# Final

# Preliminary Geotechnical Evaluation

# Technical Memorandum

# Canyon Viaduct Alternative

# Floyd Hill, Clear Creek County, Colorado

Yeh Project No.: 218-300

August 7, 2020
Prepared for:

Atkins
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# 1. PURPOSE AND SCOPE

This technical memorandum presents our preliminary geotechnical engineering investigation to support design decisions for the proposed westbound (WB) and eastbound (EB) Interstate 70 (I-70) rock cuts related to the proposed Floyd Hill highway realignment. The study was performed in accordance with project constraints and our proposal to Atkins dated January 2, 2020.

## 1.1 Purpose of Work

The I-70 Floyd Hill Realignment Project is proposed to improve highway safety and capacity from the Beaver Brook Interchange to the Veterans Memorial Tunnels. Potential improvements include bridge structures and multiple rock cuts for both eastbound and westbound highway re-alignments.

## 1.2 Scope of Work

Project limits for this scope are bounded by approximately CDOT Milepost 243.5 to Milepost 244.0, though on the south side of Clear Creek opposite the existing highway. The performed scope of work included the following:

- Research the project area using published sources and existing information for evaluating the geologic setting and existing areas of slope instability.
- Identify and characterize the geological and geotechnical hazards directly or potentially impacting I-70, including landslides, debris flows and rockfall hazard areas.
- Obtain necessary right-of-entry and access permits (Clear Creek County Open Space Permit) in preparation of field reconnaissance.
- Conduct field mapping of the exposed rock slopes and observed geological hazards.
- Conduct a preliminary analysis of the rock slopes.
- Conduct geotechnical reconnaissance, including a site visit to verify appropriate drilling access and methods.
- Field investigation preparation. Obtain necessary right-of-entry, drilling/excavation permits (Clear Creek County Open Space Permit), utility clearances, etc.
- Conduct geotechnical drilling investigation. The subsurface investigation will at a minimum follow the sampling protocol delineated in the current version of the Field Materials Manual. Investigate and characterize soils and rock for potential cut materials and pavement materials. Collect samples of soil materials for laboratory testing. Collect and log recovered rock core and evaluate for rock mass characteristics using the Rock Mass Rating system or equivalent.
- We assume all boring locations are accessible by a track-mounted drill rig, from the recreational trail. Reclaim all borings and excavations to a condition acceptable to the Clear Creek County Open Space Permit. Log and mark all exploration/sampling locations to be surveyed by others. Compile field notes, field boring logs, photos, sketches, etc. Photograph all sites of investigation.

- Laboratory testing of soil/rock for engineering properties, primarily in the Yeh and Associates, Inc. laboratory though a limited number of specialized rock tests may be performed using an outside laboratory.
- Provide a Preliminary Geotechnical Evaluation Memorandum that presents the findings of the field and subsurface investigation. The memo will summarize the geotechnical and geological conditions and mention preliminary issues/constraints.

# 2. BACKGROUND INFORMATION

The purpose of this I-70 Floyd Hill project is to perform a preliminary feasibility assessment of the proposed bridge structures and support highway design decisions. Continuing west from the base of Floyd Hill, which routinely experiences traffic backup conditions, the highway follows the contours of the slope above Clear Creek along the base of steep rock cuts.

## 2.1 Geologic Site Conditions

A wide range of geologic conditions are represented and exposed along the I-70 corridor due to the extensive period of time represented in the multiple rock formations. The geologic time reflected along the corridor ranges from recent river and debris flow deposits to Precambrian rocks between 1 and 2 billion years old. The Precambrian age metamorphic and igneous rocks are intruded by Precambrian, Tertiary and Cretaceous age stocks and numerous porphyritic dikes. The regional rock type of most relevance to the bridge structures is hornblende gneiss identified in Figure 1. The most common porphyries range in composition from Precambrian pegmatite and lamprophyres to Cretaceous quartz monzonite and granodiorite.

Most of the present configuration of the area is characterized by moderately rugged topographic relief. The mountains to the south and north are deeply incised by Clear Creek Canyon and its tributaries. The maximum local relief is about 3,000 feet. The elevation in the project area ranges from slightly over 7,000 feet along Clear Creek to more than 10,000 feet at Santa Fe Mountain to the southwest. Natural slopes are typically steep, averaging approximately 35 degrees. Topographic forms are generally influenced by minor faulting, fractures, and zones of weakness in rock. In addition, rain, snowmelt, freeze-thaw, wind and Clear Creek have created deposits of alluvium (stream deposits), talus (rockfall deposits) and alluvial fans.

Bedrock in the project area is primarily Precambrian metamorphic gneiss and migmatite. Hornblende gneiss is the predominant mapped bedrock along the proposed Canyon Viaduct Alternative alignment, however the geologic map includes feldspar gneiss, biotite gneiss, calc-silicate gneiss, and amphibolite. Locally the bedrock is also known as migmatite, a composite rock consisting of igneous and metamorphic portions. In Colorado, migmatite is generally a blend of quartz pegmatite or granite intruded into a metamorphic host rock and intensely deformed. Hornblende gneiss in the immediate area of the proposed Canyon Viaduct area varies from light gray to dark gray in color depending upon hornblende content and is fine to medium grained. Some layers predominantly consist of the amphibole mineral hornblende—therefore called amphibolite—and nearly black in color. Precambrian pegmatite dikes, lamprophyre dikes, and Cretaceous quartz monzonite porphyry or granodiorite porphyry dikes are also mapped in the project area.

Some of the rock exposed in the existing cuts along I-70 shows up to 1-ft thick parallel layering while some of the rock is more schistose with foliated biotite seams or banded bedrock. In many places, the rock is highly deformed by folding and faulting. Intense localized folding is commonly found in the project area and is typical of migmatite formations. Measurements of these folds is not feasible or indicative of regional tectonic movement. Pegmatite and folding is common in the migmatite/gneiss. The density and orientation of fracturing is highly variable in the studied area, however the general rock structure follows the same orientation as the gneissic banding or foliation. Banding generally trends northeast, and lineations generally plunge 45-55 degrees to the north-northwest based on the (Squaw Pass Quadrangle geologic map)

The west limb of the Floyd Hill Fault crosses the east portion of the study area, as shown on the Squaw Pass Quadrangle map. The Floyd Hill Fault generally trends north-northwest. Few fractures exposed along the existing rock cut of I-70 are continuous. Those that are continuous show alteration or oxidation staining. No apparent fractures associated with the Floyd Hill Fault were observed in the Canyon Viaduct study area.

The Canyon Viaduct area is located east of historical metal mining activity known as the Idaho Springs Mining District. Bedrock in the Floyd Hill Canyon Viaduct area contains few sulfide veins and displays only very weak sulfide mineralization or alteration along select fractures. At least three prospect pits have been dug in the vicinity of the Canyon Viaduct as shown on the Squaw Pass Quadrangle map, though all are on the north side of Clear Creek. No possible sites of exploration activity or borrow pits were observed in the current area of investigation.

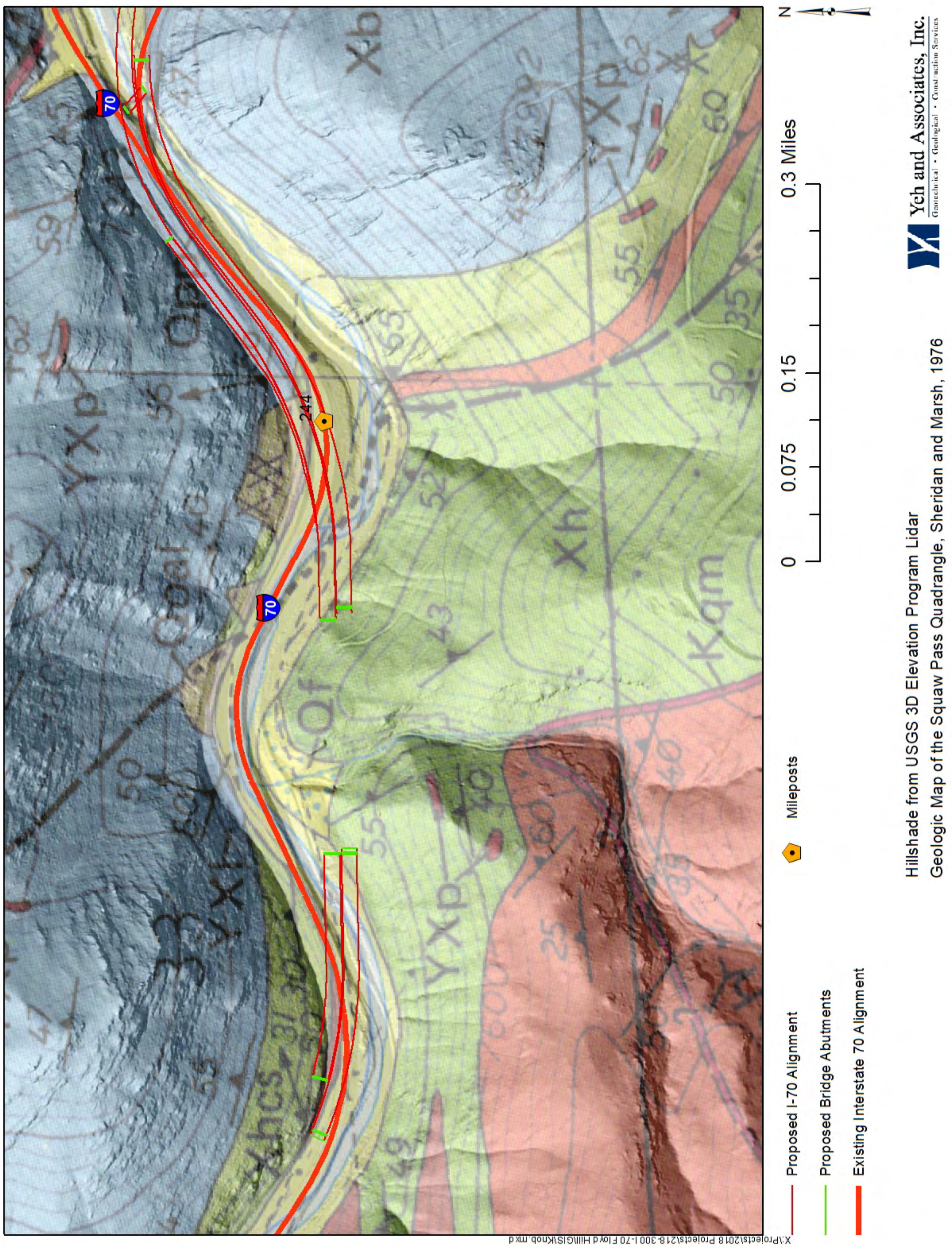

**Figure 1. Geologic Map of the Squaw Pass Quadrangle, Clear Creek, Jefferson and Gilpin Counties, Colorado by Sheridan and Marsh (1976). Published by USGS.**

# 3. GEOLOGIC EXPLORATION

The investigation program included site reconnaissance, core drilling and logging, structural mapping and geologic data interpretation.

## 3.1 Subsurface exploration

Three borings were drilled for the proposed Canyon Viaduct Alternative alignment. Borings YA-FHK-B1 and YA-FHK-B3 were drilled at or as close as feasible to proposed bridge abutments. Boring YA-FHK-B2 was drilled near the highest elevation (the proposed "saddle" area where the viaducts on the east and west sides would land on rock for a short distance) along the alignment.

The boring locations were located in the field using a handheld Garmin GPS unit. Final boring locations were dependent upon the space requirements and access for the drill rig and support equipment. The boring locations are shown in Figure 2.

YA-FHK-B1 and YA-FHK-B3 were advanced by Authentic Drilling using an Acker Renegade and CME 550X, respectively. YA-FHK-B2 on the "saddle" was advanced by Salisbury and Associates using a relatively lightweight, difficult-access drilling rig due to the soft and unstable mica-rich soils encountered along the access trail above YA-FHK-B3. HQ3 and NQ3 diamond rock core was obtained from the borings. The subsurface conditions encountered in the borings were logged by a Yeh and Associates field geologist. Driven samples of the overburden alluvium were obtained in YA-FHK-B1 and YA-FHK-B3. The recorded penetration resistance measurements were obtained by driving a split spoon sampler into the subsurface materials with an automatic hammer producing the equivalent energy of a 140-pound hammer falling 30 inches, generally according to ASTM D1586, "Standard Test Method for Standard Penetration Test SPT) and Split Barrel Sampling of Soils." The penetration resistance value (N-value) is a useful index to describe the consistency and relative density or hardness of the materials encountered.

Native alluvial soils encountered on site generally consisted of mica-rich silty sand, with gravel and occasional cobbles. Subsurface conditions varied due to elevation, weathering horizon, and extent of fracturing.

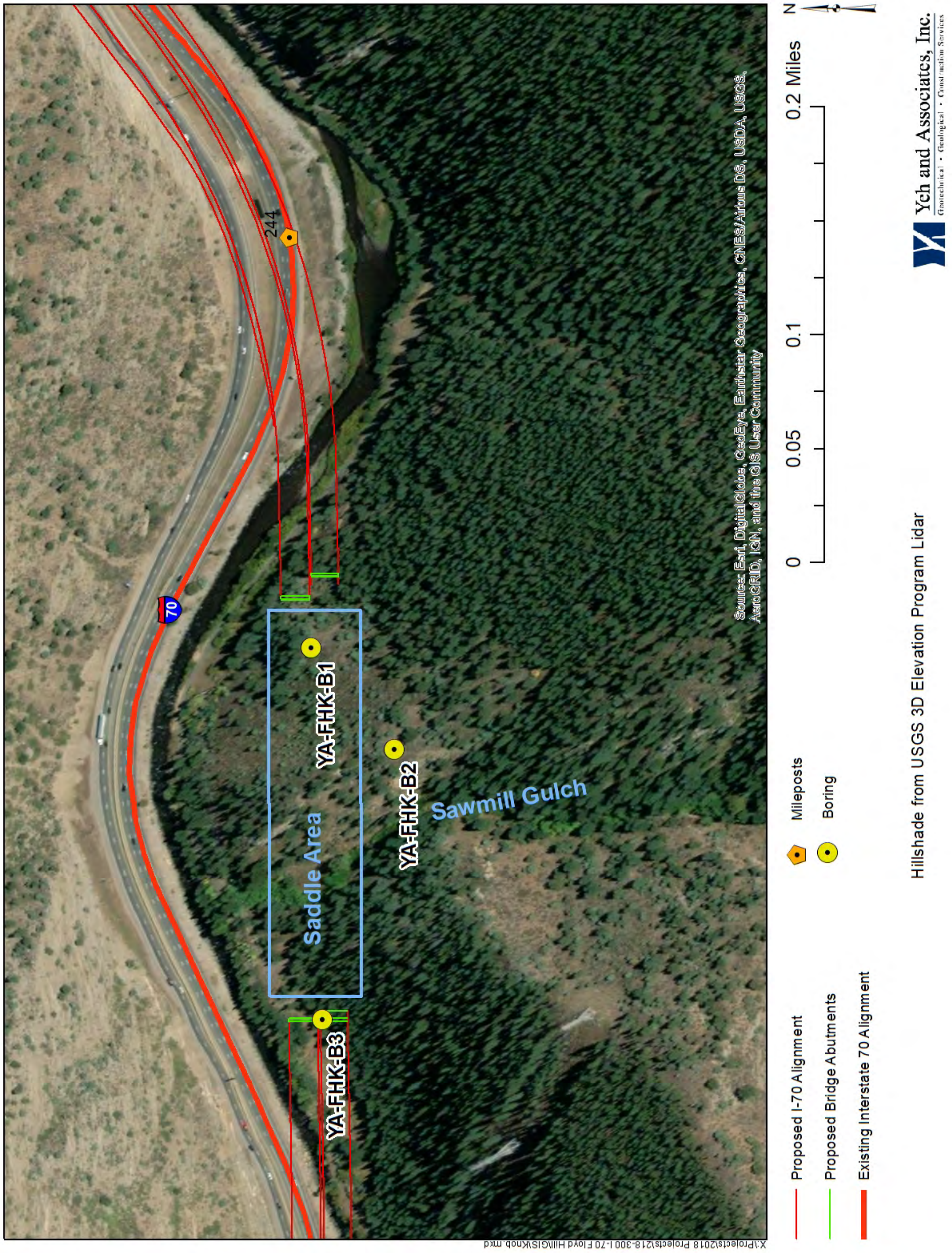

**Figure 2. Boring location map.**

A total of three borings were drilled for the Canyon Viaduct alignment. Total boring depths as well as overburden depths are shown below in Table 1.

**Table 1. Summary of boreholes.**

| Borehole | Total Depth (ft) | Depth of soil overburden (ft) |
|---|---|---|
| YA-FHK-B1 | 80.0 | 15 |
| YA-FHK-B2 | 50.3 | 4 |
| YA-FHK-B3 | 70.0 | 22 |

Rock Quality Designation (RQD) and core recovery information obtained from the core drilling program is presented in the boring logs. Prominent discontinuities are noted in the boring logs. The boring logs are found in Appendix A and core photos are found in Appendix B.

## 3.2 Geologic Mapping

Field geologic mapping was performed along the outcrops on and surrounding the area of investigation to supplement the analysis of joints in drilled core. Field geologic mapping of project site occurred over a period of one day. Measurements of rock structures were completed manually using the FieldMove Clino application on mobile devices with enabled GPS. Photos and data were downloaded to a computer and visualized using Google Earth Pro and ArcMap 10.5.

Based on our mapping and measurements of dip/dip direction, the foliation/banding planes and discontinuities (joints) would control the overall strength of the rock mass. Mappable outcrops are sparse, therefore. structure measurements were taken at the six areas of visible rock outcrop as shown in Figure 3. Most of the data was collected along the prominent ridge (saddle area) within the study area as shown in Figure 4. The structural mapping encountered a pervasive joint set dipping between 30 and 50 degrees towards the northeast, which is generally parallel to metamorphic foliation planes in this part of the Clear Creek valley. Stereonets and structure measurements are included in Appendix C.

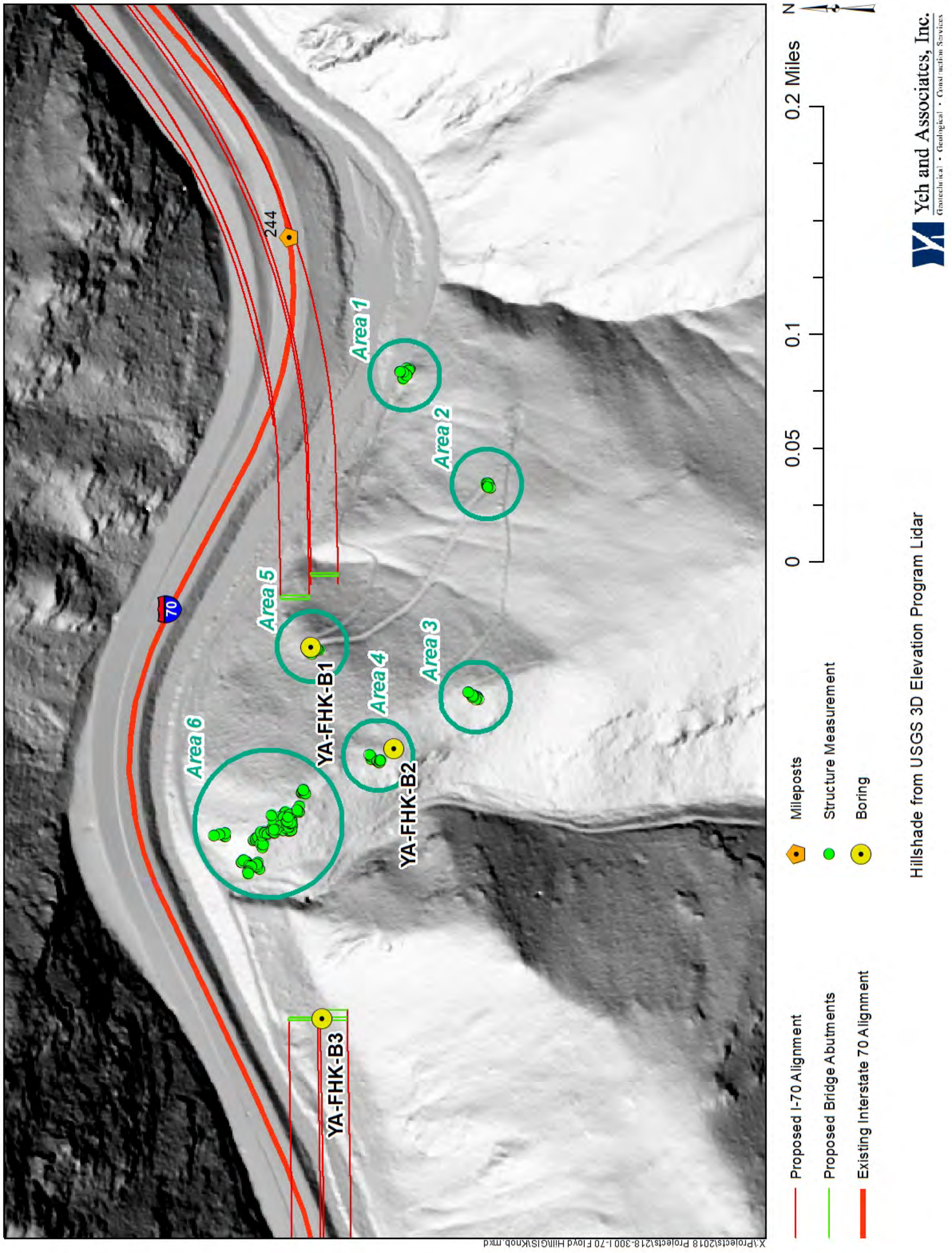

**Figure 3. Structure measurement locations.**

**Figure 4. Google Street View Image from April 2018 showing the prominent rock ridge (circled in red) in the project area. Orange arrow marks the structural trend of the area, with joints and metamorphic foliation dipping between 30 and 50 degrees to the northeast. Photo is looking east-southeast.**

## 3.3 Geologic Structure

Generally, the dip and dip direction of the rock structure and banded foliation is oriented approximately north-northeast to northeast. This orientation combines with the existing east-west canyon tend to create north facing slopes as seen in Figure 3. Regional dipping metamorphic foliation observed indicate an average apparent dip direction northeast, at a dip angle ranging between 36° to 39° below horizontal. This regional dip angle forms the northeast slope face of the saddle area as shown in Figure 5.

Fracturing and jointing is common throughout the borings. YA-FHK-B3 encountered the least-fractured rock, with relatively good recovery and RQD below about 45 feet depth. YA-FHK-B2 encountered strongly oxidized (discolored to orange) rock to a depth of approximately 40 feet. Rock beneath 40 feet was still fractured but much less oxidized as evidenced by fresh magnetite blebs observed in select layers. YA-FHK-B1 encountered minimal oxidation—with oxidation mostly confined to fracture surfaces—except for an oxidized zone from 75 to 78 feet.

Depending on bearing of the road excavation with respect to the pervasive discontinuity strike direction (i.e. perpendicular to the dip direction), this creates variable conditions for excavation in the rock mass, since the bedrock structure is dipping northeastward creating many of the natural northeast-facing slopes visible from the highway. Rock core samples indicate that the thickness of weathered bedrock varies; weathered bedrock may be minimal as evidenced by 0-2 feet at YA-FHK-B1 and YA-FHK-B3, or significant, with 40 feet of weathered bedrock encountered at YA-FHK-B2. Anticipated failure mechanisms that would influence the rock cuts are the structural rock fabric and the shear strength of weak foliations or banding. Other failure mechanisms such as cross-cutting joints or discontinuities are present, however those conditions vary considerably locally.

Preliminary rock mass discontinuity data measured manually along the exposed rock outcrops are summarized as stereonets on Figure 6. Structure measurements and kinematic analysis stereonets can be found in Appendix C.

## 4. ROCK CUTS

Based on our preliminary field investigation, and our previous experience in the corridor, the banded rock structure and bedrock composition appear to be similar. Migmatite bedrock, generally consisting of folded metamorphic gneiss and quartz pegmatite intrusions, is found along the existing I-70 road cuts. Regional dipping bedrock conditions observed in the banded structure indicate a northeasterly average dip direction, at a dip angle ranging between 36° to 39° below horizontal as previously discussed.

Intact rock strength of the rock is dependent on the degree of weathering, which ranged from intensely weathered to moderately weathered in the first 40 feet of our drilled borings, not including overlying soil or colluvium. Up to 2 repeating joint sets were visually identified in the recovered rock core. Other weak planes or discontinuities are found throughout the site. These joint sets are visually apparent but, based on analyzed data are, not as statistically persistent as the data points representing structural foliation or banding.

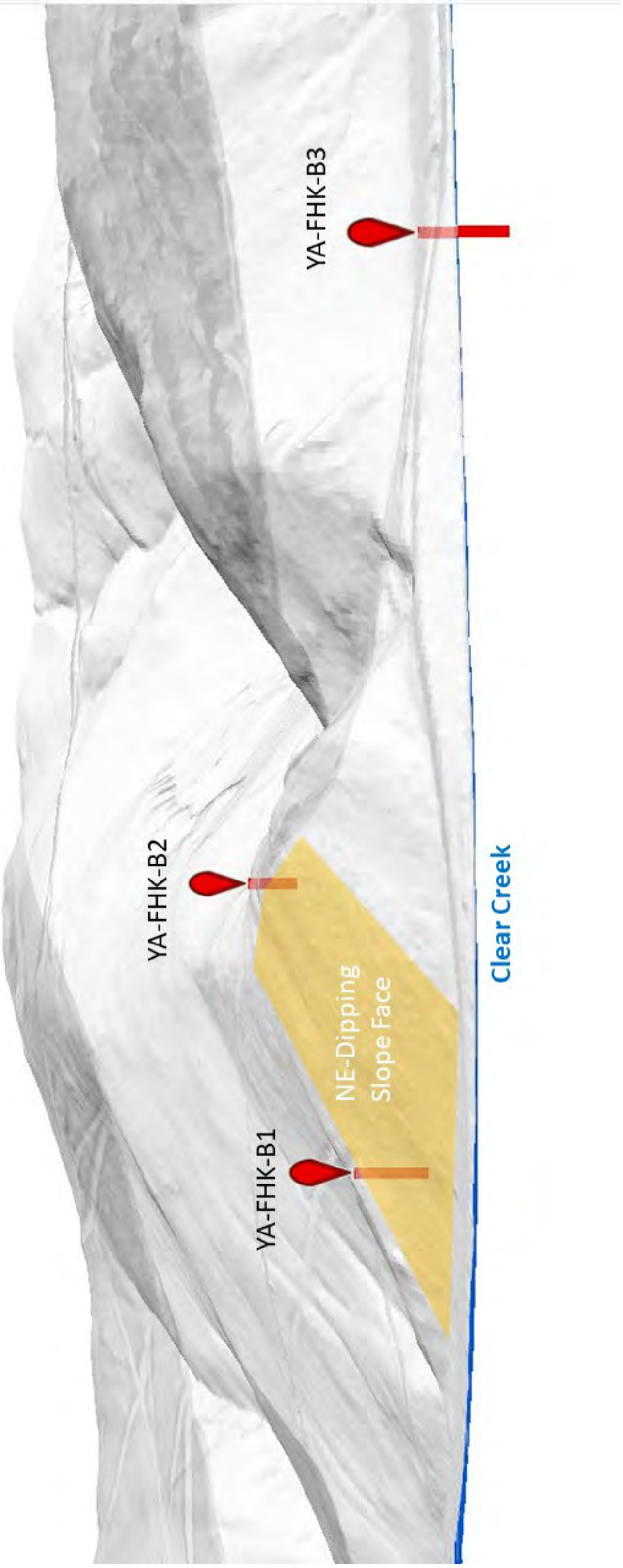

**Figure 5. Three-dimensional rendering of the Lidar hillshade showing subsurface extents of borings, Clear Creek, and the NE-dipping slope face that reflects the regional dip of the metamorphic structure. View is looking approximately southeast.**

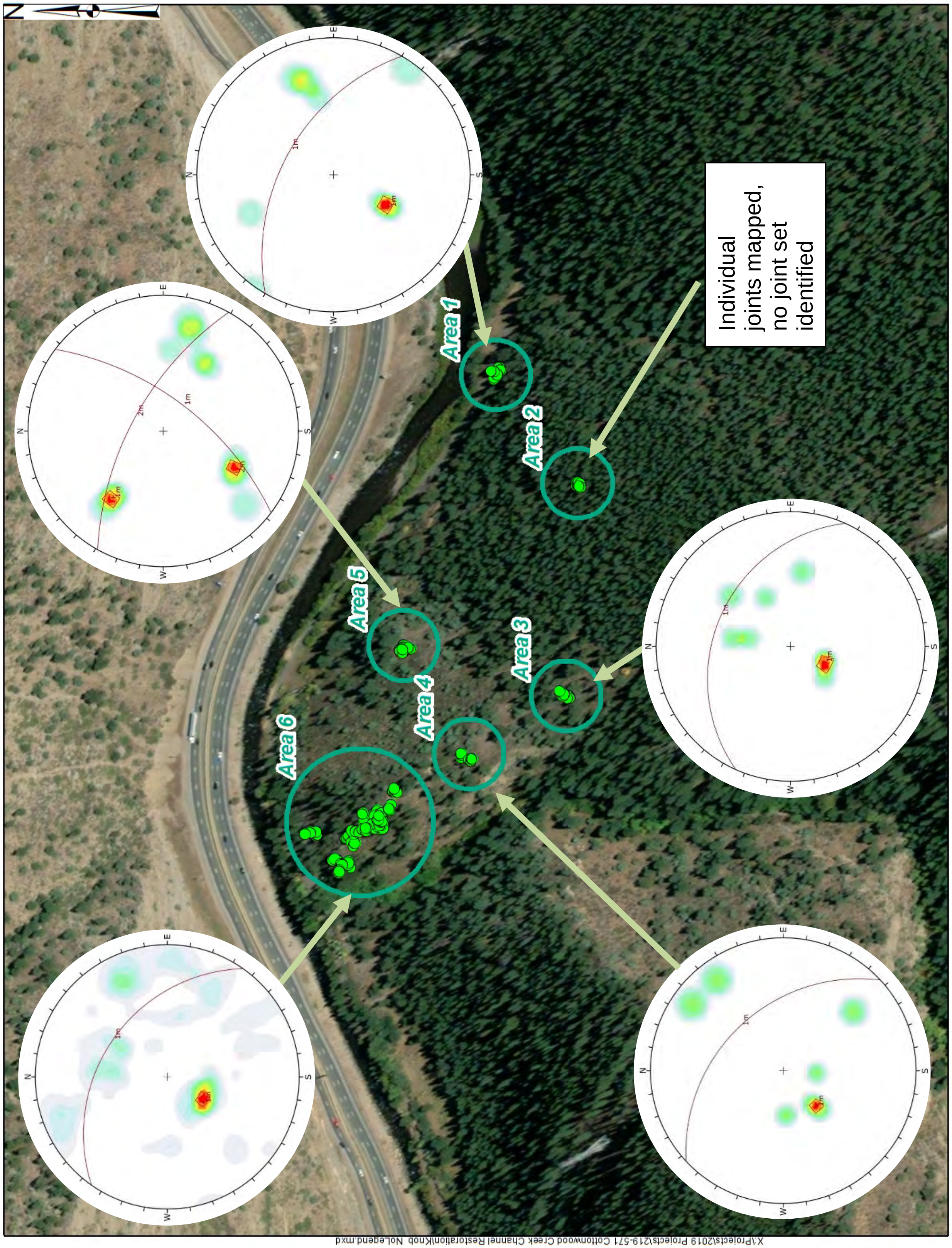

**Figure 6. Stereonets for all six mapped rock outcrop areas.**

## 4.1 Rock Cut Evaluation

Based on our current understanding the proposed rock cuts are 0.25H:1.0V (76° above horizontal) with an excavation bench roughly equal in size to the combined footprint of the eastbound and westbound roadways. Current proposed rock cut heights may exceed 140 feet on occasion.

### 4.1.1 Rock Cuts – I-70 MP 243.6 to MP 243.8 (Approximately)

There are two proposed rock cuts on the south side of Clear Creek that form a single saddle to straighten I-70 between the base of Floyd Hill and Hidden Valley (see Figure 2). The proposed roadway cuts intersect, and lie either side of, Sawmill Gulch giving rise to the question of how to maintain the Gulch without blocking its natural drainage and debris flow function. The rock cuts vary in height depending on location but at the highest points they are of the order of 140 feet high. Rock discontinuity dip and dip direction measurements where obtained at six surface outcrops locations identified in Figure 6.

## 4.2 Kinematic Analysis

The Rocscience program DIPS was used to perform a kinematic rockfall analysis of the rock cut locations. Using the dip and dip direction measurements wedge failure, planar failure and toppling failure were modeled. General interpretation and a summary are presented here. The stereonet analyses and data are presented in Appendix C.

### 4.2.1 Area 1

Results from the wedge, planar and toppling failure analyses indicate that only wedge failure has any real potential for occurring. Wedge failure analysis indicates that a few wedge failures may occur. Conventionally this usually consists of numerous small failures though isolated large wedge failures are possible.

### 4.2.2 Area 2

Results from the failure analyses indicate that wedge, planar, flexural and direct toppling failure could have the potential of occurring. Wedge failure analysis indicates such occurrences may be anticipated. Conventionally this usually consists of numerous small failures though isolated large wedge failures are possible. There is a minor possibility of planar failure. Flexural toppling is also possible, but toppling is an uncommon mode of failure in the metamorphic rocks in the corridor in this area and is not anticipated to be significant. Toppling in metamorphic rocks in this area of the I-70 corridor generally contributes to slope face raveling.

### 4.2.3 Area 3

Results for this location indicate that three modes, wedge, flexural and direct toppling failure could have the potential of occurring. As previously indicated significant toppling failure is uncommon in the metamorphic rocks in the corridor in this area. Wedge failure may occur and conventionally this usually consists of numerous small failures though a few isolated large wedge failures are possible.

### 4.2.4 Area 4

At this location only the possibility of wedge failures appears to be present and in a limited way.

### 4.2.5 Area 5

The potential for wedge failure appears to be high however conventionally this usually consists of numerous small failures though a few isolated large wedge failures are possible. The potential for direct toppling appears to be high but, as indicated before, toppling is an uncommon mode for failures of significant size in the area.

### 4.2.6 Area 6

As was the case for area 5, wedge failure potential appears to be high, however, conventionally this usually consists of numerous small failures though isolated large wedge failures are possible. The potential for direct toppling appears to be high but as indicated before toppling is an uncommon mode for failures of significant size failures in the area.

### 4.2.7 Summary of Kinematic Analysis

Based on dip of slopes, rock cuts oriented due east-west (the proposed bearing of the roadway) should not experience planar failure because of the general northeasterly dip of the rock structure. Planar failures are the most prominent mechanism of major rock slope failures in this part of the I-70 corridor.

Numerous small-scale wedge failures are possible and are common features of rock cuts in the area but are generally not major slope stability issues though some isolated large-scale wedge failures may occur.

No recent slope failures or slope distress such as visible slope creep, tension cracks or failure scarps were observed during field work in rock slopes immediately adjacent to the investigation site.

The fractured nature of the rock mass may influence the prevalence of rockfall from cut faces due to the potential availability of source rock from the faces.

## 4.3 Rock Cut Excavation

Due to the fractured nature of the rock in this area, care must be taken during the design and excavation of these cuts so as not to introduce long term maintenance issues that could be avoided. Excavation methods already proven in the I-70 corridor are conventional blasting or mechanical excavation techniques. Recent rock excavations for Colorado DOT projects have been performed primarily with drill and blast methods using conventional explosives.

Controlled blasting for the proposed rock cut should be utilized. Presplitting or trim blast methods may be used to manage overbreak of the final rock cut. Blasting of the face should be controlled so as to prevent excessive damage to the rock cut, requiring maintenance rockfall mitigation or additional excavation to bring the cut face into a serviceable condition.

Other alternative methods may be possible, provided a contractor can present current successful construction examples.

## 4.4 Rock Cut Slope Mitigation

Final desired rock cut stability may be achieved by a combination of rock cut slope mitigation treatments concurrent with a suitable excavation method. Mitigation treatments for final cut slopes includes draped double-twist mesh, spiral rope net, cable-net mesh panels, rock bolts, rock dowels and rock face scaling (manual or mechanical). One or a combination of these could be applied to the final cut slope, if required, to stabilize the face for long-term service life and to reduce potential future hazards. Rock slope treatment types, effectiveness and applicable locations are presented below in Table 2

**Table 2. Rock Slope Mitigation Treatment Options.**

| Rock Slope Treatment Option | Suitability – Effectiveness | Applicable Locations | Maintenance |
|---|---|---|---|
| Double Twist Mesh Drape | Contains small rocks and debris in shoulder | Slopes ≤200 ft<br>Stable cuts producing small rocks, debris | Regular inspection, repair wire mesh when necessary |

| | | | |
|---|---|---|---|
| Anchored Cable Net Panel | Contains moderate-size rocks in place, prevents bouncing | Slopes ≥200 ft<br>Cuts producing rocks, debris regularly | Regular inspection<br>Replacement of rockfall impacted panels |
| Anchored High Strength Spiral Rope Net | Contains moderate-size rocks in place, prevents bouncing | Slopes ≥200 ft<br>Cuts producing rocks, debris regularly | Regular Inspection<br>This does not use panels. Installation is easier. Continuous diamond mesh is less obtrusive |
| Rock Bolt/Dowel | Passive or active anchoring of rocks in place | Large rock masses requiring stabilization | Limited with proper installation and corrosion protection |
| Scaling | Removes unstable rocks | Rock cut excavations<br>Incipient rockfall hazards outside right-of-way | Perform every 3 to 5 years not including emergency rockfall response |

#### 4.4.1 Rock Cut Catchment Ditches

A properly designed rockfall ditch creates a buffer between traffic and rockfall sources and acts as a catchment for rockfall. Appropriately sized ditches should be considered not only for debris catchment but for highway clear zones and sight lines around curves. Ditch effectiveness is a function debris source height, width, storage capacity for collected debris and surface hardness as it relates bouncing rockfall. Many factors need to be considered when evaluating ditch effectiveness which can be modeled for further evaluation. Existing ditch widths along the current I-70 highway alignment are generally too small to effectively mitigate rockfall or serve as debris storage and have had to be supplemented by mesh solutions, impact berms or barriers. Any new highway alignment should incorporate sufficient ditch space/rockfall mitigation treatment to contain debris from new rock cuts, before resorting to supplementary measures like meshes or barriers.

## 5. GROUNDWATER CONDITIONS

The drilling program for the Floyd Hill Canyon Viaduct Alternative investigation used water, therefore the water observed during and after hole completion is attributed to drilling. No long-term groundwater monitoring was planned or has been performed since completion of the drilling program. Drill water circulation was lost in YA-FHK-B1 and YA-FHK-B2 which is not unusual in fractured rock. Groundwater was apparently encountered at 20 feet in YA-FHK-B3 as the drive sample was wet and drill water circulation was successful during coring. Groundwater encountered in YA-FHK-B3 is approximately at the level of Clear Creek.

The loss of drill water circulation indicates sufficient open fractures or discontinuities that can transmit water. This water is likely using fractures in the rock as a drainage pathway and may not indicate a water bearing formation.

Variations in groundwater conditions may occur seasonally. The magnitude of the variation will be largely dependent upon fluctuations in the amount of spring snowmelt, and the duration and intensity of precipitation, site grading changes, and the surface and subsurface drainage characteristics of the surrounding area. Seasonal perched areas of groundwater or fracture flow may also exist but were not confirmed in any of our test holes during the investigation. Generally, based on observation, the rock mass in the area above creek level appears relatively dry. This should be confirmed.

## 6. SUBSURFACE CONDITIONS AND SEISMICITY

The site is classified as Site Class B in accordance with Table 3.10.3.1-1 of the AASHTO LRFD Bridge Design Specifications. The Peak Ground Acceleration (PGA), and the short- and long-period spectral acceleration coefficients ($S_S$ and $S_1$ respectively) for the site were obtained using the USGS/AASHTO 2007 Seismic Parameters for an event with a 7% Probability of Exceedance (PE) in 75 years and a Site Class B (reference site). An event with the above probability of exceedance has a return period of about 1,033 years. Since the site classification (B) is the same as the reference site (B), no value adjustments were necessary. The seismic parameters for this site are shown on 3 and 4.

**Table 3. Seismic Design Parameters**

| PGA (0.0 sec) | $S_S$ (0.2 sec) | $S_1$ (1.0 sec) |
|---|---|---|
| 0.067 | 0.141 | 0.036 |

**Table 4. Seismic Design Parameters for Site Class B**

| $A_S$ (0.0 sec) | $S_{DS}$ (0.2 sec) | $S_{D1}$ (1.0 sec) | Seismic Zone |
|---|---|---|---|
| 0.067g | 0.141 g | 0.036 g | 1 |

The nearby quarry can cause ground vibrations during blasting operations. This is a potential external source of vibration.

# 7. LABORATORY TESTING

In order to determine small-sample rock strength, samples of the core obtained from the rock mass were tested by Advanced Terra Testing of Lakewood, Colorado. Unconfined compressive strength testing (ASTM D7012; Method D) was performed on the rock core. Generally, the unconfined compressive strength of intact rock specimens is high, ranging between 10,000 psi and 18,000 psi with results clustering towards the higher end of this range. Intact rock compressive strengths are not an accurate indicator of rockmass strength because rock mass strength is structurally controlled by jointing and fractures. They do, however, provide an index which, in conjunction with other rock parameters, may be used to determine an estimate of rockmass strength. Complete testing results can be found in Appendix D.

# 8. LIMITATIONS

This report transmits geotechnical data only, for use by Atkins Global Corporation and CDOT, for the proposed Canyon Viaduct Alternative approximately between Floyd Hill and Hidden Valley, Colorado. The data submitted are based on the exploratory borings, laboratory testing, field mapping and reconnaissance included in our investigation. As previously identified, the rock and its properties in the Canyon Viaduct area can vary considerably over short distances.

This Investigation has been conducted in accordance with generally accepted geotechnical engineering practices in this area. The nature and extent of subsurface variations across the site may not become evident until excavation is performed. During construction conditions may be different from those described herein. No warranty, expressed or implied, is made.

Yeh and Associates, Inc.

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Senior Project Manager

# 9. REFERENCES

Sheridan, D.M. and Marsh, S.P., 1976, Geologic Map of the Squaw Pass Quadrangle, Clear Creek, Jefferson, and Gilpin Counties, Colorado: U.S. Geological Survey Map GQ-1337.

Technical Memorandum. Draft Floyd Hill Tunnel Portal Investigation; I-70 Floyd Hill, Clear Creek County, Colorado. December 21, 2018. Yeh and Associates Project 218-300.

Widmann, B.L. and Rogers, W.P., 2002, Geologic Hazards of the Georgetown, Idaho Springs, and Squaw Pass Quadrangles, Colorado Geological Survey Open File Report 03-02.

