2-inch	100	-
1-inch	95-100	-
3/4-inch	-	100
#4	30-70	30-65
#8	-	25-55
#200	3-15	3-12

3.3. Construction Dewatering

Groundwater was encountered during the subsurface investigation. For the wet well, a watertight shoring system will be necessary for successful construction and use. In general, sump pumps may be used in order to control minor groundwater seepage. Deep wells or well points may be ineffective at this site due to the clayey nature of subsurface materials. Construction dewatering must adhere to the Colorado Department of Public Health and Environment (CDPHE) regulations and discharge permit requirements. GEI is available to assist with dewatering design if required. Dewatering should be carefully evaluated and accounted for in the construction estimate to avoid construction cost and time increases.

Low permeability backfill is commonly used around structures to minimize the movement of water through soils adjacent to foundations. Typically composed of compacted clay or specially engineered materials, this backfill reduces hydraulic conductivity, thereby limiting water infiltration and protecting structural integrity. In contrast, secant pile walls are a structural system formed by constructing overlapping concrete piles, often reinforced, to create a continuous barrier. While both approaches aim to limit water movement, secant pile walls offer higher structural strength and can withstand greater lateral earth pressures, making them ideal for complex projects where stability and groundwater exclusion for deep excavations.

3.3.1. Intake Pipeline and Structures

Depending on canal and reservoir conditions, construction near or into the reservoir may require use of cofferdams or canal diversions to facilitate construction. For the intake pipeline, open-cut pipeline construction would require progressive watertight cofferdam construction and dewatering along the entire underwater alignment to a width that allows trench excavation and shoring/support of excavation within the reservoir bottom sediments. The cofferdams are typically built to form a water barrier, providing a dry working area. Reservoir depths of approximately 37 feet at maximum operating capacity would require significant structural design and heavy construction of a water retention system to resist hydrostatic pressure and control water inflows to the working area. If the water level in the reservoir was lowered for construction, this would greatly reduce cost, risk, and resources required for open-cut intake pipeline construction. It is our understanding that the reservoir will be lowered from September 26 to March 27. GEI recommends construction occurring during this timeframe.

After cofferdam construction and dewatering, a conventional trench excavation and pipeline installation would be completed "in the dry". The area enclosed by the cofferdam should be minimized for construction costs but should be consistent with construction requirements, including space to accommodate berms, access roads/ramps, internal drainage features, and a reasonable working area. A soil berm may be constructed along the interior edge of the dam to provide additional sliding and overturning resistance. The berm will also serve to lengthen the seepage path and decrease the upward seepage gradients on the interior of the dam. However, a berm will require a larger cofferdam enclosure, increasing associated construction costs and durations. If a double wall or cellular cofferdam is selected, it may be advantageous to increase the width of the dam rather than constructing an interior berm.

Cofferdam sheet piles should be embedded into the claystone bedrock to provide a low permeability base to the cutoff. The depths of overburden into which the cells and cutoff wall are driven should be limited to about 30 feet in order to prevent driving the pile out of interlock. Cofferdams are often connected to land at a sloping shoreline. The cofferdam should be designed to integrate into the support of excavation system utilized at the pump station shaft.

3.3.1.1. Cellular Cofferdam Construction (Maximum Water Level)

Cellular sheet pile cofferdams consist of sheet pile "cells" filled with earth materials. Cellular cofferdams are widely used in large marine construction projects to retain water with hydrostatic pressures greater than 30 feet and up to over 100 feet of head. Planning, design, and construction of these structures must be accomplished by the same procedures and with the same high level of engineering competency as those required for permanent features of the work.

Almost all modern cellular sheet pile structures are designed based on the assumption that a free draining granular fill will be available near the construction site. Materials encountered during the geotechnical investigation may not be suitable for granular free draining fill. The performance of the sheet pile structure is directly related to the drainage characteristics of the cell fill. If lower quality fill is to be used, the effects on structural design should be considered against the cost of the fill.

Diver inspection of the interlocks, after filling of the cells, should be required. The cofferdam cells should be located a sufficient distance from open excavations such as a trench required for intake pipeline installation.

3.3.1.2. Double Wall Sheet Pile Cofferdam (Moderate Water Level)

Double wall sheet pile cofferdams consist of parallel continuous sheet pile walls, connected with either wales, bracing, or tie rods, often filled with earth materials. Double wall sheet pile cofferdams are commonly used to resist hydrostatic pressures up to approximately 30 feet of head.