Widmann, B.L., Kirkham, R.M., Morgan, M.L., and Rogers, W.P., with contributions by Crone, A.J., Personius, S.F., and Kelson, K.I., and GIS/Web design by Morgan, K.S., Pattyn, G.R., and Phillips, R.C., 2002, Colorado Late Cenozoic fault and fold database and internet map server: Colorado Geological Survey Bulletin 64a, https://pubs.er.usgs.gov/publication/70031679

The National Map, United States Geological Survey: https://viewer.nationalmap.gov/basic/

# Appendix A

## BORING LOGS AND LEGEND

**Yeh and Associates, Inc.**
Geotechnical • Geological • Construction Services

*Project Name:* Floyd Hill Canyon Viaduct Alternative

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*Project Number:* 218-300 *Boring No.:* **YA-FHK-B1**

**Boring Began: 5/6/2020**
**Boring Completed: 5/7/2020**
Drilling Method(s): ODEX / Coring
Driller: Authentic Drilling
Drill Rig: Acker Renegade
Hammer: Automatic (hydraulic), ER: 96%

**Total Depth: 80.0 ft**
Ground Elevation: 7338
Coordinates: Lat: 39.742212 Long: -105.446846
Location: East abutment
Logged By: M. Boyd
Final By: M. Boyd

Weather Notes: Sunny, cool
Inclination from Horiz.: Vertical
Night Work: ☐

| Groundwater Levels: Not Observed | | | |
|---|---|---|---|
| Symbol | | | |
| Depth | - | - | - |
| Date | - | - | - |

| Elevation (feet) | Depth (feet) | Sample Type/Depth | Drilling Method | Rock Recovery (%) | Rock RQD (%) | Soil Samples Blows per 6 in | Soil Samples Penetration Resistance | Lithology | Material Description | Moisture Content (%) | Dry Density (pcf) | Fines Content (%) | Atterberg Limits Liquid Limit | Atterberg Limits Plasticity Index | AASHTO & USCS Classifi-cations | Field Notes and Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7335 | 0 - 5 | | ODEX | | | | | | **0.0 - 15.0 ft. Silty SAND with gravel, cobbles, and boulders**, brown to gray - brown, dry, mica-rich sand. | | | | | | | |
| 7330 | 5 - 10 | | ODEX | | | | | | | | | | | | | |
| 7325 | 10 | ☒ | ODEX | | | 16-16-19 | 35 | | | | | | | | | |
| | 15 | ☒ | | | | 50:2" | 50:2" | | | | | | | | | |
| 7320 | 15 - 20 | | Coring | 50 | 8 | | | | **15.0 - 29.0 ft. GNEISS**, dark gray, fresh, dark gray hornblende gneiss with white and pink quartz-rich +/- feldspar layers and migmatite; high angle joints with orange oxidation infilling; some moderate to low angle joints; undulating and smooth to rough surfaces; stockwork green clay veins, moderate to high angle, 15 to 19 ft.. | | | | | | | |
| 7315 | 20 - 25 | | Coring | 56 | 8 | | | | | | | | | | | |
| 7310 | 25 - 30 | | Coring | 66 | 8 | | | | **29.0 - 33.0 ft. MIGMATITE**, white to pink, appears fresh but is highly broken, crumbly. Mixed with gneiss. Notable undulating high angle fracture at 32-33 ft.. | | | | | | | |
| 7305 | 30 - 35 | | Coring | 78 | 16 | | | | **33.0 - 46.0 ft. GNEISS**, dark gray, fresh, dark gray hornblende gneiss with minimal white and pink quartz-rich +/- feldspar | | | | | | | |

| Elevation (feet) | Depth (feet) | Sample Type/Depth | Drilling Method | Rock: Recovery (%) | Rock: RQD (%) | Soil Samples: Blows per 6 in | Soil Samples: Penetration Resistance | Lithology | Material Description | Moisture Content (%) | Dry Density (pcf) | Fines Content (%) | Atterberg Limits: Liquid Limit | Atterberg Limits: Plasticity Index | AASHTO & USCS Classifi-cations | Field Notes and Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | migmatite; moderate ot high angle foliation and banding; highly fractured, some orange oxidation infilling on surfaces. | | | | | | | |
| 7300 | | | | 46 | 0 | | | | | | | | | | | |
| | 40 | | | | | | | | | | | | | | | |
| 7295 | | | | 38 | 0 | | | | | | | | | | | |
| | 45 | | | | | | | | | | | | | | | |
| | | | | | | | | | **46.0 - 57.0 ft. MIGMATITE**, dark gray and white, fresh to slightly weathered, white quartz +/- felsdpar mixed with dark gray hornblende gneiss, highly fractured, moderate angle fractures with some broken zones, orange oxidiation infilling and coating of surfaces. | | | | | | | |
| 7290 | | | | 52 | 0 | | | | | | | | | | | |
| | 50 | | | | | | | | | | | | | | | |
| 7285 | | | | 48 | 16 | | | | | | | | | | | |
| | 55 | | | | | | | | | | | | | | | |
| | | | | | | | | | **57.0 - 74.8 ft. GNEISS**, dark gray, hornblende gneiss with few layers of quartz +/- feldspar; highly fractured except 65.5 to 69 ft; moderate angle to high angle fractures common, orange oxidation infilling; 64-65.5 is quartz-rich, likley due to compositional layering of original sediments. | | | | | | | |
| 7280 | | | | 86 | 16 | | | | | | | | | | | |
| | 60 | | | | | | | | | | | | | | | |
| 7275 | | | | 94 | 23 | | | | | | | | | | | |
| | | | | 113 | 33 | | | | | | | | | | | |
| | 65 | | | | | | | | | | | | | | | |
| 7270 | | | | 100 | 70 | | | | | | | | | | | |
| | | | | 110 | 60 | | | | | | | | | | | |
| | 70 | | | | | | | | | | | | | | | |
| 7265 | | | | 78 | 36 | | | | | | | | | | | |
| | 75 | | | | | | | | **74.8 - 78.0 ft. SHEAR ZONE**, orange-brown, decomposed, strongly oxidized, broken zone; three inch clay zone at 77.5. | | | | | | | |
| | | | | 63 | 10 | | | | | | | | | | | |

| Elevation (feet) | Depth (feet) | Sample Type/Depth | Drilling Method | Rock |  | Soil Samples |  | Lithology | Material Description | Moisture Content (%) | Dry Density (pcf) | Fines Content (%) | Atterberg Limits |  | AASHTO & USCS Classifi-cations | Field Notes and Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  |  | Recovery (%) | RQD (%) | Blows per 6 in | Penetration Resistance |  |  |  |  |  | Liquid Limit | Plasticity Index |  |  |
| 7260 |  |  |  | 65 | 0 |  |  |  | **78.0 - 80.0 ft. GNEISS**, dark gray, slightly weathered, broken zone with some orange oxidation on fracture surfaces. |  |  |  |  |  |  |  |
|  | 80 |  |  |  |  |  |  |  | Bottom of Hole at 80.0 ft. |  |  |  |  |  |  |  |

7255

7250

7245

7240

7235

7230

7225

7220

**Yeh and Associates, Inc.**
Geotechnical • Geological • Construction Services

*Project Name:* Floyd Hill Canyon Viaduct Alternative

*PAGE 1 of 2*

*Project Number:* 218-300

*Boring No.:* **YA-FHK-B2**

**Boring Began: 5/19/2020**

**Boring Completed: 5/19/2020**

Drilling Method(s): Coring

Driller: Salisbury & Associates

Drill Rig: Saitech Viper

Hammer: , ER: %

**Total Depth: 50.3 ft**

Ground Elevation: 7448

Coordinates: Lat: 39.74278 Long: -105.44768

Location: Top of hill

Logged By: M. Boyd

Final By: M. Boyd

Weather Notes: Sunny, 75F

Inclination from Horiz.: Vertical

Night Work: ☐

| Groundwater Levels: Not Observed | | | |
|---|---|---|---|
| Symbol | | | |
| Depth | - | - | - |
| Date | - | - | - |

| Elevation (feet) | Depth (feet) | Sample Type/Depth | Drilling Method | Rock: Recovery (%) | Rock: RQD (%) | Soil Samples: Blows per 6 in | Soil Samples: Penetration Resistance | Lithology | Material Description | Moisture Content (%) | Dry Density (pcf) | Fines Content (%) | Atterberg Limits: Liquid Limit | Atterberg Limits: Plasticity Index | AASHTO & USCS Classifi-cations | Field Notes and Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | **0.0 - 4.0 ft. Silty SAND**, brown to orange-brown, dry. | | | | | | | |
| 7445 | | | | | | | | | | | | | | | | |
| | | | | | | | | | **4.0 - 41.0 ft. GNEISS**, orange-brown to dark gray, moderately weathered to predominantly decomposed, pervasive oxidation; moderate angle banding/foliation; moderate angle joints parallel to banding, planar; some broken zones. | | | | | | | |
| | 5 | | | | | | | | | | | | | | | |
| 7440 | | | | 22 | 0 | | | | | | | | | | | |
| | 10 | | | | | | | | | | | | | | | |
| 7435 | | | | 16 | 0 | | | | | | | | | | | |
| | 15 | | | | | | | | | | | | | | | |
| | | | | 44 | 0 | | | | | | | | | | | |
| | | | | 43 | 0 | | | | | | | | | | | |
| 7430 | | | | 96 | 44 | | | | | | | | | | | |
| | 20 | | | | | | | | | | | | | | | |
| | | | | 94 | 0 | | | | | | | | | | | |
| 7425 | | | | | | | | | | | | | | | | |
| | | | | 100 | 88 | | | | | | | | | | | |
| | 25 | | | | | | | | | | | | | | | |
| | | | | 58 | 29 | | | | | | | | | | | |
| 7420 | | | | | | | | | | | | | | | | |
| | 30 | | | 89 | 50 | | | | | | | | | | | |
| 7415 | | | | | | | | | | | | | | | | |
| | | | | 100 | 12 | | | | | | | | | | | |

| Elevation (feet) | Depth (feet) | Sample Type/Depth | Drilling Method | Rock | | Soil Samples | | Lithology | Material Description | Moisture Content (%) | Dry Density (pcf) | Fines Content (%) | Atterberg Limits | | AASHTO & USCS Classifi-cations | Field Notes and Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Recovery (%) | RQD (%) | Blows per 6 in | Penetration Resistance | | | | | | Liquid Limit | Plasticity Index | | |
| | | | | 42 | 0 | | | | | | | | | | | |
| 7410 | | | | 50 | 0 | | | | | | | | | | | **38.6 ft** - mislatch 38.6-40 ft |
| | 40 | | | 40 | 0 | | | | | | | | | | | |
| | | | | 110 | 100 | | | | Migmatite 40-41 ft and 42-42.5 ft. | | | | | | | UCCS=18340 psi<br>UCCS=17089 psi |
| 7405 | | | | 100 | 50 | | | | **41.0 - 50.3 ft. GNEISS**, dark gray, fresh, dark gray hornblende gneiss with white and pink quartz-rich +/- feldspar layers; moderate angle foliation and banding; miderate angle planar joints parallel to banding; magnetite blebs in select layers, some larger than 1/4 inch.<br>Recovery of mica-rich sand from 45.7 to 49.5; driller says it drilled like hard rock. Due to mislatch at 38.6 ft, difficulty retrieving core barrel below 50 ft, and fresh nature of the rock on either side of this zone, it is assumed that this is the result of mechanical issues during drilling. | | | | | | | |
| | 45 | | | 100 | 100 | | | | | | | | | | | |
| | | | | 75 | 0 | | | | | | | | | | | |
| | | | | 100 | 100 | | | | | | | | | | | |
| 7400 | 50 | | | 48 | 13 | | | | | | | | | | | |

Bottom of Hole at 50.3 ft.

7395

7390

7385

7380

7375

**Boring Began: 5/4/2020**

**Boring Completed: 5/5/2020**

Drilling Method(s): ODEX /
Coring

Driller: Authentic Drilling

Drill Rig: CME 550 Rubber Tire

Hammer: Automatic (hydraulic), ER: 96%

**Total Depth: 70.0 ft**

Ground Elevation: 7282

Coordinates: Lat: 39.74327 Long: -105.45005

Location: West abutment

Logged By: M. Boyd

Final By: M. Boyd

Weather Notes: Windy, cool

Inclination from Horiz.: Vertical

Night Work: ☐

| Groundwater Levels: | | | |
|---|---|---|---|
| Symbol | ▽ | | |
| Depth | 20.0 ft | - | - |
| Date | 5/4/20 | - | - |

| Elevation (feet) | Depth (feet) | Sample Type/Depth | Drilling Method | Rock: Recovery (%) | Rock: RQD (%) | Soil Samples: Blows per 6 in | Soil Samples: Penetration Resistance | Lithology | Material Description | Moisture Content (%) | Dry Density (pcf) | Fines Content (%) | Atterberg Limits: Liquid Limit | Atterberg Limits: Plasticity Index | AASHTO & USCS Classifi-cations | Field Notes and Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | **0.0 - 0.2 ft. TOPSOIL**, grass and roots. | | | | | | | |
| | | | | | | | | | **0.2 - 2.0 ft. GRAVEL and SAND**. | | | | | | | |
| 7280 | | | | | | | | | **2.0 - 10.0 ft. Silty SAND**, brown to reddish brown, dry, mica-rich. | | | | | | | |
| | 5 | | | | | 3-3-4 | 7 | | | | | | | | | |
| 7275 | | | | | | | | | | | | | | | | |
| | 10 | | | | | 50:3" | 50:3" | | **10.0 - 12.0 ft. BOULDER**, white to gray. | | | | | | | |
| 7270 | | | | | | | | | **12.0 - 14.0 ft. Silty SAND**, brown, dry. | | | | | | | |
| | | | | | | | | | **14.0 - 22.0 ft. BOULDERS with COBBLES, GRAVEL, and SAND**, minor clay and silt, brown to gray, dry to wet, possibly very weathered bedrock. | | | | | | | |
| | 15 | | | | | 50:0" | 50:0" | | | | | | | | | |
| 7265 | | | | | | | | | | | | | | | | |
| | ▽20 | | | | | 19-22-50:2" | 72:8" | | | | | | | | | |
| 7260 | | | | | | | | | **22.0 - 24.0 ft. GNEISS**, gray to brown. | | | | | | | |
| | | | | 100 | 0 | | | | **24.0 - 42.0 ft. GNEISS**, dark gray to pink, slightly weathered, dark gray hornblende gneiss with white and pink quartz-rich +/- feldspar layers and possible migmatite; moderate angle foliation and banding; high angle joints common with red-orange oxidation staining, stepped to undulating adn rough surfaces; some moderate to low angle joints. | | | | | | | |
| | 25 | | | 100 | 40 | | | | | | | | | | | |
| 7255 | | | | 80 | 30 | | | | | | | | | | | |
| | | | | 100 | 52 | | | | | | | | | | | |
| | 30 | | | | | | | | | | | | | | | |
| 7250 | | | | 68 | 0 | | | | | | | | | | | |
| | | | | 92 | 48 | | | | Fractures healed with clay and orange oxidation infilling. | | | | | | | |

| Elevation (feet) | Depth (feet) | Sample Type/Depth | Drilling Method | Rock | | Soil Samples | | Lithology | Material Description | Moisture Content (%) | Dry Density (pcf) | Fines Content (%) | Atterberg Limits | | AASHTO & USCS Classifi-cations | Field Notes and Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Recovery (%) | RQD (%) | Blows per 6 in | Penetration Resistance | | | | | | Liquid Limit | Plasticity Index | | |
| | | | | | | | | | | | | | | | | |
| | | | | 80 | 40 | | | | | | | | | | | |
| 7245 | | | | 110 | 80 | | | | | | | | | | | |
| | 40 | | | 77 | 23 | | | | Vertical fractures making drilling difficult. | | | | | | | |
| 7240 | | | | 110 | 50 | | | | **42.0 - 58.0 ft. GNEISS**, dark gray, fresh, dark gray hornblende gneiss with white and pink quartz-rich +/- feldspar layers; moderate angle foliation and banding; moderate angle joints generally parallel to banding, surfaces generally planar, orange oxidation on some fracture surfaces; high angle to vertical fractures healed with white quartz veins, stepped surfaces..<br>Green clay alteration along some banding/foliation. | | | | | | | |
| | | | | 120 | 0 | | | | | | | | | | | |
| | 45 | | | 93 | 60 | | | | | | | | | | | |
| 7235 | | | | 140 | 80 | | | | | | | | | | | |
| | | | | 107 | 20 | | | | | | | | | | | |
| 7230 | 50 | | | 100 | 77 | | | | | | | | | | | |
| 7225 | 55 | | | 100 | 82 | | | | | | | | | | | UCCS=14292 psi |
| 7220 | 60 | | | 102 | 71 | | | | **58.0 - 58.5 ft. SHEAR ZONE**, light green, broken zone, clay alteration.<br>**58.5 - 70.0 ft. GNEISS**, dark gray, fresh, dark gray hornblende gneiss with white and pink quartz-rich +/- feldspar layers; moderate angle foliation and banding; moderate angle joints parallel to and perpendicular to banding, minor orange oxidation on some fracture surfaces; high angle to vertical fractures healed with white quartz veins; planar to undulating fracture surfaces; abundant garnets. | | | | | | | |
| 7215 | 65 | | | 100 | 96 | | | | | | | | | | | UCCS=10827 psi |
| | | | | 96 | 84 | | | | | | | | | | | |
| | 70 | | | | | | | | Bottom of Hole at 70.0 ft. | | | | | | | |
| 7210 | | | | | | | | | | | | | | | | |
| 7205 | | | | | | | | | | | | | | | | |

**Yeh and Associates, Inc.**
Geotechnical • Geological • Construction Services

*Project:* Floyd Hill Canyon Viaduct Alternative

*Project Number:* 218-300

# Legend for Symbols Used on Borehole Logs

## Sample Types

| | |
|---|---|
| ▮ | Rock core |
| ☒ | Standard Penetration Test (ASTM D1586) |

## Drilling Methods

| | |
|---|---|
| ▮ | CORING |
| ▮ | CORING |
| ▮ | CORING |

## Lithology Symbols

(see Boring Logs for complete descriptions)

| | |
|---|---|
| ▨ | Boulders |
| ▨ | Cobbles and gravel |
| ▨ | USCS Silty Sand |
| ▨ | Topsoil |
| ▨ | Gneiss |
| ▨ | Weathered Bedrock |

## Lab Test Standards

| | |
|---|---|
| Moisture Content | ASTM D2216 |
| Dry Density | ASTM D7263 |
| Sand/Fines Content | ASTM D421, ASTM C136, ASTM D1140 |
| Atterberg Limits | ASTM D4318 |
| AASHTO Class. | AASHTO M145, ASTM D3282 |
| USCS Class. | ASTM D2487 |

(Fines = % Passing #200 Sieve
Sand = % Passing #4 Sieve, but not passing #200 Sieve)

## Other Lab Test Abbreviations

| | |
|---|---|
| pH | Soil pH (AASHTO T289-91) |
| S | Water-Soluble Sulfate Content (AASHTO T290-91, ASTM D4327) |
| Chl | Water-Soluble Chloride Content (AASHTO T291-91, ASTM D4327) |
| S/C | Swell/Collapse (ASTM D4546) |
| UCCS | Unconfined Compressive Strength (Soil - ASTM D2166, Rock - ASTM D7012) |
| R-Value | Resistance R-Value (ASTM D2844) |
| DS (C) | Direct Shear cohesion (ASTM D3080) |
| DS (phi) | Direct Shear friction angle (ASTM D3080) |
| Re | Electrical Resistivity (AASHTO T288-91) |
| PtL | Point Load Strength Index (ASTM D5731) |

## Notes

1. Visual classifications are in general accordance with ASTM D2488, "Standard Practice for Description and Identification of Soils (Visual-Manual Procedures)".
2. "Penetration Resistance" on the Boring Logs refers to the uncorrected N value for SPT samples only, as per ASTM D1586. For samples obtained with a Modified California (MC) sampler, drive depth is 12 inches, and "Penetration Resistance" refers to the sum of all blows. Where blow counts were > 50 for the 3rd increment (SPT) or 2nd increment (MC), "Penetration Resistance" combines the last and 2nd-to-last blows and lengths; for other increments with > 50 blows, the blows for the last increment are reported.
3. The Modified California sampler used to obtain samples is a 2.5-inch OD, 2.0-inch ID (1.95-inch ID with liners), split-barrel sampler with internal liners, as per ASTM D3550. Sampler is driven with a 140-pound hammer, dropped 30 inches per blow.
4. "ER" for the hammer is the Reported Calibrated Energy Transfer Ratio for that specific hammer, as provided by the drilling company.

# Appendix B

## CORE PHOTOS

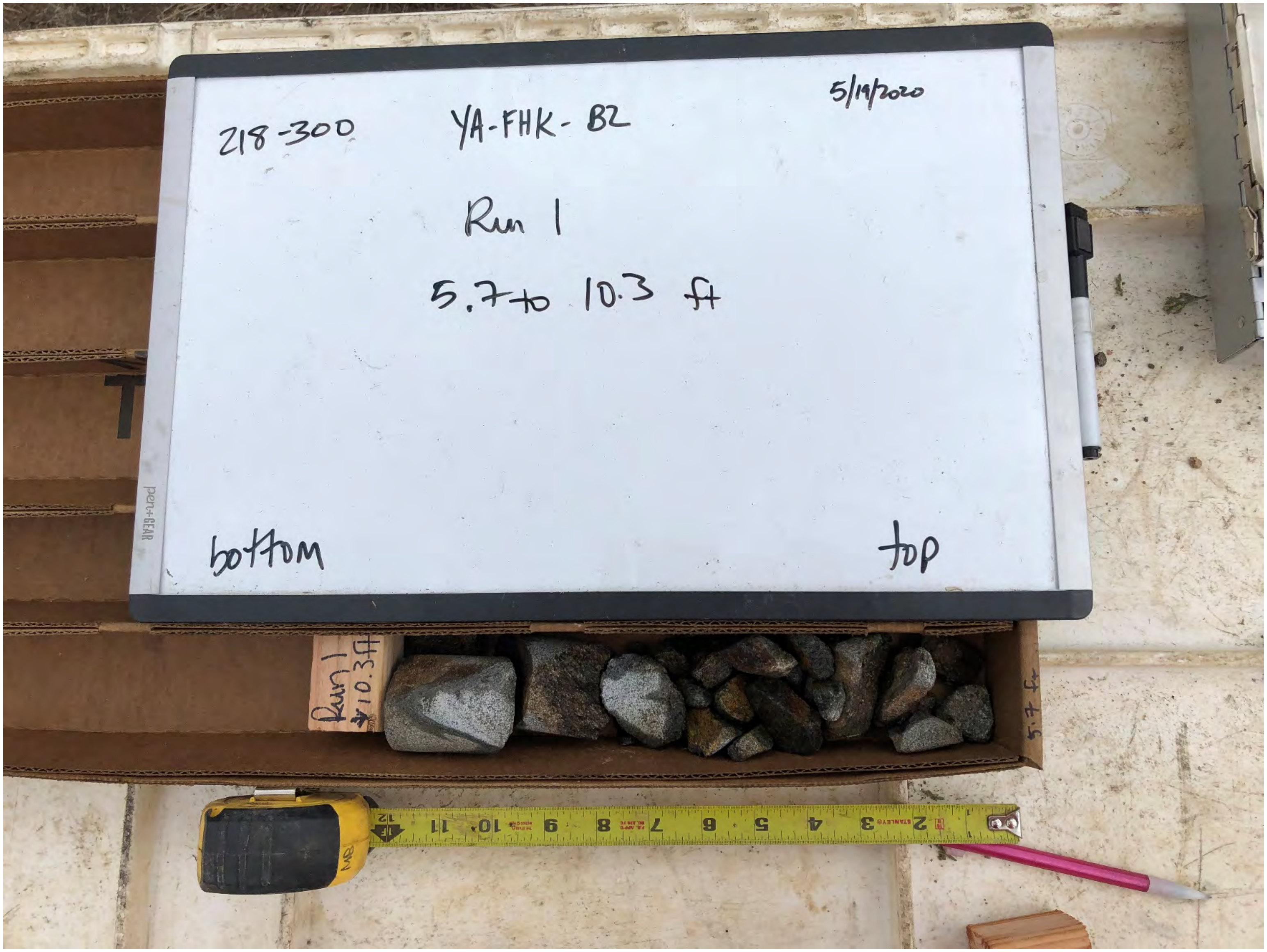
Box 1, 15 to 30 feet.

Box 2, 30 to 46 feet.

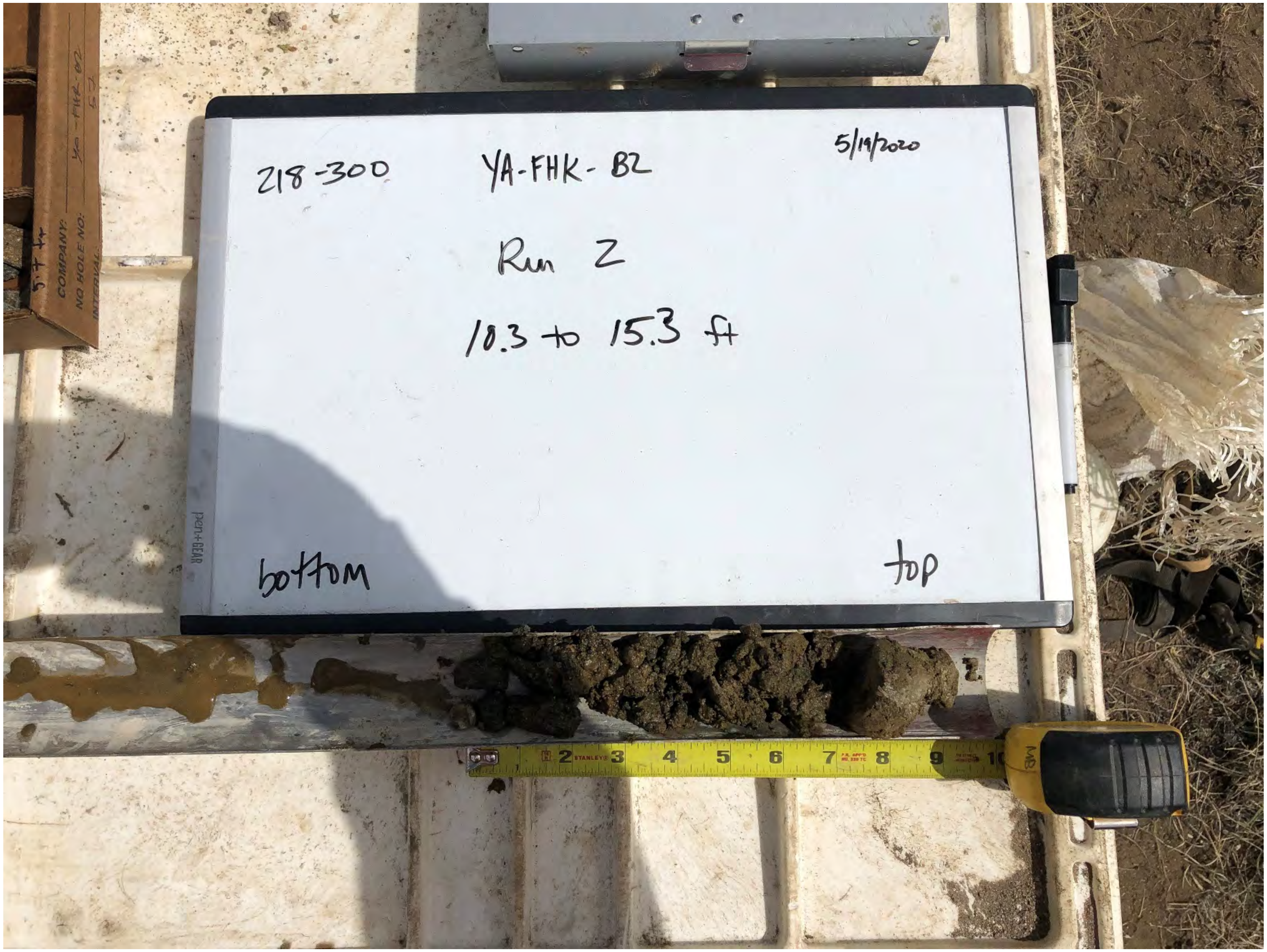
Box 3, 46 to 60.5 feet.

Box 4, 60.5 to 69 feet.

Box 5, 69 to 80 feet.



Run 1, 5.7 to 10.3 feet.



Run 2, 10.3 to 15.3 feet.

Run 3, 15.3 to 16.9 feet.

Run 4, 16.9 to 17.6 feet.

Run 5, 17.6 to 20.3 feet.

Run 6, 20.3 to 23.7 feet.

Run 7, 23.7 to 25.3 feet.

Box 2, 24.3 to 34 feet.

Box 3, 34 to 44.6 feet.

Box 4, 44.6 to 50.3 feet.

Box 1, 24 to 33 feet.

Box 2, 33 to 43 feet.

Box 3, 43 to 50.8 feet.

Box 4, 50.8 to 58.5 feet.

Box 5, 58.5 to 66 feet.

Box 6, 66 to 70 feet.

# Appendix C

## ROCK DISCONTINUITY MEASUREMENTS AND KINEMATIC ANALYSIS

### Area 1

| Dip | Dip Direction |
|---|---|
| 70 | 254 |
| 42 | 30 |
| 67 | 155 |
| 59 | 259 |
| 52 | 29 |
| 74 | 248 |
| 49 | 31 |
| 87 | 305 |

### Area 2

| Dip | Dip Direction |
|---|---|
| 51 | 45 |
| 61 | 60 |
| 64 | 166 |
| 73 | 253 |
| 67 | 268 |
| 76 | 343 |

### Area 3

| Dip | Dip Direction |
|---|---|
| 30 | 21 |
| 32 | 26 |
| 38 | 43 |
| 47 | 188 |
| 34 | 191 |
| 60 | 220 |
| 44 | 243 |
| 58 | 279 |

### Area 4

| Dip | Dip Direction |
|---|---|
| 28 | 4 |
| 44 | 46 |
| 36 | 49 |
| 37 | 86 |
| 80 | 216 |
| 80 | 234 |
| 69 | 320 |

### Area 5

| Dip | Dip Direction |
|---|---|
| 59 | 21 |
| 64 | 29 |
| 61 | 32 |
| 78 | 42 |
| 60 | 127 |
| 63 | 127 |
| 71 | 128 |
| 62 | 275 |
| 79 | 283 |
| 74 | 287 |
| 61 | 298 |
| 60 | 306 |

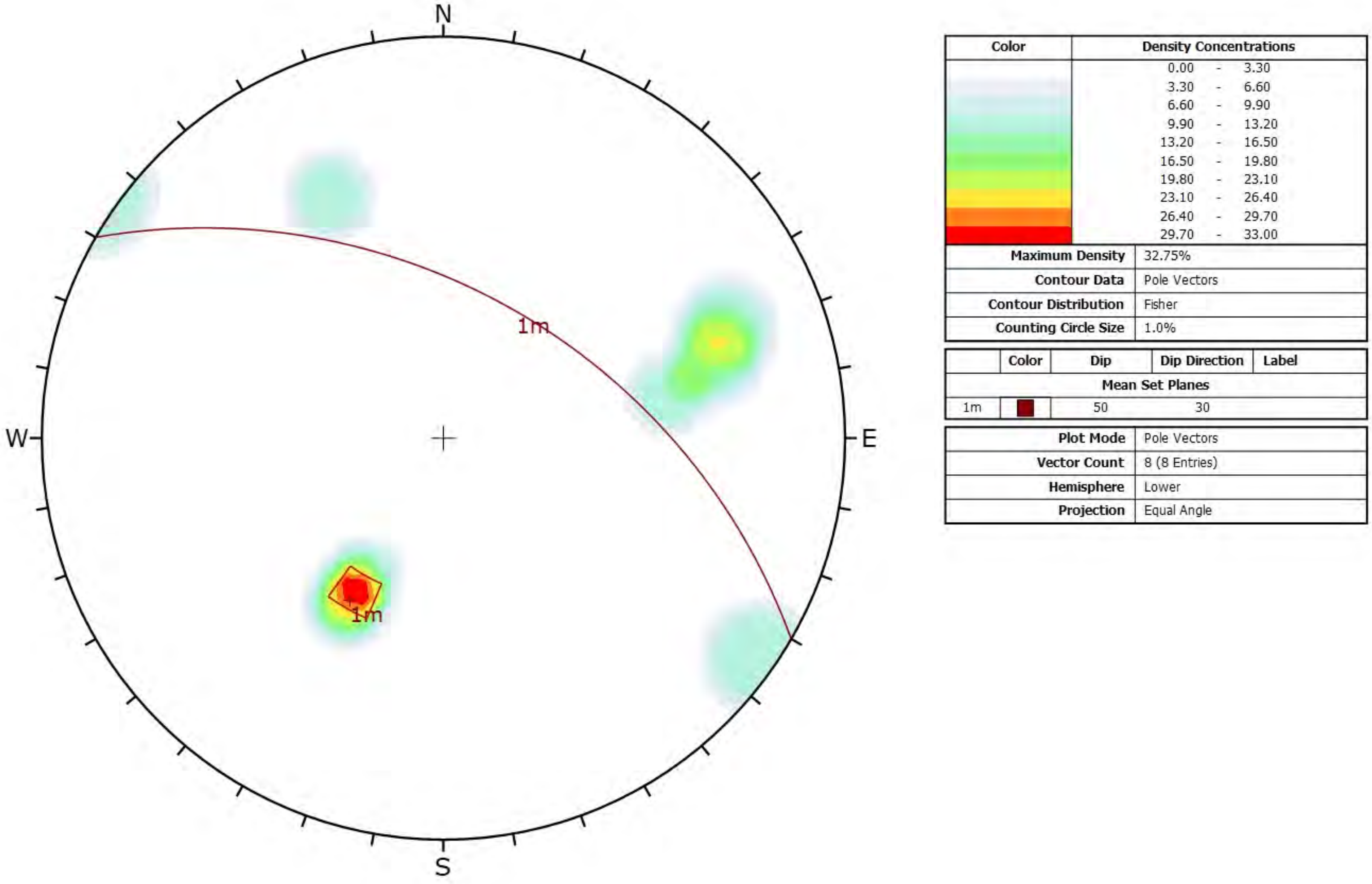
### Area 6

| Dip | Dip Direction |
|---|---|
| 41 | 66 |
| 74 | 221 |
| 68 | 338 |
| 73 | 241 |
| 86 | 145 |
| 70 | 44 |
| 20 | 50 |
| 84 | 168 |

| Dip | Dip Direction |
|---|---|
| 58 | 211 |
| 26 | 20 |
| 81 | 238 |
| 37 | 38 |
| 82 | 155 |
| 40 | 207 |
| 37 | 45 |
| 68 | 202 |
| 30 | 238 |
| 30 | 18 |
| 39 | 53 |
| 87 | 35 |
| 83 | 156 |
| 42 | 36 |
| 44 | 216 |
| 37 | 173 |
| 51 | 281 |
| 25 | 28 |
| 53 | 289 |
| 50 | 162 |
| 48 | 23 |
| 71 | 250 |
| 80 | 343 |
| 55 | 359 |
| 85 | 300 |
| 38 | 33 |
| 55 | 214 |
| 39 | 36 |
| 67 | 162 |
| 77 | 49 |
| 61 | 300 |
| 84 | 269 |
| 72 | 322 |
| 30 | 21 |
| 82 | 270 |
| 61 | 341 |
| 87 | 254 |
| 28 | 62 |
| 46 | 271 |
| 81 | 326 |

| Dip | Dip Direction |
|---|---|
| 34 | 16 |
| 69 | 300 |
| 44 | 210 |
| 85 | 280 |
| 74 | 290 |
| 43 | 181 |
| 27 | 256 |
| 54 | 90 |
| 85 | 248 |
| 56 | 84 |
| 59 | 320 |
| 74 | 300 |
| 38 | 36 |
| 57 | 240 |
| 55 | 180 |
| 38 | 14 |
| 51 | 256 |
| 59 | 106 |
| 51 | 187 |
| 75 | 282 |
| 62 | 179 |
| 40 | 187 |
| 74 | 257 |
| 40 | 17 |
| 31 | 29 |
| 35 | 1 |
| 62 | 190 |
| 47 | 190 |
| 31 | 62 |
| 54 | 194 |
| 76 | 244 |
| 75 | 161 |
| 77 | 246 |
| 46 | 213 |
| 38 | 37 |
| 36* | 34 |
| 36* | 352 |
| 39* | 9 |

*Starred measurements are foliation/banding planes



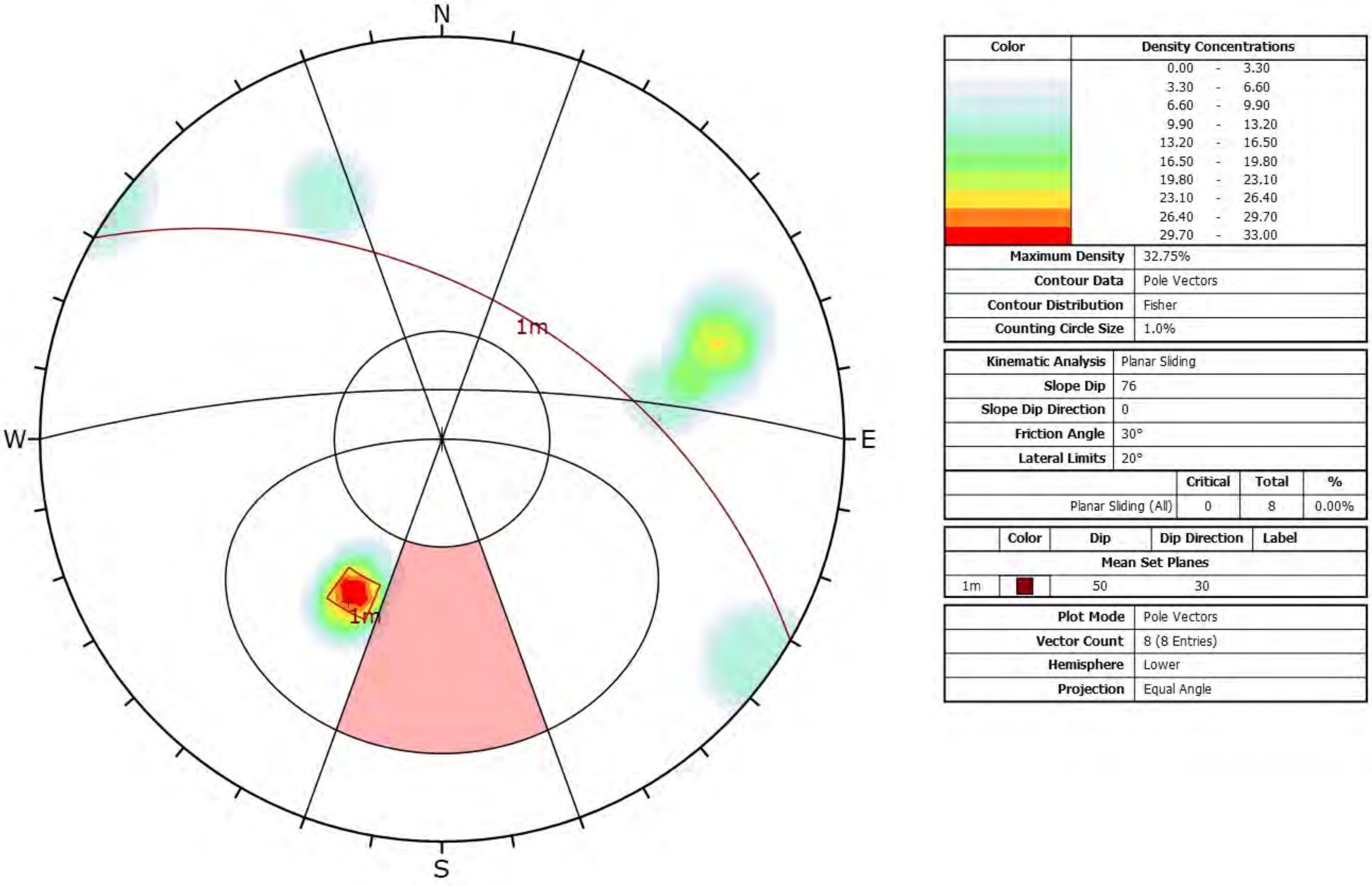
| Color | Density Concentrations |
|---|---|
| | 0.00 - 3.30 |
| | 3.30 - 6.60 |
| | 6.60 - 9.90 |
| | 9.90 - 13.20 |
| | 13.20 - 16.50 |
| | 16.50 - 19.80 |
| | 19.80 - 23.10 |
| | 23.10 - 26.40 |
| | 26.40 - 29.70 |
| 🟥 | 29.70 - 33.00 |

| | |
|---|---|
| **Maximum Density** | 32.75% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| | | Mean Set Planes | | |
| 1m | 🟥 | 50 | 30 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 8 (8 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 1, Dip and Dip Direction of the Mean Set Plane.



| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 3.30 |
| ⬜ | 3.30 - 6.60 |
| ⬜ | 6.60 - 9.90 |
| 🟦 | 9.90 - 13.20 |
| 🟩 | 13.20 - 16.50 |
| 🟩 | 16.50 - 19.80 |
| 🟩 | 19.80 - 23.10 |
| 🟨 | 23.10 - 26.40 |
| 🟧 | 26.40 - 29.70 |
| 🟥 | 29.70 - 33.00 |

| | |
|---|---|
| **Maximum Density** | 32.75% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

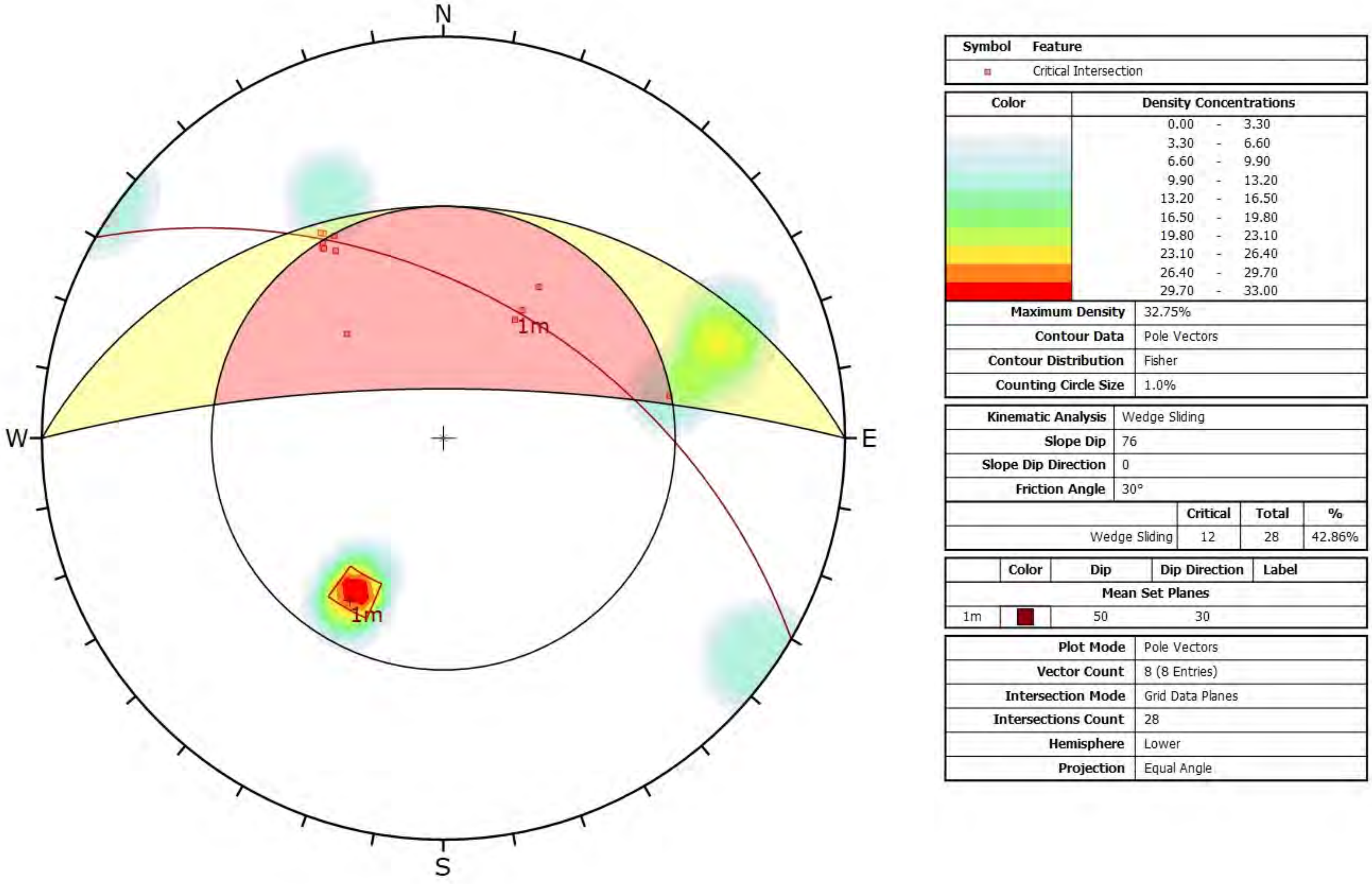
| | |
|---|---|
| **Kinematic Analysis** | Planar Sliding |
| **Slope Dip** | 76 |
| **Slope Dip Direction** | 0 |
| **Friction Angle** | 30° |
| **Lateral Limits** | 20° |

| | Critical | Total | % |
|---|---|---|---|
| Planar Sliding (All) | 0 | 8 | 0.00% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| **Mean Set Planes** | | | | |
| 1m | 🟥 | 50 | 30 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 8 (8 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 1, Planar Failure Analysis.



| Symbol | Feature |
|---|---|
| ▪ | Critical Intersection |

| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 3.30 |
| ⬜ | 3.30 - 6.60 |
| 🟦 | 6.60 - 9.90 |
| 🟦 | 9.90 - 13.20 |
| 🟩 | 13.20 - 16.50 |
| 🟩 | 16.50 - 19.80 |
| 🟩 | 19.80 - 23.10 |
| 🟨 | 23.10 - 26.40 |
| 🟧 | 26.40 - 29.70 |
| 🟥 | 29.70 - 33.00 |

| | |
|---|---|
| Maximum Density | 32.75% |
| Contour Data | Pole Vectors |
| Contour Distribution | Fisher |
| Counting Circle Size | 1.0% |

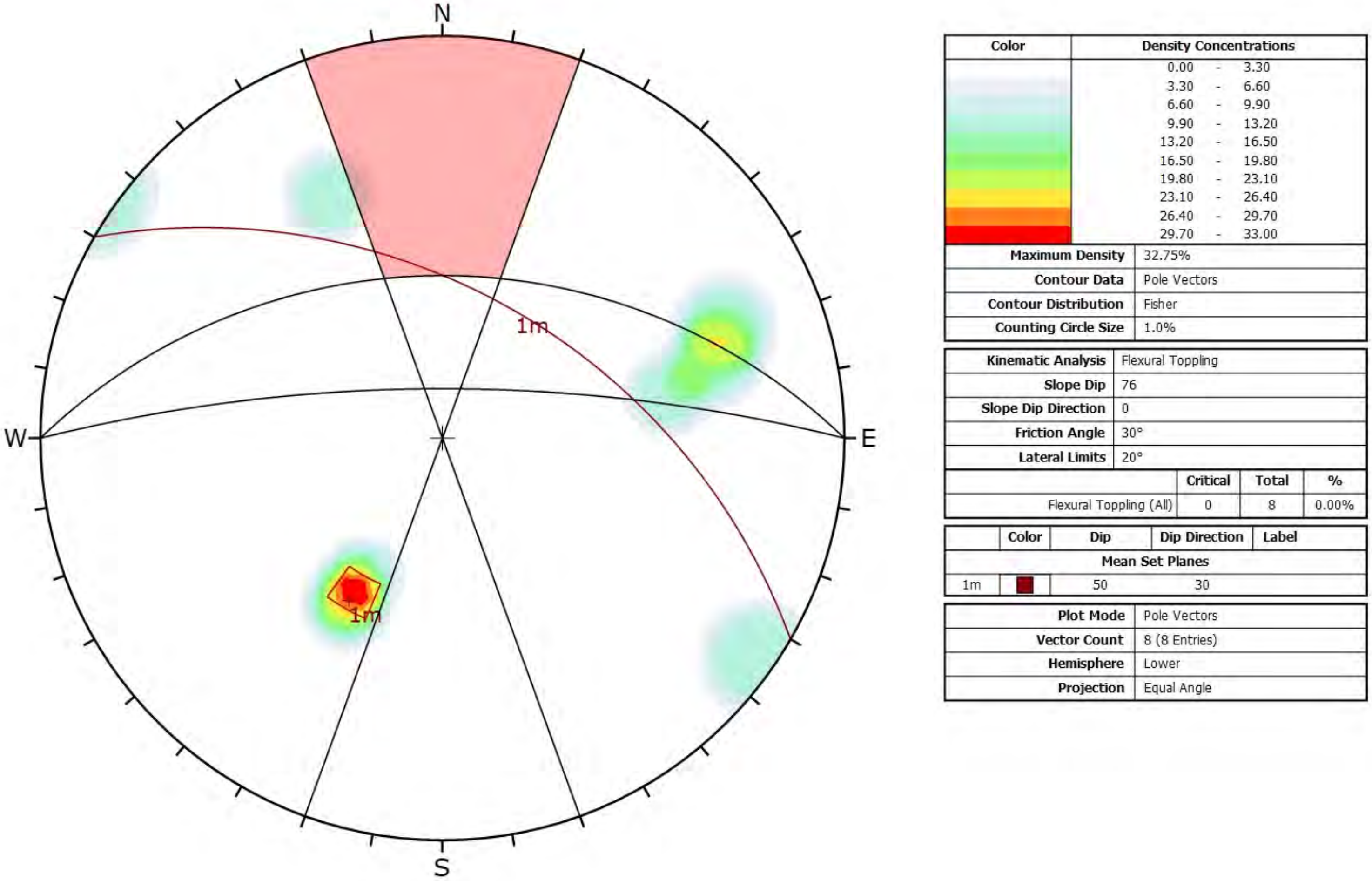
| | |
|---|---|
| Kinematic Analysis | Wedge Sliding |
| Slope Dip | 76 |
| Slope Dip Direction | 0 |
| Friction Angle | 30° |

| | Critical | Total | % |
|---|---|---|---|
| Wedge Sliding | 12 | 28 | 42.86% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| Mean Set Planes | | | | |
| 1m | 🟥 | 50 | 30 | |

| | |
|---|---|
| Plot Mode | Pole Vectors |
| Vector Count | 8 (8 Entries) |
| Intersection Mode | Grid Data Planes |
| Intersections Count | 28 |
| Hemisphere | Lower |
| Projection | Equal Angle |

Area 1, Wedge Failure Analysis.



| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 3.30 |
| ⬜ | 3.30 - 6.60 |
| 🟦 | 6.60 - 9.90 |
| 🟦 | 9.90 - 13.20 |
| 🟩 | 13.20 - 16.50 |
| 🟩 | 16.50 - 19.80 |
| 🟩 | 19.80 - 23.10 |
| 🟨 | 23.10 - 26.40 |
| 🟧 | 26.40 - 29.70 |
| 🟥 | 29.70 - 33.00 |

| | |
|---|---|
| Maximum Density | 32.75% |
| Contour Data | Pole Vectors |
| Contour Distribution | Fisher |
| Counting Circle Size | 1.0% |

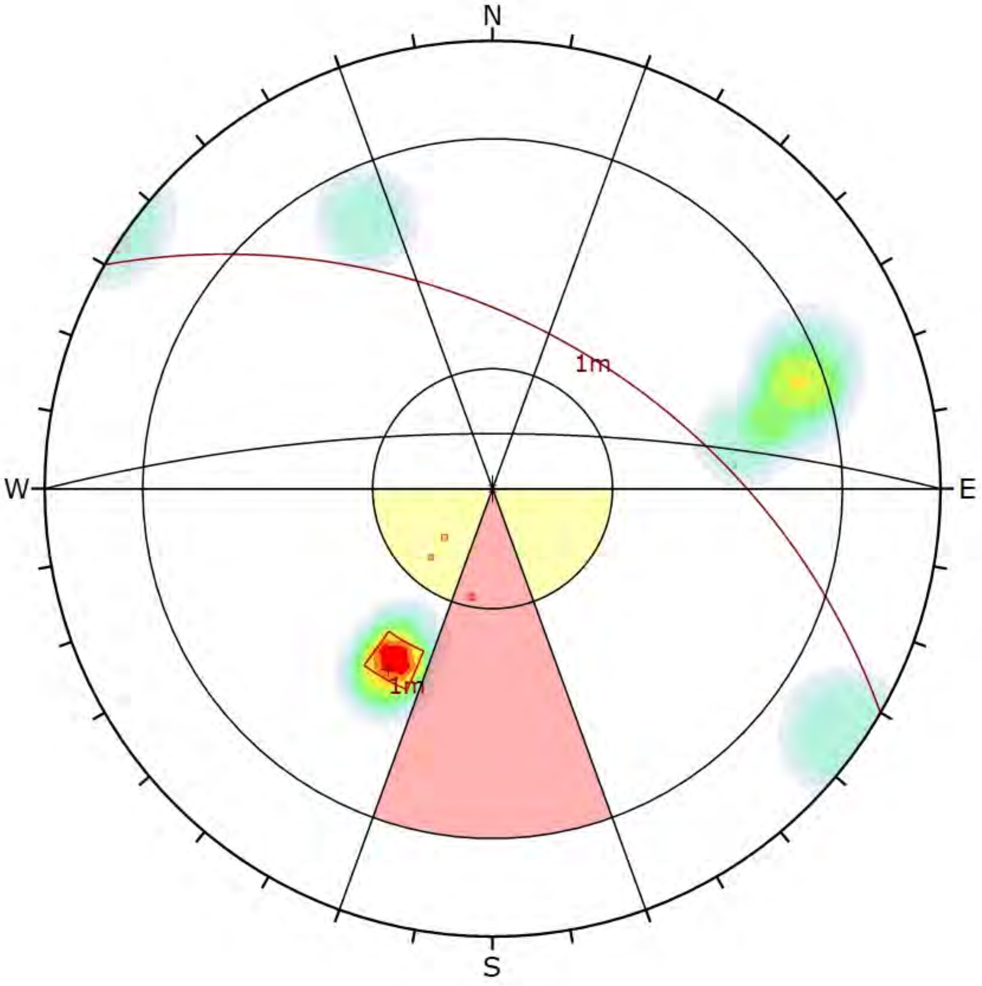
| | |
|---|---|
| Kinematic Analysis | Flexural Toppling |
| Slope Dip | 76 |
| Slope Dip Direction | 0 |
| Friction Angle | 30° |
| Lateral Limits | 20° |

| | Critical | Total | % |
|---|---|---|---|
| Flexural Toppling (All) | 0 | 8 | 0.00% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| Mean Set Planes | | | | |
| 1m | 🟥 | 50 | 30 | |

| | |
|---|---|
| Plot Mode | Pole Vectors |
| Vector Count | 8 (8 Entries) |
| Hemisphere | Lower |
| Projection | Equal Angle |

Area 1, Flexural Toppling Analysis.



| Symbol | Feature |
|---|---|
| □ | Critical Intersection |

| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 3.30 |
| ⬜ | 3.30 - 6.60 |
| ⬜ | 6.60 - 9.90 |
| 🟦 | 9.90 - 13.20 |
| 🟩 | 13.20 - 16.50 |
| 🟩 | 16.50 - 19.80 |
| 🟩 | 19.80 - 23.10 |
| 🟨 | 23.10 - 26.40 |
| 🟧 | 26.40 - 29.70 |
| 🟥 | 29.70 - 33.00 |

| | |
|---|---|
| **Maximum Density** | 32.75% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

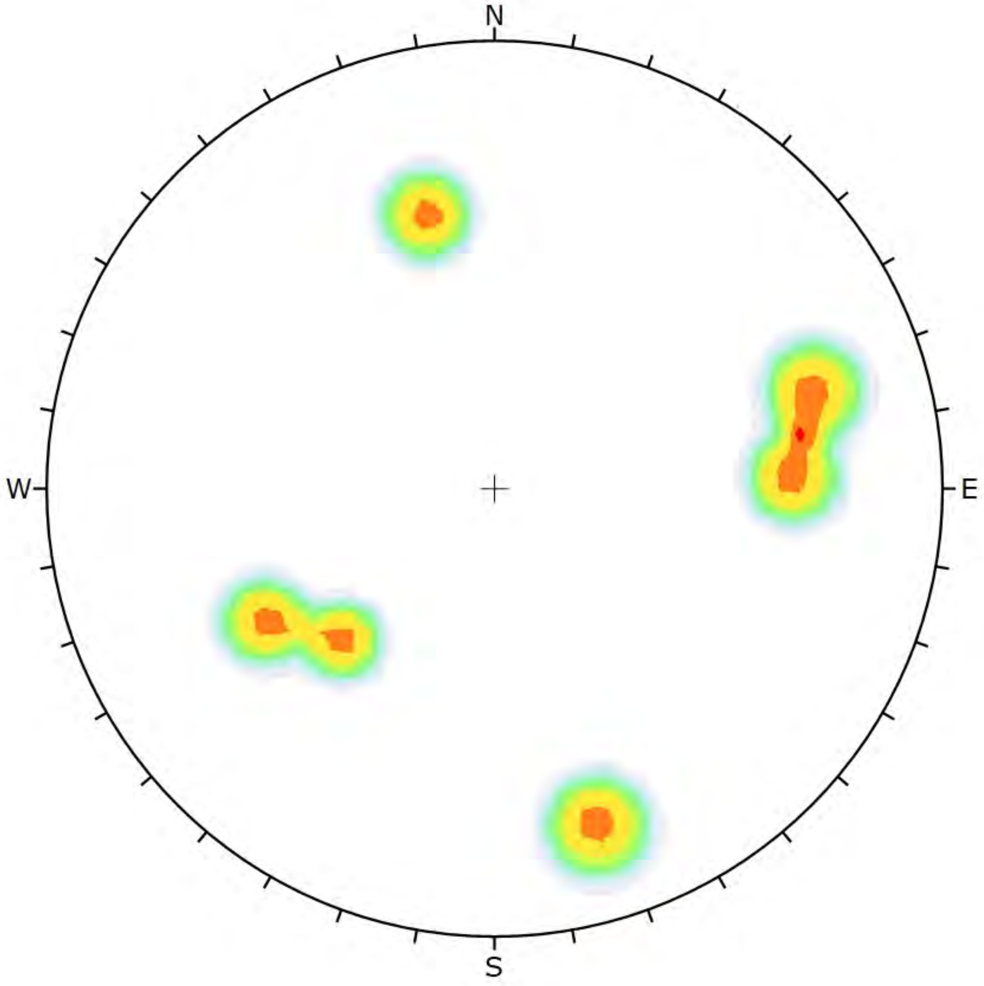
| | |
|---|---|
| **Kinematic Analysis** | Direct Toppling |
| **Slope Dip** | 76 |
| **Slope Dip Direction** | 0 |
| **Friction Angle** | 30° |
| **Lateral Limits** | 20° |

| | Critical | Total | % |
|---|---|---|---|
| Direct Toppling (Intersection) | 1 | 28 | 3.57% |
| Oblique Toppling (Intersection) | 2 | 28 | 7.14% |
| Base Plane (All) | 0 | 8 | 0.00% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| | | **Mean Set Planes** | | |
| 1m | 🟥 | 50 | 30 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 8 (8 Entries) |
| **Intersection Mode** | Grid Data Planes |
| **Intersections Count** | 28 |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 1, Direct Toppling Analysis.

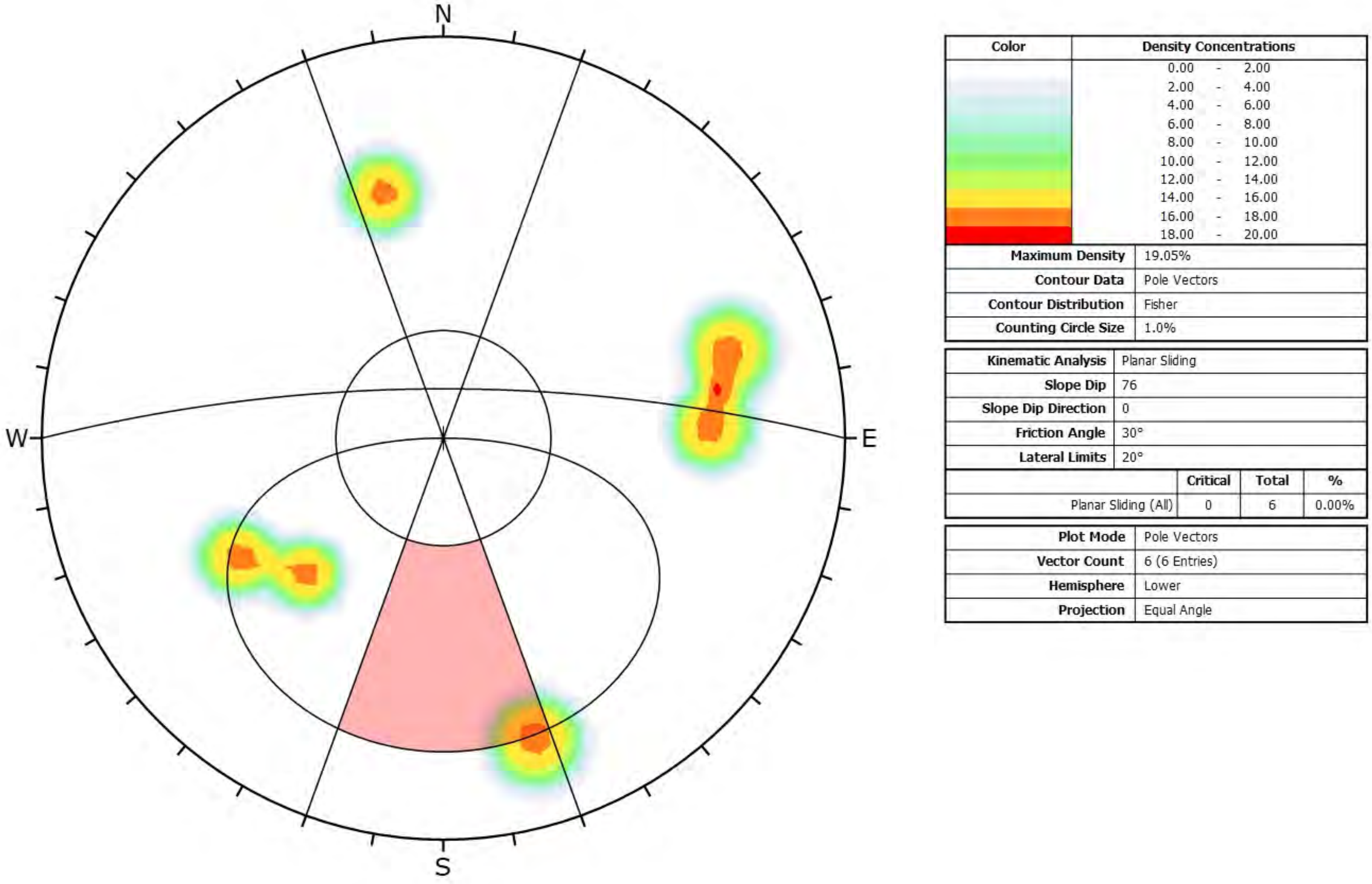


| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.00 |
| ⬜ | 2.00 - 4.00 |
| 🟦 | 4.00 - 6.00 |
| 🟦 | 6.00 - 8.00 |
| 🟩 | 8.00 - 10.00 |
| 🟩 | 10.00 - 12.00 |
| 🟩 | 12.00 - 14.00 |
| 🟨 | 14.00 - 16.00 |
| 🟧 | 16.00 - 18.00 |
| 🟥 | 18.00 - 20.00 |

| | |
|---|---|
| **Maximum Density** | 19.05% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 6 (6 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 2 Stereonet.



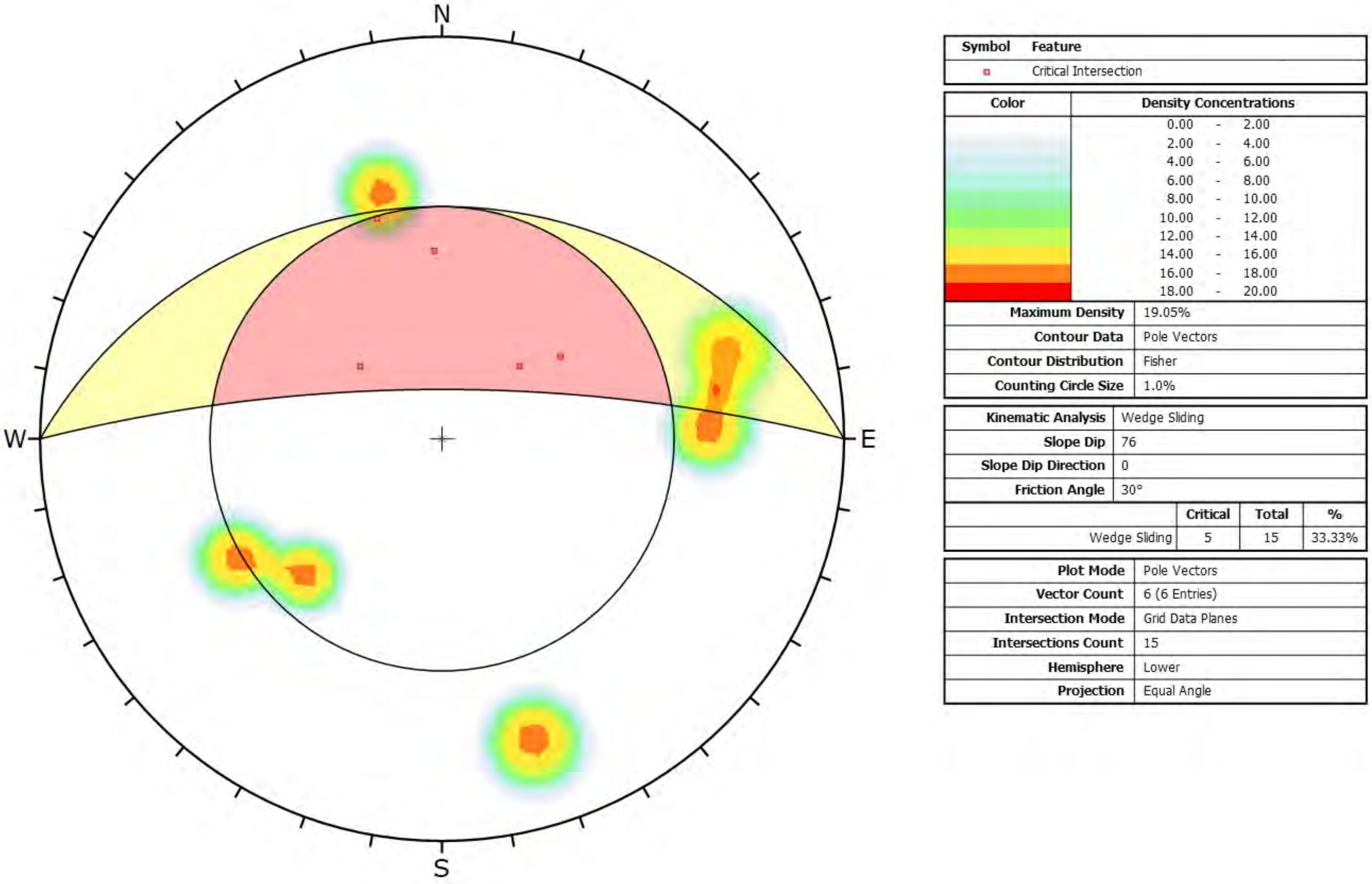
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.00 |
| ⬜ | 2.00 - 4.00 |
| 🟦 | 4.00 - 6.00 |
| 🟦 | 6.00 - 8.00 |
| 🟩 | 8.00 - 10.00 |
| 🟩 | 10.00 - 12.00 |
| 🟩 | 12.00 - 14.00 |
| 🟨 | 14.00 - 16.00 |
| 🟧 | 16.00 - 18.00 |
| 🟥 | 18.00 - 20.00 |

| | |
|---|---|
| **Maximum Density** | 19.05% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | | | |
|---|---|---|---|
| **Kinematic Analysis** | Planar Sliding | | |
| **Slope Dip** | 76 | | |
| **Slope Dip Direction** | 0 | | |
| **Friction Angle** | 30° | | |
| **Lateral Limits** | 20° | | |
| | **Critical** | **Total** | **%** |
| Planar Sliding (All) | 0 | 6 | 0.00% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 6 (6 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 2, Planar Failure Analysis.



| Symbol | Feature |
|---|---|
| ▫ | Critical Intersection |

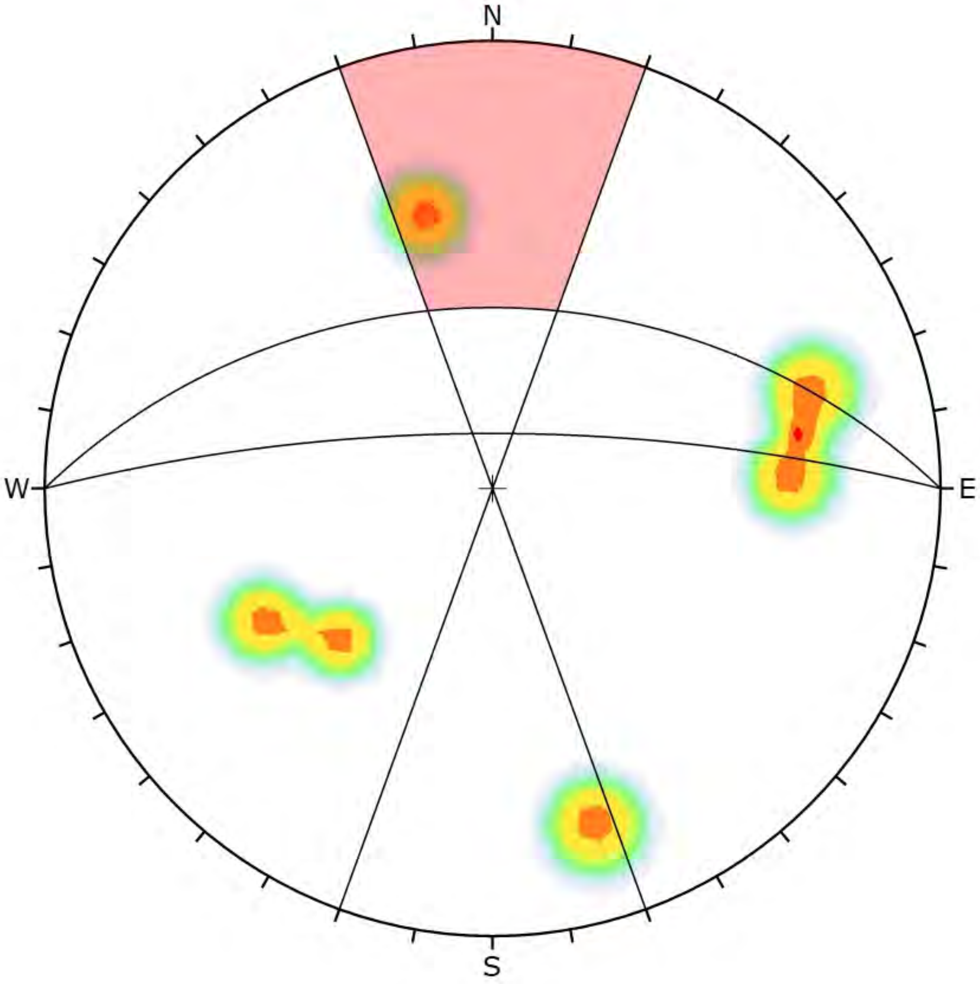
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.00 |
| ⬜ | 2.00 - 4.00 |
| ⬜ | 4.00 - 6.00 |
| ⬜ | 6.00 - 8.00 |
| 🟩 | 8.00 - 10.00 |
| 🟩 | 10.00 - 12.00 |
| 🟩 | 12.00 - 14.00 |
| 🟨 | 14.00 - 16.00 |
| 🟧 | 16.00 - 18.00 |
| 🟥 | 18.00 - 20.00 |

| | |
|---|---|
| **Maximum Density** | 19.05% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | | | |
|---|---|---|---|
| **Kinematic Analysis** | Wedge Sliding | | |
| **Slope Dip** | 76 | | |
| **Slope Dip Direction** | 0 | | |
| **Friction Angle** | 30° | | |
| | **Critical** | **Total** | **%** |
| Wedge Sliding | 5 | 15 | 33.33% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 6 (6 Entries) |
| **Intersection Mode** | Grid Data Planes |
| **Intersections Count** | 15 |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 2, Wedge Failure Analysis.



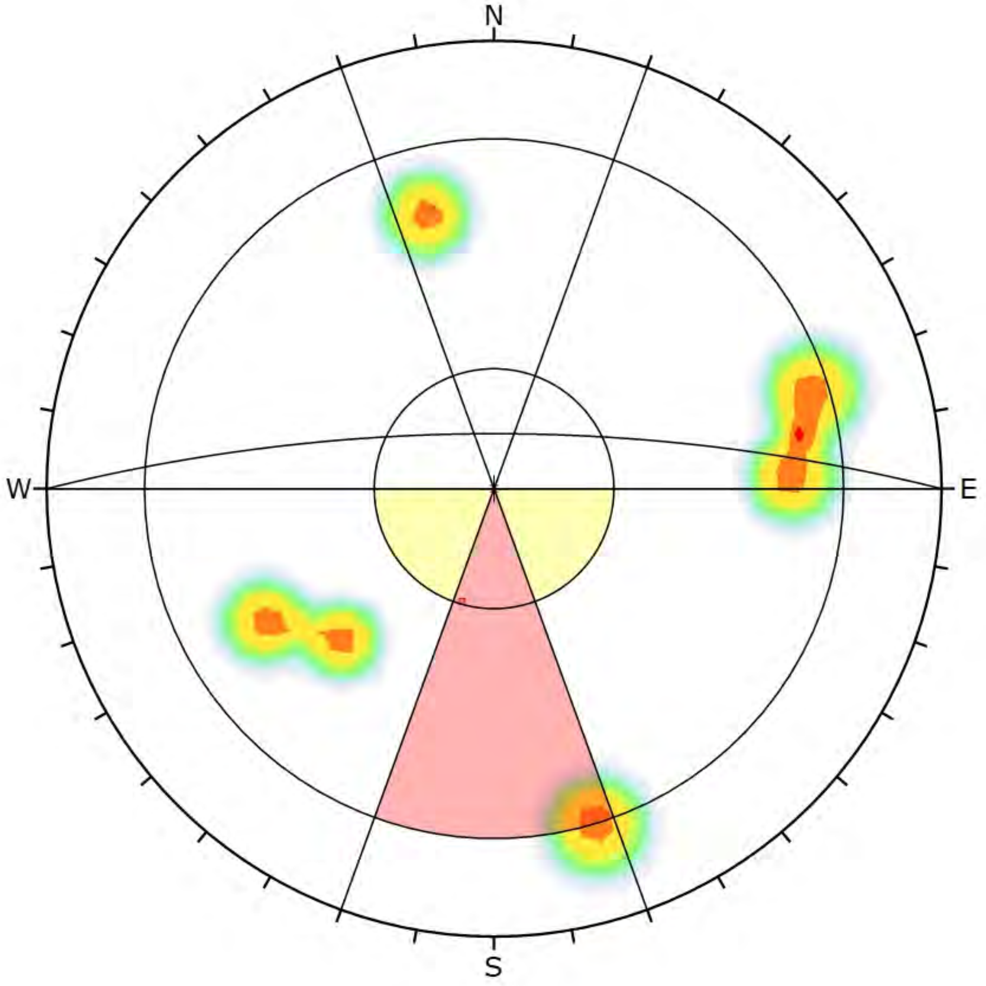
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.00 |
| ⬜ | 2.00 - 4.00 |
| ⬜ | 4.00 - 6.00 |
| 🟦 | 6.00 - 8.00 |
| 🟩 | 8.00 - 10.00 |
| 🟩 | 10.00 - 12.00 |
| 🟩 | 12.00 - 14.00 |
| 🟨 | 14.00 - 16.00 |
| 🟧 | 16.00 - 18.00 |
| 🟥 | 18.00 - 20.00 |

| | |
|---|---|
| **Maximum Density** | 19.05% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | | | |
|---|---|---|---|
| **Kinematic Analysis** | Flexural Toppling | | |
| **Slope Dip** | 76 | | |
| **Slope Dip Direction** | 0 | | |
| **Friction Angle** | 30° | | |
| **Lateral Limits** | 20° | | |
| | **Critical** | **Total** | **%** |
| Flexural Toppling (All) | 1 | 6 | 16.67% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 6 (6 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 2, Flexural Toppling Analysis.



| Symbol | Feature |
|---|---|
| ▪ | Critical Intersection |

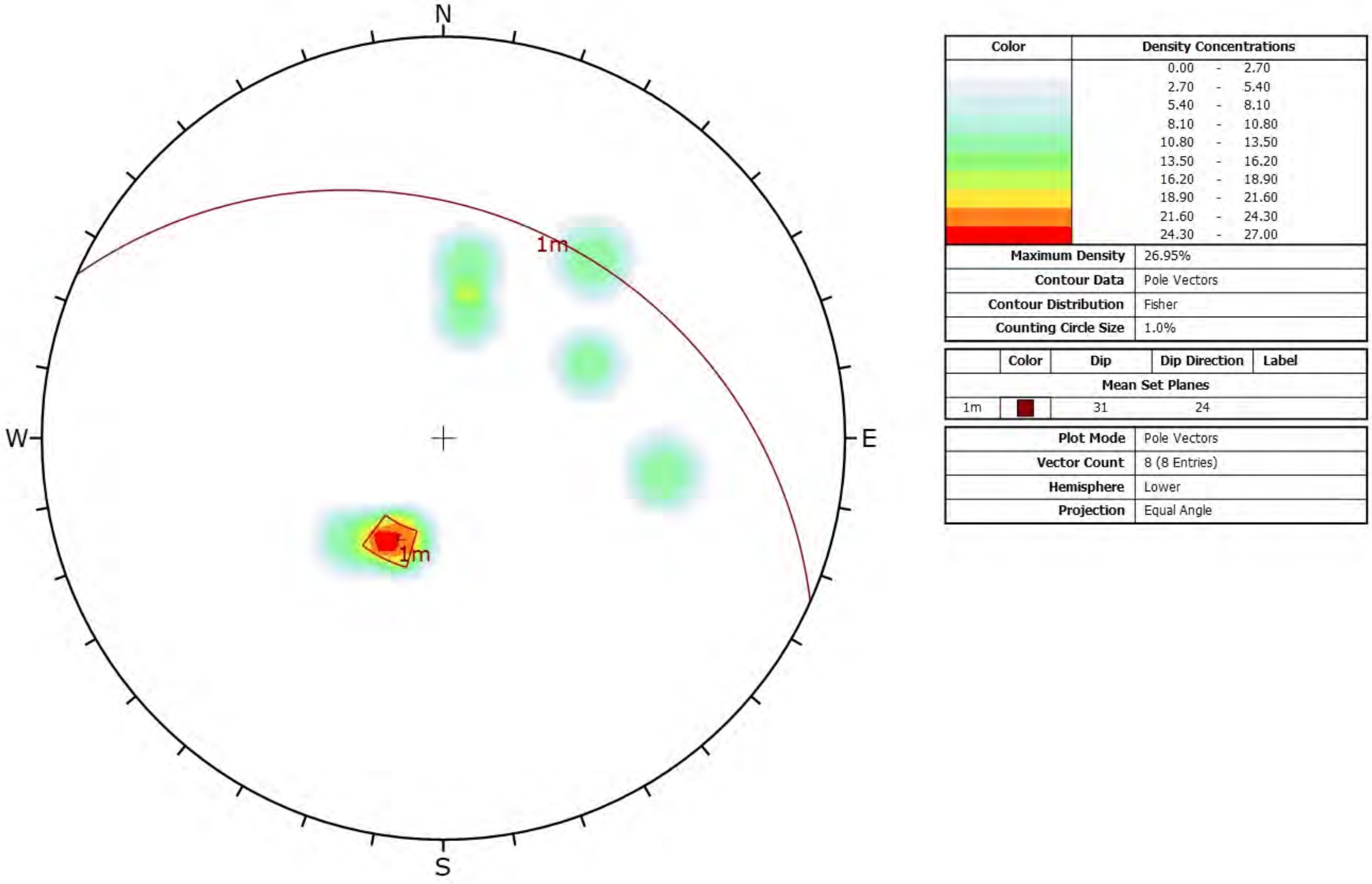
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.00 |
| ⬜ | 2.00 - 4.00 |
| 🟦 | 4.00 - 6.00 |
| 🟦 | 6.00 - 8.00 |
| 🟩 | 8.00 - 10.00 |
| 🟩 | 10.00 - 12.00 |
| 🟩 | 12.00 - 14.00 |
| 🟨 | 14.00 - 16.00 |
| 🟧 | 16.00 - 18.00 |
| 🟥 | 18.00 - 20.00 |
| **Maximum Density** | 19.05% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | |
|---|---|
| **Kinematic Analysis** | Direct Toppling |
| **Slope Dip** | 76 |
| **Slope Dip Direction** | 0 |
| **Friction Angle** | 30° |
| **Lateral Limits** | 20° |

| | **Critical** | **Total** | **%** |
|---|---|---|---|
| Direct Toppling (Intersection) | 1 | 15 | 6.67% |
| Oblique Toppling (Intersection) | 0 | 15 | 0.00% |
| Base Plane (All) | 1 | 6 | 16.67% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 6 (6 Entries) |
| **Intersection Mode** | Grid Data Planes |
| **Intersections Count** | 15 |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 2, Direct Toppling Analysis.



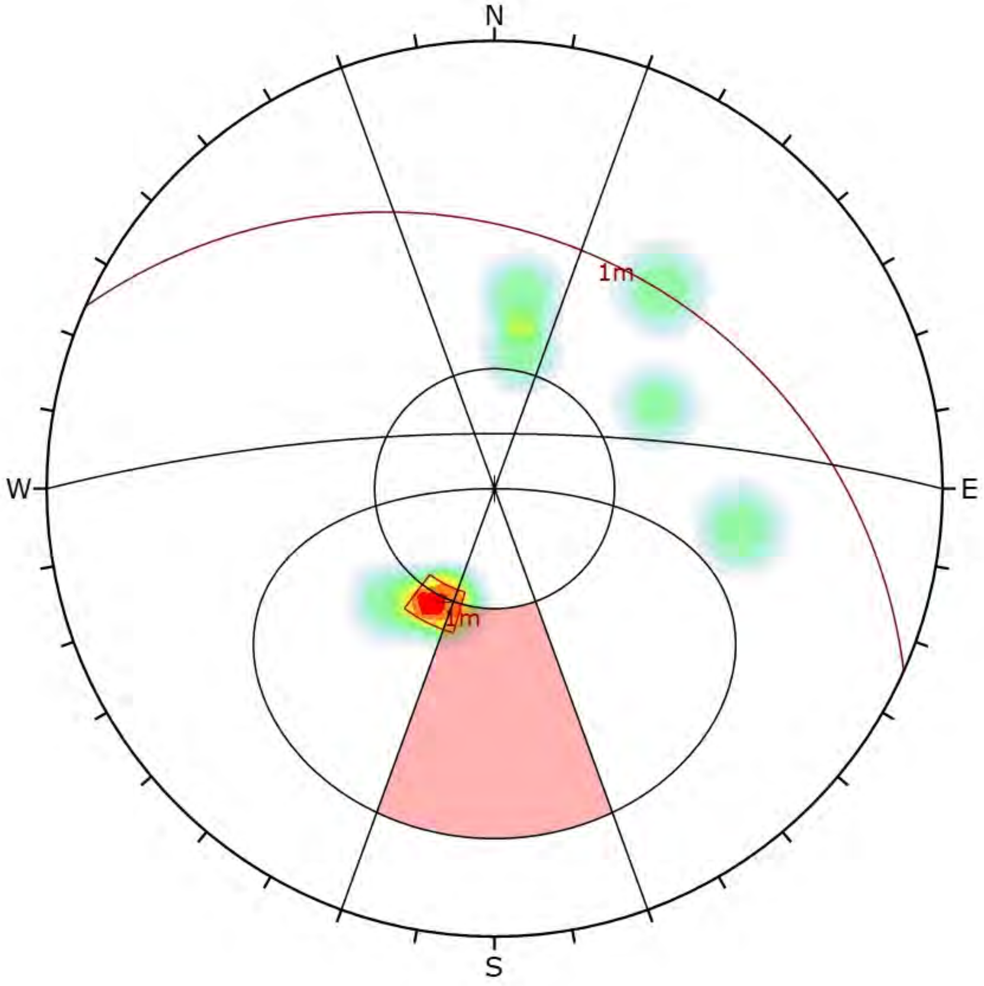
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.70 |
| ⬜ | 2.70 - 5.40 |
| ⬜ | 5.40 - 8.10 |
| 🟦 | 8.10 - 10.80 |
| 🟩 | 10.80 - 13.50 |
| 🟩 | 13.50 - 16.20 |
| 🟩 | 16.20 - 18.90 |
| 🟨 | 18.90 - 21.60 |
| 🟧 | 21.60 - 24.30 |
| 🟥 | 24.30 - 27.00 |

| | |
|---|---|
| Maximum Density | 26.95% |
| Contour Data | Pole Vectors |
| Contour Distribution | Fisher |
| Counting Circle Size | 1.0% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| Mean Set Planes | | | | |
| 1m | 🟥 | 31 | 24 | |

| | |
|---|---|
| Plot Mode | Pole Vectors |
| Vector Count | 8 (8 Entries) |
| Hemisphere | Lower |
| Projection | Equal Angle |

Area 3, Dip and Dip Direction of the Mean Set Plane.



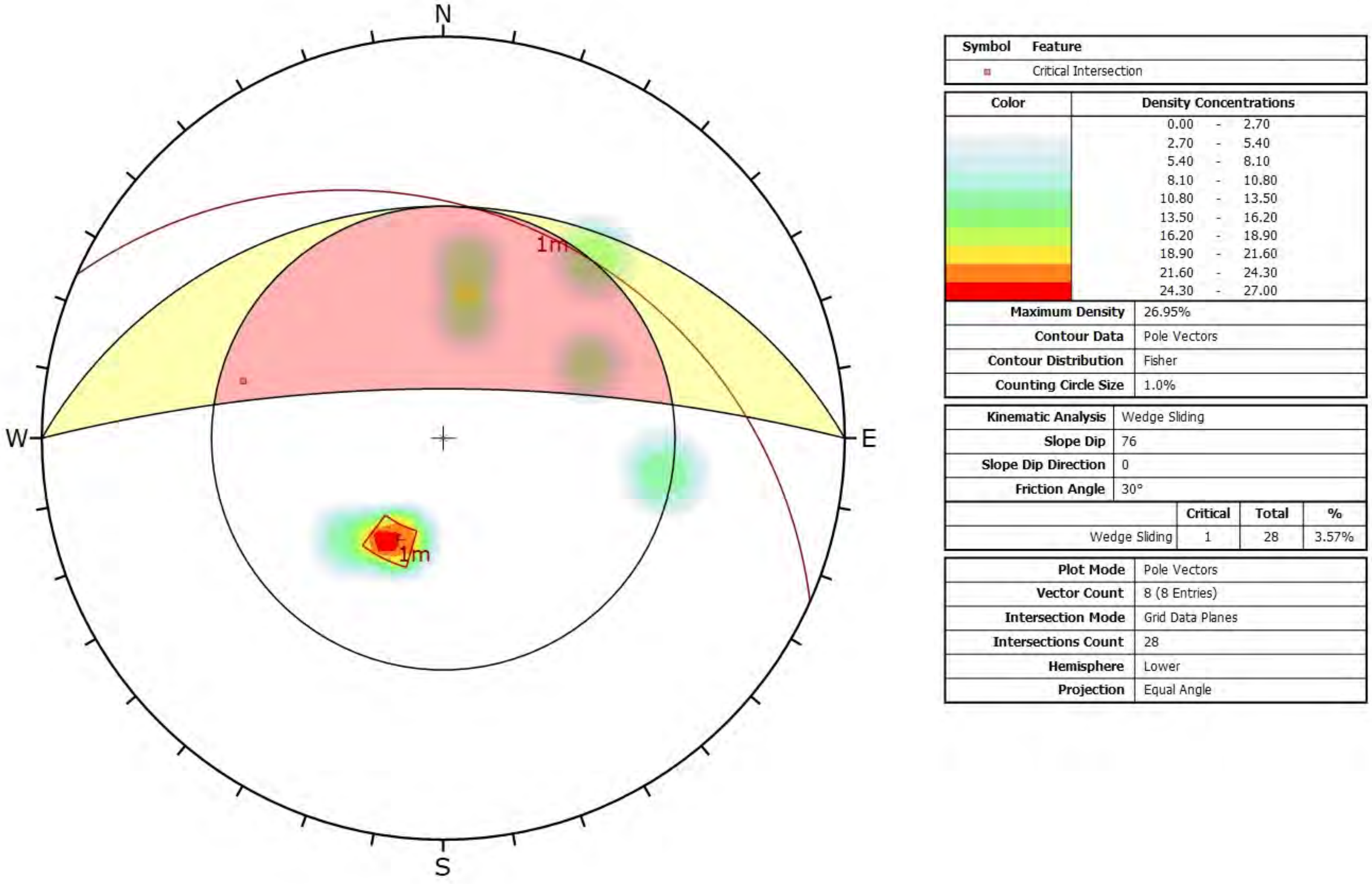
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.70 |
| ⬜ | 2.70 - 5.40 |
| ⬜ | 5.40 - 8.10 |
| 🟦 | 8.10 - 10.80 |
| 🟩 | 10.80 - 13.50 |
| 🟩 | 13.50 - 16.20 |
| 🟩 | 16.20 - 18.90 |
| 🟨 | 18.90 - 21.60 |
| 🟧 | 21.60 - 24.30 |
| 🟥 | 24.30 - 27.00 |

| | |
|---|---|
| **Maximum Density** | 26.95% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | | | |
|---|---|---|---|
| **Kinematic Analysis** | Planar Sliding | | |
| **Slope Dip** | 76 | | |
| **Slope Dip Direction** | 0 | | |
| **Friction Angle** | 30° | | |
| **Lateral Limits** | 20° | | |
| | **Critical** | **Total** | **%** |
| Planar Sliding (All) | 0 | 8 | 0.00% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 8 (8 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 3, Planar Failure Analysis.



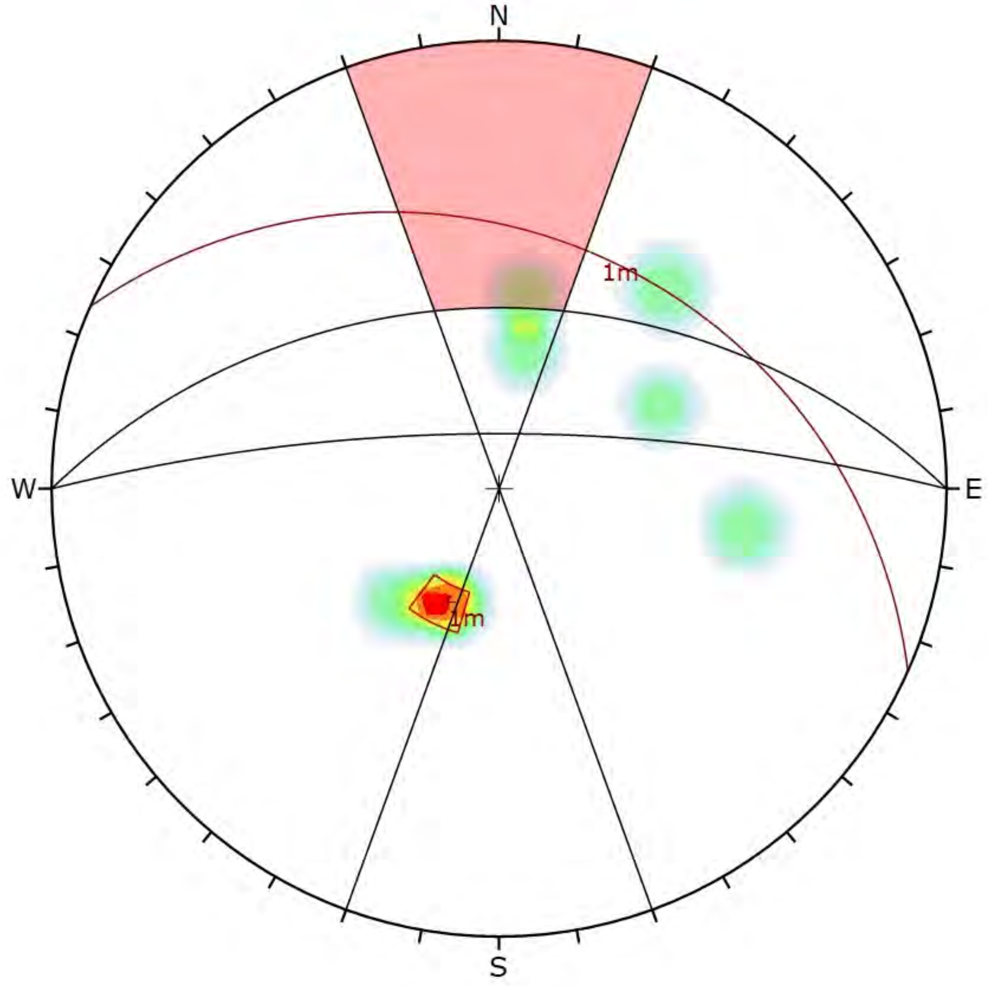
| Symbol | Feature |
|---|---|
| ▪ | Critical Intersection |

| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.70 |
| ⬜ | 2.70 - 5.40 |
| 🟦 | 5.40 - 8.10 |
| 🟦 | 8.10 - 10.80 |
| 🟩 | 10.80 - 13.50 |
| 🟩 | 13.50 - 16.20 |
| 🟩 | 16.20 - 18.90 |
| 🟨 | 18.90 - 21.60 |
| 🟧 | 21.60 - 24.30 |
| 🟥 | 24.30 - 27.00 |
| **Maximum Density** | 26.95% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | | | |
|---|---|---|---|
| **Kinematic Analysis** | Wedge Sliding | | |
| **Slope Dip** | 76 | | |
| **Slope Dip Direction** | 0 | | |
| **Friction Angle** | 30° | | |
| | **Critical** | **Total** | **%** |
| Wedge Sliding | 1 | 28 | 3.57% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 8 (8 Entries) |
| **Intersection Mode** | Grid Data Planes |
| **Intersections Count** | 28 |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 3, Wedge Failure Analysis.