Similar to cellular sheet pile structures, many double wall sheet pile cofferdams are designed based on the assumption that a free draining granular fill will be available near the construction site. Materials encountered during the geotechnical investigation may not be suitable for granular free draining fill. The performance of the sheet pile structure is directly related to the drainage characteristics of the cell fill. If lower quality fill is to be used, the effects on structural design should be considered against the cost of the fill.

3.3.1.3. Cantilevered Single Wall Sheet Pile Floodwalls (Low Water Level)

Cantilevered sheet pile floodwalls rely on passive earth pressures within the embedded length for structural stability and are typically limited to low wall heights of less than 10 to 15 feet. The above ground portion of the sheet piles will leak through the interlocks and will need to be sealed.

Combined wall systems with king piles and intermediate piles may be utilized to increase the horizontal capacity of the wall. In combined wall systems, the intermediate piles transfer loads to the king piles, which carry the majority of the bending moment and shear and are typically designed with H-pile or pipe sections. A connector is welded to the king pile in order to provide interlock between the king pile and sheet pile.

Single wall sheet pile cofferdams are often built with upstream and downstream berms. Similar to most dams, berms serve to provide additional sliding and overturning resistance, lengthen the seepage path, and generally increase capacity of sheet piles, but require free-draining granular fill material, often in large volumes for construction, which as discussed herein may not be available onsite.

3.3.1.4. Water Inflow Control

The bedrock encountered at the site provides a relatively suitable foundation material for cofferdam construction given the low permeability and in-situ strength. The maximum thickness of soil overlying bedrock of 24 feet along the intake alignment is suitable for watertight sheet pile embedment.

Dewatering systems intended to lower groundwater levels such as pumping wells will be difficult to implement due to the fines content of the soil and bedrock on site. Seepage rates should be estimated and accounted for during design of temporary shoring and construction methods. The high fines content and low permeability of the soil and rock will assist in resisting inflows, but watertight construction methods should be planned as inflow rates will be difficult to lower when encountered.

Care should be taken to prevent loss of sheet pile interlocks during driving, especially at greater depths, and depending on cofferdam design, sealing of the interlocks may be necessary to mitigate seepage, and diver inspection to check for interlock separation may be required. With any cofferdam design, there will be seepage and inflows requiring continuous dewatering after lowering the water level within the working area.

3.4. Unstable Subgrade Conditions

Unstable subgrades may be encountered for the intake structure. If soft/loose pockets or undesirable materials are encountered in the excavations, the proposed excavations may be re-established by backfilling after the soft/loose material has been removed. At locations where the subgrade material is too soft to facilitate adequate workmanship for the proposed pipeline or results in undesirable deflections, we recommend an over excavation of 12-inches beneath the bottom of the pipe bedding zone and replacing with crowding aggregate. Crowding aggregate material up to 3- to 4-inches in diameter, such as recycled concrete, is recommended to stabilize the soft subgrade beneath the pipe bedding zone. We recommend placing a separating geofabric such as Mirafi 150N between the stabilization aggregate and pipe bedding to prevent material migration. Should an over excavation be undesirable, a stabilization fabric such as a Tensar Biaxial Geogrid BX1200 or Mirafi Woven Geotextile HP270 may be utilized in substitute of over excavation and crowding aggregate. Field observations by a geotechnical engineer and testing of the open excavations should be performed prior to concrete placement to verify the subgrade conditions and/or to make corrective recommendations, as necessary.

3.5. Drilled Shaft Construction

An experienced and competent contractor should be retained to install the drilled shafts. The contractor will select the appropriate equipment to drill the shafts based on the site conditions and design parameters. These machines are capable of reaching significant depths while maintaining hole stability and verticality. Standard shaft depths range from 20 to 150 feet. Specialized drilling equipment is required for depths greater than 250 feet. Heavy-duty hydraulic rotary drilling rigs equipped with high-torque capabilities are used to penetrate hard rock formations. Drill bits are selected based on the specific rock type, with roller-cone, core barrel, or rock auger bits being common choices for strong bedrock.