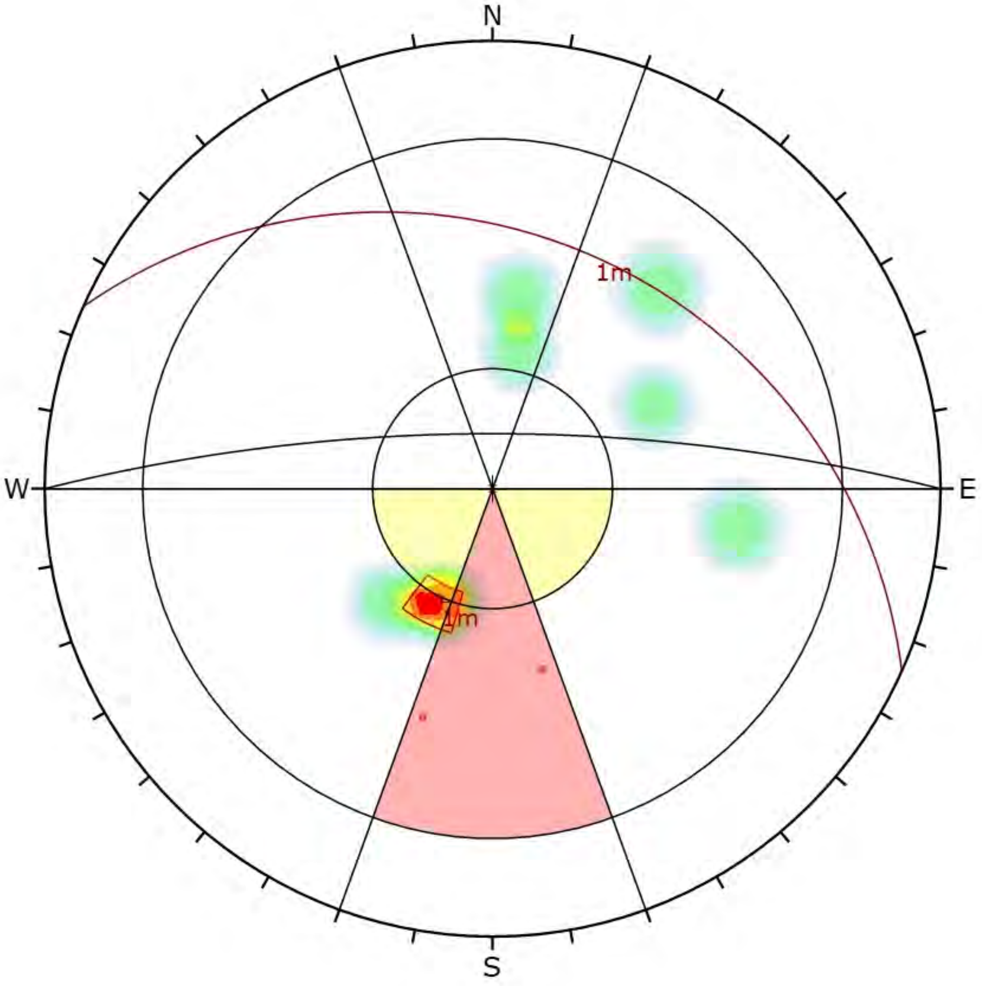


| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.70 |
| ⬜ | 2.70 - 5.40 |
| ⬜ | 5.40 - 8.10 |
| 🟦 | 8.10 - 10.80 |
| 🟩 | 10.80 - 13.50 |
| 🟩 | 13.50 - 16.20 |
| 🟩 | 16.20 - 18.90 |
| 🟨 | 18.90 - 21.60 |
| 🟧 | 21.60 - 24.30 |
| 🟥 | 24.30 - 27.00 |
| **Maximum Density** | 26.95% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | | | |
|---|---|---|---|
| **Kinematic Analysis** | Flexural Toppling | | |
| **Slope Dip** | 76 | | |
| **Slope Dip Direction** | 0 | | |
| **Friction Angle** | 30° | | |
| **Lateral Limits** | 20° | | |
| | **Critical** | **Total** | **%** |
| Flexural Toppling (All) | 1 | 8 | 12.50% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 8 (8 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 3, Flexural Toppling Analysis.



| Symbol | Feature |
|---|---|
| ▪ | Critical Intersection |

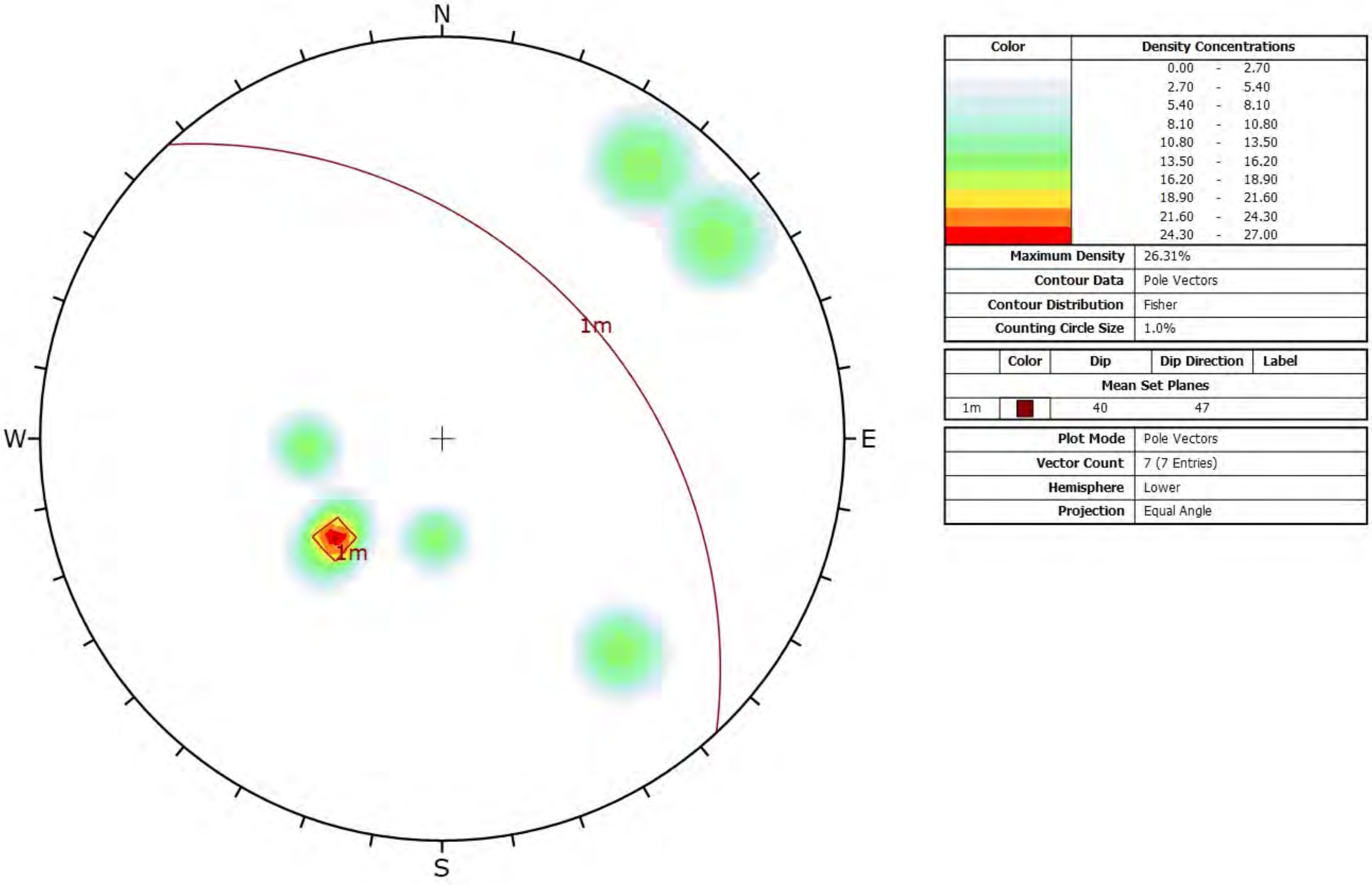
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.70 |
| 🟦 | 2.70 - 5.40 |
| 🟦 | 5.40 - 8.10 |
| 🟦 | 8.10 - 10.80 |
| 🟩 | 10.80 - 13.50 |
| 🟩 | 13.50 - 16.20 |
| 🟩 | 16.20 - 18.90 |
| 🟨 | 18.90 - 21.60 |
| 🟧 | 21.60 - 24.30 |
| 🟥 | 24.30 - 27.00 |
| **Maximum Density** | 26.95% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | |
|---|---|
| **Kinematic Analysis** | Direct Toppling |
| **Slope Dip** | 76 |
| **Slope Dip Direction** | 0 |
| **Friction Angle** | 30° |
| **Lateral Limits** | 20° |

| | **Critical** | **Total** | **%** |
|---|---|---|---|
| Direct Toppling (Intersection) | 2 | 28 | 7.14% |
| Oblique Toppling (Intersection) | 0 | 28 | 0.00% |
| Base Plane (All) | 1 | 8 | 12.50% |
| Base Plane (Set 1) | 1 | 2 | 50.00% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 8 (8 Entries) |
| **Intersection Mode** | Grid Data Planes |
| **Intersections Count** | 28 |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 3, Direct Toppling Analysis.



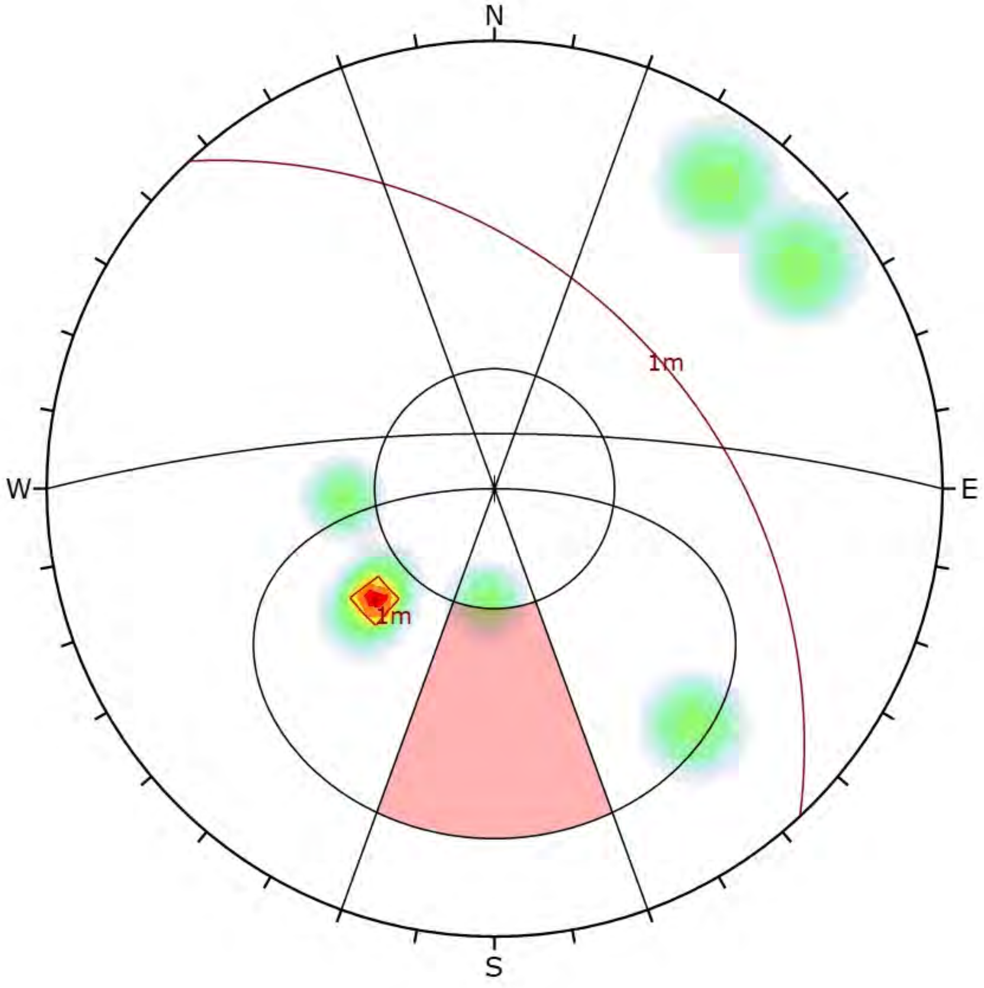
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.70 |
| ⬜ | 2.70 - 5.40 |
| ⬜ | 5.40 - 8.10 |
| 🟦 | 8.10 - 10.80 |
| 🟩 | 10.80 - 13.50 |
| 🟩 | 13.50 - 16.20 |
| 🟩 | 16.20 - 18.90 |
| 🟨 | 18.90 - 21.60 |
| 🟧 | 21.60 - 24.30 |
| 🟥 | 24.30 - 27.00 |

| | |
|---|---|
| **Maximum Density** | 26.31% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| | | Mean Set Planes | | |
| 1m | 🟥 | 40 | 47 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 7 (7 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 4, Dip and Dip Direction of the Mean Set Plane.



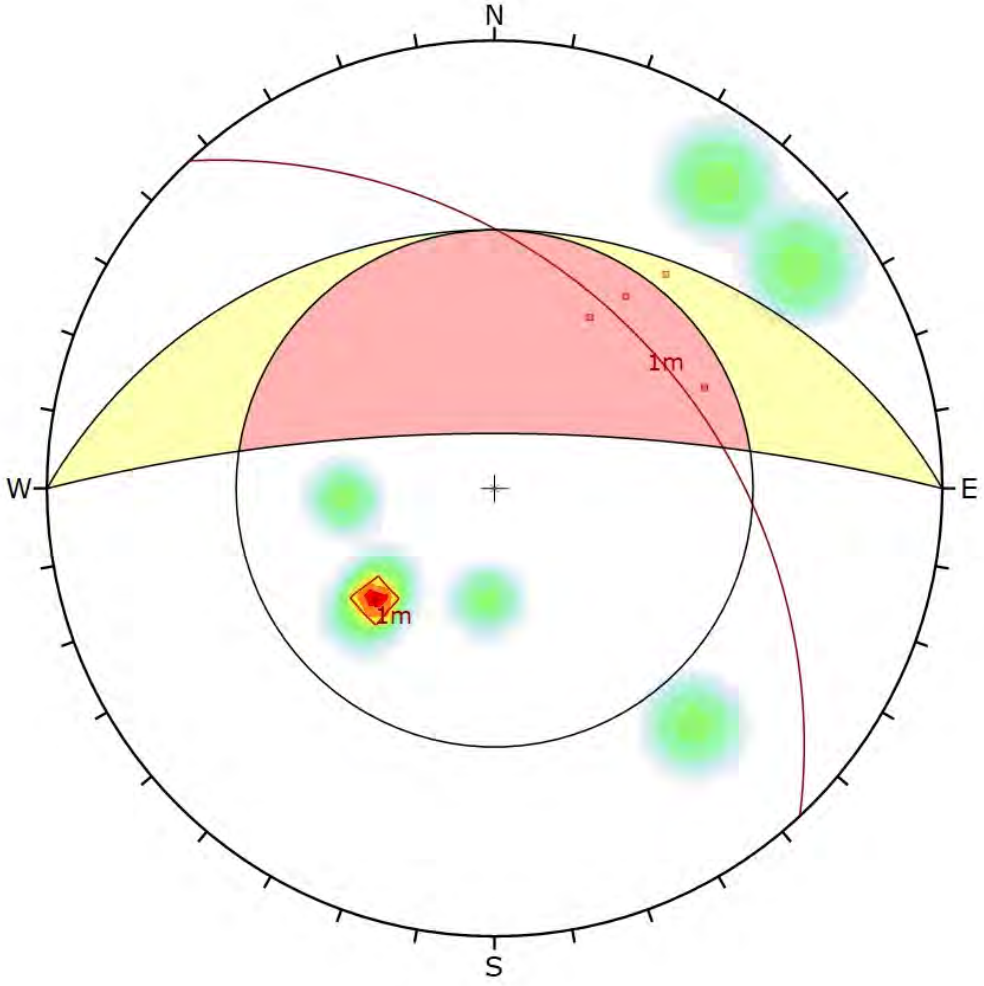
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.70 |
| ⬜ | 2.70 - 5.40 |
| ⬜ | 5.40 - 8.10 |
| ⬜ | 8.10 - 10.80 |
| 🟩 | 10.80 - 13.50 |
| 🟩 | 13.50 - 16.20 |
| 🟩 | 16.20 - 18.90 |
| 🟨 | 18.90 - 21.60 |
| 🟧 | 21.60 - 24.30 |
| 🟥 | 24.30 - 27.00 |
| **Maximum Density** | 26.31% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | | | |
|---|---|---|---|
| **Kinematic Analysis** | Planar Sliding | | |
| **Slope Dip** | 76 | | |
| **Slope Dip Direction** | 0 | | |
| **Friction Angle** | 30° | | |
| **Lateral Limits** | 20° | | |
| | **Critical** | **Total** | **%** |
| Planar Sliding (All) | 0 | 7 | 0.00% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| **Mean Set Planes** | | | | |
| 1m | 🟥 | 40 | 47 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 7 (7 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 4, Planar Failure Analysis.



| Symbol | Feature |
|---|---|
| □ | Critical Intersection |

| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.70 |
| ⬜ | 2.70 - 5.40 |
| 🟦 | 5.40 - 8.10 |
| 🟦 | 8.10 - 10.80 |
| 🟩 | 10.80 - 13.50 |
| 🟩 | 13.50 - 16.20 |
| 🟩 | 16.20 - 18.90 |
| 🟨 | 18.90 - 21.60 |
| 🟧 | 21.60 - 24.30 |
| 🟥 | 24.30 - 27.00 |

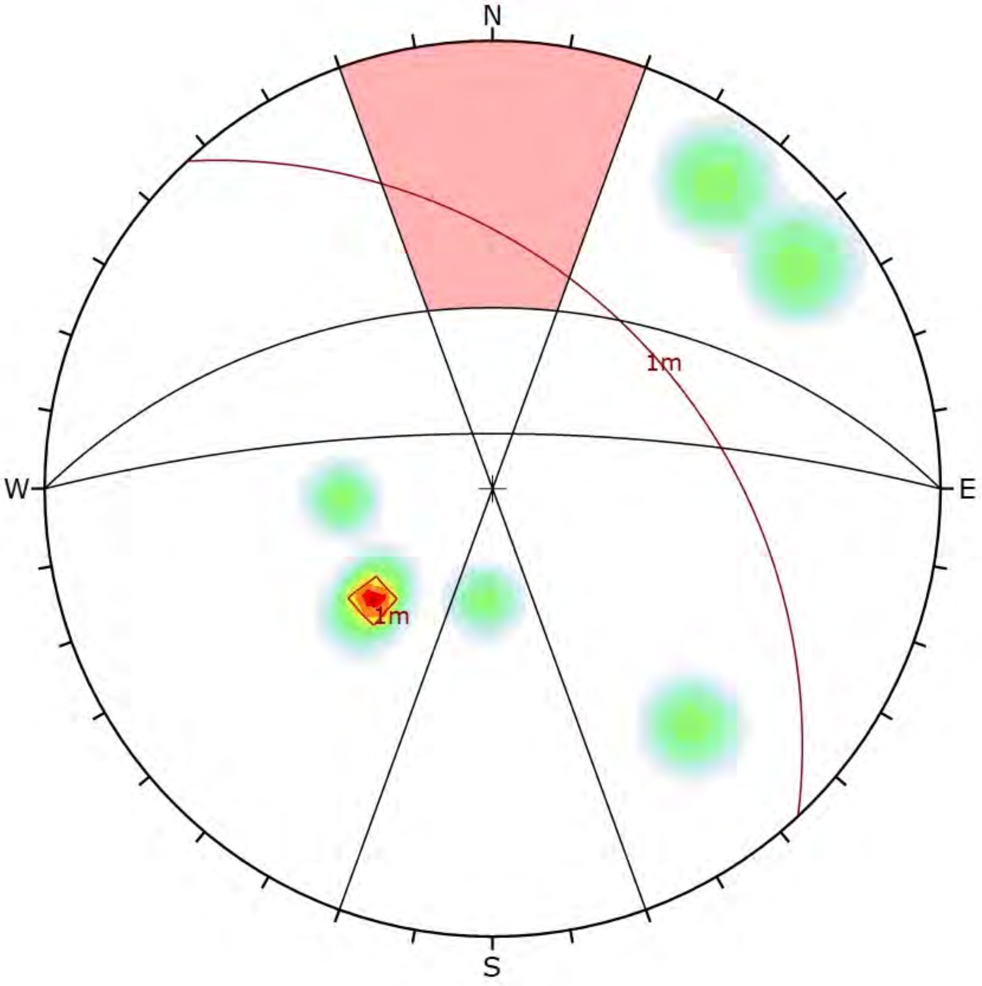
| | |
|---|---|
| Maximum Density | 26.31% |
| Contour Data | Pole Vectors |
| Contour Distribution | Fisher |
| Counting Circle Size | 1.0% |

| | | | |
|---|---|---|---|
| Kinematic Analysis | Wedge Sliding | | |
| Slope Dip | 76 | | |
| Slope Dip Direction | 0 | | |
| Friction Angle | 30° | | |
| | Critical | Total | % |
| Wedge Sliding | 4 | 21 | 19.05% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| Mean Set Planes | | | | |
| 1m | 🟥 | 40 | 47 | |

| | |
|---|---|
| Plot Mode | Pole Vectors |
| Vector Count | 7 (7 Entries) |
| Intersection Mode | Grid Data Planes |
| Intersections Count | 21 |
| Hemisphere | Lower |
| Projection | Equal Angle |

Area 4, Wedge Failure Analysis.



| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.70 |
| ⬜ | 2.70 - 5.40 |
| ⬜ | 5.40 - 8.10 |
| 🟦 | 8.10 - 10.80 |
| 🟩 | 10.80 - 13.50 |
| 🟩 | 13.50 - 16.20 |
| 🟩 | 16.20 - 18.90 |
| 🟨 | 18.90 - 21.60 |
| 🟧 | 21.60 - 24.30 |
| 🟥 | 24.30 - 27.00 |
| **Maximum Density** | 26.31% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

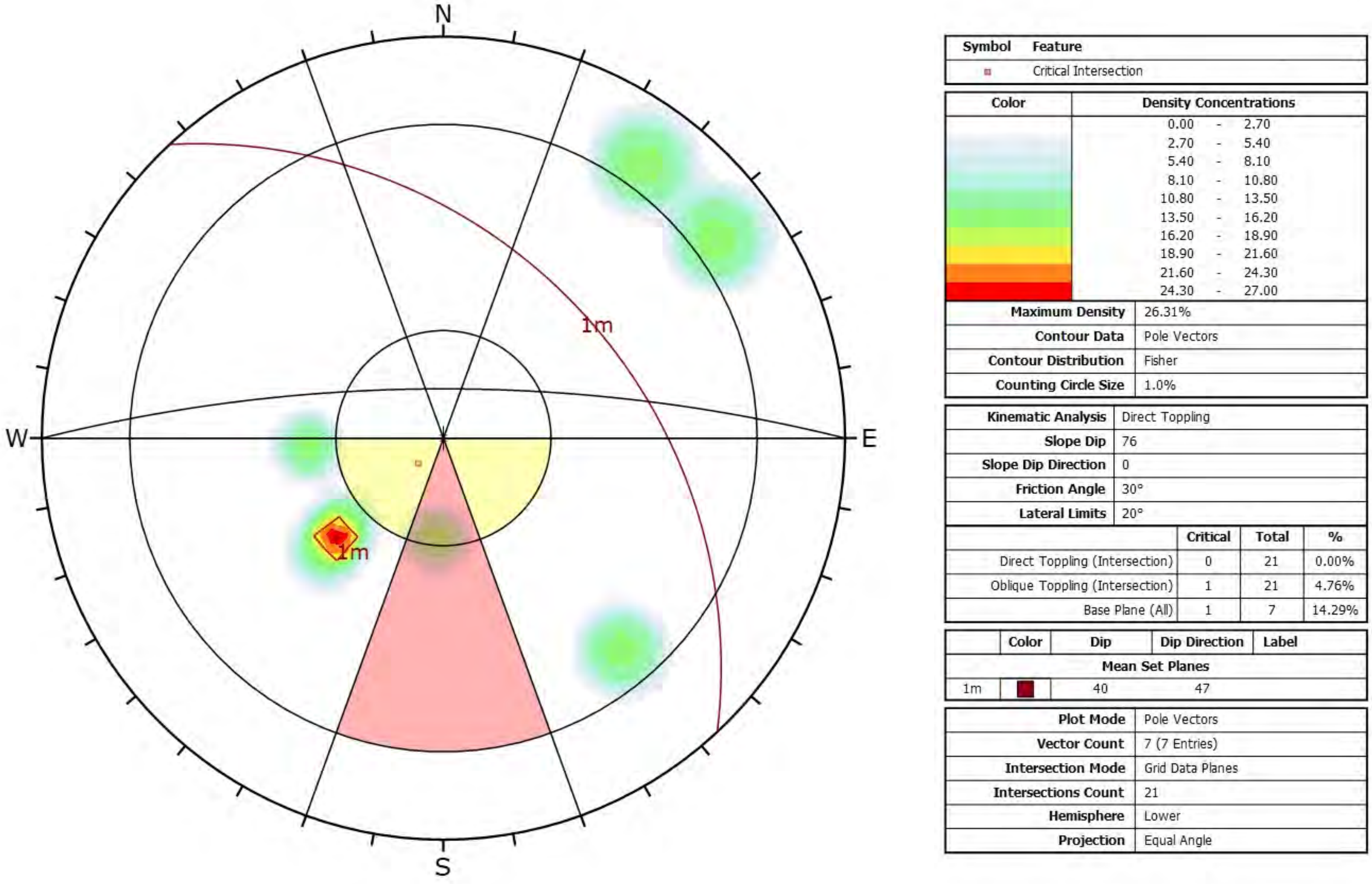
| | |
|---|---|
| **Kinematic Analysis** | Flexural Toppling |
| **Slope Dip** | 76 |
| **Slope Dip Direction** | 0 |
| **Friction Angle** | 30° |
| **Lateral Limits** | 20° |

| | Critical | Total | % |
|---|---|---|---|
| Flexural Toppling (All) | 0 | 7 | 0.00% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| | | Mean Set Planes | | |
| 1m | 🟥 | 40 | 47 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 7 (7 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 4, Flexural Toppling Analysis.



| Symbol | Feature |
|---|---|
| ■ | Critical Intersection |

| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.70 |
| ⬜ | 2.70 - 5.40 |
| ⬜ | 5.40 - 8.10 |
| 🟦 | 8.10 - 10.80 |
| 🟩 | 10.80 - 13.50 |
| 🟩 | 13.50 - 16.20 |
| 🟩 | 16.20 - 18.90 |
| 🟨 | 18.90 - 21.60 |
| 🟧 | 21.60 - 24.30 |
| 🟥 | 24.30 - 27.00 |

| | |
|---|---|
| Maximum Density | 26.31% |
| Contour Data | Pole Vectors |
| Contour Distribution | Fisher |
| Counting Circle Size | 1.0% |

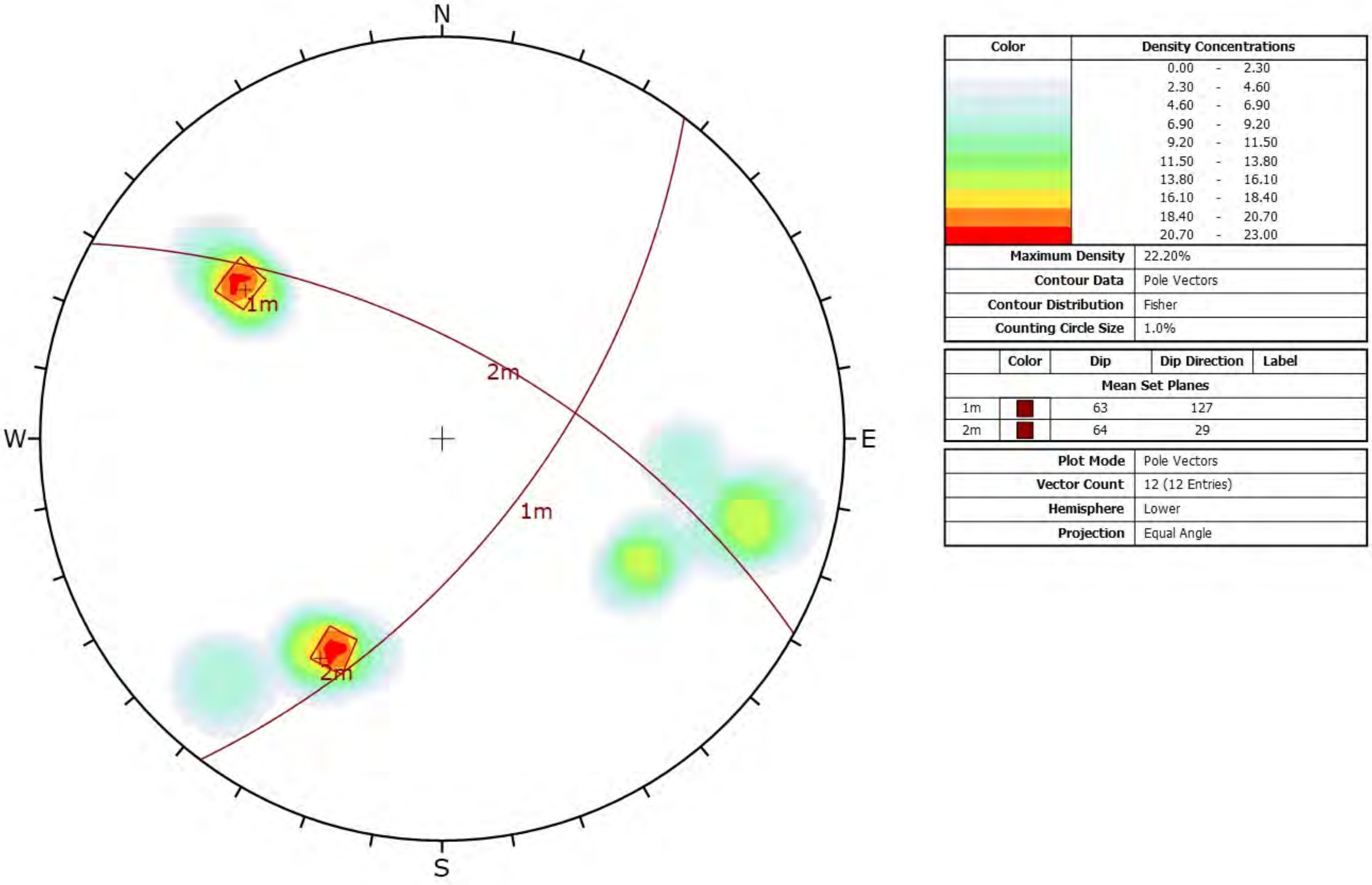
| | |
|---|---|
| Kinematic Analysis | Direct Toppling |
| Slope Dip | 76 |
| Slope Dip Direction | 0 |
| Friction Angle | 30° |
| Lateral Limits | 20° |

| | Critical | Total | % |
|---|---|---|---|
| Direct Toppling (Intersection) | 0 | 21 | 0.00% |
| Oblique Toppling (Intersection) | 1 | 21 | 4.76% |
| Base Plane (All) | 1 | 7 | 14.29% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| Mean Set Planes | | | | |
| 1m | 🟥 | 40 | 47 | |

| | |
|---|---|
| Plot Mode | Pole Vectors |
| Vector Count | 7 (7 Entries) |
| Intersection Mode | Grid Data Planes |
| Intersections Count | 21 |
| Hemisphere | Lower |
| Projection | Equal Angle |

Area 4, Direct Toppling Analysis.



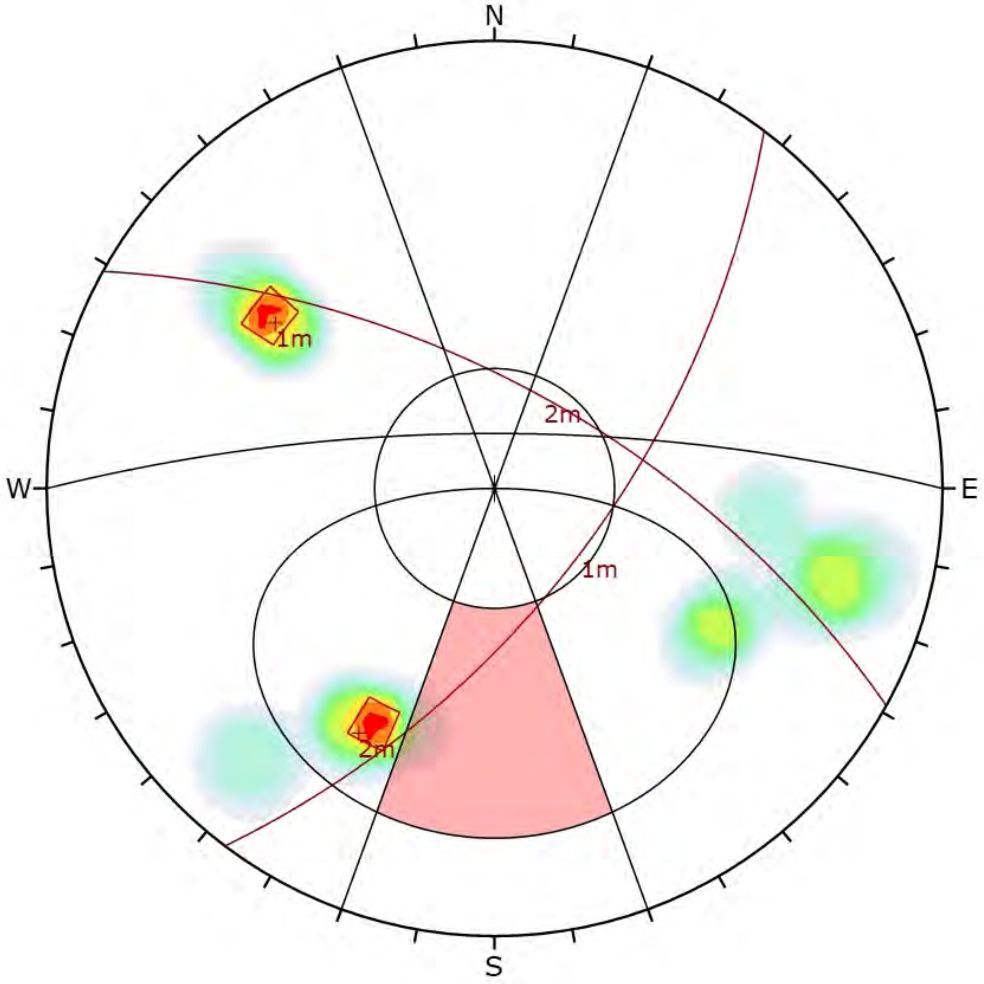
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.30 |
| ⬜ | 2.30 - 4.60 |
| 🟦 | 4.60 - 6.90 |
| 🟦 | 6.90 - 9.20 |
| 🟩 | 9.20 - 11.50 |
| 🟩 | 11.50 - 13.80 |
| 🟨 | 13.80 - 16.10 |
| 🟨 | 16.10 - 18.40 |
| 🟧 | 18.40 - 20.70 |
| 🟥 | 20.70 - 23.00 |

| | |
|---|---|
| **Maximum Density** | 22.20% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| **Mean Set Planes** | | | | |
| 1m | 🟥 | 63 | 127 | |
| 2m | 🟥 | 64 | 29 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 12 (12 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 5, Dip and Dip Direction of the Mean Set Planes.



| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.30 |
| ⬜ | 2.30 - 4.60 |
| ⬜ | 4.60 - 6.90 |
| 🟦 | 6.90 - 9.20 |
| 🟩 | 9.20 - 11.50 |
| 🟩 | 11.50 - 13.80 |
| 🟩 | 13.80 - 16.10 |
| 🟨 | 16.10 - 18.40 |
| 🟧 | 18.40 - 20.70 |
| 🟥 | 20.70 - 23.00 |

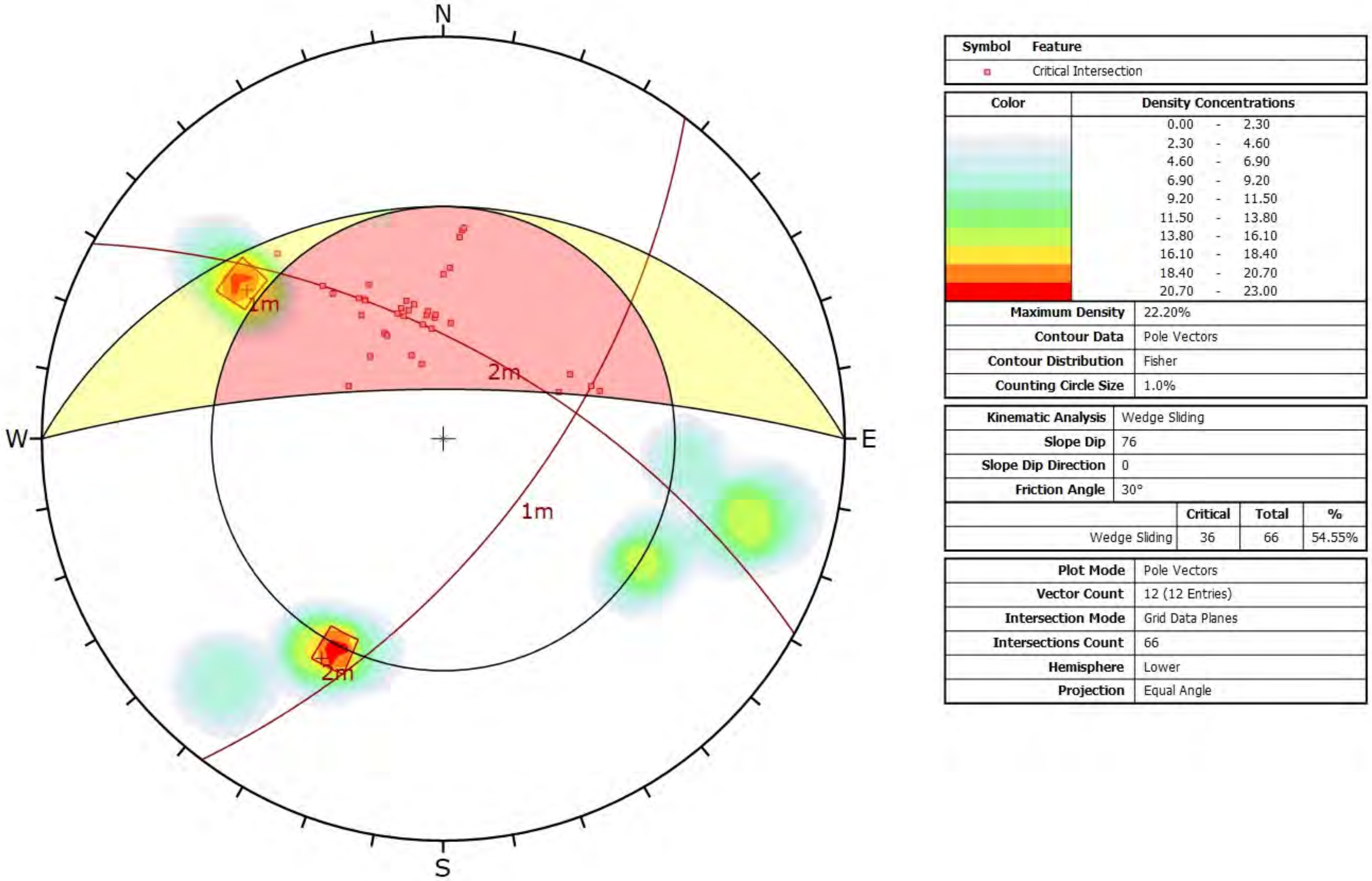
| | |
|---|---|
| **Maximum Density** | 22.20% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | |
|---|---|
| **Kinematic Analysis** | Planar Sliding |
| **Slope Dip** | 76 |
| **Slope Dip Direction** | 0 |
| **Friction Angle** | 30° |
| **Lateral Limits** | 20° |

| | **Critical** | **Total** | **%** |
|---|---|---|---|
| Planar Sliding (All) | 0 | 12 | 0.00% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 12 (12 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 5, Planar Failure Analysis.



| Symbol | Feature |
|---|---|
| ▫ | Critical Intersection |

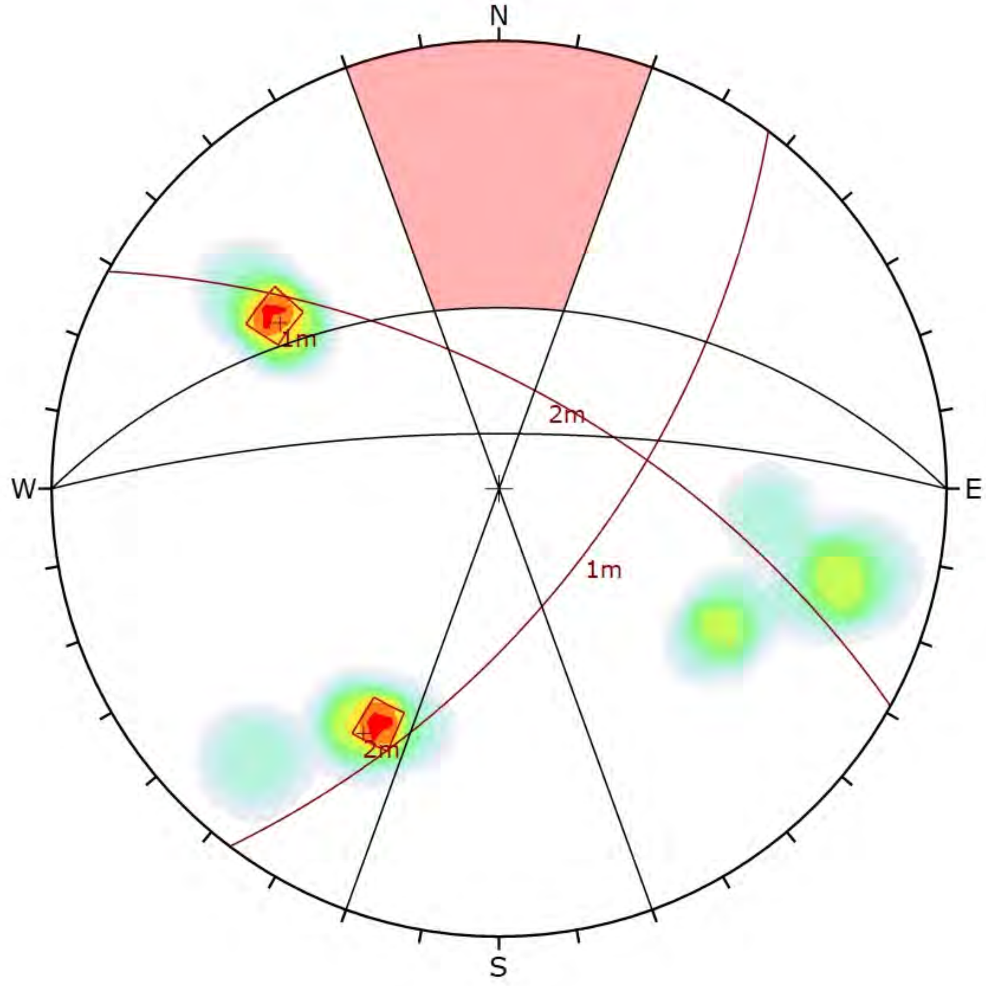
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.30 |
| ⬜ | 2.30 - 4.60 |
| 🟦 | 4.60 - 6.90 |
| 🟦 | 6.90 - 9.20 |
| 🟩 | 9.20 - 11.50 |
| 🟩 | 11.50 - 13.80 |
| 🟩 | 13.80 - 16.10 |
| 🟨 | 16.10 - 18.40 |
| 🟧 | 18.40 - 20.70 |
| 🟥 | 20.70 - 23.00 |

| | |
|---|---|
| **Maximum Density** | 22.20% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | | | |
|---|---|---|---|
| **Kinematic Analysis** | Wedge Sliding | | |
| **Slope Dip** | 76 | | |
| **Slope Dip Direction** | 0 | | |
| **Friction Angle** | 30° | | |
| | **Critical** | **Total** | **%** |
| Wedge Sliding | 36 | 66 | 54.55% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 12 (12 Entries) |
| **Intersection Mode** | Grid Data Planes |
| **Intersections Count** | 66 |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 5, Wedge Failure Analysis.

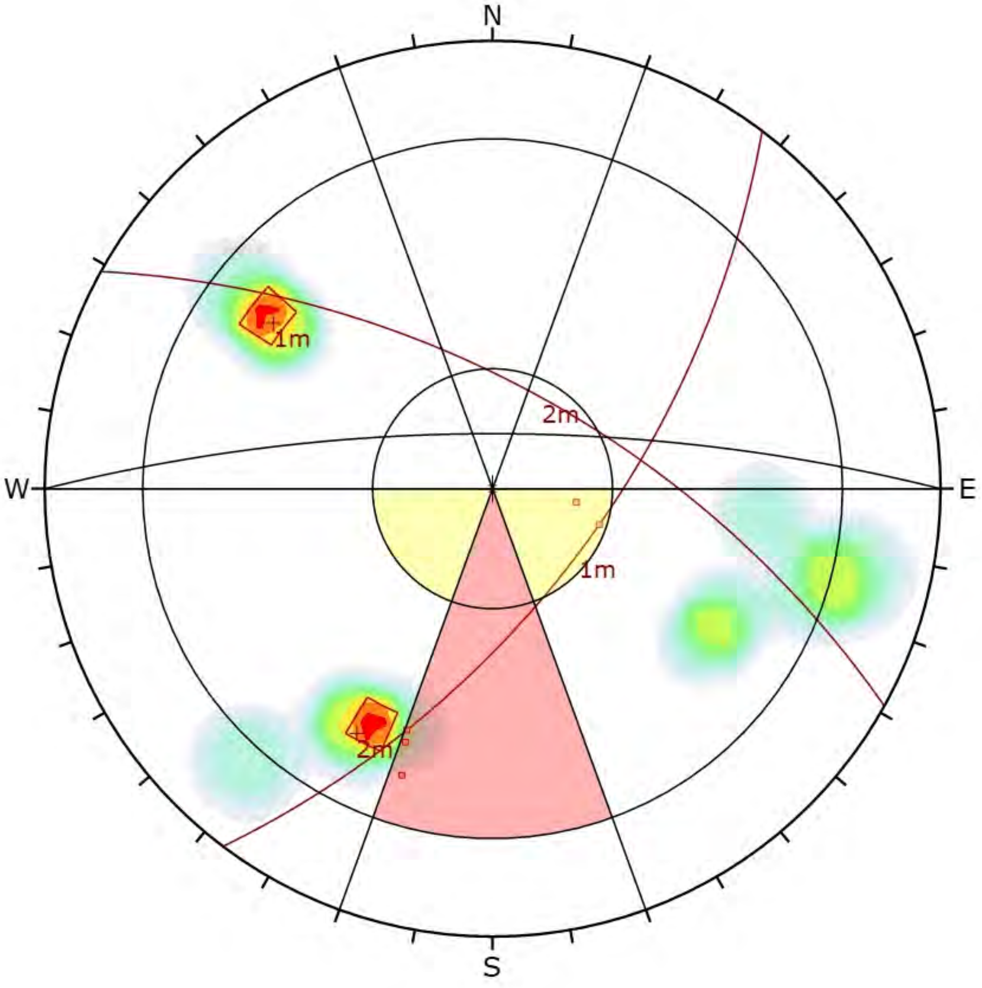


| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.30 |
| ⬜ | 2.30 - 4.60 |
| 🟦 | 4.60 - 6.90 |
| 🟦 | 6.90 - 9.20 |
| 🟩 | 9.20 - 11.50 |
| 🟩 | 11.50 - 13.80 |
| 🟩 | 13.80 - 16.10 |
| 🟨 | 16.10 - 18.40 |
| 🟧 | 18.40 - 20.70 |
| 🟥 | 20.70 - 23.00 |
| **Maximum Density** | 22.20% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | | | |
|---|---|---|---|
| **Kinematic Analysis** | Flexural Toppling | | |
| **Slope Dip** | 76 | | |
| **Slope Dip Direction** | 0 | | |
| **Friction Angle** | 30° | | |
| **Lateral Limits** | 20° | | |
| | **Critical** | **Total** | **%** |
| Flexural Toppling (All) | 0 | 12 | 0.00% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 12 (12 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 5, Flexural Toppling Analysis.



| Symbol | Feature |
|---|---|
| ■ | Critical Intersection |

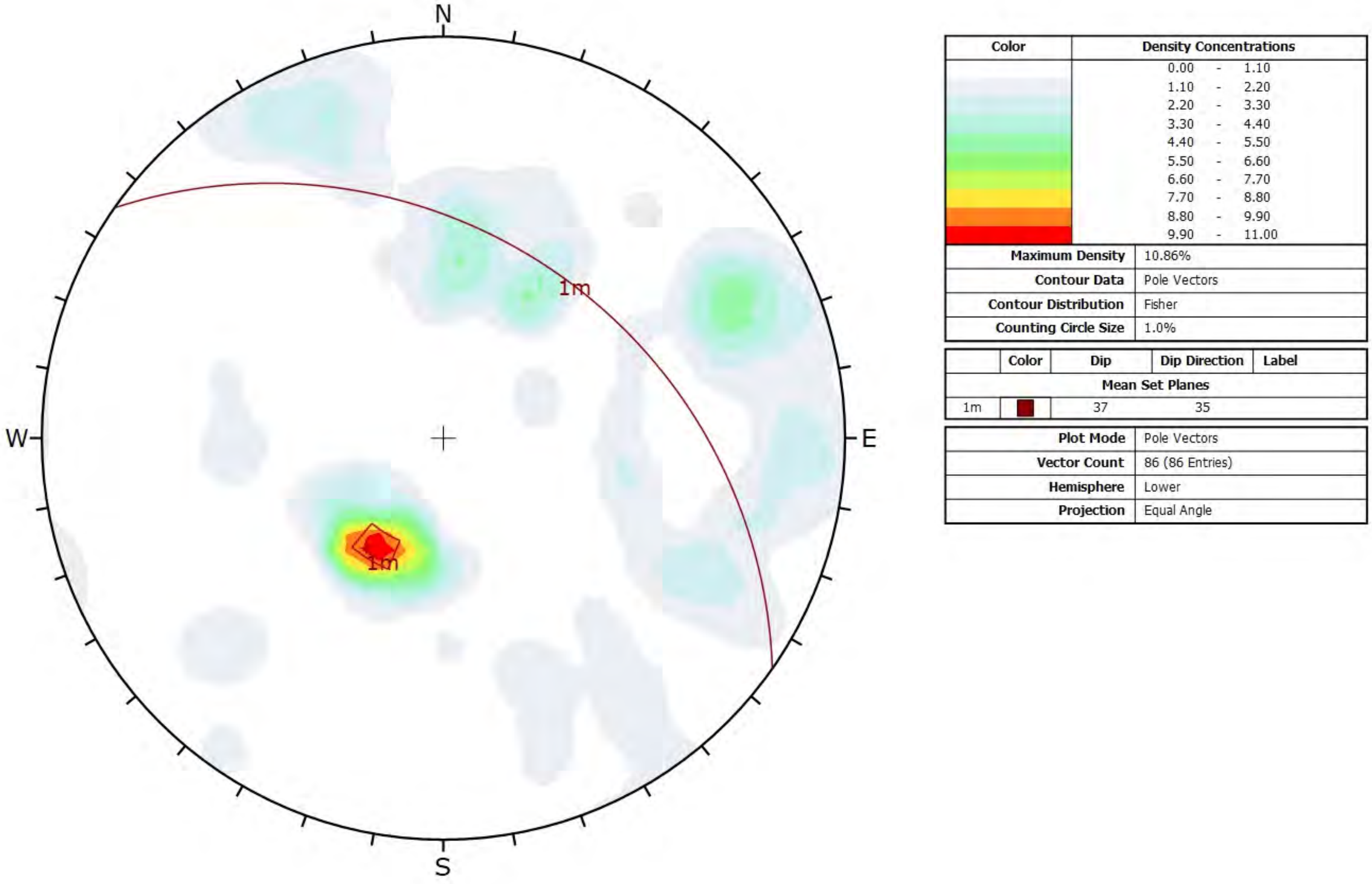
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 2.30 |
| ⬜ | 2.30 - 4.60 |
| 🟦 | 4.60 - 6.90 |
| 🟦 | 6.90 - 9.20 |
| 🟩 | 9.20 - 11.50 |
| 🟩 | 11.50 - 13.80 |
| 🟩 | 13.80 - 16.10 |
| 🟨 | 16.10 - 18.40 |
| 🟧 | 18.40 - 20.70 |
| 🟥 | 20.70 - 23.00 |
| **Maximum Density** | 22.20% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | |
|---|---|
| **Kinematic Analysis** | Direct Toppling |
| **Slope Dip** | 76 |
| **Slope Dip Direction** | 0 |
| **Friction Angle** | 30° |
| **Lateral Limits** | 20° |

| | Critical | Total | % |
|---|---|---|---|
| Direct Toppling (Intersection) | 3 | 66 | 4.55% |
| Oblique Toppling (Intersection) | 2 | 66 | 3.03% |
| Base Plane (All) | 0 | 12 | 0.00% |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 12 (12 Entries) |
| **Intersection Mode** | Grid Data Planes |
| **Intersections Count** | 66 |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 5, Direct Toppling Analysis.



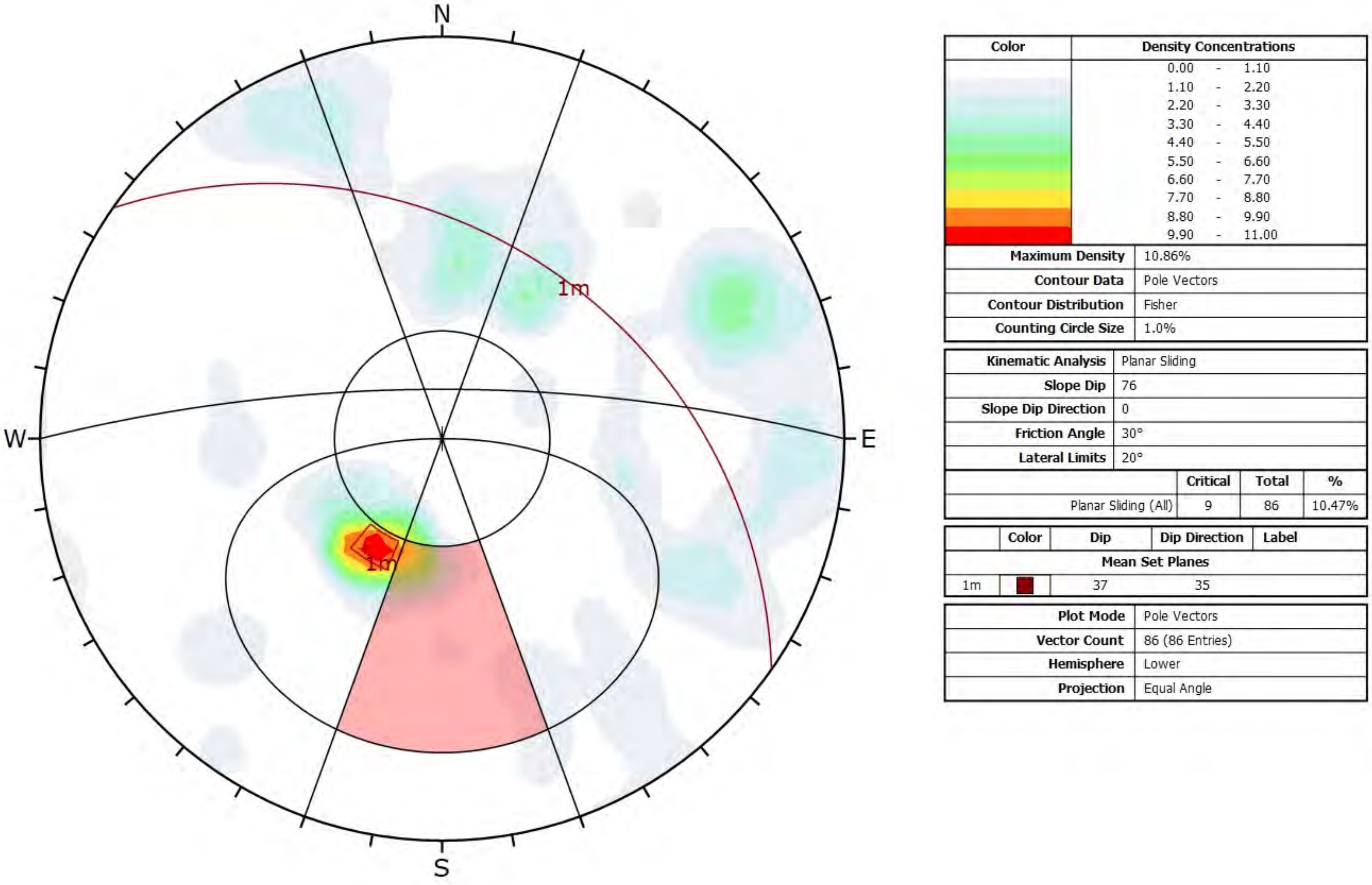
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 1.10 |
| ⬜ | 1.10 - 2.20 |
| ⬜ | 2.20 - 3.30 |
| 🟩 | 3.30 - 4.40 |
| 🟩 | 4.40 - 5.50 |
| 🟩 | 5.50 - 6.60 |
| 🟨 | 6.60 - 7.70 |
| 🟨 | 7.70 - 8.80 |
| 🟧 | 8.80 - 9.90 |
| 🟥 | 9.90 - 11.00 |

| | |
|---|---|
| **Maximum Density** | 10.86% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| **Mean Set Planes** | | | | |
| 1m | 🟥 | 37 | 35 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 86 (86 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 6, Dip and Dip Direction of the Mean Set Plane.



| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 1.10 |
| ⬜ | 1.10 - 2.20 |
| ⬜ | 2.20 - 3.30 |
| 🟦 | 3.30 - 4.40 |
| 🟩 | 4.40 - 5.50 |
| 🟩 | 5.50 - 6.60 |
| 🟩 | 6.60 - 7.70 |
| 🟨 | 7.70 - 8.80 |
| 🟧 | 8.80 - 9.90 |
| 🟥 | 9.90 - 11.00 |
| **Maximum Density** | 10.86% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

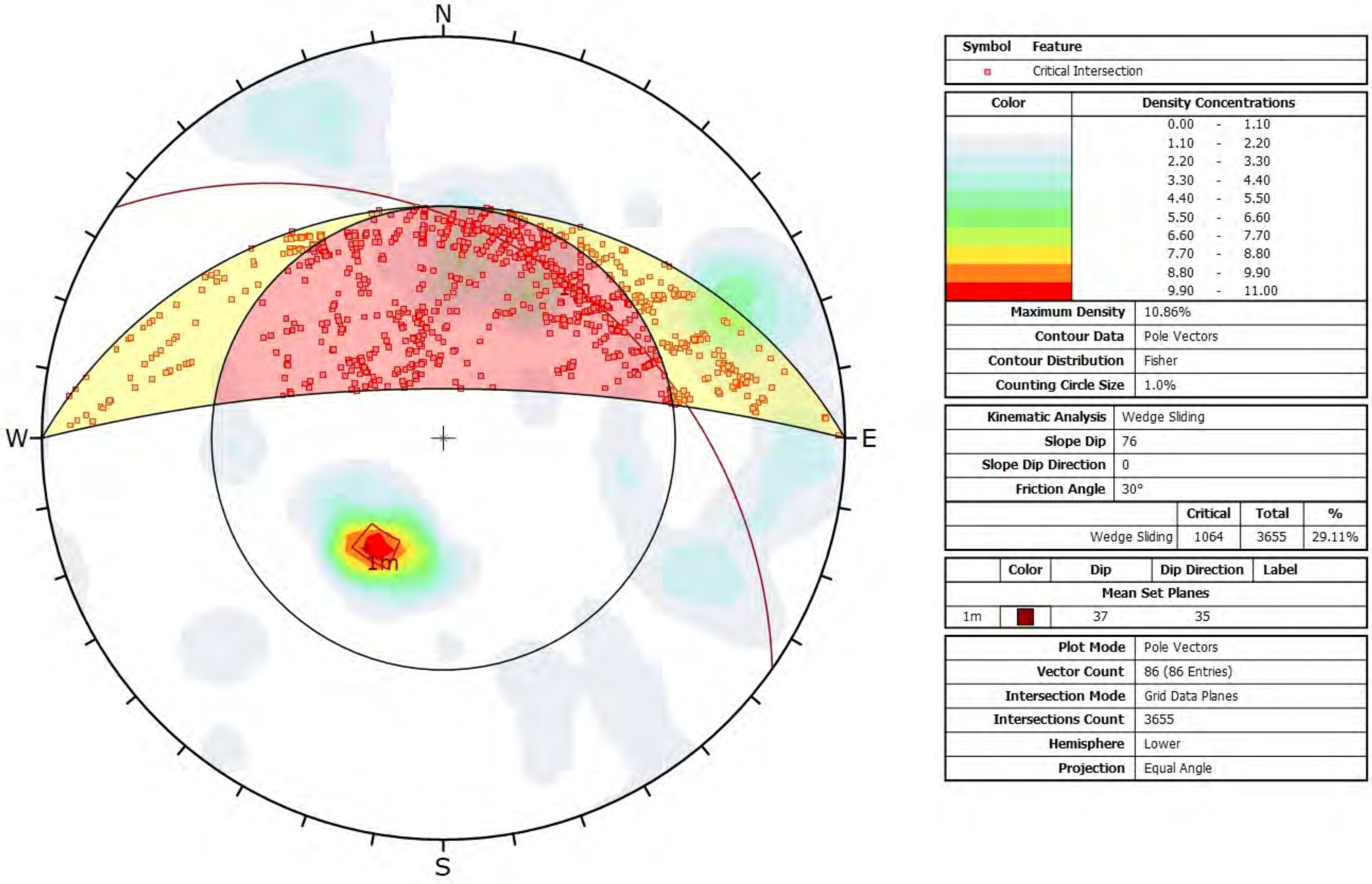
| | |
|---|---|
| **Kinematic Analysis** | Planar Sliding |
| **Slope Dip** | 76 |
| **Slope Dip Direction** | 0 |
| **Friction Angle** | 30° |
| **Lateral Limits** | 20° |

| | Critical | Total | % |
|---|---|---|---|
| Planar Sliding (All) | 9 | 86 | 10.47% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| **Mean Set Planes** | | | | |
| 1m | 🟥 | 37 | 35 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 86 (86 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 6, Planar Failure Analysis.



| Symbol | Feature |
|---|---|
| ▫ | Critical Intersection |

| Color | Density Concentrations |
|---|---|
| | 0.00 - 1.10 |
| | 1.10 - 2.20 |
| | 2.20 - 3.30 |
| | 3.30 - 4.40 |
| | 4.40 - 5.50 |
| | 5.50 - 6.60 |
| | 6.60 - 7.70 |
| | 7.70 - 8.80 |
| | 8.80 - 9.90 |
| 🟥 | 9.90 - 11.00 |

| | |
|---|---|
| Maximum Density | 10.86% |
| Contour Data | Pole Vectors |
| Contour Distribution | Fisher |
| Counting Circle Size | 1.0% |

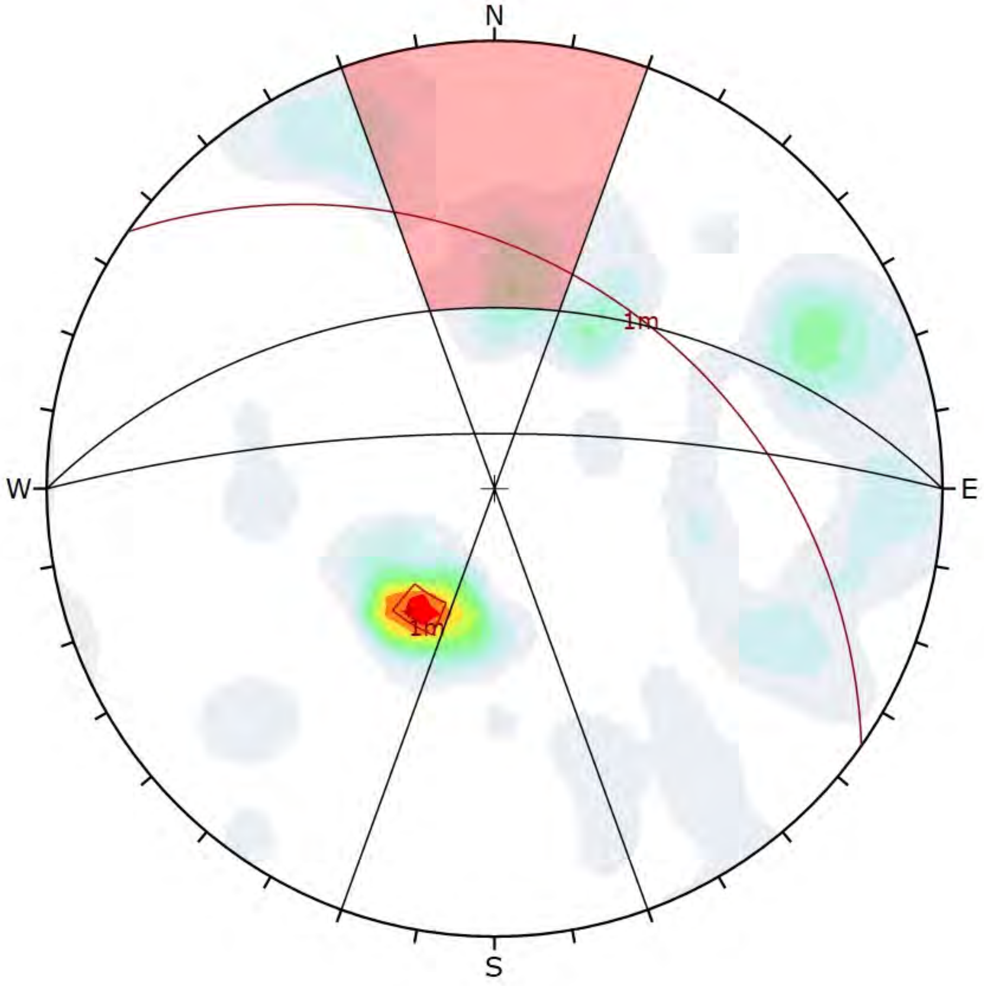
| | |
|---|---|
| Kinematic Analysis | Wedge Sliding |
| Slope Dip | 76 |
| Slope Dip Direction | 0 |
| Friction Angle | 30° |

| | Critical | Total | % |
|---|---|---|---|
| Wedge Sliding | 1064 | 3655 | 29.11% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| Mean Set Planes | | | | |
| 1m | ■ | 37 | 35 | |

| | |
|---|---|
| Plot Mode | Pole Vectors |
| Vector Count | 86 (86 Entries) |
| Intersection Mode | Grid Data Planes |
| Intersections Count | 3655 |
| Hemisphere | Lower |
| Projection | Equal Angle |

Area 6, Wedge Failure Analysis.



| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 1.10 |
| ⬜ | 1.10 - 2.20 |
| ⬜ | 2.20 - 3.30 |
| 🟦 | 3.30 - 4.40 |
| 🟩 | 4.40 - 5.50 |
| 🟩 | 5.50 - 6.60 |
| 🟩 | 6.60 - 7.70 |
| 🟨 | 7.70 - 8.80 |
| 🟧 | 8.80 - 9.90 |
| 🟥 | 9.90 - 11.00 |

| | |
|---|---|
| **Maximum Density** | 10.86% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

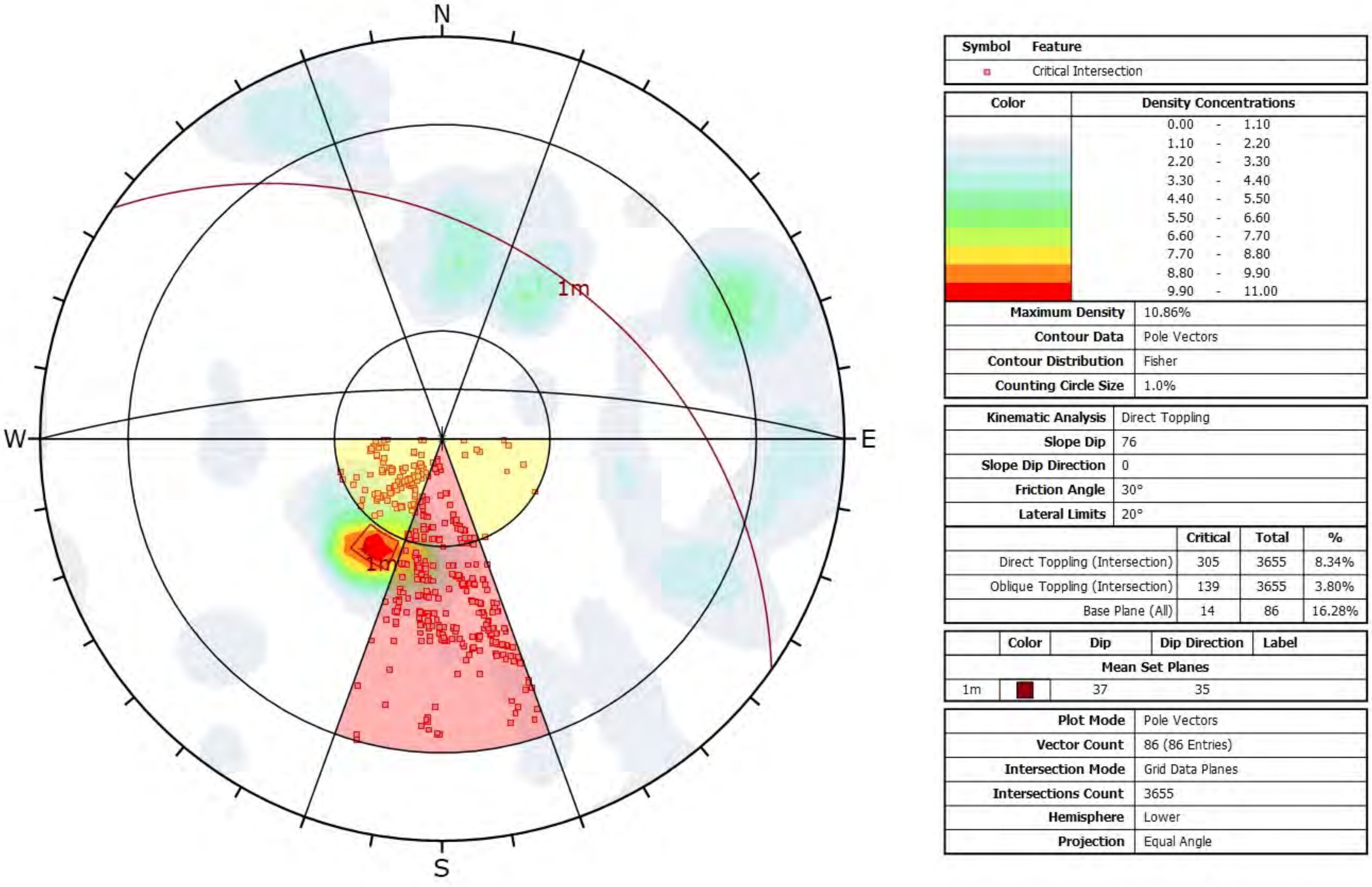
| | |
|---|---|
| **Kinematic Analysis** | Flexural Toppling |
| **Slope Dip** | 76 |
| **Slope Dip Direction** | 0 |
| **Friction Angle** | 30° |
| **Lateral Limits** | 20° |

| | Critical | Total | % |
|---|---|---|---|
| Flexural Toppling (All) | 10 | 86 | 11.63% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| **Mean Set Planes** | | | | |
| 1m | 🟥 | 37 | 35 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 86 (86 Entries) |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Area 6, Flexural Toppling Analysis.



| Symbol | Feature |
|---|---|
| □ | Critical Intersection |

| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 1.10 |
| ⬜ | 1.10 - 2.20 |
| 🟦 | 2.20 - 3.30 |
| 🟦 | 3.30 - 4.40 |
| 🟩 | 4.40 - 5.50 |
| 🟩 | 5.50 - 6.60 |
| 🟩 | 6.60 - 7.70 |
| 🟨 | 7.70 - 8.80 |
| 🟧 | 8.80 - 9.90 |
| 🟥 | 9.90 - 11.00 |

| | |
|---|---|
| Maximum Density | 10.86% |
| Contour Data | Pole Vectors |
| Contour Distribution | Fisher |
| Counting Circle Size | 1.0% |

| | |
|---|---|
| Kinematic Analysis | Direct Toppling |
| Slope Dip | 76 |
| Slope Dip Direction | 0 |
| Friction Angle | 30° |
| Lateral Limits | 20° |

| | Critical | Total | % |
|---|---|---|---|
| Direct Toppling (Intersection) | 305 | 3655 | 8.34% |
| Oblique Toppling (Intersection) | 139 | 3655 | 3.80% |
| Base Plane (All) | 14 | 86 | 16.28% |

| | Color | Dip | Dip Direction | Label |
|---|---|---|---|---|
| Mean Set Planes | | | | |
| 1m | 🟥 | 37 | 35 | |

| | |
|---|---|
| Plot Mode | Pole Vectors |
| Vector Count | 86 (86 Entries) |
| Intersection Mode | Grid Data Planes |
| Intersections Count | 3655 |
| Hemisphere | Lower |
| Projection | Equal Angle |

Area 6, Direct Toppling Analysis.

# Appendix D

## LABORATORY TEST RESULTS

ADVANCED TERRA TESTING

# Unconfined Compressive Strength
## ASTM D7012 Method C

| | | | |
|---|---|---|---|
| CLIENT | Yeh and Associates | JOB NO. | 2546-123 |
| PROJECT | Floyd Hill Structured Lanes | LOCATION | -- |
| PROJECT NO. | 218-300 | | |

| | | | | |
|---|---|---|---|---|
| BORING NO. | YA-FHK-B2 | YA-FHK-B2 | YA-FHK-B3 | YA-FHK-B3 |
| DEPTH | 40.3 | 41.3 | 53.0 | 63.0 |
| SAMPLE NO. | | | | |
| DATE SAMPLED | | | | |
| DATE TESTED | 06/09/20 | 06/09/20 | 06/09/20 | 06/09/20 |
| TECHNICIAN | BFUTCH | BFUTCH | BFUTCH | BFUTCH |
| ROCK TYPE | | | | |
| Diameter (in): | 1.764 | 1.764 | 2.392 | 2.394 |
| Height (in): | 3.877 | 4.028 | 5.176 | 5.371 |
| Mass of Wet Rock (g): | 405.5 | 441.6 | 1039.3 | 1166.5 |
| Wet Density (lbs/ft³): | 163.0 | 170.9 | 170.2 | 183.8 |
| Wet Density (g/cm³): | 2.61 | 2.74 | 2.73 | 2.94 |
| Peak Load (lbs): | 44822 | 41763 | 64226 | 48734 |
| Compressive Strength (psi) | 18340 | 17089 | 14292 | 10827 |
| Compressive Strength (MPa) | 126 | 118 | 99 | 75 |
| Failure Type: | Fracture / Bedding | Fracture / Bedding | Fracture / Bedding | Fracture / Bedding |

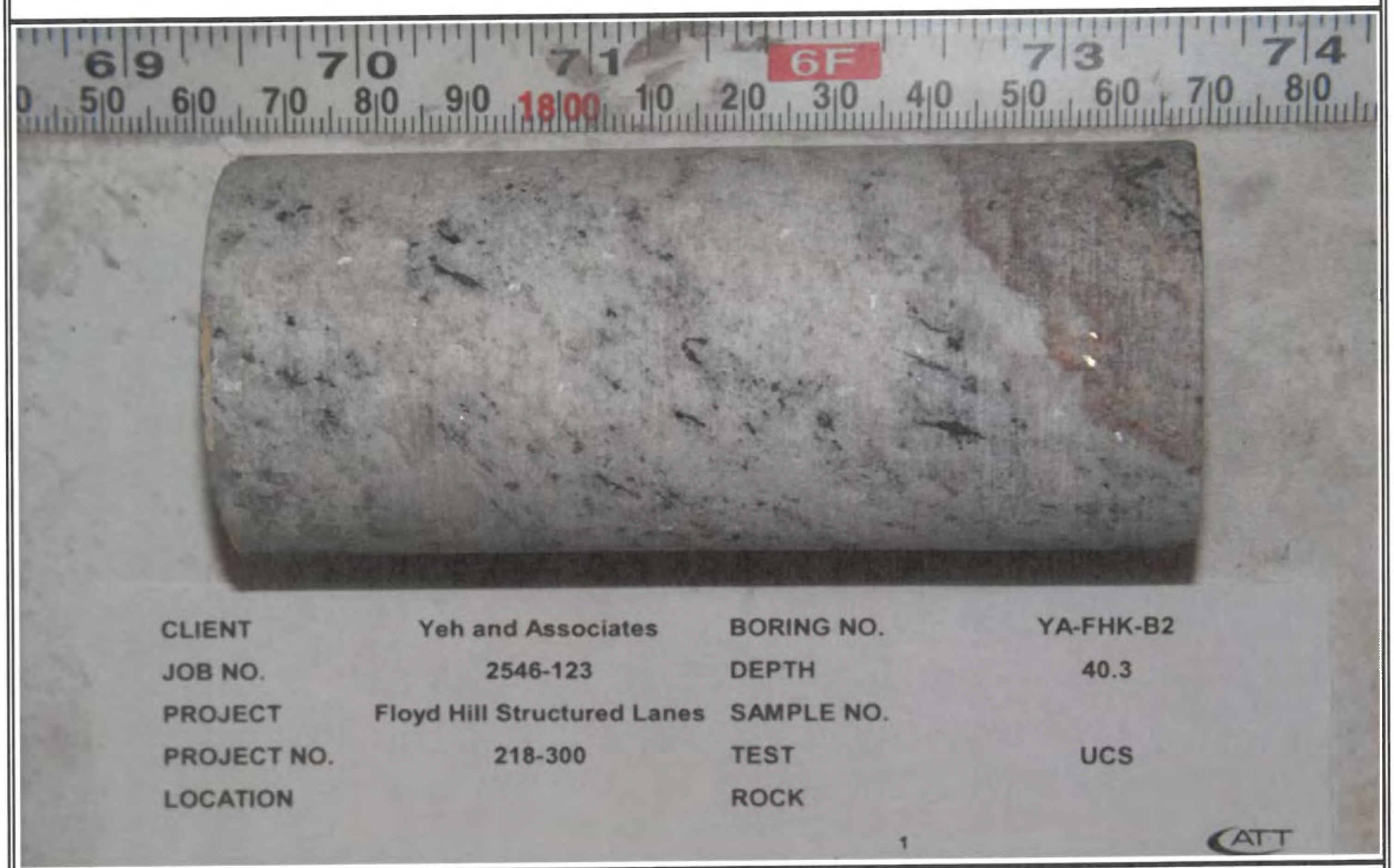
| | |
|---|---|
| BORING NO. | |
| DEPTH | |
| SAMPLE NO. | |
| DATE SAMPLED | |
| DATE TESTED | |
| TECHNICIAN | |
| ROCK TYPE | |
| Diameter (in): | |
| Height (in): | |
| Mass of Wet Rock (g): | |
| Wet Density (lbs/ft³): | |
| Wet Density (g/cm³): | |
| Peak Load (lbs): | |
| Compressive Strength (psi) | |
| Compressive Strength (MPa) | |
| Failure Type: | |
| NOTES | |

Data entry by: BFUTCH Date: 6/9/2020
Checked by: HN Date: 6/10/2020
File name: 2546123_Rock UCS-TCS ASTM D7012 Method A and C_0.xlsm

ADVANCED TERRA TESTING

# Image Attachment

| | | | |
|---|---|---|---|
| CLIENT | Yeh and Associates | BORING NO. | YA-FHK-B2 |
| JOB NO. | 2546-123 | DEPTH | 40.3 |
| PROJECT | Floyd Hill Structured Lanes | SAMPLE NO. | |
| PROJECT NO. | 218-300 | TEST TYPE | UCS |
| LOCATION | -- | ROCK TYPE | |



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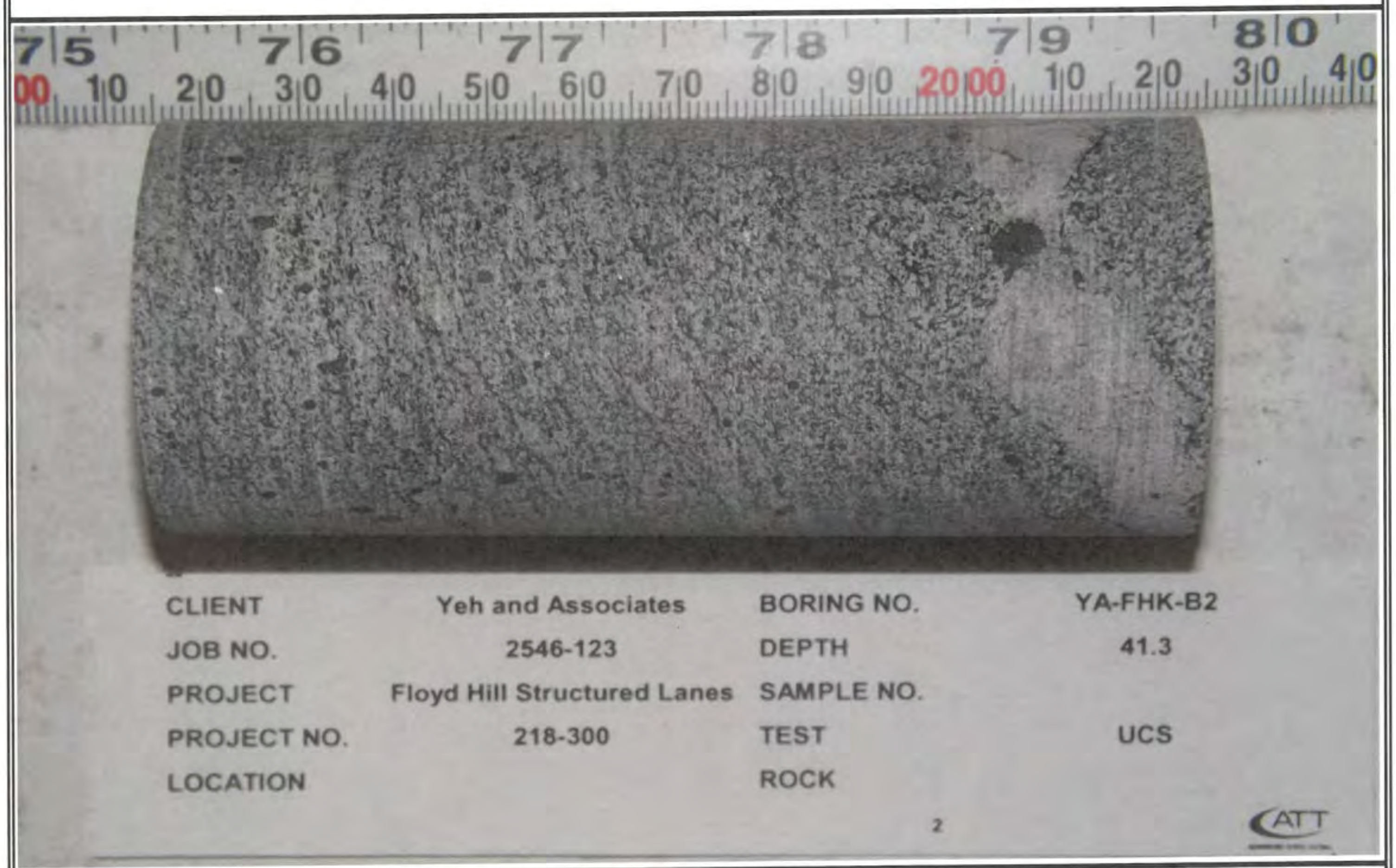

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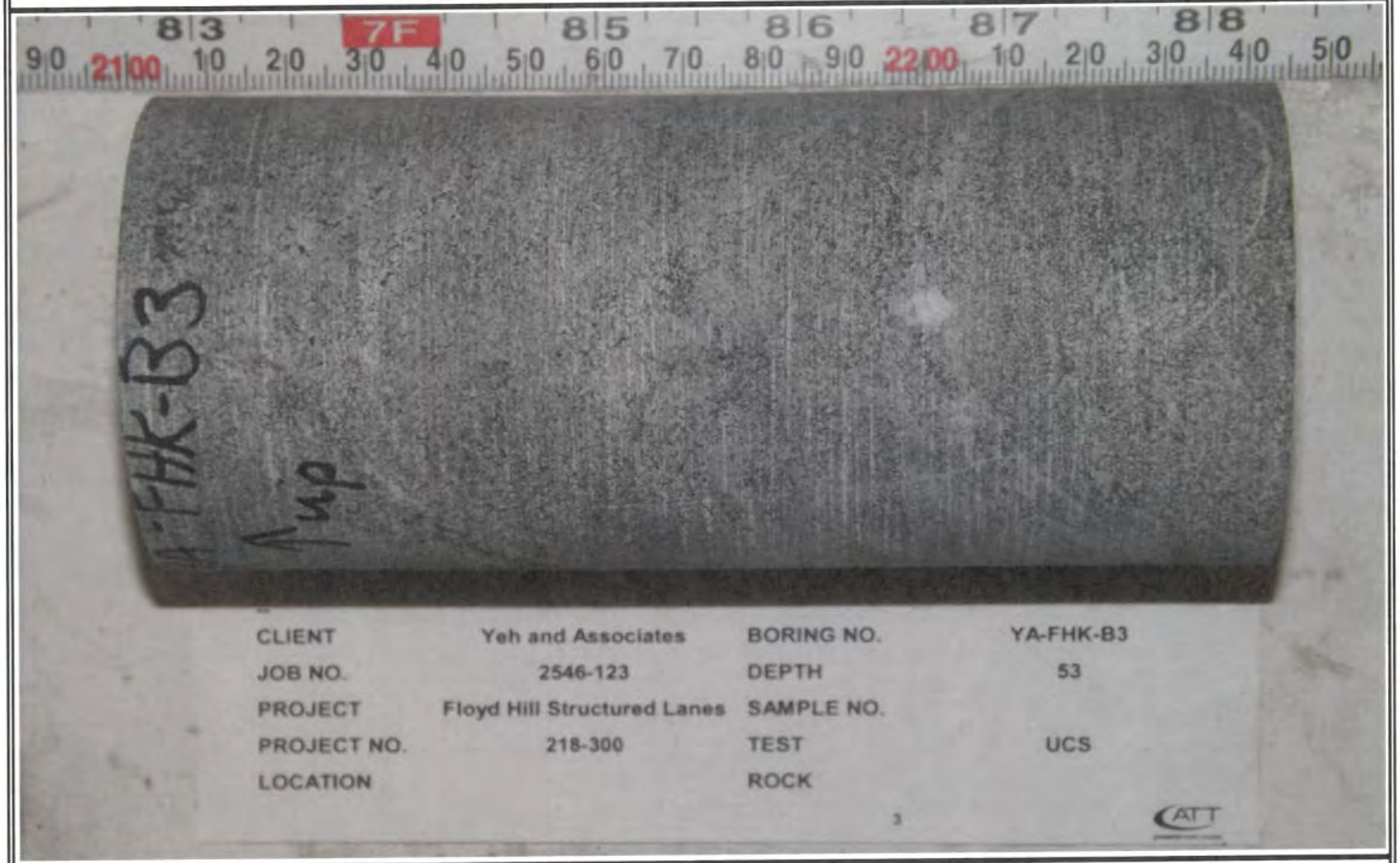

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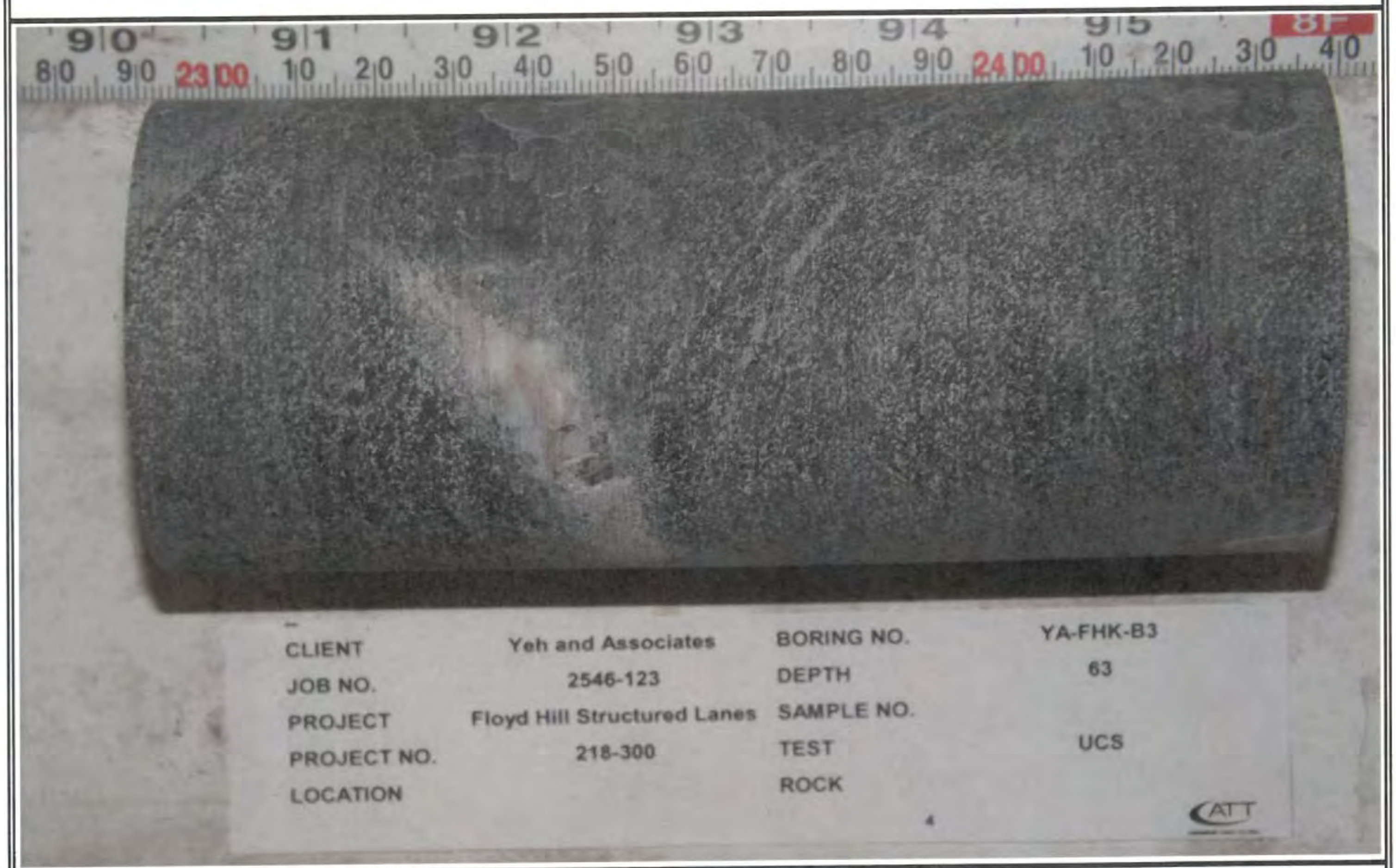