Furthermore, it is recommended that a competent geotechnical engineer be retained for full time inspection during drilled shaft installation per the International Building Code. The shaft holes should be kept as plumb as possible; a maximum deviation of one percent variation from plumb in relation to shaft length is recommended. Bedrock unconfined compressive strength testing produced values as high as approximately 4 ksi which should be considered when selecting auger teeth, auger taper, and anticipated excavation rates. Prior to concrete placement, the lower ten feet of each drilled shaft should be roughened with a “saw-tooth” to create a roughened surface and increase skin friction of the shafts to the recommended value. The concrete should be placed in such a manner as to prevent hitting the sides of the shaft hole and also to prevent segregation of the aggregate. The use of a funnel and chute (elephant trunk) for concrete placement is recommended. The use of a tremie or concrete pump lowered to the placement depth will also reduce segregation of the concrete. Concrete should be placed at a high slump (6- to 8-inches) in order to reduce the possibility of necking and to help ensure water displacement. Concrete for each shaft should be formed at the top of the shaft, if necessary, to ensure a uniform diameter at the top of the shaft and to prevent “mushrooms” from forming. We do not anticipate the need for casing to hold the hole open prior to pouring concrete.

4. Construction Observation and Quality Assurance

Site observations during construction specifically to address the geotechnical components of the project by GEI are recommended including trench and structure excavations, placement of pipe bedding, and backfill placement and compaction. Quality assurance of compaction and backfill materials are necessary to ensure long-term performance of the proposed waterline and to avoid differential settlements. GEI recommends that a qualified testing agency is retained during construction to monitor backfill materials and compaction. GEI should be notified if unanticipated subsurface conditions are encountered during construction in order to re-evaluate and potentially change the design and/or construction recommendations presented in this report.

5. Limitations

This study was conducted in accordance with generally accepted geotechnical engineering and engineering geologic practices and principles; no warranty, express or implied is made. The subsurface conditions described in this report were based on data obtained from widely spaced exploratory borings, geotechnical laboratory testing, information provided by the client, engineering judgement, and our experience with similar subsurface conditions and projects. Subsurface conditions are typically variable, both laterally and vertically, and the nature and extent of the subsurface variations across the site may not become evident until construction. The boundaries between different soil types and bedrock presented in this report are approximate and, in some cases, may be more abrupt or gradational than described herein. Groundwater levels may vary with time, adjacent surface water levels, precipitation, and changes to the hydrogeological conditions at or surrounding the project site.

This report has been prepared exclusively for our client for design purposes for the subject project. GEI Consultants is not responsible for technical interpretations by others of the data presented in this report or use of this report by others for the subject project or other projects. If differing site conditions are encountered during further evaluation of the subsurface conditions by others or during construction, GEI Consultants should be notified immediately to determine if any changes to our recommendations presented in this report are warranted.

The recommendations presented in this report are only intended for the proposed design and construction as understood by GEI Consultants at the time of issuing this report. If the proposed design and construction changes, GEI Consultants should be notified immediately and given the opportunity to review the proposed changes and if necessary, modify our recommendations presented herein.

An environmental assessment was not included in GEI Consultants scope of work for this project. Any statements regarding the absence or presence of hazardous and/or toxic substances presented herein are only intended for informational purposes. If the client is concerned about the environmental conditions at the site, GEI Consultants recommends the client and/or owner retain a qualified environmental firm to conduct an environmental site assessment.

6. References

American Association of State Highway and Transportation Officials. (n.d.). AASHTO Standards. Washington, D.C.

American Society for Testing and Materials. (n.d.). ASTM Standards. West Conshohocken, Pennsylvania: ASTM International.

American Water Works Association. (2017). *AWWA M11 Steel Pipe - A guide for Design and Installation.* Denver.

American Water Works Association. (n.d.). AWWA Standards. Denver, Colorado.

Colorado Association of Geotechnical Engineers. (1996). *Guideline for Slab Performance Risk Evaluation and Residential basement Floor System Recommendations (Denver Metropolitan Area).* Denver.

Colorado Department of Transportation. (2023). Section 703 Aggregates. *Standard Specifications for Road and Bridge Construction*. Denver, Colorado.

Larimer County. (2025). Structural Design Information.
https://www.larimer.gov/sites/default/files/uploads/2022/structural_design_information_0.pdf

Ocupational Safety and Hazard Administration. (2025). *29 CFR Part 1926.* Washington, D.C.

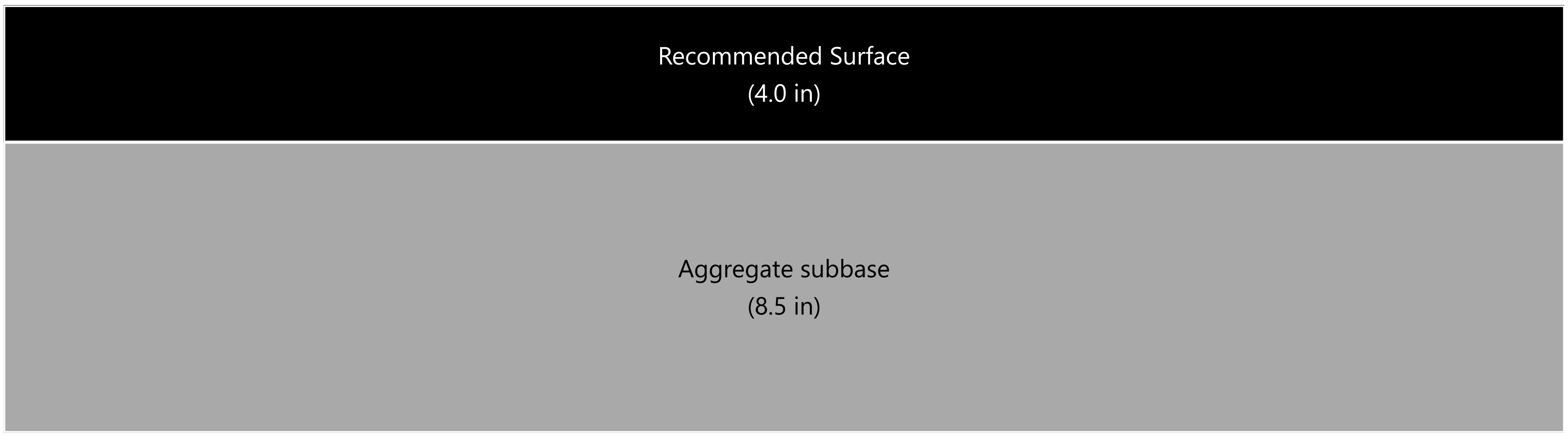
Appendix A - Pavement Section Alternatives and Thicknesses Parameters

Project: Larimer County Source Water Pump Station

Emergency Access Road - Composite Section

AASHTO '93/'98: Flexible Pavement Design

Pavement Diagram



Required minimum design SN: 2.65

Layer Thicknesses (in)

Recommended Surface: 4.0 in

Aggregate subbase: 8.5 in

Total SN: 2.70

Print

Details

Scenario: Emergency Access Road - Composite Section

Created By: Robert Johanson, rjohanson@geiconsultants.com

Last Modified: January 15, 2025 3:22:17 pm

Design Parameters

Design Period: 20 years

Reliability Level (R): 75%

Combined Standard Error (S_0): 0.5

Initial Servicability Index (p_i): 4.5

Terminal Servicability Index (p_t): 2

Delta Servicability Index (ΔPSI): 2.5

Total Design ESALs (W_{18}): 73000

Layers

Recommended Surface - Asphalt

Thickness: 4 in

Aggregate subbase - Base

Thickness: 8.5 in

Structural Coefficient: 0.11

Drainage Coefficient: 1

Project: Larimer County Source Water Pump Station

Emergency Access Road - Full Depth Asphalt

AASHTO '93/'98: Flexible Pavement Design

Pavement Diagram

Recommended Surface
(6.5 in)