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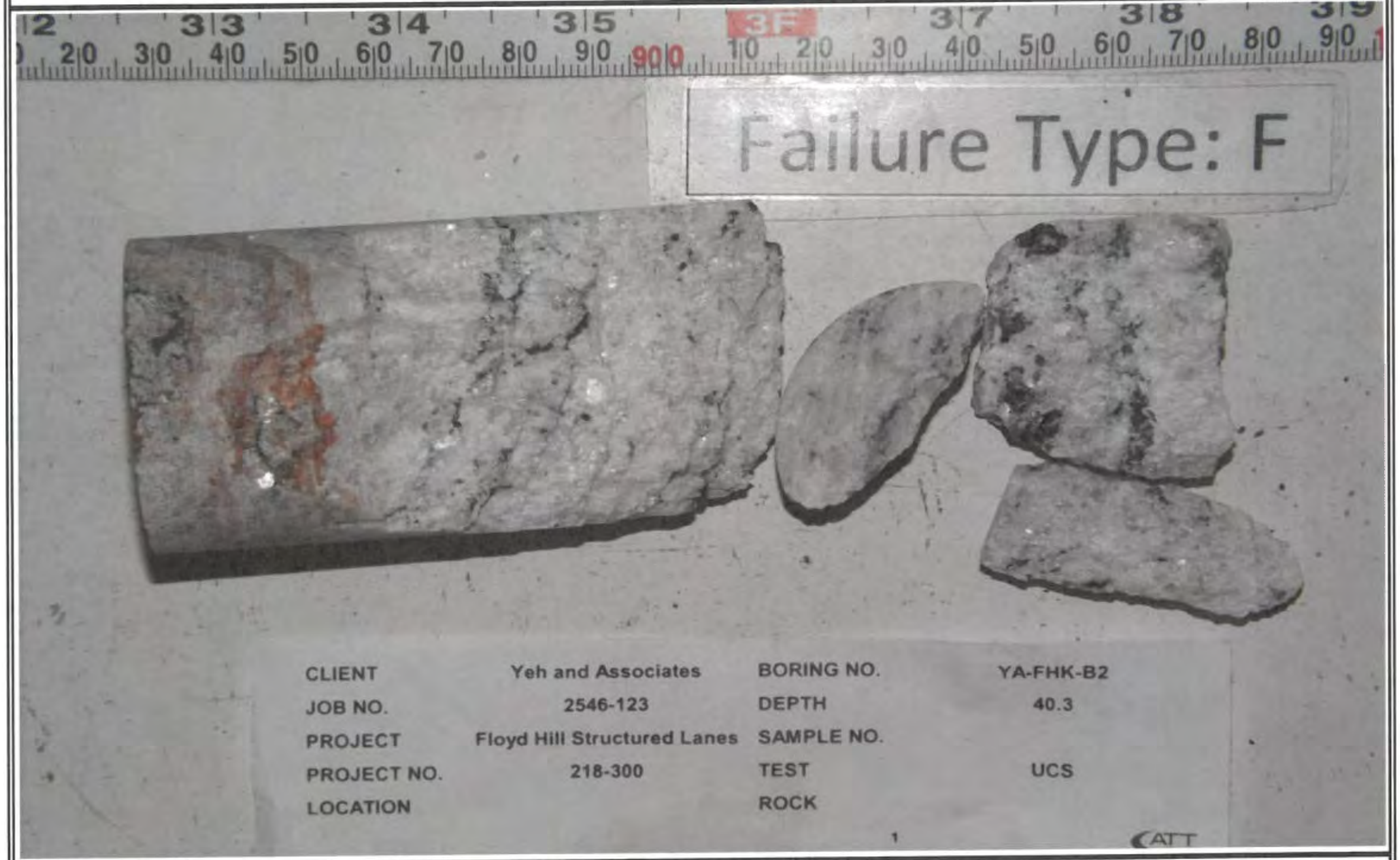

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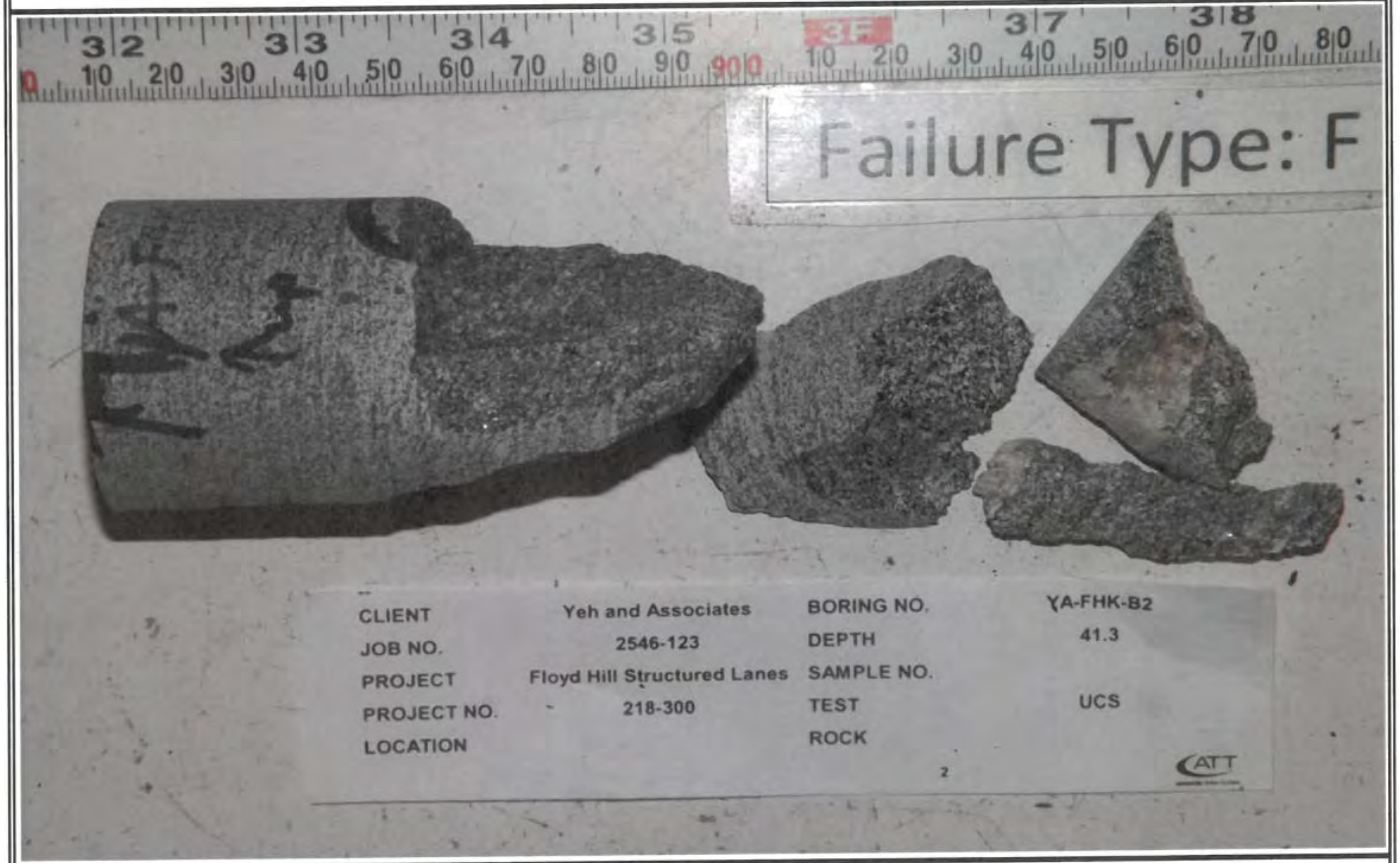

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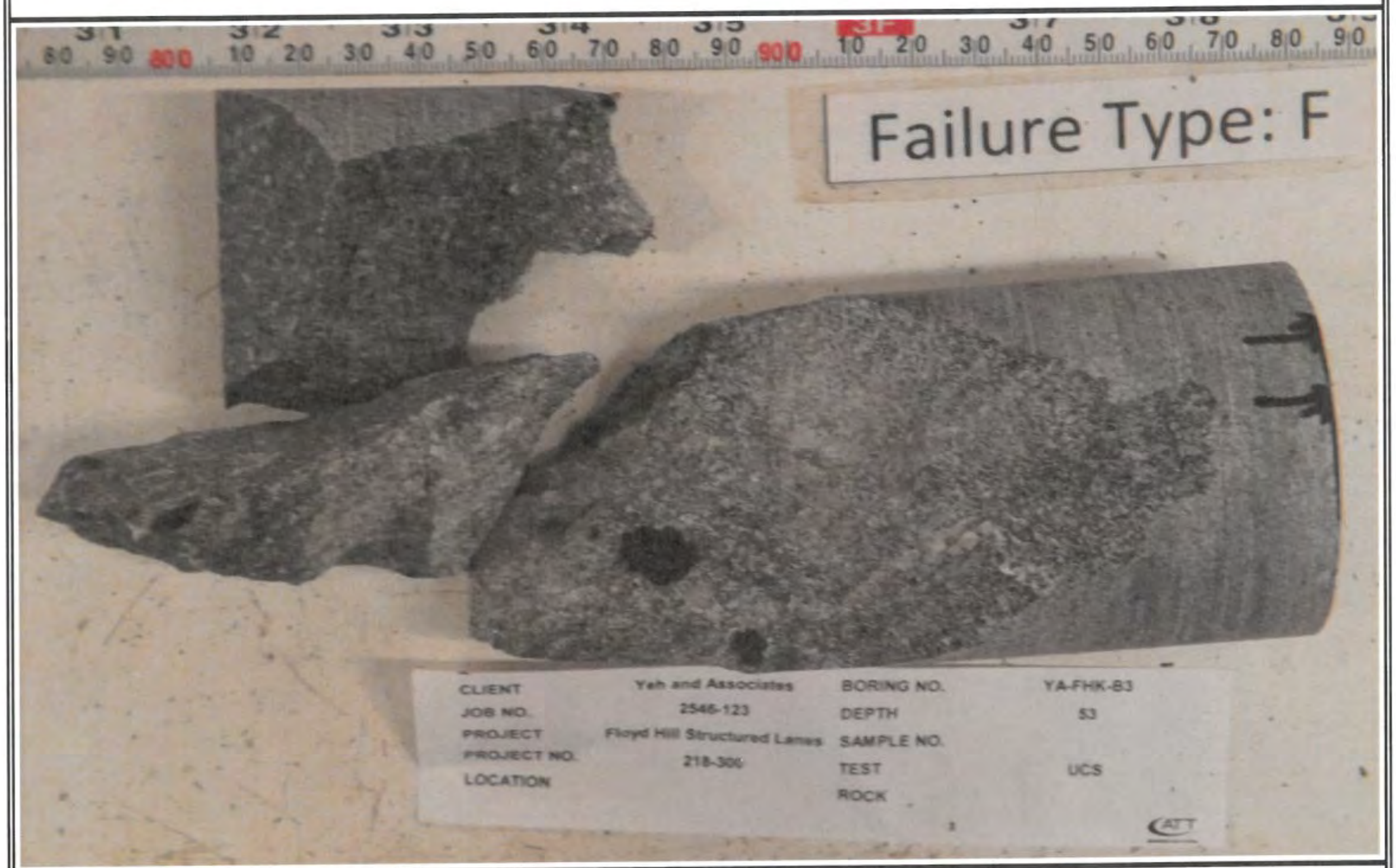

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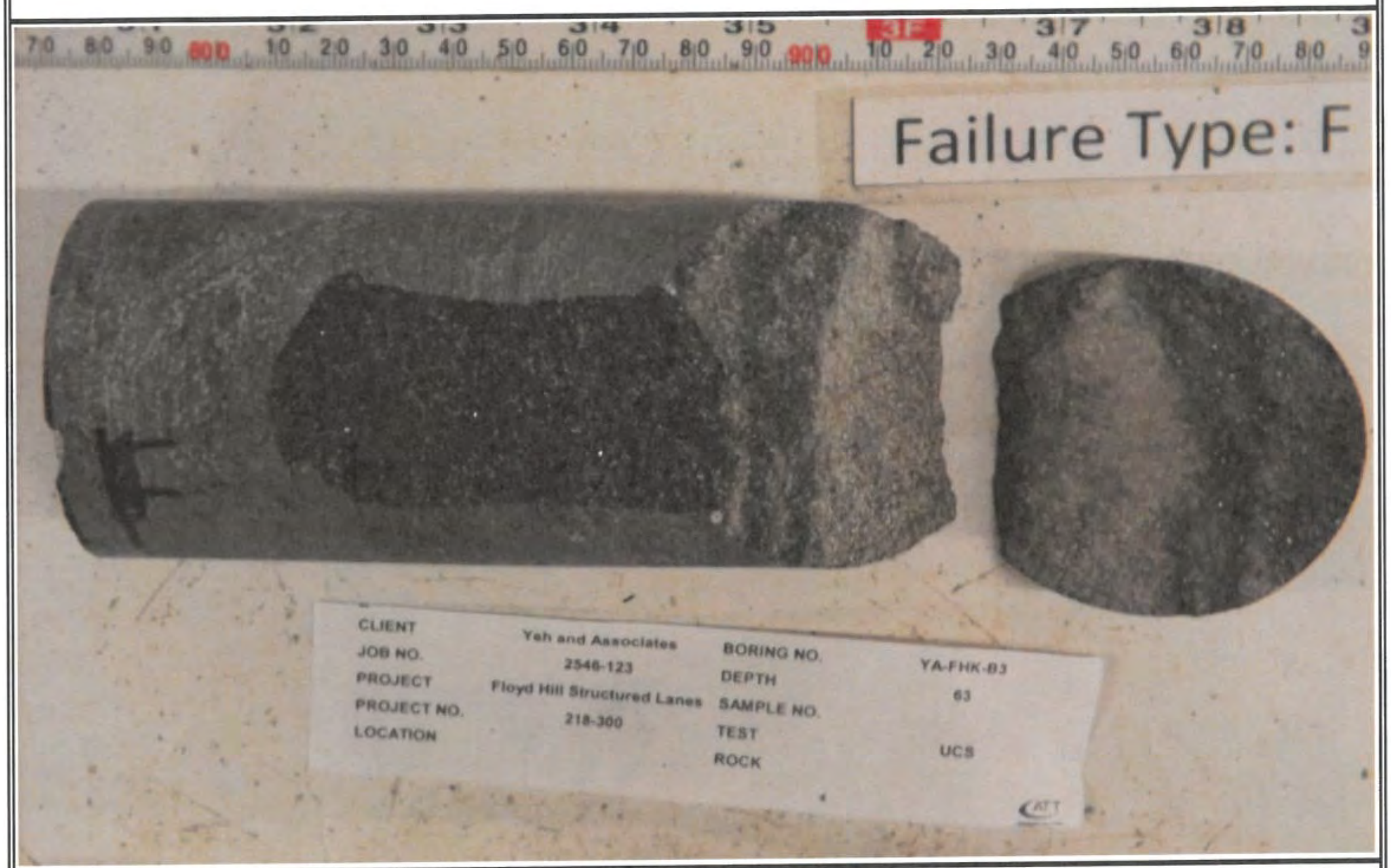

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| LOCATION | | ROCK | |

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# I-70 Floyd Hill Tunnel

## Tunnel Feasibility Study Report

**Colorado Department of Transportation**

11 February 2019

# Notice

This document and its contents have been prepared and are intended solely as information for Colorado Department of Transportationation (CDOT) and use in relation to the I-70 Floyd Hill Tunnel Feasibility Study.

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## Document History

| Revision | Purpose description | Originated | Checked | Reviewed | Authorized | Date |
|---|---|---|---|---|---|---|
| Rev 1.0 | Initial issue for review by CDOT | Alex Green | Luke Marriott | Ian Gee | Luke Marriott | 02/11/19 |
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## Client Signoff

| | |
|---|---|
| Client | Colorado Department of Transportation |
| Project | I-70 Floyd Hill Tunnel |
| Job number | 5182219 |
| Client signature / date | |

# Executive Summary

## Part A – Project Background

### Project Background

The purpose of the I-70 Floyd Hill to Veterans Memorial Tunnels Project (Project) is to improve travel time reliability, safety, and mobility, and address the deficient infrastructure on westbound I-70 through the Floyd Hill area of the I-70 Mountain Corridor (see Figure 0.1). This will be achieved through realignment of roads, addition of a third lane in both Eastbound and Westbound directions; and improvements to a number of a key interchanges.

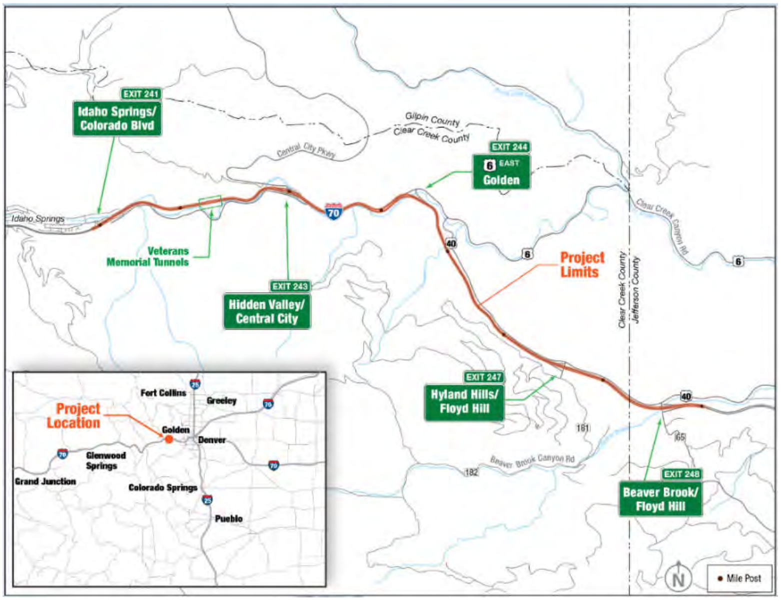


Figure 0.1 - Project Location

A tunnel will be constructed between Exit 244 and Exit 243 to enable realignment of the Westbound I-70. The tunnel will be constructed through the Northern side of Sawmill Gulch, as shown in Figure 0.2. The tunnel is required to accommodate three lanes of unidirectional traffic (Westbound) during normal operational conditions.

Figure 0.2 - Floyd Hill Tunnel Indicative Route

This feasibility study considered the initial design of the tunnel, including where appropriate, selection between different options. The study did not consider changes to the proposed alignment of the tunnel, which was produced outside of the scope of this study, and was therefore assumed fixed throughout this report.

# Part B – Tunnel Operational Requirements

## Tunnel Operations

As a unidirectional tunnel, a range of simplified operational modes are to be considered, especially for the tunnel ventilation system, with the tunnel presenting an ability to self-ventilate (i.e. system inactive) with vehicles travelling in excess of 10mph and a minimum number of fans to be consistently activated for standstill traffic or emergency conditions respectively. The tunnel is provided with lighting appropriate to this unidirectional traffic profile and modulated to ambient luminosity to prevent impact on drivers' vision as they enter and exit the tunnel portals. The activation, control and monitoring of in-tunnel systems will be performed as part of a wider operational strategy specific to Floyd Hill Tunnel and appropriate to the operational status of the tunnel.

An Operational Control Centre will be required to initiate appropriate protocols in the tunnel depending on the nature of the incident and predefined sequential activation of the in-tunnel systems. Traffic management, tunnel ventilation, mechanical, electrical and plumbing (MEP) services systems and access for fire and rescue services are critical parameters that will require integration in the overall operational protocols to allow informed decisions to be made for an incident.

## Tunnel Operating Design Parameters

The design speed for Floyd Hill Tunnel is 55mph, for the design year 2040. The Annual Average Daily Traffic (AADT) was derived from the traffic data projections and the peak hourly traffic rates. The traffic profile is expected to vary seasonally with potentially higher traffic observed during winter due to the I-70 Mountain Corridor serving as a major route for winter recreational activities and/or increased summer traffic for outdoor activities in the Rocky Mountains.

It was assumed that there will be no additional vehicle restrictions for the tunnel and therefore any vehicle allowed to use the I-70 Mountain Corridor will be able to use Floyd Hill Tunnel; therefore, the tunnel will allow uncontrolled vehicle types to travel through it, including dangerous goods vehicles (HazMat).

## NFPA 502 Requirements

The concepts, strategies and systems will be required to achieve, as a minimum, the mandatory requirements of National Fire Protection Association (NFPA) Standard 502 (2017) for Category C tunnels with conditionally mandatory requirements (CMRs) such as the consideration of a water based fixed fire-fighting system (FFFS) being subject to engineering judgment and approval by the Authority Having Jurisdiction (AHJ). NFPA 502 is the governing standard for fire protection and fire life safety requirements for road tunnels in North America. The key objectives of the adopted systems and strategies must be able to fulfil the following key safety objectives in accordance with NFPA 502:

- Permit the users of the Floyd Hill Tunnel to evacuate in reasonable safety;
- Reduce the life safety risk to as low as reasonably practicable;
- To facilitate fire-fighting operations.

# Part C – Tunnel Configuration Options & Initial Evaluation

## Tunnel Configuration Options

A total of nine configurations were considered for Floyd Hill Tunnel. Variations between the nine configurations considered provisions for egress from the tunnel and the number of bores used to accommodate the minimum three traffic lanes. The alignment of the tunnel was fixed for all nine options.

## Initial Evaluation of Tunnel Configuration Options

These options were initially evaluated using two key criteria:

- Compliance with NFPA 502;
- Commonality with other tunnel configuration options – grouping of options that were functionally similar/the same, followed by a qualitative assessment of cost and constructability to identify a single preferred option within the grouping.

The initial evaluation removed five of the nine initial configurations, leaving a total of four configurations that were shortlisted for further quantitative assessment at later stages in the study. These are:

- **Option A** – single bore, dedicated egress route provided by a compartmentalized corridor within the main highway tunnel;
- **Option B** – single bore, dedicated egress route provided by tunnels mined perpendicular to the main highway tunnel;
- **Option C** – single bore, dedicated egress route provided by tunnels mined parallel and on the same level as the main highway tunnel;
- **Option D** – twin bore, with interconnecting cross passages (egress is through evacuation into the other bore).

# Part D – Fire Risk Assessment and Tunnel Ventilation

## General Considerations

The potential maximum design fire heat release rate (FHRR) for the ventilation system sizing was assessed based on the following parameters:

- Historical precedence;
- Requirements of standards;
- The expected usage of the tunnel as forecast through the traffic profile and vehicle categories;
- Initial probabilistic assessment of fire likelihood.

Current international practice shows that that the choice of a design fire varies widely from 102 to 1,024MBTU/hr (≈ 30 to 300MW), depending on the type of traffic in the tunnel and in some cases the strategy of the ventilation system.

## Assessment of Likelihood of Fire

The assessment method for the likelihood of tunnel fire incidents used within this report was based on that presented in the output of the major research programme ‘Durable and Reliable Tunnel Structures’ (DARTS) by the European Thematic Network programme. A design peak heat release rate (HRR) of 550MBTU/hr (≈ 160MW) without FFFS and 135MBTU/hr (≈ 40MW) with FFFS was recommended, appropriate for 99.99% of the expected vehicle fire incidents in Floyd Hill Tunnel, with the following considerations:

- Traffic projections for the design year 2040;
- National statistics on vehicle fire incidents;
- Vehicle categories travelling through Floyd Hill Tunnel, including unrestricted access of Dangerous Goods (HazMat).

The derived peak HRR from the engineering analysis were found to be in good agreement with the range of values published by NFPA for representative peak HRRs for HGVs.

## Water Based Fixed Fire-Fighting Systems

The study considered the concept of integrating a FFFS for fire life safety purposes, to limit fire growth to a peak of 275MBTU/hr (≈ 80MW) and to further limit the residual convective heat release rate (HRR) visible by the ventilation system to 135MBTU/hr (≈ 40MW). Nevertheless, FFFS was not recommended for Floyd Hill Tunnel due to its unidirectionality and the tenable environment upstream of the fire for self-rescue by virtue of the ventilation system.

## Tunnel Ventilation

A longitudinal forced mechanical ventilation system was proposed to control smoke for emergency scenarios by preventing the back layering of heat and smoke in the case of fire, and limit the concentration of vehicle pollutants (CO and $NO_2$) to acceptable levels in the tunnel for normal or congested scenarios. A total of 24 jet fans were envisaged, arranged in 6 banks with 4 fans per bank. The 64” (≈ 1,600mm) diameter fans would be located at tunnel soffit level, clear of the traffic envelope and flapping zone. The ventilation system was sized for smoke control due to more stringent (safety) requirement than that for pollution. The performance of the system was assessed in the absence of FFFS. The traffic profile of Floyd Hill Tunnel presents an ability of the tunnel to self-ventilate for traffic speeds greater than 10mph and the requirement for the tunnel ventilation system to be active for standstill or slow-moving traffic (<10mph). Portal emissions were beyond the scope of this feasibility study and therefore were not considered.

# Part E – Tunnel Services

## Mechanical, Electrical & Plumbing Systems

The Floyd Hill Tunnel mechanical and electrical (M&E) systems were designed to be compliant with all mandatory requirements for a Category C tunnel as defined in NFPA 502 Section 7.2 (tunnels 1000ft to 3280ft not affected by heavy traffic). The systems provided to meet the mandatory requirements include:

- An Operational Control Center (with an ability to control the tunnel from an alternative location in the event of disruption at the primary control center). It was envisaged the Operational Control Center will be remote from the tunnel and may be integrated with other tunnel control facilities in a common Colorado Department of Transportation (CDOT) tunnels control center;
- A means to close the tunnel to traffic on the approach and within the tunnel complex. This will be achieved using Lane Use Signs and variable message signs;
- The tunnel will be provided with a system of stand pipes and water storage to ensure that the local emergency response teams are able to fight fires in the tunnel complex. The water supply will support fire-fighting operations for a minimum of one hour;
- Portable fire extinguishers will be provided in wall cabinets no further than 300ft apart;
- The drainage system will be designed to manage the control of water ingress and to safely manage the discharge of hazardous or flammable liquids as a result of an incident. The drainage system will also have the capacity to cope with the water used during fire-fighting operations;
- To support the emergency egress provisions, a mechanical ventilation system will be provided;
- A mechanical tunnel ventilation system will be provided to manage pollution and fire smoke;
- A fully resilient, dual-fed electrical system will be provided to all tunnel mechanical and electrical systems. For critical systems such as emergency lighting, an uninterruptible power source will be provided.

In addition, NFPA 502 requires a number of systems to be subject to agreement with the Authority Having Jurisdiction (AHJ) or as Conditionally Mandatory Requirements (CMRs) and subject to an ‘engineering analysis’. Three systems fall into this category – their possible provision should be assessed in consultation with the AHJ. These features are:

- Automatic fire detection and Closed-Circuit Television (CCTV) systems are conditionally mandatory, and an engineering analysis recommends that these features are provided. If agreed otherwise, the omission of these systems should be agreed with the AHJ;
- The Emergency Communication system comprising the emergency two-way radio system, the commercial radio rebroadcast and the Highway Advisory Radio (HAR) system. The current position taken, subject to agreement by the AHJ, is that these systems are required due to the remoteness of the location and the likely delays in getting rescue services on the scene;
- The provision of a FFFS will need to be considered in consultation with the AHJ. The work done to date demonstrates that a tenable environment for self-rescue and fire-fighting can be achieved by the tunnel ventilation system alone.

# Part F – Tunnel & Portals Design Development

## Geology and Ground Conditions

A wide range of geological conditions are represented and exposed along the I-70 corridor. The regional rock type of most relevance to the tunnel is biotite gneiss. In terms of local geology, the bedrock in the project area is primarily Precambrian metamorphic gneiss and migmatite. The proposed alignment of the tunnel is planned to pass between two mapped limbs of the Floyd Hill Fault, which generally trends North-Northwest. According to the American Society for Civil Engineers (ASCE) classification, the level of seismicity in the area local to the proposed Floyd Hill Tunnel area is considered to be 'very low'.

A preliminary Ground Investigation (GI) exercise was carried out for the tunnel. This consisted of the drilling of five boreholes (two at the East Portal, three at the West Portal). Only two boreholes struck water; likely to be from discrete, unconnected bodies of water located in joints collecting from rainwater and snow, with infiltration into the rock mass. The ground around the tunnel is expected to be relatively dry with low flows of water.

The GI provided some initial data on the characteristics of the rock local to the proposed location of Floyd Hill Tunnel. This data was used to produce a preliminary classification for the rock mass across the length of the tunnel; it was categorized as 'poor' rock (Q = 1-4) according to the Q-system and 'fair' (RMR = 41-60) according to the rock mass rating (RMR) system. The Q value for the rock mass was used to determine a number of additional rock design parameters, in order to inform the production of preliminary designs for the tunnel lining.

Following classification of the rock mass, a further assessment of the general rock quality and distribution was made. Locally in all boreholes there are indications of zones of weakness where there is a distinct local drop in the quality of rock. The single weakness zone where the Q is below 1.0 is of significant concern, and is located towards the end of borehole EP-2. Based on the available information this feature does pose a significant risk to the construction of both the tunnel and the rock walls for the portals. A detailed assessment of this needs to be undertaken once more complete information on the nature and geometry of this particular weakness zone becomes available.

## Spaceproofing

Options A, B and C, which utilize a single bore tunnel, accommodate the three required lanes of traffic in a single tunnel bore. Option D utilizes two separate highway spaces, with two highway lanes in each of the twin bore tunnels (providing one lane of traffic over the required three). An initial profile was drawn for Options A, B/C (same tunnel profile) and D, based on the form used for the Veterans Memorial Tunnels (VMT) Project – vertical side walls, with a transitional curve (shoulders) connecting the side walls to the main curved roof profile (crown). The tunnel profiles were revised in order to minimize the amount of rock excavation, with the cross-sections of Option A and B/C reduced through lowering the roof, thus creating a flatter roof profile. Option D experienced no further reduction in size, due to the limited space in the soffit for M&E equipment (mainly ventilation fans).

## Tunnel Lining Development

A tunnel lining concept was developed, based on best practice and the geological information obtained to date. The recommendations for the key features of the tunnel lining are outlined in Table 0.1.

Table 0.1 - Summary of the Recommended Tunnel Lining Features

| **Tunnel Lining Feature** | **Selection** | | |
|---|---|---|---|
| | **Option A** | **Option B/C** | **Option D** |
| Design life | Assumed to be a minimum of 100 years | | |
| Structural form | Primary SCL with a cast *in situ* concrete secondary lining (with conventional bar reinforcement) | | |
| Waterproofing | Drained lining. Dimpled drainage membrane (drained system) located between primary SCL and *in situ* secondary lining | | |
| Fireproofing | Fireboards fixed to intrados of *in situ* secondary lining | | |

The design of the primary ground support was carried out using the Q-system, which bases the thickness of the reinforced sprayed concrete lining (SCL) and the length/spacing of the rock bolts on empirical data. The design of the *in situ* concrete secondary lining was based on structural analyses undertaken for each of the different options using a 2-dimensional beam-spring model utilizing a structural design software. Predicted lining actions (bending moments, shear forces and axial forces) were determined following an assessment of the likely loads acting on the secondary lining. The secondary lining thickness and reinforcement requirements were determined from the outputs of these analyses.

## Portal Design Development

The portal locations were estimated by determining the required rock cover (rock brow thickness) over the crown of the tunnel and comparing it with the longitudinal profile of the tunnel. As a result, the portal cut excavation volumes were found through use of a three-dimensional solid modelling program (using surveyed terrain data). The portal will take the form of a cutting cut into the side of the ridge that the tunnel passes through. A gradient of 1:¼ was proposed for the headwalls at the portals, whilst side ditches were proposed at either side of the portal opening to provide rockfall protection for the I-70 highway. SCL and rock bolts were recommended as the main rock support for the portal headwall. During portal excavation into the headwall, a canopy tube array (grouted tubes) was suggested to provide additional support to the initial tunnel drive.

Potential factors to consider during the construction phase include: difference in constructability of the East and West Portal; excavated muck removal limited due to possible disruption to existing I-70 highway; and construction of a new viaduct in order to construct the East Portal. Rockfall protection, in the form a concrete canopy structure at the portal, was identified as a potential design requirement.

# Part G – Tunnel Configuration Final Evaluation

## Tunnel Configuration Options Assessment

The shortlisted four tunnel configurations were further evaluated using the following criteria:

- **Cost** – estimated capital cost for construction of the tunnel;
- **Operational benefit** – qualitative consideration of any additional benefits (over and above the minimum requirements as specified by CDOT and as required by relevant codes and standards) for normal operation of the tunnel, maintenance works and emergency scenarios;
- **Construction duration** – estimated time required to excavate and build the tunnel;
- **Construction risk** – qualitative consideration of constructability issues, considering the size of the excavation required for the main highway tunnel; the size of portals required at each end of the tunnel; additional tunneling and civil works required (e.g. for mined or isolated egress tunnels/compartments); and inherent health and safety hazards.

Following the evaluation of Tunnel Configuration Options A, B, C and D based on the assessment criteria, a summary of the options assessment can be found below (Table 0.2). Operational benefit and construction risk were rated as either low, moderate or high (considered relative to the other options).

Table 0.2 - Summary of Options Assessment

| Tunnel Configuration Option | Operational Benefit | Construction Risk | Cost (Nearest $1,000) | Construction Duration |
|---|---|---|---|---|
| Option A | High | Moderate | $177,279,000 | 46 months<br>(3 years, 10 months) |
| Option B | Low | High | $164,552,000* | 46 months<br>(3 years, 10 months) |
| Option C | Moderate | Moderate | $160,754,000 | 44 months<br>(3 years, 8 months) |
| Option D | High | High | $193,134,000 | 43 months<br>(3 years, 7 months) |

** The cost shown for Option B includes an approximate cost estimate of $15,000,000 for construction of the egress tunnel portals*

Evaluation of the shortlisted options identified two preferred configurations for the I-70 Floyd Hill Tunnel:

- **Option A** – single bore, dedicated egress route provided by a compartmentalized corridor within the main highway tunnel;
- **Option C** – single bore, dedicated egress route provided by tunnels mined parallel and on the same level as the main highway tunnel.

Both options met the requirements of NFPA 502 and the operational requirements outlined by CDOT. Selection of a single preferred option from these two is likely to be decided by CDOT (on the basis of operational benefits) and the Contractor appointed to build the tunnel (on the basis of preferences on construction methodologies).

## Recommendations & Further Work

It was found that a number of matters within NFPA 502 (and additional design issues) require the opinion of the AHJ – these discussion topics are summarized in Table 0.3.

Table 0.3 - Topics for Discussion with the AHJ

| Report Reference | Discussion Point | |
|---|---|---|
| | Topic | Decision |
| 2.4.1 | Emergency protocol | To implement automatically, semi automatically or manually. |
| 2.4.4 | Evacuation | The harsh winter ambient conditions of Floyd Hill may also suggest the need for some sheltering for external assembly points. |
| 3.1.2 | Tunnel strategies for contraflow operation | Contraflow traffic through the tunnel in situations when the eastbound carriageway is closed. |
| 4.6 & 11.3 | Fire alarm and detection | Manual fire alarm boxes or emergency telephones. |
| 4.6 | Fire alarm and detection | 24/7 CCTV monitoring. |
| 4.7 & 11.4 | Emergency communications | Two-way radio communication enhancement systems. |
| 4.7 | Emergency communications | HAR and re-broadcasting of AM/FM commercial radio with overrides. |
| 4.9 | Fire apparatus | Water should be made available to supplement the Colorado Fire Department's water. This can be either by the Fire Department or the tunnel facility. |
| 4.10 | Standpipes | Fire hydrant hose connections should be provided such that no location on the protected roadway is more than 150ft from the connection and a maximum spacing of 275ft, or less if required by the AHJ. |
| 4.16 | Means of egress | Tunnel roadway surface upstream of fire as means of escape with longitudinal ventilation system, exit doors serving as alternative access for fire rescue services. |
| 4.17 | FFFS | Provision of fixed fire-fighting system. |
| 4.20 | Emergency response | Emergency response plan. |
| 13.2 | Emergency exits | Emergency exit provision. |
| 8.4 & 11.8 | Fire likelihood analysis | Likelihood of fire events. |
| 9.2.4 | FFFS | Details of activation times and running times. |
| 14.2.4 | Passive fire protection to the structure | Acceptance of RWS time-temperature curve. |
| 15.3.3 | Evacuation | Provision of assembly points and portal configuration. |

Areas for optimization and further investigation within the design of Floyd Hill Tunnel were identified and should be explored during the subsequent stages of design. They are as follows:

- **Tunnel Operations** – CDOT need to develop a control strategy for Floyd Hill Tunnel (and their other tunnels) so that they can be controlled in a safe and efficient manner;
- **Tunnel Services** – The requirements for drainage and utilities supplies need to be established, including the provision of a tunnel services building;
- **Tunnel Ventilation** – The likelihood of fires in the tunnel, the recommendation of the peak HRR and the conclusions on the ventilation strategy require analysis of latest information sourced from local authorities and state departments;
- **Tunnel Lining and Portals** – A more detailed GI must be undertaken across the entire alignment, in order to further characterize potential weakness zones identified in the rock mass conditions along the route. Understand the feasibility of removing reinforcement bars from the secondary lining design, instead using SFRC if possible. The impact of assembly points and a tunnels services building on the size of the portal rock cut will have to be determined (if necessary), as well as more detailed understanding of the impacts on the existing I-70 highway during construction of the portals. The requirement of a canopy portal structure to provide sufficient rockfall protection should be explored.

# Contents

# Figures

## Tables

# List of Abbreviations

| | |
|---|---|
| ACSE | American Society for Civil Engineers |
| AHJ | Authority Having Jurisdiction |
| AID | Automatic Incident Detection |
| BLEVE | Boiling Liquid Expanding Vapor Explosion |
| BSI | British Standards Institute |
| CCTV | Closed-Circuit Television |
| CDOT | Colorado Department of Transportation |
| CMR | Conditionally Mandatory Requirement |
| CNG | Compressed Natural Gas |
| CO | Carbon Monoxide |
| DARTS | Durable and Reliable Tunnel Structures |
| DGV | Dangerous Goods Vehicle |
| ELV | Extra-Low Voltage |
| ESR | Excavation Support Ratio |
| FFFS | Fixed Fire-Fighting System |
| FFSS | Fixed Fire Suppression System |
| FHRR | Fire Heat Release Rate |
| FHWA | Federal Highway Administration |
| GI | Ground Investigation |
| HAR | Highway Advisory Radio |
| HazMat | Hazardous Materials |
| HRR | Heat Release Rate |
| LED | Lighting-Emitting Diode |
| LNG | Liquified Natural Gas |
| LP-Gas | Liquified Petroleum Gas |
| LSOH | Low Sulfur, Zero Halogen |
| LV | Low Voltage |
| MEP | Mechanical, Electrical and Plumbing |
| MR | Mandatory Requirement |
| NFPA | National Fire Protection Association |
| $NO_2$ | Nitrogen Dioxide |
| $NO_x$ | Nitrogen Oxide |
| OA | Outside Air |
| OAD | Outside Air Demand |
| PA | Public Address |
| PE | Probability of Exceedance |
| PGA | Peak Ground Acceleration |
| PIARC | World Road Association |
| PLC | Programmable Logic Controller |
| RMi | Rock Mass index |
| RMR | Rock Mass Rating |
| RQD | Rock Quality Designation |
| RWS | Rijkswaterstaat |
| SCADA | Supervisory Control And Data Acquisition |
| SCL | Sprayed Concrete Lining |
| SFRC | Steel Fibre Reinforced Concrete |
| SRF | Stress Reduction Factor |
| TRL | Transport Research Laboratory |
| UCS | Unconfined Compressive Strength |
| US | United States |
| USCRN | United States Climate Reference Network |
| U.S.NRC | United States Nuclear Regulatory Commission |
| VMT | Veterans Memorial Tunnels |

# Introduction

This report presents the outcomes of a feasibility study for the proposed Interstate 70 (I-70) Floyd Hill Tunnel. The tunnel has been assessed within the wider context of the I-70 Floyd Hill to Veterans Memorial Project.

The Floyd Hill Tunnel Feasibility Study focused on four key issues:

- **Provisions for in-tunnel emergencies** – egress routes, smoke control and firefighting provisions;
- **Mechanical, electrical and plumbing systems** – recommendations on provision of fire detection, alarm and suppression systems; communication and traffic control; ventilation; drainage and lighting;
- **Tunnel structural lining** – strategy for supporting the ground around the tunnel opening and for controlling ground water. This included specification of spaceproofing, primary support provisions, tunnel lining composition, lining thicknesses and concrete reinforcement;
- **Recommendations on optimum tunnel configurations** – evaluation of a total of nine permutations of tunnel configuration, primarily varying provisions for egress from the tunnel in emergency conditions. Evaluation of these options considered compliance with key codes and standards, operational benefits, construction durations, constructability and cost.

The report concludes with a list of suggested actions to be undertaken to facilitate further development of the design for Floyd Hill Tunnel.

# Report Structure

The report is presented in seven parts, as detailed below:

## Part A – Project Background

High level overview of the report context and background information on the project.

## Part B – Tunnel Operational Requirements

Outlines the principles of a range of normally expected operational tunnel modes, with reference made to key system technical features and characteristics that demonstrate how the in-tunnel systems are expected to respond for normal and emergency modes. The criteria that the tunnel must satisfy in order to meet the minimum requirements are also determined, in order to achieve compliance with key clauses of the codes and standards (e.g. NFPA 502) referenced in this report, as appropriate to Floyd Hill Tunnel.

## Part C – Tunnel Configuration Options & Initial Evaluation

Presentation of nine configurations (cross sections and means of refuge/egress) initially considered for the tunnel and a subsequent high-level evaluation of these options. Nine initial configurations reduced to four, which were taken forward for further quantitative evaluation.

## Part D – Fire Risk Assessment and Tunnel Ventilation

Provides an assessment of the likelihood of potential fire in Floyd Hill Tunnel based on available traffic data, national statistics and guidance from international research; a peak heat release rate is recommended following appropriate analysis. The types, key characteristics and operational principles of water based fixed fire-fighting systems are discussed, leading to a recommendation for the use of such systems for Floyd Hill Tunnel. Various design conditions that govern the performance of the tunnel ventilation system are considered – the concept and components of a longitudinal mechanical forced ventilation system are presented. A further study is performed to assess the conditions under which the tunnel can self-ventilate to dilute vehicle exhaust gases.

## Part E – Tunnel Services

Definition of required mechanical and electrical systems, including: control centers, fire alarms and detection, emergency communication systems, traffic control systems, firefighting provisions, ventilation, drainage and lighting.

## Part F – Tunnel and Portals Design Development

Overview of the regional/local geology and the expected ground conditions. Spaceproofing exercise undertaken in order to refine the shape/size of the shortlisted tunnel configuration options. Development of the recommended tunnel lining design for Options A-D, including: primary ground support (primary lining and rock bolts); secondary lining form and thickness; and waterproofing/drainage and fireproofing details. Development of East and West Portal designs, including: location; definition of geometry; and rock support required.

## Part G – Tunnel Configuration Final Evaluation

Final assessment and evaluation of the shortlisted tunnel configuration options, based on four criteria: cost; operational benefit; construction duration; and construction risk. Recommendations for further work and key actions that are required to enable the further development of designs for Floyd Hill Tunnel are provided.

# PART A - Project Background

# 1. Project Background

## 1.1. Project Aim

The purpose of the I-70 Floyd Hill to Veterans Memorial Tunnels Project (Project) is to improve travel time reliability, safety, and mobility, and address the deficient infrastructure on Westbound I-70 through the Floyd Hill area of the I-70 Mountain Corridor. The Proposed Action addresses specific highway improvements defined in the ROD, including providing three-lane capacity for westbound I-70 from Floyd Hill to the Veterans Memorial Tunnels; a multimodal trail and frontage road between U.S. Highway 6 (US 6) and Idaho Springs; and physical and/or operational improvements to four interchanges: the Floyd Hill/Beaver Brook exit (Exit 248) near the top of Floyd Hill; the Floyd Hill/Hyland Hills exit (Exit 247); the junction with US 6 (Exit 244) near the base of Floyd Hill; and the Hidden Valley/Central City exit (Exit 243). The project would also improve curves through the corridor, consistent with the recommended 55 miles per hour (mph) design speed from the 2016 *I-70 Mountain Corridor Design Speed Study*.

## 1.2. Project Location

The project is located on the I-70 between milepost (MP) 248 (just East of the Beaver Brook interchange and Exit 241 (East Idaho Springs, west of the Veterans Memorial Tunnels). It is mostly located within Clear Creek County with the Eastern end located within Jefferson County. See Figure 1.1.

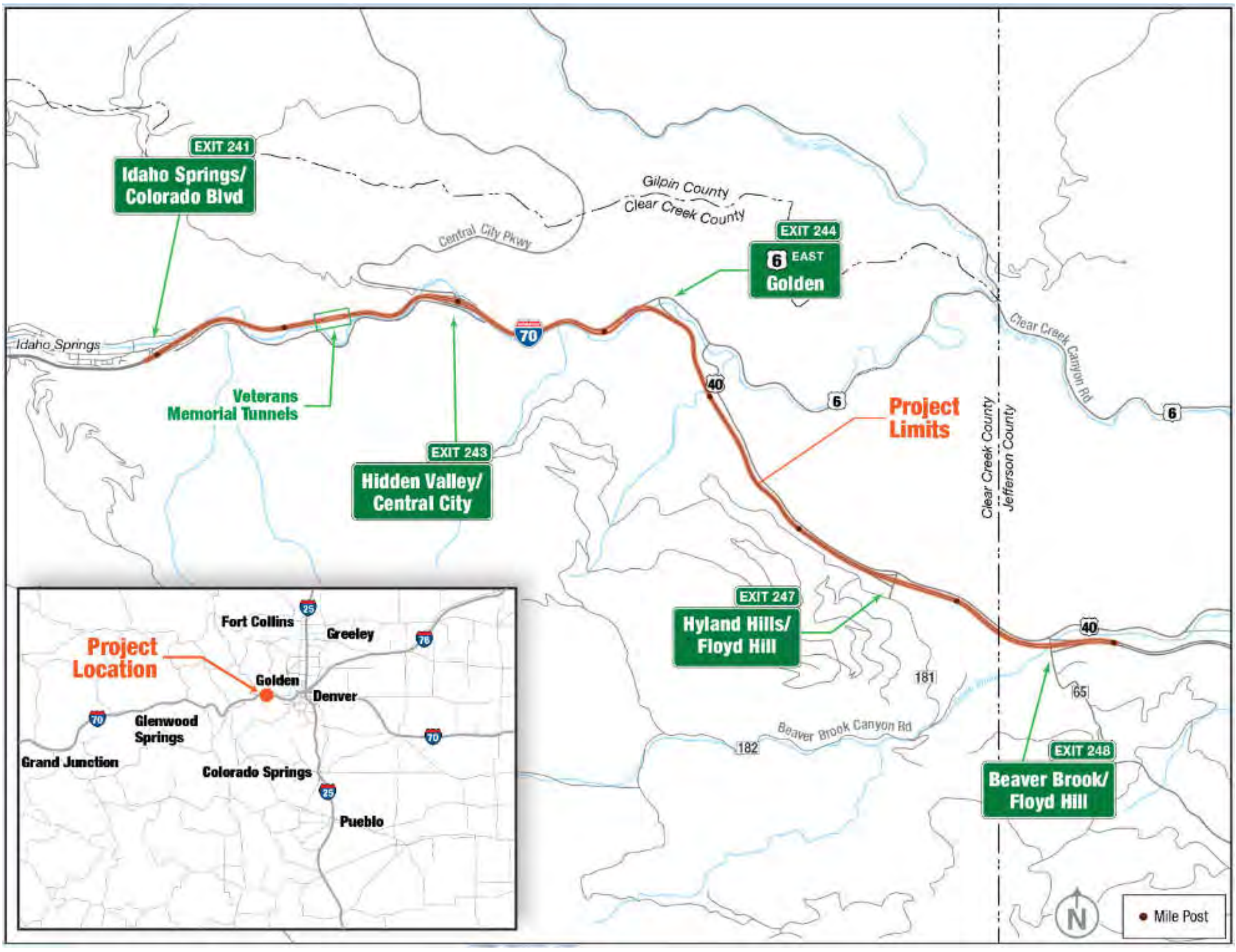


Figure 1.1 - Project Location

## 1.3. Proposed Action

The key elements included in the Proposed Action include:

- Adding a third westbound travel lane to the two-lane section of I-70 from the current three- to two-lane drop (approximately MP 246) through the Veterans Memorial Tunnels;
- Constructing a new frontage road between US 6 and the Hidden Valley Interchange;
- Improving interchanges and intersections throughout the project area;
- Improving design speeds and stopping sight distance on horizontal curves;
- Improving the multimodal trail (Clear Creek Greenway) between US 6 and the Veterans Memorial Tunnels;
- Reducing animal-vehicle conflicts and improving wildlife connectivity with new and/or improved wildlife overpasses or underpasses.

A detailed description of the Proposed Action and other design concepts considered can be found in the *I-70 Floyd Hill to Veterans Memorial Tunnels: Alternatives Analysis Technical Report*.

## 1.4. Floyd Hill Tunnel

A tunnel will be constructed in the Floyd Hill area to enable realignment of the Westbound I-70. The tunnel will be constructed through the Northern side of Sawmill Gulch, as shown in Figure 1.2.

Figure 1.2 - Floyd Hill Tunnel Indicative Route

The tunnel is required to accommodate three lanes of unidirectional traffic during normal operational conditions. During exceptional circumstances (e.g. construction/maintenance works or a major incident on the surface Eastbound Lane) contraflow operation of the tunnel will be required with two lanes of traffic running in either direction with a commensurate reduction in speed limit.

The tunnel will be approximately 2200 feet (ft) long and will be located between Westbound stations 2080+50 and 2102+90. It is envisaged that the tunnel will require a full *in situ* concrete lining to provide support to the rock overburden and to provide a structure through which ground water can be routed to appropriate drainage channels. The highway grade throughout the tunnel is generally inclined uphill in the Westbound direction of traffic.

This feasibility study considers the initial design of the tunnel, including where appropriate, selection between different options. The study does not consider changes to the proposed alignment of the tunnel, which has been produced outside of the scope of this study, and has therefore been assumed fixed throughout this report.

# PART B – Tunnel Operational Requirements

# 2. Tunnel Operations

## 2.1. Overview

Tunnel operations affect the risk profile of the tunnel, the equipment provision and the implemented strategies, as well as the size of the tunnel to accommodate the traffic and equipment. This section sets out to document these requirements that will need to be agreed prior to start of the next design stage, to assist in forecasts of project costs. For emergency operations that involve a vehicle fire incident, the basis of reference will be the National Fire Protection Association (NFPA) Standard 502 (2017) [1] with interpretation of key clauses provided in Section 4. The same section highlights the limitations of NFPA 502 to non-fire incidents. This section of the report presents the considerations given to such modes, as guidance for the next design phase.

## 2.2. Typical Modes

The operation of Floyd Hill Tunnel is characterized by a range of modes based on the type of activity in the tunnel. Each of these operational modes carries a specific set of protocols that consist of a predefined set of actions by the in-tunnel systems that can be automatic or manually-initiated by the tunnel operator. These modes are summarized below:

- Normal Operation for normal free-flowing traffic where no action is expected from the in-tunnel systems apart from routine monitoring. The tunnel will only require intervention by the operator to manage breakdowns or incidents. Systems will be designed to operate automatically in the background under normal circumstances and raise an alert to the operator if their action is required.
- Congested Operation for standstill or slow-moving traffic that can yield high pollution in the tunnel and require activation of the tunnel ventilation system, complemented by appropriate responses by traffic management systems and warning signs. The tunnel ventilation system will be operated automatically in the background based on feedback by in-tunnel monitoring stations, with continuous feedback provided for the operator and alerts given if their action is required.
- Degraded Operation for incidents that do not classify the tunnel to be in a state of emergency (e.g. non-evacuation) but require expert judgment and decision making by the operator (e.g. due to major accident, chemical/oil spill, ice, snowstorm) that could potentially develop into a life-threatening situation. Following feedback from in-tunnel systems, the operator would be required to assess the severity of the incident and decide on whether existing non-emergency protocols can be followed (e.g. traffic management) or an emergency response would need to be activated to evacuate people if there is plausible risk of an imminent life-threatening event, such as risk of fire.
- Emergency Operation where an emergency in the tunnel has been declared that requires evacuation of the tunnel occupants in accordance with the emergency response plans. Under the prescribed action sequence, in-tunnel systems would support evacuees in reaching a point of safety and emergency services to intervene and tackle the incident.
- Maintenance Operation for periods where the system, its assets or the tunnel itself needs to be maintained or replaced, where manual or local operation may be required but with the operator maintaining full control of the tunnel operation. This mode is mostly expected to be during scheduled maintenance of the system or the tunnel itself at times with low traffic (e.g. night-time); or with appropriate measures in place to minimize risk, disruption of traffic and tunnel operation for other times.
- Testing and Commissioning Operation for periods when the system or individual assets need to be tested and commissioned for their performance and system integration, either during installation or post-opening of the tunnel (e.g. if a new fan has been installed).

## 2.3. Normal – Congested Operation

### 2.3.1. Ventilation

Due to the unidirectional profile of free-flowing traffic, the tunnel can present an ability to self-ventilate due to the piston effect from travelling vehicles at posted speed, by drawing in outside air through the entry portal that dilutes and pushes contaminated air towards the exit portal. Where conditions such as strong opposing headwind, slow moving or stalled traffic counteract the benefits of the piston effect, the air pollution in the tunnel could increase and require activation of the tunnel ventilation system. Air quality monitors measuring carbon monoxide (CO), nitrogen oxide ($NO_x$) and visibility, will be strategically positioned in the tunnel to control the tunnel ventilation system to maintain the air quality in the tunnel to acceptable levels.

### 2.3.2. Drainage

Some water may enter the tunnel from leakage, vehicle runoff or driving rain/snow. The drainage system will normally dispose of this water by directing this to the foul sewer system. This water will be monitored for pollutants and automatically switch the effluent to storage tanks if required. This effluent can then be tankered away for processing/disposal. Not all pollutants can be automatically detected; therefore, if there is a spillage, the operator may need to manually override the diverter valve.

### 2.3.3. Lighting

Tunnel lighting is provided to reduce the impact on drivers' vision as they approach the tunnel through various ambient light conditions. This is particularly important on bright sunny days where the tunnel entrance can appear as a 'black hole' against a bright background. Photometer sensors will be installed on the approach near the tunnel to sense the lighting conditions at the portal from a drivers' perspective. The tunnel lighting will be automatically controlled to reflect the ambient to ensure that the entrance lighting is matched to the approach. This will ensure that the drivers are not brought into a light condition significantly different from the ambient.

## 2.4. Emergency Operation

### 2.4.1. Objectives

The primary objectives of an emergency protocol for Floyd Hill Tunnel can be described as follows:

- Fast detection, alert and response;
- Provide a safe egress route for self-rescue;
- Allow emergency services to safely intervene;
- Limit potential damage to infrastructure.

The decision as to whether the system activation and response to a fire incident should be fully automatic, manual and automatic, or fully manual is subject to future design and consultation with key stakeholders, the AHJ, the emergency services, and the tunnel operator.

### 2.4.2. Ventilation

Longitudinal forced mechanical ventilation will generate air flow in the direction of normal traffic in the tunnel that maintains tenable conditions upstream of the fire location, as discussed in Section 10. The proposed ventilation system for Floyd Hill Tunnel allows evacuees to reach a point of safety, whether that is back towards the entry portal or through an available exit door.

### 2.4.3. Traffic Management

For emergency incidents, the Floyd Hill Tunnel bore(s) will be closed to oncoming traffic with appropriate warning signs displayed on signs of the I-70 Mountain Corridor at a certain distance upstream of the tunnel to prevent and divert further oncoming Westbound traffic.

### 2.4.4. Evacuation and Intervention

Tunnel occupants are expected to evacuate their vehicles and use evacuation directional aids (wayfinding, exit door signs) to walk towards a point of safety, whether that is through an available exit door or the entry portal, by benefit of a tenable environment upstream of the fire location as provided by the tunnel ventilation system.
Common design practice encourages the identification of a safe assembly area for evacuees that exit the tunnel facility, be that just outside the entry portal or in a predefined assembly area. This is a matter for future design, taking into account site specific considerations. The harsh winter ambient conditions of Floyd Hill may also suggest the need for some sheltering, which should be considered as part of this planning exercise, with stakeholders and AHJ consultation.

Fire and rescue services will require access to the fire location to assist with the evacuation process, provide medical assistance and tackle the fire. Both Floyd Hill Tunnel and the management of highway traffic must therefore allow adequate means of access for emergency vehicles to travel past stalled traffic while the in-tunnel systems provide a tenable environment for fire fighters to approach and tackle the fire. The tunnel ventilation system will be able to provide such conditions upstream of the fire location, with access ensured through the west entry portal or through the egress paths as an alternative.

Appendix A provides examples of possible evacuation and intervention strategies, used as part of the considerations taken in the assessment of the different tunnel configurations.

## 2.5. Maintenance Operation

### 2.5.1. Access

Tunnel equipment should be designed and installed to allow safe maintenance. This would normally be undertaken in tunnel closures with traffic either diverted onto an alternative route or in contraflow on the Eastbound carriageway. Both of these options require significant planning and implementation from an operational perspective, so consideration should be given to ensuring that equipment is highly reliable and installed out of the bore, where possible, to minimize closures. Consideration may be given to partial tunnel closures, allowing maintenance activities whilst traffic operates in a single lane at reduced speed under low-flow conditions (e.g. at night-time).

### 2.5.2. Cleaning and Preventative Maintenance

The tunnel will require regular cleaning and preventative maintenance. Walls, lighting luminaires and Closed-Circuit Television (CCTV) cameras will need to be regularly cleaned. All mechanical, electrical and plumbing (MEP) equipment will need to be regularly inspected and serviced to ensure that it is ready for use on demand. These operations are undertaken during tunnel access restrictions with diversions and risk mitigation put in place. The tunnel maintenance team will need to be based within a reasonable distance from the tunnel so that equipment is readily available. Consideration should be given to combining with the Colorado Department of Transportation (CDOT) maintenance depot near I-70 Exit 243.

### 2.5.3. Winter Maintenance

During the winter, the normal open road procedures for snow clearing will need to be adapted to take account of the tunnel systems and evacuation routes. The escape doors and passages should be kept clear of any snow that is blown into the tunnel and is cleared from the carriageway. Consideration needs to be given to avoiding corrosive chemicals or salt from being spread in the tunnel as it will accelerate the corrosion of MEP equipment.

### 2.5.4. Routine Maintenance

Routine maintenance will need to be carried out to inspect, clean and service equipment. This would normally take place in planned tunnel closures typically on a three-month cycle.

### 2.5.5. Reactive maintenance

Reactive maintenance may need to be carried out to repair or replace any equipment that fails prematurely. The design should consider equipment redundancy to minimize unplanned closures.

## 2.6. Tunnel Control and Monitoring

The tunnel will need to be operated from an Operational Control Center which should be manned 24/7. The location is yet to be defined but a study should be undertaken to determine the best strategy for this and other CDOT tunnels in the area.

The Operational Control Center should be equipped with CCTV, incident detection, fire detection and tunnel control systems. The operator(s) should monitor the free flowing of traffic through the tunnel and be ready to respond to any incident or vehicle breakdown. The inclusion of an incident detection system will reduce the workload on the operator(s) by highlighting abnormal vehicle behavior and alerting the operator to look at the relevant monitors. The operator can then assess what action to take; for example, closing a lane using the Lane Use Signs to protect a broken-down vehicle. The operator will be able to use the Dynamic Message Signs on the portal and be able to broadcast messages on the Highway Advisory Radio to warn drivers of any unusual situations ahead.

# 3. Tunnel Operating Design Parameters

## 3.1. Traffic Speed and Direction

### 3.1.1. Traffic Speed

The design speed is 55mph for the design year 2040.

### 3.1.2. Traffic Direction

The feasibility study is based upon unidirectional westbound traffic through Floyd Hill Tunnel.

There may be circumstances of an incident in the eastbound open highway that can result in its part or full closure with diversion of eastbound traffic through Floyd Hill Tunnel for periods of time. Highway design is expected to accommodate crossovers near the portals to facilitate temporary contraflow traffic through the tunnel during construction or in the event of a major incident on the eastbound highway.

Contraflow traffic through the tunnel elevates the safety risk to tunnel users, may increase the complexity of in-tunnel systems and their operational strategies and require specific operational measures to be adopted. Such risk mitigation measures adopted to manage the safety of the tunnel is to be addressed under the management and operational procedures for the tunnel. Such circumstances lend themselves to advance consultation with the AHJ and key stakeholders, before adapting tunnel strategies for contraflow operation.

Considering the above, no provisions for contraflow traffic through Floyd Hill Tunnel are considered at this time in this respect. Conditions under which contraflow will be permissible for these cases should be determined at the time and may be allowable with the proposed system configurations with appropriate measures, procedures and operational safety protocols in place.

## 3.2. Number of Lanes

The scheme outlined in Section 1 of this report requires three lanes of traffic through the tunnel, one of which may be a dedicated tolled Express Lane. The scheme requires a buffer zone between the normal lanes and the Express Lane, and a shoulder on the nearside.

## 3.3. Annual Average Daily Traffic (AADT)

Traffic data projections in vehicles per hour for the design year of 2040 are considered as the best currently available for Floyd Hill Tunnel for a Westbound traffic direction, just before the merge with Westbound US 6 road network, and will form the basis of establishing the following (seasonal and diurnal variations estimated as appropriate):

- Tunnel category in accordance with NFPA 502;
- Fire frequency analysis to obtain a recommended design peak heat release rate (HRR);
- Density of traffic backlog upstream of the fire location for system performance under emergency conditions;
- In-tunnel vehicle emissions to assess the outside air demand required to maintain acceptable pollution levels in the tunnel.

The projected traffic volumes for 18-hourly (i.e. 04:00 - 21:00) traffic volume data on winter Saturday and summer Sunday are presented in Table 3.1. The remaining 6-hour period (22:00 to 03:00) has been interpolated as a constant average of the first and last hour (4:00 and 21:00) to represent night traffic and complete the 24-hr traffic profile. This is considered a reasonable assumption for the purposes of the feasibility study and is subject to further refinement when more detailed traffic profiles are developed in the later design stages of the project.

Table 3.1 - Hourly Traffic Data Forecast for Design Year 2040

| **Hour** | **Daily Count (vehicles/hour)** | | **Hour** | **Peak Hourly Traffic (vehicles/hour/lane)** | |
|---|---|---|---|---|---|
| | Winter Saturday | Summer Sunday | | Winter Saturday | Summer Sunday |
| 04:00 | 236 | 164 | 04:00 | 79 | 55 |
| 05:00 | 1746 | 445 | 05:00 | 582 | 148 |
| 06:00 | 3369 | 836 | 06:00 | 1123 | 279 |
| 07:00 | 3847 | 1499 | 07:00 | 1282 | 500 |
| 08:00 | 3969 | 1900 | 08:00 | 1323 | 633 |
| 09:00 | 3159 | 2294 | 09:00 | 1053 | 765 |
| 10:00 | 2890 | 2987 | 10:00 | 963 | 996 |
| 11:00 | 3447 | 3253 | 11:00 | 1149 | 1084 |
| 12:00 | 3155 | 3207 | 12:00 | 1052 | 1069 |
| 13:00 | 3085 | 3095 | 13:00 | 1028 | 1032 |
| 14:00 | 3067 | 2776 | 14:00 | 1022 | 925 |
| 15:00 | 2904 | 2425 | 15:00 | 968 | 808 |
| 16:00 | 2440 | 2290 | 16:00 | 813 | 763 |
| 17:00 | 2010 | 1972 | 17:00 | 670 | 657 |
| 18:00 | 1626 | 1896 | 18:00 | 542 | 632 |
| 19:00 | 1243 | 1410 | 19:00 | 414 | 470 |
| 20:00 | 925 | 1082 | 20:00 | 308 | 361 |
| 21:00 | 704 | 795 | 21:00 | 235 | 265 |
| 22:00 | 470 | 480 | 22:00 | 157 | 160 |
| 23:00 | 470 | 480 | 23:00 | 157 | 160 |
| 00:00 | 470 | 480 | 00:00 | 157 | 160 |
| 01:00 | 470 | 480 | 01:00 | 157 | 160 |
| 02:00 | 470 | 480 | 02:00 | 157 | 160 |
| 03:00 | 470 | 480 | 03:00 | 157 | 160 |

The Annual Average Daily Traffic (AADT) is derived from the traffic datasets of each day. From the above projections, distributed annually as equal periods of 7 days/week and 6 months each, i.e. winter Saturday and summer Sunday traffic data are the same for 6 months. Table 3.2 provides an overview of the derived AADT data used for the purposes of the fire frequency analysis.

Table 3.2 - I-70 Floyd Hill Traffic Overview

| **Variable** | **Value** | **Comments** |
|---|---|---|
| Tunnel length | 2,200ft (670m) | |
| AADT (vehicles/day) | 46,642 | **Default winter traffic** for 7 days/week for 26 weeks |
| | 37,206 | **Default summer traffic** for 7 days/week for 26 weeks |
| Vehicle miles per year<br>(Vehicle km per year) | 6,370,617<br>(10,252,514) | (Summer AADT + Winter AADT) x 7 days/week x 26 weeks |

The traffic profile is expected to vary seasonally with potentially higher traffic observed during winter due to the I-70 Mountain Corridor serving as a major route for winter recreational activities and/or increased summer traffic for outdoor activities in the Rocky Mountains. Therefore, the results and considerations of the feasibility study must be appropriately reviewed for their impact on conclusions, when new data become available during the course of the project.

## 3.4. Vehicle Categories

Given that the I-70 route is an interstate highway, it is anticipated that there will be no additional vehicle restrictions for the tunnel and therefore any vehicle allowed to use the I-70 Mountain Corridor will be able to use the tunnel, including vehicles carrying hazardous goods and hazardous materials (HazMat), such as fuel tankers.

For the percentage of dangerous goods vehicles and in the absence of any specific data for I-70 Mountain Corridor, it has been assumed that the percentage of HazMat vehicles is similar to the United States (US) road network [2] of 7%, that reflects the total number of trucks carrying hazardous materials. Therefore, out of the percentage of Heavy Goods Vehicles (3.5%), 7% are considered to carry hazardous materials, as tabulated below.

Following the above, the vehicle categories forecasted to travel through the I-70 Mountain Corridor and therefore the Floyd Hill Tunnel are summarized below and outlined in Table 3.3:

a) 1% of medium vehicles (single unit trucks and buses), refined further to 0.5% HGVs (as attributes of single unit trucks) and 0.5% buses;
b) Total of 3.5% of HGVs considering 3% heavy vehicles (tractor trailers) and 0.5% from above assumption;
c) Out of the above 3.5% of HGVs, 7% is assigned for dangerous goods vehicles resulting in 0.245% of Hazmat and 3.255% of HGVs;
d) Remaining vehicle fleet as passenger cars.

Table 3.3 - Vehicle Categories

| Vehicle Category | Value | Comments |
|---|---|---|
| %Buses | 0.5% | See Point a above |
| %Trucks / HGVs | 3.255% | See Point b and c above |
| %Dangerous Goods (HazMat) | 0.245% | See Point d above |
| %Cars | 96% | = 100% - %Buses - %HGVs - %HazMat |

It is worth noting that having a greater percentage of buses in the tunnel will increase the population in the tunnel that need to be considered for evacuation during an emergency.

# 4. NFPA 502 Requirements

## 4.1. Overview

The purpose of this section is to provide an understanding and interpretation of relevant NFPA fire protection and fire life safety clauses and requirements that need to be considered in the feasibility study and future design stages. It should be noted that NFPA 502 has not been adopted by State of Colorado but CDOT intend to follow it as their design standard. All observations and interpretations are based on NFPA 502 (2017).

As clarified in NFPA 502 Clause 1.3.1, *"the provisions of this standard are the minimum necessary to provide protection from loss of life and property from fire*"; with a fire emergency defined under Clause 3.3.27 as "*the existence of, or threat of, fire or the development of smoke or fumes, or any combination thereof, that demands immediate action to mitigate the condition or situation to emergency conditions*". Based on the above, interpretation of clauses and requirements are only applicable for emergency scenarios that involve a fire incident, whereas the ability of the tunnel and/or systems to accommodate events that do not involve a fire (e.g. vehicle collision or breakdown, traffic congestion, chemical spill, etc.) are not in the scope of this feasibility study.

It is noted that a number of clauses relate to operational characteristics of the proposed systems that do not affect the tunnel profile or system strategy (for the purposes of the feasibility study) and are not discussed herein. The following elements are not in the scope of the feasibility study thus their interpretation or reference is not offered in this report, as these would be expected to be demonstrated during the later design stages:

- Demonstrate the requirements for compliance to all clauses of NFPA 502;
- Investigate compliance to other NFPA codes as referenced in NFPA 502;
- Discuss compliance to other international, national or state codes and industry practices.

## 4.2. Terminology and Definitions

A selection of definitions provided in NFPA 502 and NFPA Glossary of Terms (2018) is provided for clarity on the terminology and interpretation of clauses identified in this report (Table 4.1). For definitions of other specific clauses, reference should be made to the General Definitions section of NFPA 502 and the NFPA Glossary of Terms.

Table 4.1 - NFPA Definitions

| Definition | Description |
|---|---|
| Shall | Indicates a mandatory requirement. |
| Should | Indicates a recommendation or that which is advised but not required. |
| Mandatory Requirement (MR) | A requirement prefaced by the word "shall" within the standard. |
| Conditionally Mandatory Requirement (CMR) | A requirement that is based on the results of an engineering analysis. |
| Authority Having Jurisdiction (AHJ) | An organization, office, or individual responsible for enforcing the requirements of a code or standard, or for approving equipment, materials, an installation, or a procedure. |
| Back-layering | The movement of smoke and hot gases counter to the direction of the ventilation airflow. |
| Critical Velocity | The minimum steady-state velocity of the ventilation airflow moving toward the fire, within a tunnel or passageway, that is required to prevent back-layering at the fire site. |
| Point of Safety | For road tunnels […], an exit enclosure that leads to a public way or safe location outside the structure, or an at-grade point beyond any enclosing structure, or another area that affords adequate protection for motorists. |
| Engineering Analysis | An analysis that evaluates all factors that affect the fire safety of a facility or a component of a facility |

Water based fixed fire-fighting systems (FFFSs) are not intended or designed to extinguish a fire but to suppress the growth to a design peak HRR. Although a more appropriate reference to such a system is considered to be a water based fixed fire suppression system (FFSS), it is noted that NFPA 502 defines a fixed water based FFFS under Clause 3.1 as "*a system permanently attached to the tunnel that is able to spread a water-based extinguishing agent in all or part of the tunnel*"; with additional clarification offered in Annex A.3.3.30: "*this term includes sprinkler systems, water spray systems, and water mist systems*". Therefore, in compliance with NFPA terminology, the report will make reference to such systems as a water based FFFS.

## 4.3. Tunnel Category

NFPA 502 provides guidance relating to the equipping of the tunnels in relation to their length and traffic flow. The dataset of project traffic flows presented in Section 3.3 of this report has identified a peak hourly traffic for a winter Saturday of 1,323 vehicles per hour per lane (based on 3-lane configuration). Considering a tunnel length of approximately 2,200ft (≈ 670m), the tunnel would be classified as Category C in accordance with NFPA 502 Figure A.7.2, reproduced below in Figure 4.1.

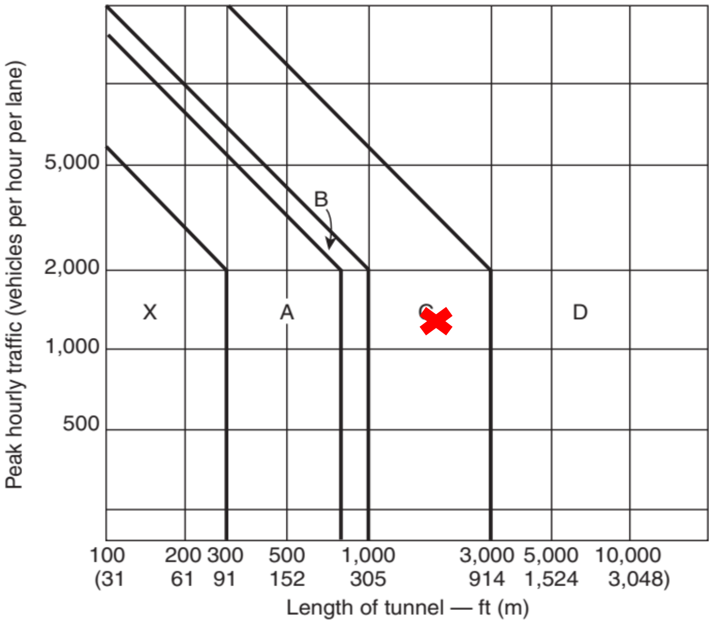


Figure 4.1 - Floyd Hill Tunnel Category (reproduced from NFPA 502)

NFPA 502 Table A.7.2 defines the minimum system provisions required for a Category C tunnel, with immediate objectives being to safely evacuate tunnel occupants and protect the tunnel structure. The report discusses further the key tunnel systems that will need to be considered, reiterating the limitations of clause interpretations and compliance checks referenced in Section 4.1.

## 4.4. Protection of Structural Elements

NFPA 502 Clause 7.3.4 specifies the following performance criteria in the event of a fire within a tunnel:

- '*The concrete is protected such that fire-induced spalling is prevented*';
- '*The temperature of the concrete surface does not exceed 380°C (716°F)*';
- '*The temperature of the steel reinforcement within the concrete [assuming a minimum cover of 25mm (1in.)] does not exceed 250°C (482°F)*'.

Mitigation measures should be employed to ensure that the three conditions above are satisfied in the event of a high energy fire (e.g. a hydrocarbon fire). If no system is employed to either put out or control the heat output and possible flame impingement due to the fire, then control measures would have to be incorporated into the structural design of the tunnel lining. These measures could include insulation on the intrados (internal face) of the tunnel lining (e.g. fire proofing boards or intumescent paint) or the incorporation of fire mitigation measures into the structural lining itself e.g. use of polypropylene fibers as an admixture to control explosive spalling. Further discussion on the possible solutions for protecting the tunnel structure in the event of fires are provided in Section 14.2.

It is important to highlight the context of Clause 4.3 where "*the requirements for ensuring human safety during the evacuation and rescue phases are substantially different from the requirements to protect the structural components of the facility and shall be separately defined*". Therefore, the design fire load established for structural protection should not be confused with the design fire load for fire life safety that defines amongst others the strategies and equipment of the tunnel ventilation system.

## 4.5. Operational Control Centre

NFPA 502 does not explicitly state that an Operational Control Center is mandatory; however, it is inferred from NFPA 502 Clause 7.2 (requiring that for tunnels in categories B through D, all requirements of NFPA 502 should be applied unless stated otherwise). In NFPA 502 Section 13.5, there is a requirement for an Operational Control Center. This has been interpreted as that the tunnel should have one. In addition, NFPA 502 Clause 13.5.6 requires that alternative location(s) shall be provided in the event that the Operational Control Center for any reason is unavailable.

## 4.6. Fire Alarm and Detection

NFPA 502 Clause 7.4 requires at least one manual means of identifying and locating a fire by the use of manual fire alarm boxes or emergency telephones. Discussions with Denver Fire Department on the Central 70 Project have revealed that manual fire alarm boxes are not favored on Floyd Hill Tunnel due to false alarms experienced in other projects. It is also understood that these boxes are likely to be removed from the next update of NFPA 502. It is therefore proposed that the tunnel be equipped with emergency telephones only, with recommendations to discuss this option with the AHJ.

NFPA 502 Clause 7.4 requires that if the tunnel is not supervised 24 hours a day or if a FFFS is installed, then an automatic fire detection system shall be installed. It is therefore recommended that these are discussed with the AHJ. NFPA Clause 7.4.3 states that CCTV can be used to identify and locate fires if the tunnel is supervised 24 hours a day. Both automatic fire detection and CCTV are CMRs; therefore, the engineering analyses performed in Section 8 of this report will address the requirements of such systems for Floyd Hill Tunnel.

A fire alarm panel control panel is required, according to NFPA 502 Clause 7.4.8.

## 4.7. Emergency Communication Systems

NFPA 502 Clause 7.5 states that two-way radio communication enhancement systems shall be installed where required by the AHJ. In view of the remoteness of the site and likely poor cellular coverage in the area due to the mountainous surrounding terrain, it is recommended that two-way radio communications systems are provided in the tunnel.

The explanatory material found in NFPA 502 Annex A (specifically Annex A.7.5), states that the Highway Advisory Radio and re-broadcasting of AM/FM commercial radio with overrides can be provided. It is therefore recommended that this is discussed with the AHJ.

## 4.8. Tunnel Closure and Traffic Control

NFPA 502 Clauses 7.6.1 and 7.6.2 place a number of requirements on tunnel operators. Clause 7.6.1 requires all tunnels, regardless of length, to have a means to stop traffic entering the tunnel. As a result, the tunnel shall have Lane Use Signs at the tunnel portal. Clause 7.6.2 is broken into three separate requirements:

1. Traffic on the direct approaches to the tunnel shall be closed. In this case, the closest off-ramp is 1400ft to the east of the portal and it would provide an ideal location to close the direct approach to regular traffic. It is therefore recommended that additional signals be provided at this location.
2. Means be provided to allow traffic in the tunnel, upstream of an incident, to be stopped. Lane Use Signs shall therefore be installed at regular intervals throughout the tunnel.
3. Means be provided downstream to expedite the flow of traffic away from the tunnel in the event of an incident. As the I-70 downstream of the tunnel is a free flow, grade separated, road with no significant flows of traffic joining or leaving the road downstream of the tunnel, there is no recommended treatment required to meet this requirement.

## 4.9. Fire Apparatus

NFPA 502 Annex K provides information regarding the provision of fire apparatus. The Fire Department should provide fire trucks suitable for fighting the fire for 30 minutes. Water should be made available during this 30 minutes to extend the period by a further 45 minutes. This can be either by the Fire Department or the tunnel facility. It is therefore recommended that this is discussed with the AHJ.

## 4.10. Standpipe and Water Supply

NFPA 502 Section 10 requires Class I standpipes to be "*either wet or dry depending on the climatic conditions, the fill times, the requirements of the authority having jurisdiction, or any combination thereof*". In view of the climatic conditions anticipated and the risk of freezing, it is possible that the standpipe be a dry system, or a wet system insulated and trace heated, subject to future engineering design development.

Fire hydrant hose connections should be provided such that no location on the protected roadway is more than 150ft from the connection and a maximum spacing of 275ft, or less if required by the AHJ. Water supplies capable of providing the system demand for 1 hour should be provided. If the required supply is not available, local tanks and/or pumps will be required.

## 4.11. Portable Fire Extinguishers

Portable fire extinguishers should be provided in wall cabinets at intervals not greater than 300ft.

## 4.12. Tunnel Drainage Systems

The tunnel rises to the West with no proposed low point. The tunnel drainage design will need to consider tanker spillage, fire-fighting water, cleaning waste and ingress water. Drainage retention tanks may be required along with a pumping station.

## 4.13. Alternative Fuels

NFPA 502 Annex G.1 highlights that "*most vehicles currently used in the United States are powered by either spark-ignited engines (gasoline) or compression-ignited engines (diesel). Vehicles that use alternative fuels such as compressed natural gas (CNG), liquefied petroleum gas (LP-Gas), and liquefied natural gas (LNG) are entering the vehicle population, but the percentage of such vehicles is still not large enough to significantly influence the design of road tunnel ventilation with regard to vehicle emissions. However, it is possible that growing concerns regarding the safety of some alternative-fuel vehicles that operate within road tunnels will affect the fire-related life safety design aspects of highway tunnels*".

Currently there is insufficient research from international programs and directives to dictate a change in patterns of peak HRRs or airborne toxic particles from a burning vehicle with alternative fuel (most tests are based on gasoline passengers cars). But given there will be no restrictions on vehicles in Floyd Hill Tunnel, these issues will require consideration in the next design stage.

## 4.14. Control of Hazardous Materials

The drainage system will naturally remove some of the combustible or flammable liquids on the tunnel floor; however, it will be not considered as a means of reducing the design peak HRR as offered by NFPA 502 Clause 7.14.1, due to significant uncertainties in such an event. The drainage system and tunnel gradients will be suitably aligned to allow fuel or spillage to be naturally removed from the tunnel roadway.

## 4.15. Flammable and Combustible Environmental Hazards

No naturally occurring and constructed environmental hazards from outside Floyd Hill will be considered for the purposes of this feasibility study.

## 4.16. Means of Egress

NFPA 502 Clause 7.16.6.1 states that "*emergency exits shall be provided throughout the tunnel*", with minimum spacing defined by a number of conditions that are normally captured in risk assessment and emergency response exercises. NFPA 502 Clause 7.16.6.6.2 outlines these conditions while clearly stating that the maximum "*spacing between exits for protection of tunnel occupants shall not exceed 300m (1000ft)*".

NFPA 502 Clause 3.3.20 offers a definition of emergency exits as "*doors, egress stairs, or egress corridors leading to a point of safety*"*.* The definition of a point of safety is provided under NFPA 502 Clause 3.3.44 (specifically for road tunnels, bridges and elevated highways) as "*an exit enclosure that leads to a public way or safe location outside the structure, or an at-grade point beyond any enclosing structure, or another area that affords adequate protection for motorists*". Therefore, the tunnel options will consider tunnel/egress configurations such as cross passages leading to an adjacent non-incident tunnel and portals, dedicated egress corridors within or adjacent to the core tunnel area, with all intervening or terminal areas along the vertical or horizontal travel leading towards a point of safety.

For options with egress corridors (including cross passages) as emergency exits, a pressurization system shall be provided in accordance with NFPA 502 Clauses 7.16.6.5 and 7.16.5.6, to demonstrate that the tandem operation of the pressurization and tunnel ventilation systems will result in a force "*to open the doors fully […] as low as possible but shall not exceed 50lb. This opening force shall not be exceeded under the worst-case ventilation differential pressure*".

In addition, and under NFPA 502 Clause 7.16.6.7, "*where cross-passageways are used as an emergency exit, provisions shall be included that stop all traffic operation in the adjacent tunnel*".

Considering that a longitudinal tunnel ventilation system for a unidirectional tunnel provides a clear egress path upstream with an empty tunnel downstream of the fire location as described in NFPA 502 Section 20, a possible interpretation of Clause 3.3.44 could be that tunnel occupants upstream of a fire location are already at a point of safety that also consists the emergency exit towards the portal. On this basis, it can be suggested that people upstream of the fire would be able to escape through the tunnel roadway itself without the need for egress provisions outside of the tunnel structure; an interpretation that is supported by NFPA 502 Clause 7.16.6.3.1, where "*the tunnel roadway surface, when supported by a traffic management system, shall be considered as a part of the egress pathway*". Nevertheless, as this is one possible interpretation of NFPA 502 which may cause significant impacts and potential risks, especially for temporary bidirectional traffic through the tunnel, it is only highlighted for information purposes to allow discussion with the AHJ and relevant stakeholders. Conclusively, such an option will be considered as non-compliant and emergency exit requirements as per NFPA 502 will be considered for the purposes of the feasibility study.

## 4.17. Fixed Fire-Fighting Systems

A key objective for the fire-fighting system is to suppress a fire of potential to grow to the design fire size, to an extent that provides conditions that are manageable by the tunnel ventilation system and suitable for effective self-rescue. The operating principles for a fire-fighting system should be such that the system is activated as early as practicable after fire detection.

A fixed water-based FFFS is a CMR for Category C tunnels with the primary objective to "*mitigate the impact of fire to improve tenability for tunnel occupants during a fire condition, enhance the ability of first responders to aid in evacuation and engage in manual fire-fighting activities, and/or protect the major structural elements of a tunnel*", according to NFPA 502 Clause 9.2.1.

In terms of fire life safety, the unidirectional mode of Floyd Hill Tunnel and especially the tunnel ventilation strategy presented in Section 10, provides a tenable environment upstream of the fire location that allows tunnel occupants to safely escape the tunnel even without a FFFS, using the appropriate egress paths available across the tunnel length.

The decision to include or exclude a FFFS needs careful consideration due to fundamental impacts on tunnel design and operation, deployment under winter conditions of people in the spray zone, MEP systems and capital and operational costs. This report notes CDOT's position during the project kick-off meeting to consider FFFS in the feasibility study, and is therefore addressed in Section 9. It is important that key stakeholders, the AHJ and the participating civil agencies are consulted, with the reasons behind the chosen option being recorded prominently and shared with relevant parties.

## 4.18. Emergency Ventilation

### 4.18.1. Overview

NFPA 502 requirements for an emergency situation involve a fire incident and evacuation of tunnel occupants; therefore, pollution control and control of accumulated noxious gases for non-emergency situations do not form part of NFPA 502 requirements. Nevertheless, in-tunnel pollution control is addressed in Section 10**,** in line with international guidance and best design practices. Although NFPA 502 requires an emergency ventilation system as conditionally mandatory for Category C tunnels, its consideration resides on engineering judgment and analysis appropriate for the operational conditions of the Floyd Hill Tunnel.

In the event of a fire incident in a unidirectional tunnel, it is reasonable to assume that vehicles downstream (in front) of the incident will continue to travel and exit the tunnel, while those upstream (behind) the incident will come to a standstill. This assumption is supported by NFPA 502 Clause 7.6.2 on mandatory traffic control measures "*[…] to stop traffic from entering the direct approaches to the tunnel, to control traffic within the tunnel, and to clear traffic downstream of the fire site following activation of a fire alarm within the tunnel*".

This report discusses a mechanical forced longitudinal ventilation system as a common strategy for unidirectional tunnels (Section 10). Such systems achieve the above design objectives by generating a longitudinal flow in the direction of traffic, overcoming: hydraulic losses from prevailing wind; vehicle drag; tunnel/MEP service resistances; and fire pressure drop.

The system will consist of banks of jet fans strategically arranged along the tunnel length, to provide a tenable environment upstream of the fire location for a range of credible fire scenarios. The study will establish the key NFPA 502 design criterion of achieving critical velocity for the design peak HRR and to inform system specifications for MEP tunnel services and spatial requirements for the tunnel profile. The analysis also considers fan unavailability and redundancy requirements as stipulated in NFPA 502, with further guidance from international research and standards on road tunnel safety where appropriate.

### 4.18.2. Design Fire Load

The design fire size or design HRR is not necessarily the worst fire that can occur. NFPA 502 Section 11.4 identifies the basis of design of the tunnel ventilation system being governed by credible fire scenarios and design HRR, representative of the: tunnel conditions; intended usage of the tunnel; types of vehicles; and material allowed to be transported through the tunnel.

NFPA 502 Table A.11.4.1 presents experimental and representative HRR values for a variety of vehicle types. These will be considered for comparison purposes only, with results from a preliminary risk assessment and engineering judgment governing the recommendation of a design peak HRR, based on the probability of occurrence and the ability to achieve practical solutions for the tunnel ventilation system. This engineering analysis will consist of a fire frequency assessment to establish the probability of a fire incident and a selection of a representative peak HRR, based on available and appropriate statistical information on traffic flows and vehicle categories that are expected to use Floyd Hill Tunnel.

As per Section 4.4, the requirements for life safety are different to those for structural protection – therefore the respective design HRR should not be confused.

### 4.18.3. Compliance Criteria

In principle the tunnel ventilation system must be capable of preventing the back-layering of smoke and providing “*a tenable environment […] in the means of egress during the evacuation phase in accordance with the emergency response plan for a specific incident”* [NFPA 502, Clause 7.16.2], whilst maintaining reasonable conditions post-evacuation for the intervention and rescue operations. For a tunnel with unidirectional traffic, the ventilation system will be required to control smoke for an incident “*where motorists are likely to be located upstream of the fire site”* [NFPA 502, Clause 11.2]. The basis of the ventilation strategy for the longitudinal system described in Section 10 is determined by the primary criteria summarized in Table 4.2.

Table 4.2 - Key Compliance Criteria for Longitudinal Ventilation System

| NFPA 502 Clause | Description |
|---|---|
| 11.2.4.1(a) | Prevent back-layering by producing a longitudinal air velocity that is calculated on the basis of critical velocity in the direction of traffic flow. |
| 11.2.4.1(b) | Avoid disruption of the smoke layer initially by not operating jet fans that are located near the fire site. Operate fans that are farthest away from the site first. |
| 11.3.2 | Longitudinal airflow rates are produced to prevent back-layering of smoke in a path of egress away from a fire. |
| 11.3.1 | A stream of noncontaminated air is provided to motorists in path(s) of egress in accordance with the anticipated emergency response plan. |
| 11.4.3 | Failure or loss of availability of emergency ventilation equipment shall be considered. |
| 11.5.2 | Tunnel ventilation fans, such as jet fans, that can be directly exposed to fire within the tunnel roadway shall be considered expendable. |
| 11.5.3 | The design of ventilation systems where fans can be directly exposed to a fire shall incorporate fan redundancy. |

## 4.19. Electrical Systems

NFPA 502 Clause 12.4 requires the electrical systems for critical systems to be provided with emergency power to ensure that they are operational in the event of a fire or a power failure. Enquiries are being made of the utility company to see if redundant power supplies are available in the area. If they are not available, a generator will need to be provided.

## 4.20. Emergency Response

The importance of active engagement with participating agencies and tunnel operators for the development of emergency response plans is emphasized in NFPA 502 Clause 13.4 for a range of typical incidents (NFPA 502 Clause 13.2), that do not necessary involve fire emergencies but may still result in deficiencies in the operation of the tunnel (e.g. vehicle collision, non-fire casualties, extreme weather conditions) and intervention by civil services. Although the emergency response plan reflects the required operation of the in-tunnel systems for a variety of incidents, its development is a collective effort that is performed as the design matures.