Required minimum design SN: 2.65

Layer Thicknesses (in)
Recommended Surface: 6.5 in

Total SN: 2.86

Print

Details

Scenario: Emergency Access Road - Full Depth Asphalt

Created By: Robert Johanson, rjohanson@geiconsultants.com

Last Modified: January 15, 2025 3:20:48 pm

Layers

Recommended Surface - Asphalt

Thickness: 6.5 in

Design Parameters

Design Period: 20 years

Reliability Level (R): 75%

Combined Standard Error (S_0): 0.5

Initial Servicability Index (p_i): 4.5

Terminal Servicability Index (p_t): 2

Delta Servicability Index (ΔPSI): 2.5

Total Design ESALs (W_{18}): 73000

Project: Larimer County Source Water Pump Station

Pump Station - Full Depth Asphalt

AASHTO '93/'98: Flexible Pavement Design

Pavement Diagram

Required minimum design SN: 2.65

Layer Thicknesses (in)
Recommended Surface: 9.0 in

Total SN: 2.70

Print

Details

Scenario: Pump Station - Full Depth Asphalt

Created By: Robert Johanson, rjohanson@geiconsultants.com

Last Modified: May 19, 2025 1:11:06 pm

Layers

Recommended Surface - Asphalt

Thickness: 9 in

Design Parameters

Design Period: 20 years

Reliability Level (R): 75%

Combined Standard Error (S_0): 0.5

Initial Servicability Index (p_i): 4.5

Terminal Servicability Index (p_t): 2

Delta Servicability Index (ΔPSI): 2.5

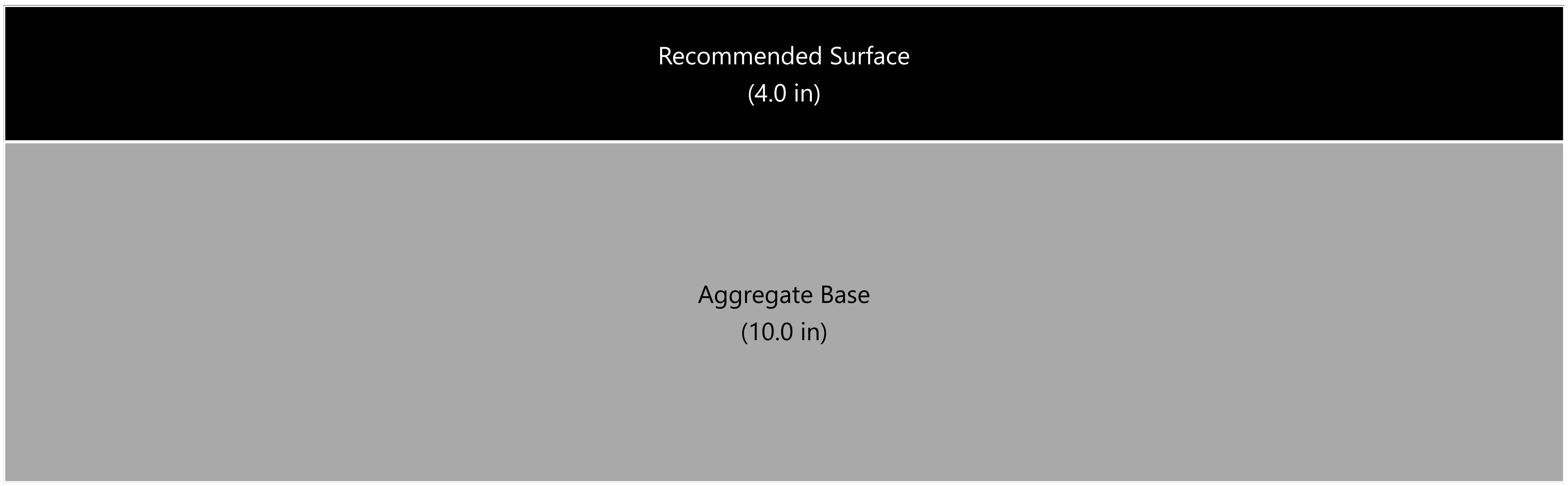
Total Design ESALs (W_{18}): 73000

Project: Larimer County Source Water Pump Station

Pump Station - Composite Section

AASHTO '93/'98: Flexible Pavement Design

Pavement Diagram



Required minimum design SN: 2.65

Layer Thicknesses (in)

Recommended Surface: 4.0 in
Aggregate Base: 10.0 in

Total SN: 2.70

Print

Details

Scenario: Pump Station - Composite Section

Created By: Robert Johanson, rjohanson@geiconsultants.com

Last Modified: May 19, 2025 1:09:55 pm

Design Parameters

Design Period: 20 years

Reliability Level (R): 75%

Combined Standard Error (S_0): 0.5

Initial Servicability Index (p_i): 4.5

Terminal Servicability Index (p_t): 2

Delta Servicability Index (ΔPSI): 2.5

Total Design ESALs (W_{18}): 73000

Layers

Recommended Surface - Asphalt

Thickness: 4 in

Aggregate Base - Base

Thickness: 10 in

Structural Coefficient: 0.15

Drainage Coefficient: 1