The systems investigated under this feasibility study will capture the principles that constitute an emergency response plan (e.g. detection, evacuation, access) to allow preliminary discussions with the relevant stakeholders, the AHJ and the appropriate authorities in establishing an initial response plan, while forming a basis on operational aspects of the proposed systems for the next design stage.

# PART C – Tunnel Configuration Options & Initial Evaluation

# 5. Tunnel Configuration Options

A total of nine configuration options were considered for Floyd Hill Tunnel. The configuration options were formed using a generic ‘D’ shaped tunnel structure, with vertical side walls, an arched roof and a flat invert. These configuration options are described below in Table 5.1, Table 5.2 and Table 5.3; and are shown in detail in Appendix B.

Table 5.1 - Tunnel Configuration Options

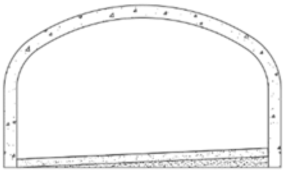
| Tunnel Layout | Description |
|---|---|
| Option 1 | Single bore, no specific refuge or egress provisions |
| Option 2 | Single bore, dedicated refuge chambers, no specific egress provisions |
| Option 3 | Single bore, dedicated egress route provided by a compartmentalized corridor within the main highway tunnel |
| Option 4 | Single bore, dedicated egress route provided by tunnels mined perpendicular to the main highway tunnel |
| Option 5 | Single bore, dedicated egress route provided by tunnel mined parallel and on the same level as the main highway tunnel |
| Option 6 | Single bore, dedicated egress route provided by tunnel mined parallel and beneath the main highway tunnel |
| Option 7 | Single bore with two segregated highway compartments provided by an internal dividing wall (egress is through evacuation into the other highway compartment) |
| Option 8 | Twin bore, with interconnecting passages (egress is through evacuation into the other bore) |
| Option 9 | Twin bore, with interconnecting passages and a dedicated egress tunnel located between the two bores |

Table 5.2 - Principal Features of the Tunnel Configuration Options

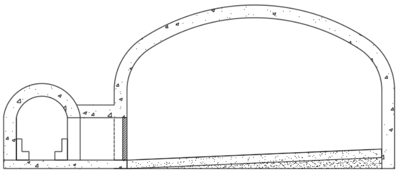
| Tunnel Layout | Tunnel Configuration | | Tunnel Operational Life Safety Provision | | |
|---|---|---|---|---|---|
| | | | | Egress Route | |
| | Single bore | Twin bore | Refuge chamber | Dedicated pedestrian tunnel or isolated pedestrian walkway | Adjacent highway tunnel or segregated highway cell |
| Option 1 | ▨ | | | | |
| Option 2 | ▨ | | ▨ | | |
| Option 3 | ▨ | | | ▨ | |
| Option 4 | ▨ | | | ▨ | |
| Option 5 | ▨ | | | ▨ | |
| Option 6 | ▨ | | | ▨ | |
| Option 7 | ▨ | | | | ▨ |
| Option 8 | | ▨ | | | ▨ |
| Option 9 | | ▨ | | | ▨ |

Table 5.3 - Schematic Drawings of the Tunnel Configuration Options (layouts are illustrative only)

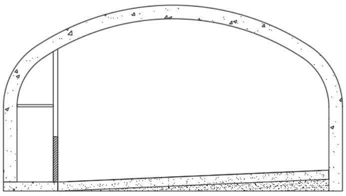
**Option 1**



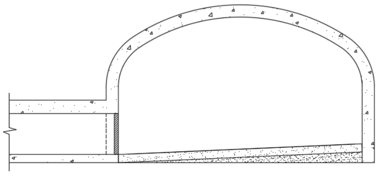
**Option 2**



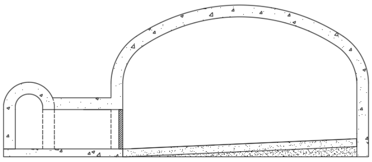
**Option 3**



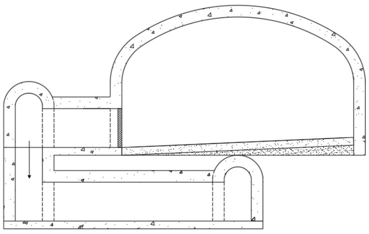
**Option 4**



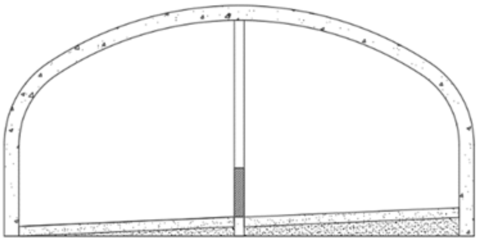
**Option 5**



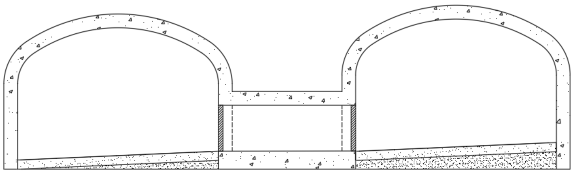
**Option 6**



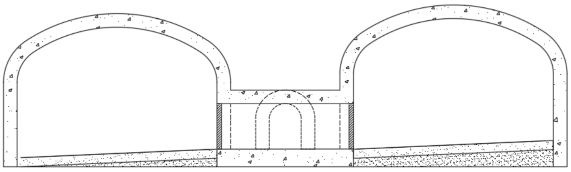
**Option 7**



**Option 8**



**Option 9**



# 6. Initial Evaluation of Tunnel Configuration Options

## 6.1. Initial Options Evaluation Criteria

Two primary criteria were used in the initial options evaluation to eliminate options from further consideration in the study. These criteria were:

- Compliance with codes and standards (principally NFPA 502);
- Commonality with other options.

### 6.1.1. Compliance with Codes and Standards

NFPA 502 Section 7.16.6 establishes the requirements for Emergency Exits in Road Tunnels. NFPA 502 states:

- '*Emergency Exits shall be provided throughout the tunnel*' [Clause 7.16.6.1];
- '*Spacing between exits for protection of tunnel occupants shall not exceed 300m (1000ft)*' [Clause 7.16.6.2].

The first evaluation criterion for the tunnel configurations was compliance with codes and standards, principally NFPA 502. Options that did not meet the minimum egress criteria established in Section 7.16.6 were discounted from further consideration in the study.

### 6.1.2. Commonality with Other Tunnel Layout Options

The nine concept layouts considered initially in the study included a certain level of commonality between options, i.e. they contained common features that made them similar from a functional perspective. Concepts were assessed against each other to identify potential areas of commonality. Where options were functionally similar/the same, a single preferred option was retained for further consideration, based on a qualitative assessment of cost and constructability.

## 6.2. Outcome of Initial Options Evaluation

### 6.2.1. Compliance with Codes and Standards

Option 1 and Option 2 both fail to comply with the requirements of NFPA 502 with respect to emergency egress provisions. Neither tunnel concept provides a dedicated means of egress from the tunnel in emergency situations, contravening the requirements of Clause 7.16.6.1: '*Emergency Exits shall be provided throughout the tunnel*'. These options were thus eliminated from further consideration in the options evaluation exercise. The refuges in Option 2 are not considered to constitute emergency exits as defined by NFPA 502.

All other options presented provide dedicated emergency exits from the tunnel and satisfy the requirements of Clause 7.16.6.1. All concepts can be developed to meet the requirements of Clause 7.16.6.2, i.e. egress points can be spaced at intervals less than 1000ft (≈ 300m). Options 3-9 were thus retained for further consideration in the options evaluation exercise.

It should be noted that whilst these options are discounted for non-compliance with NFPA 502, a risk-based approach to safety design may offer opportunity to bring these options back into consideration. This may be a consideration to discuss with the fire services and the AHJ at the appropriate time, and may include a demonstration that a place of relative safety can be provided inside the tunnel bore, upstream of the fire location by means of protection by the tunnel ventilation system. From this location, evacuees may be able to be provided a safe route of egress in the tunnel to the upstream portal. Nevertheless, for the purposes of progressing compliant options for feasibility assessment, Options 1 and 2 are not proposed for progression at this stage.

### 6.2.2. Commonality with Other Tunnel Configuration Options

For the assessment of commonality, similar concepts were grouped together and the options within these groups were tested against each other on the basis of: a) commonality of function; and b) cost and constructability (Table 6.1).

Table 6.1 - Commonality between Tunnel Configuration Options

| **Tunnel Layout** | **Tunnel Configuration** | | **Tunnel Operational Life Safety Provision** | | | **Commonality with Other Options?** |
|---|---|---|---|---|---|---|
| | | | Dedicated refuge | **Egress Route** | | |
| | Single bore | Twin bore | | Dedicated pedestrian tunnel or isolated pedestrian walkway | Adjacent highway tunnel or segregated highway cell | |
| ~~Option 1~~ | | | | | | |
| ~~Option 2~~ | | | | | | |
| Option 3 | ■ | | | ■ | | **No** |
| Option 4 | ■ | | | ■ | | **Yes** – Group 1 |
| Option 5 | ■ | | | ■ | | **Yes** – Group 1 |
| Option 6 | ■ | | | ■ | | **Yes** – Group 1 |
| Option 7 | ■ | | | | ■ | **Yes** – Group 2 |
| Option 8 | | ■ | | | ■ | **Yes** – Group 2 |
| Option 9 | | ■ | | | ■ | **Yes** – Group 2 |

A discussion on how options were eliminated from the two groups is provided below.

Group 1 (single bore tunnel with mined passage to tunnel portals/valley side used for emergency evacuation)

- **Option 4** – single bore highway tunnel with two/three perpendicular egress tunnels;
- **Option 5** – single bore highway tunnel with parallel egress tunnel, adjacent to main tunnel;
- **Option 6** – single bore highway tunnel with parallel egress tunnel, underneath main tunnel.

While Options 5 and 6 utilize a mined egress tunnel that parallels the main highway tunnel, Option 4 utilizes perpendicular tunnels. This changes the emergency escape strategy as the escape tunnels would lead to sites of safety on the sides of Sawmill Gulch, rather than at the Floyd Hill Tunnel portals. The overall length of mining required with the Option 4 solution could also be significantly different from the Option 5 and 6 solutions. For these reasons Option 4 was considered to utilize a sufficiently distinct solution from Options 5 and 6 and it has been retained for further consideration in the study.

Option 5 and Option 6 provide very similar solutions from a functional perspective. For Option 6, the egress tunnel is positioned beneath the main highway tunnel. This requires passengers to transition between vertical levels when escaping from the main highway tunnel. Option 6 requires a larger excavation size to allow the egress tunnel to be constructed in the same advance face as the main highway tunnel. It is likely to be more expensive to construct than the Option 5 solution and its overall construction duration could be longer. Passages connecting the egress tunnel to the main highway tunnel are relatively long given restrictions of minimum pillar widths, maximum gradients of ramps, levels of risers with stairs, further adding to the cost and complexity of the construction. For these reasons, Option 5 was deemed to provide a more efficient solution than Option 6 and as such Option 6 was eliminated from further study.

Group 2 (twin highway tunnels/compartments)

- **Option 7** – single bore tunnel with internal wall used to form two highway cells;
- **Option 8** – twin bore tunnel with connecting cross passages;
- **Option 9** – twin bore tunnel with connecting cross passages and dedicated egress route accessed through connecting cross passages.

Option 7 and Option 8 are functionally very similar. They both provide two highway compartments, with evacuation in emergency conditions being through the passage into the other compartment (a place of relative safety). For both Option 7 and 8, an incident in one compartment requires temporary closure of the other compartment to allow people to evacuate safely by foot without facing hazards from moving vehicles. From a cost and constructability perspective, Option 8 is favorable, as the total excavated volume for Option 8 is likely to be significantly lower and the two tunnels can be driven in parallel – factors which would allow the Option 8 tunnel to be constructed quicker and for a lower cost than with the larger single bore tunnel. Option 7 may also present a challenge from a constructability perspective due to the size of excavation required for the large single bore. For these reasons Option 7 was eliminated from further investigation in the study.

In Option 9 a dedicated egress tunnel is provided between the two tunnel bores and evacuation in emergency situations would be through this tunnel. Although it is likely that the non-incident tunnel would still be closed in an emergency scenario, a dedicated egress tunnel would still provide additional operational benefits. The egress tunnel provides a safe place for emergency phones in case of vehicle breakdowns within the highway tunnels and it also provides an accessible place for mechanical and electrical equipment outside of the tunnel bore, potentially offering significant operational benefits and cost savings.

Option 9 is substantially more expensive to build than Option 8, given that it requires the construction of a 3rd tunnel (albeit one that it is much smaller than the main highway tunnels). To ensure stability of the rock excavation around the egress tunnels the main tunnels must maintain a moderate amount of separation from the main highway tunnels (minimum pillar width). This makes the overall width of the tunnels arrangement relatively high, which could cause significant issues with the connections of the roads into the surface network at either of the portal locations. The additional benefit of having a dedicated egress and mechanical & electrical (M&E) systems tunnel is not considered sufficient to justify the increased cost and additional constructability issues associated with Option 9. For this reason, Option 9 was eliminated from further consideration in the study, and Option 8 was retained for further investigation.

### 6.2.3. Options Retained for Further Consideration

The options evaluation process resulted in a total of five tunnel layout concepts being eliminated from further consideration in the study. A summary of the options evaluation process is shown below in Table 6.2.

Table 6.2 - Results of Initial Tunnel Configuration Options Evaluation

| **Tunnel Layout** | **Tunnel Configuration** | | **Tunnel Operational Life Safety Provision** | | | **Retained for Further investigation?** |
|---|---|---|---|---|---|---|
| | | | Dedicated refuge | **Egress Route** | | |
| | Single bore | Twin bore | | Dedicated pedestrian tunnel or isolated pedestrian walkway | Adjacent highway tunnel or segregated highway cell | |
| ~~Option 1~~ | | | | | | No – not compliant with NFPA 502 |
| ~~Option 2~~ | | | | | | No – not compliant with NFPA 502 |
| Option 3 | ▨ | | | ▨ | | **Yes** |
| Option 4 | ▨ | | | ▨ | | **Yes** |
| Option 5 | ▨ | | | ▨ | | **Yes** |
| ~~Option 6~~ | | | | | | No – high level of commonality with Option 5 |
| ~~Option 7~~ | | | | | | No – high level of commonality with Option 8 |
| Option 8 | | | | | | **Yes** |
| ~~Option 9~~ | | | | | | No – high level of commonality with Option 8 |

A total of four tunnel configuration options were thus carried forward from the initial evaluation process and will be taken forward for further quantitative consideration in the feasibility study. These options are:

- **Option 3** – single bore, dedicated egress route provided by a compartmentalized corridor within the main highway tunnel;
- **Option 4** – single bore, dedicated egress route provided by tunnels mined perpendicular to the main highway tunnel;
- **Option 5** – single bore, dedicated egress route provided by tunnels mined parallel and on the same level as the main highway tunnel;
- **Option 8** – twin bore, with interconnecting cross passages (egress is through evacuation into the other bore).

For clarity in the next stage of assessment these options are renamed as:

- **Option A** (previously Option 3) – single bore, dedicated egress route provided by a compartmentalized corridor within the main highway tunnel;
- **Option B** (previously Option 4) – single bore, dedicated egress route provided by tunnels mined perpendicular to the main highway tunnel;
- **Option C** (previously Option 5) – single bore, dedicated egress route provided by tunnels mined parallel and on the same level as the main highway tunnel;
- **Option D** (previously Option 8) – twin bore, with interconnecting cross passages (egress is through evacuation into the other bore).

Each option is fundamentally different in terms of its configuration and/or its operational life safety provision. To help differentiate between the options, a plan view sketch of each option showing main highway tunnel(s), approximate portal locations, cross passages and egress tunnels (if applicable), including approximate distances for each, has been provided (Figure 6.1). The distances provided are subject to change and will be finalized in the subsequent design stages of the project once the portal locations have been set.

The sketches for Option B show two possible routes for the egress tunnels: the standard option requires two portals to be excavated at the end of each egress tunnel, whilst the alternative brings each egress tunnel out to the same point at Sawmill Gulch (i.e. on egress portal required). For the purposes of this feasibility study, the standard option (two separate egress portals) has been considered throughout the remainder of this report. If it is found that Option B is the preferred tunnel choice, further investigation may be required to understand the potential benefits of the alternative option – one egress portal rather than two.

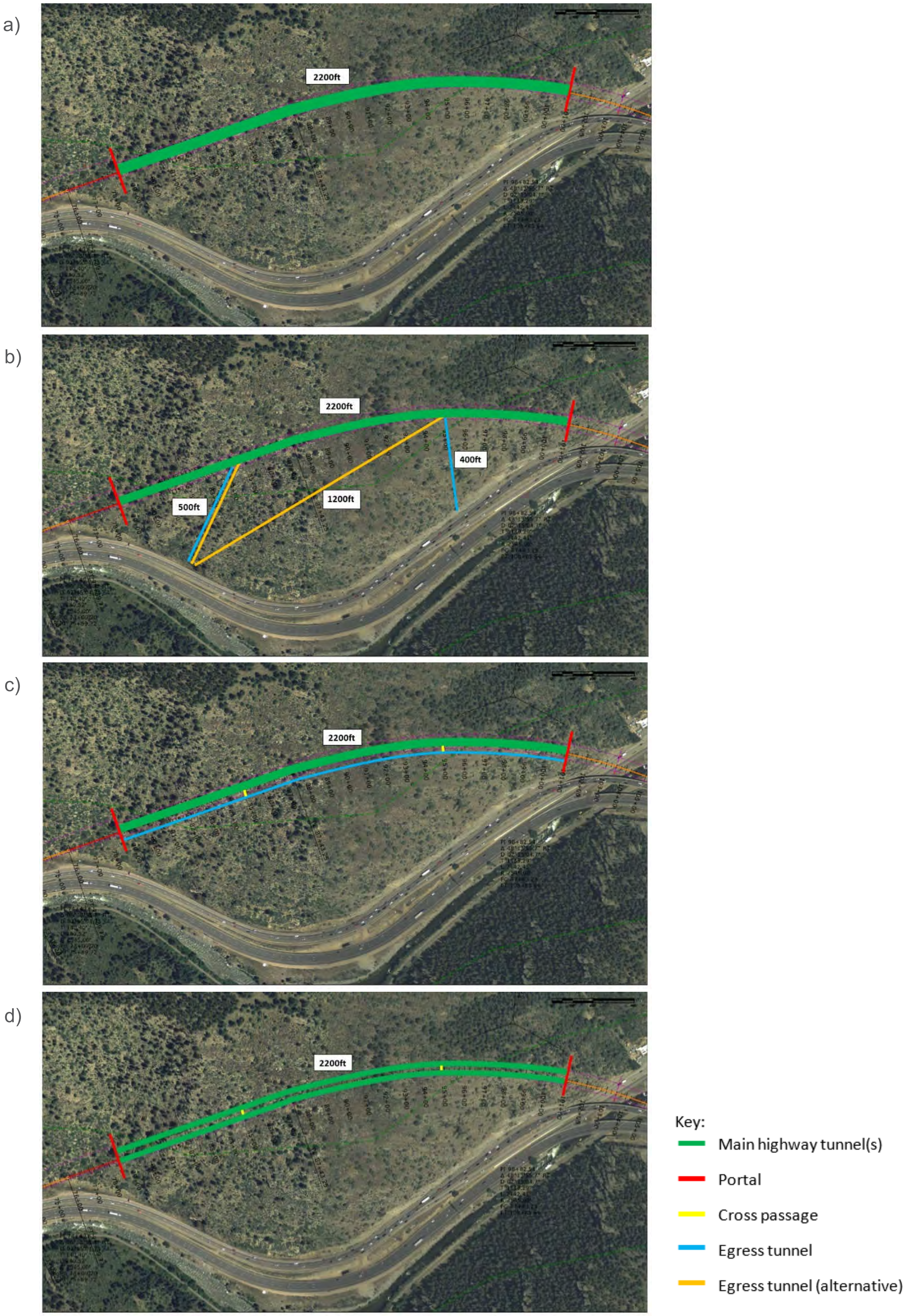


Figure 6.1 - Plan View Sketch for: a) Option A; b) Option B; c) Option C; and d) Option D

# PART D – Fire Risk Assessment and Tunnel Ventilation

# 7. General Considerations

## 7.1. Overview

The design fire size (peak HRR) provides the fire characteristic used to inform the design, sizing and configuration of the tunnel fire safety equipment and facilities, based on the operational design parameters described in Section 3 of this report.

In determining the design maximum fire heat release rate (FHRR) for fire life safety and tunnel ventilation systems, it is necessary to consider the types of vehicles that are expected to use the tunnel including what materials those vehicles will be allowed to transport through the tunnel. If appropriate, the design FHRR may be reduced through the implementation of operational measures or alternative safety systems, such as dangerous goods management or through the introduction of a FFFS to suppress the growth and the potential maximum FHRR. For the purposes of this feasibility study, the potential maximum design FHRR for the ventilation system sizing will be assessed based on the following parameters:

- Historical precedence;
- Requirements of standards;
- The expected usage of the tunnel as forecast through the traffic profile and vehicle categories;
- Initial probabilistic assessment of fire likelihood.

For incidents where the cause of the fire is due to an accident, typically more than one vehicle is involved. In such events, the fire size is likely to be larger and depends on the composition of the vehicles involved in the incident. Nevertheless, this does not mean that the fire size will never be large in the case of a vehicle breakdown, and conversely will never be small in the case of an accident. The above information is only intended to provide context on the diversity of circumstances that can be reasonably expected.

## 7.2. Historical Precedence

During the mid-1990s design fires for HGVs were typically taken as 68MBTU/hr (≈ 20MW), as given in standards applicable at the time. The tunnel fire events in European tunnels [Channel Tunnel (1996), Isola delle Femmine motorway (1996), Mont Blanc (1999), St Gotthard (2001), etc.] during the mid-1990s and early 2000s led to a series of research and development projects (including a number of full-scale fire tests). These have resulted in modified guidance and changes to the measures used to manage and mitigate the hazards from fire and smoke in tunnels. There are two important areas of improved understanding: the potential magnitude of peak HRRs discussed in this section; and enhanced technologies for fire protection and safety, such as fire detection, fire-fighting and evacuation systems discussed separately in the report.

## 7.3. References to Standards

### 7.3.1. NFPA 502

NFPA 502 is not prescriptive in its specification of design fire but states that the defined fire scenario for emergency ventilation will be based on the operational risk associated with the type of vehicles expected to use the tunnel. NFPA 502 refers to test data from HGV fire tests [3] reaching average peak HRRs of up to 683MBTU/hr (≈ 200MW). NFPA 502 states that the design fire is not necessarily the worst fire that may occur, but that engineering judgement is required to be used to establish the probability of occurrence and the ability to achieve practical solutions.

### 7.3.2. International Guidance of PIARC (World Road Association)

In 1999, the World Road Association (PIARC) [4] recommended a tunnel design fire size of 102MBTU/hr (≈ 30MW). However, that report acknowledged that significantly higher HRRs from HGVs and vehicles with dangerous goods are possible based on the experimental measurements undertaken during the EUREKA fire tests with fire loads up to 410MBTU/hr (≈ 120MW). The HRRs from fires arising from petrol tanker fuel spills were considered to be in the range of 683-1,024MBTU/hr (≈ 200-300MW).

Further, published work by PIARC [5] explores design fire criteria in some depth, concluding that events and tests in recent years have shown fire sizes in excess of those generally found in codes and standards. The report notes that solid fuel fires can potentially be even more severe than liquid pool fires resulting from a single tanker incident, depending on the drainage provisions; and that a fire will become difficult for the fire service to approach once it reaches between 170MBTU/hr (≈ 50 MW) and 341MBTU/hr (≈ 100MW).

### 7.3.3. NCHRP 2011

In 2011, the National Cooperative Highway Research Program in the U.S. published a report on 'Design Fires in Road Tunnels', reflecting the state of knowledge and practice. No prescriptive way of defining fire size is proposed, but it highlights that the fire growth rate is more important than the peak HRR when investigating the safety of people in road tunnels. This is true where people may reasonably be expected to have evacuated before the fire reaches its peak HRR. The report also highlights that the design peak HRR may be influenced by the effects of tunnel geometry and the presence of FFFS.

## 7.4. Current Practice

Selection of international and national practices are shown in Table 7.1. This data provides a view of fire sizes internationally, as adopted in projects and standards. Data may not be a full summary of current practice, following more recent fire tests, incidents, data and industry knowledge, but nevertheless serves as a useful summary of practices worldwide. It is evident that the choice of a design fire varies widely from 102 to 1,024MBTU/hr (≈ 30 to 300MW), depending on the type of traffic in the tunnel and in some cases the strategy of the ventilation system – with design fire loads possibly higher than those previously prescribed by codes and standards. This observation only reinforces the argument that the assessment of the design fire criterion needs to be tailored to the specific conditions of Floyd Hill Tunnel, with international practices used for reference only.

Table 7.1 - Guidance on Fire Sizes Adopted Internationally

| **Country** | **Design Fire** | | **Additional Information** |
|---|---|---|---|
| | MBTU/hr | MW | |
| Australia | 170 | 50 | With FFFS (deluge system), for ventilation |
| Austria | 102 | 30 | High risk category: 170MBTU/hr (≈ 50MW) |
| France | 102 - 683 | 30 - 200 | 683MBTU/hr (≈ 200MW) when transport of dangerous goods allowed but only applied for longitudinal ventilation |
| Germany | 102 - 341 | 30 - 100 | Depending on length and HGVs in tunnel |
| Greece | 341 | 100 | Longitudinal ventilation |
| Italy | 68 - 683 | 20 - 200 | |
| Japan | 102 | 30 | |
| Netherlands | 341 - 683 | 100 - 200 | 341MBTU/hr (≈ 100MW) if tankers are not allowed, otherwise 683MBTU/hr (≈ 200MW). For ventilation system |
| Norway | 68 - 341 | 20 - 100 | Depending on risk class, always longitudinal ventilation |
| Portugal | 34 - 341 | 10 - 100 | Based on traffic type |
| Russia | 170 - 341 | 50 - 100 | |
| Singapore | 102 - 683 | 30 - 200 | Depends on vehicle types allowed |
| Spain | ≥ 102 | ≥ 30 | |
| Sweden | 341 | 100 | Longitudinal ventilation [170MBTU/hr (≈ 50MW) with FFFS under some circumstances] |
| Switzerland | 102 | 30 | Smoke extraction equals 590-670fpm (≈ 3.3-4m/s) times cross section |

# 8. Assessment of Likelihood of Fire

## 8.1. Methodology

The assessment method for the likelihood of tunnel fire incidents used within this feasibility study is based on that presented in the output of the major research program 'Durable and Reliable Tunnel Structures' (DARTS) by the European Thematic Network program. The base data for the DARTS study is derived from an analysis of tunnel fire events to provide the best possible up-to-date information in terms of real fire scenarios in tunnels. This method takes into consideration registered real fire scenarios, rather than trying to derive a range of possible theoretical causes and effects as considered by other methods.

The DARTS method uses the information above in a detailed analysis to determine frequencies for a range of vehicle fires. The methodology acknowledges that only a small proportion of fires develop into serious fires. Most serious fires start in trucks, with only a small percentage starting in cars. Of these serious fires a proportion can go out of control, which is defined by DARTS as an event with duration of more than five hours (i.e. fire spread to other vehicles).

The approach taken for Floyd Hill Tunnel study is to derive a likelihood 'event tree' for a representative range of scenarios that are defined, considering:

- Traffic characteristics and volumes;
- Fire probability;
- Severity of fire (small, moderate, large/serious fire);
- Whether fires are in or out of control (for serious fires).

The classification of the severity of fire and its potential of being out of control has a vital role in the assignment of representative peak HRRs depending on the vehicle category.

## 8.2. Fire Rate Statistics

The expected fire rates (fire/vehicle miles) are obtained from NFPA and Federal Highway Administration (FHWA) statistical data [6,7,8] for travelled distance for cars, buses, HGVs and for Dangerous Goods Vehicles (DGVs) [2] and summarized in Table 8.1.

Table 8.1 - Estimated Fire Rates per Vehicle Category for US

| | **Car** | **Buses** | **HGV** | **DGV** | **Comments** |
|---|---|---|---|---|---|
| Highway vehicle fires | 174,000 | | | | [6] For Year 2015 |
| Fire incidents | 91.12% | 0.70% | 7.28% | - | [7,8] |
| Fires/year | 158,549 | 1,218 | 12,668 | - | (e.g.= 174,000 x 91.12%)<br>Estimated for 2015 |
| Travelled distance in million miles/year (million km/year) | 2,849,718<br>(4,586,177) | 16,350<br>(26,312) | 287,895<br>(463,322) | - | [4] |
| Fire rate in fire per vehicle-mile (fire per vehicle-km) | 5.56E-08<br>(3.46E-08) | 7.45E-08<br>(4.63E-08) | 4.40E-08<br>(2.73E-08) | 1.64E-09<br>(1.02E-09) | = fires/year/travelled distance |

## 8.3. Severity of Fire Relative to Vehicle Categories

The expected rate of serious fires [≥ 51MBTU/hr (≈ 15MW)] for large vehicles (HGVs and buses) is estimated as 2.55% by DARTS [9]. For smaller vehicle category fires this figure of serious fires is taken as 0.23%. HGV fires greater than 51MBTU/hr (≈ 15MW) may be broken down further into categories of reducing likelihood with increasing fire size. The probability that a serious HGV fire will develop into a relatively high peak HRR will depend on whether the HGV is carrying a load, the likelihood of the fire involving the load and the probability that the load is combustible. Simplifying assumptions need to be made to generate broad estimates of possible fire return periods for the purposes of the fire frequency analysis.

The percentage of HGV serious fires that further develop into more severe fires is rooted on assumptions of the Transport Research Laboratory (TRL) report [10], where it is considered that 50% fires involving the fire load, and that 67% of combustible products are carried by HGVs. With a percentage of 72% of laden HGVs (i.e. 28% of HGVs run empty) a percentage of 24% (50% x 67% x 72%) is further applied to the 2.55% (resulting in 0.62% for HGVs) of serious fires to further derive the more severe fire categories 239MBTU/hr (≈ 70MW) onwards. The remaining 1.93% (2.55-0.62%) remains as fire sizes of greater than 51MBTU/hr (≈ 15MW) fires. There is a degree of judgement and subjectivity in this categorization, but the process provides a means to represent the full range of fire risk scenarios for the tunnel.

With regards to HazMat, the United States Nuclear Regulatory Commission (U.S.NRC) has recently undertaken analysis on roadway accidents involving long duration fires [11]. Their analysis is based upon twelve years of HazMat accidents registered (1997-2008) and has identified all fires and those within the ‘severe’ category. Their definition of a ‘severe’ fire complies with the following: persistent fire for an extended period of time, more than one vehicle involved, fuel was flammable liquid that could pool under another vehicle. In this study 8% of the fires involved flammable gas, whereas the rest were flammable liquid (69%), oxidiser (5%), radioactive (1%). These have been grouped on the event tree as liquid and others, and gaseous flammable fires.

For gaseous flammable fires, and in the absence of other data, the following DARTS assumptions have been taken for distribution of:

- Jet fires: 30%
- Flash fires: 40%
- Boiling Liquid Expanding Vapor Explosion (BLEVE): 30%

For the event tree, the number of fires per year in each fire size category is the product of:

- The total traffic in the tunnel (vehicle-miles/year);
- The fraction of vehicles per vehicle class (% vehicles of each class);
- The traffic mode proportion (% flow in each traffic mode), assumed as unidirectional in this study;
- The expected fire rate in the tunnel (fire/vehicle-miles);
- The proportion of fires developing to different fire sizes (%) and for the serious fires;
- The proportion of out of control fires (%) for Hazmat Category.

## 8.4. Fire Likelihood Analysis

The fire return period (years) is the inverse of the calculated fire frequency from Table 8.1 and distributed across the various categories based on severity classification of the fire and vehicle categories previously described. The estimates of the assessed fire return periods are listed in Table 8.2 for a range of fire classifications.

Table 8.2 - Return Period per Vehicle Type

| Severity of Fire | Traffic Type | Fire Load | | Return Period (years) per Vehicle Type | | | |
|---|---|---|---|---|---|---|---|
| | | MBTU/hr | MW | Car | Bus | HGV | DGV |
| Small & moderate | Normal | 17 | 5 | 3 | | | |
| | Normal | 51 | 15 | | 432 | 127 | |
| | Normal & Liquid Fires | 102 | 30 | | | | 53,030 |
| | Normal & Liquid Fires | 239 | 70 | | | | 243,120 |
| Large (serious) | Normal | 51 | 15 | 1,278 | | | |
| | Normal | 102 | 30 | | 16,526 | 1,168 | |
| | Normal | 171 | 50 | | | 1,168 | |
| Large (severe) | Normal | 239 | 70 | | | 11,990 | |
| | Normal | 341 | 100 | | | 11,990 | 1,848,557 |
| | Normal | 682 | 200 | | | 11,990 | |
| Large (out of control) | Normal & Gas Fired | 1,024 | 300 | | | | 16,637,013 |
| Jet | Normal & Gas Fired | 1,024 | 300 | | | | 1,626,268 |
| Flash | Normal & Gas Fired | 1,024 | 300 | | | | 1,219,701 |
| BLEVE | Normal & Gas Fired | 1,024 | 300 | | | | 1,626,268 |

The return periods assigned to each representative design fire load are presented in Table 8.3 along with a cumulative percentage towards a number of fire scenarios that are likely to occur in the tunnel.

Table 8.3 - Return Period per Fire Load

| Fire Load | | Fire/year | Return Period (years) | % Fires | Cumulative % of Fires |
|---|---|---|---|---|---|
| ≤ 17MBTU/hr | ≤ 5MW | 4.05E-01 | 3 | 96.3137% | 96.3137% |
| ≤ 51MBTU/hr | ≤15MW | 1.31E-02 | 91 | 3.1051% | 99.4188% |
| ≤ 102MBTU/hr | ≤30MW | 1.12E-03 | 1,069 | 0.2654% | 99.6842% |
| ≤ 171MBTU/hr | ≤50MW | 1.02E-03 | 1,168 | 0.2429% | 99.9271% |
| ≤ 239MBTU/hr | ≤70MW | 1.04E-04 | 11,427 | 0.0248% | 99.9519% |
| ≤ 341MBTU/hr | ≤100MW | 1.00E-04 | 11,913 | 0.0238% | 99.9757% |
| ≤ 682MBTU/hr | ≤200MW | 9.96E-05 | 11,990 | 0.0237% | 99.9994% |
| ≤ 1,024MBTU/hr | ≤300MW | 2.52E-06 | 473,981 | 0.0006% | 100.0000% |

By interpolation of the cumulative percentages of fires from above, a peak HRR of 550MBTU/hr (≈ 160MW) will be used for the purposes of this feasibility study; this is estimated to accommodate 99.99% of the fire scenarios that are likely occur in the Floyd Hill Tunnel. It is emphasized that this recommendation is made based on the information presented and conditioned in this feasibility study and will need to be revisited for any changes during the course of the project. It should also be noted that the level of 99.99% is a judgement made for the purposes of current assessment and is subject to discussion and agreement with the fire services and the AHJ.

The final decision on the appropriate level for the design fire will also need to be informed by the feasibility and practicability of meeting the proposed level, without disproportionate cost.

## 8.5. Influence of FFFS

FFFS provides benefits by way of two main mechanisms: cooling and oxygen displacement to limit fire growth; and water heating and phase change to limit the residual convective heat release. A key objective for FFFS is to suppress a fire (with potential to grow to the design fire size) to an extent that provides conditions that are manageable by the tunnel ventilation system and suitable for effective self-rescue. The operating principles for a FFFS should be such that the system is activated as early as practicable after fire detection. An option may be a delayed-automatic activation (countdown) that commences after fire detection. Within the countdown period the operator may choose to abort or delay the activation (for a false alarm or for a condition under which activation would be inappropriate) or confirm the activation at any point.

Non-dangerous-goods fires would be expected to grow slowly initially, a period during which FFFS activation would be expected to be very effective at controlling further fire growth; and then if uncontrolled, fire growth would accelerate to a fast growth period where FFFS is less effective. Tests on solid fuel and liquid pan-fires have shown that FFFS is effective, if designed, installed, operated and activated appropriately, to control fire HRRs to levels that are more manageable with regards to ventilation and fire service intervention. The extent of fire growth control depends on the type of system (pressure, water flow rates), the fire growth rate and the delay before activation of the FFFS.

The potential peak (total) FHRR is dependent on activation time and fire growth rate. Early activation of the FFFS would be expected to limit HRR to low MBTU/hr (MW) values [12]. Considering practical aspects of deployment, it can be assumed that (as a reasonable worst case) the peak FHRR may be limited to 50% of its (unsuppressed) peak. In addition to HRR control, further heat will be lost to the tunnel walls through radiation and to the FFFS water through water heating and phase change. Research has indicated that up to 50% of the residual fire heat release may be lost to radiation and water effects, leaving 25% (i.e. 50% x 50%) convective heat load to be managed by the longitudinal ventilation system [13].

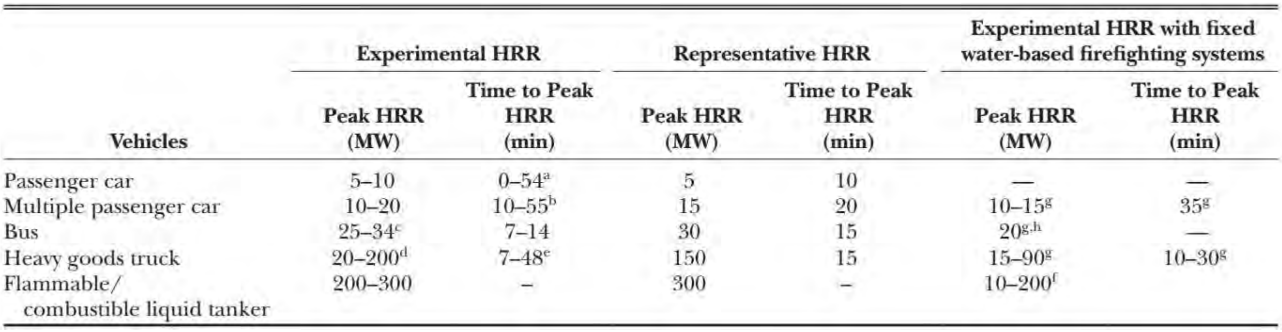
Following the above, the proposed design fire load of 550MBTU/hr (≈ 160MW) for the ventilation may be reduced to 135MBTU/hr (≈ 40MW) convective heat load by integrating FFFS in the tunnel; meaning that the ventilation system design would only need to be effective for a 135MBTU/hr (≈ 40MW) peak convective HRR. Nevertheless, any benefits on the tunnel ventilation system with a reduced design fire load carry commensurate costs on MEP systems and civil structures. Those impacts are discussed in Section 9 of this report, to allow informed decisions to be made on whether a FFFS is a practicable solution for Floyd Hill Tunnel with unidirectional traffic.

## 8.6. Summary of Findings

Following the above engineering analysis as summarized in the flow chart of Figure 8.2, the following peak HRRs will be used for the assessment of the tunnel ventilation system performance:

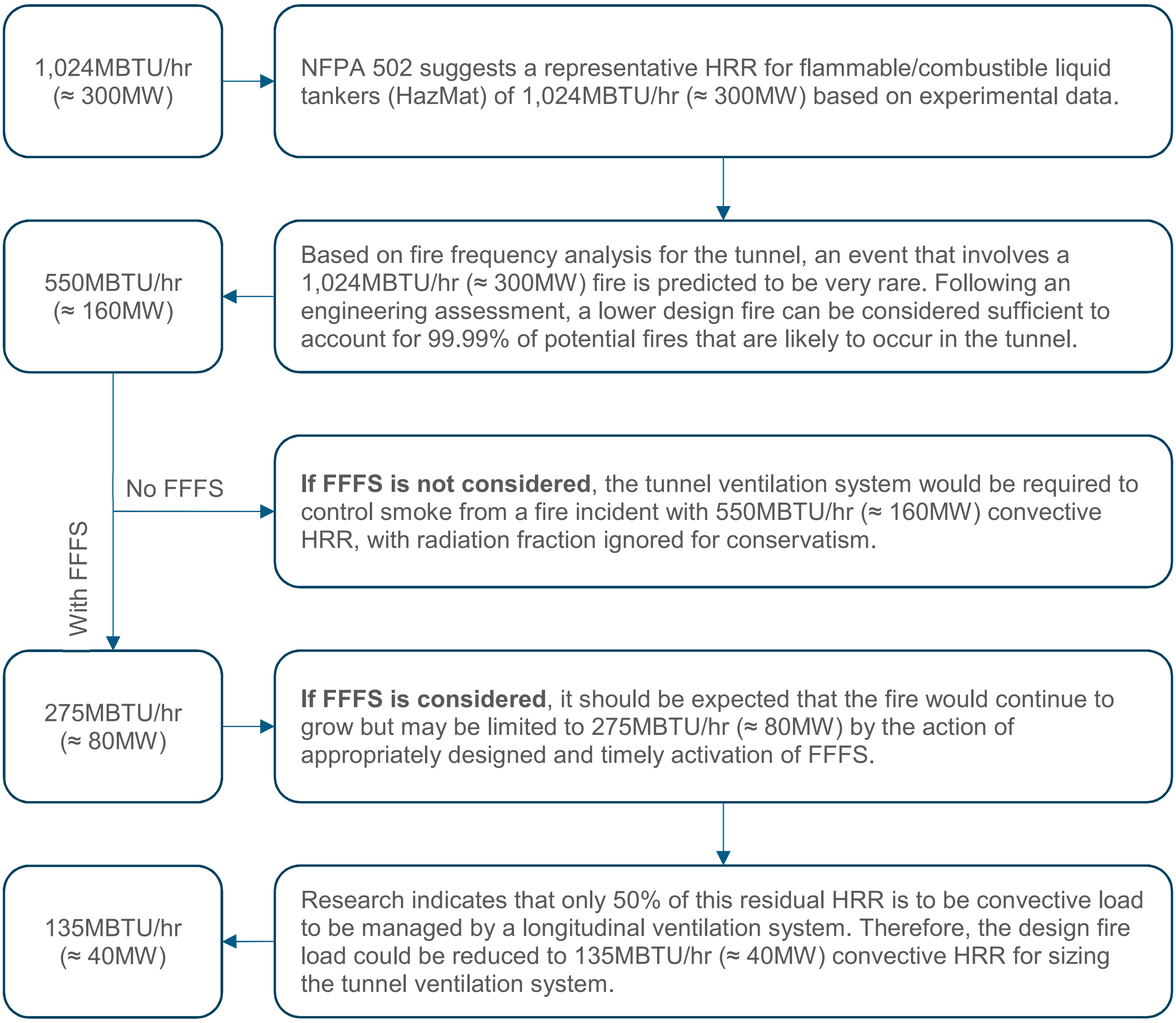
- Equipment design performance based on a peak HRR of 550MBTU/hr (≈ 160MW) without FFFS;
- Sensitivity analysis on potential system performance for 135MBTU/hr (≈ 40MW) with FFFS.

Figure 8.1 reproduces NFPA 502 Table A.11.4.1, where it can be seen that the derived peak HRR are in good agreement with the range of values published by NFPA for representative peak HRRs for HGVs.



| | Experimental HRR | | Representative HRR | | Experimental HRR with fixed water-based firefighting systems | |
|---|---|---|---|---|---|---|
| Vehicles | Peak HRR (MW) | Time to Peak HRR (min) | Peak HRR (MW) | Time to Peak HRR (min) | Peak HRR (MW) | Time to Peak HRR (min) |
| Passenger car | 5–10 | 0–54[a] | 5 | 10 | — | — |
| Multiple passenger car | 10–20 | 10–55[b] | 15 | 20 | 10–15[g] | 35[g] |
| Bus | 25–34[c] | 7–14 | 30 | 15 | 20[g,h] | — |
| Heavy goods truck | 20–200[d] | 7–48[e] | 150 | 15 | 15–90[g] | 10–30[g] |
| Flammable/ combustible liquid tanker | 200–300 | – | 300 | – | 10–200[f] | |

Figure 8.1 - NFPA 502 Table A.11.4.1



NFPA 502 suggests a representative HRR for flammable/combustible liquid tankers (HazMat) of 1,024MBTU/hr (≈ 300MW) based on experimental data.

Based on fire frequency analysis for the tunnel, an event that involves a 1,024MBTU/hr (≈ 300MW) fire is predicted to be very rare. Following an engineering assessment, a lower design fire can be considered sufficient to account for 99.99% of potential fires that are likely to occur in the tunnel.

**If FFFS is not considered**, the tunnel ventilation system would be required to control smoke from a fire incident with 550MBTU/hr (≈ 160MW) convective HRR, with radiation fraction ignored for conservatism.

**If FFFS is considered**, it should be expected that the fire would continue to grow but may be limited to 275MBTU/hr (≈ 80MW) by the action of appropriately designed and timely activation of FFFS.

Research indicates that only 50% of this residual HRR is to be convective load to be managed by a longitudinal ventilation system. Therefore, the design fire load could be reduced to 135MBTU/hr (≈ 40MW) convective HRR for sizing the tunnel ventilation system.

Figure 8.2 - Flowchart of Recommended Design HRRs

# 9. Water Based Fixed Fire-Fighting Systems

## 9.1. Performance Objectives

### 9.1.1. System Objectives

A key objective for a FFFS is to suppress a fire of potential to grow to the peak design HRR to an extent that provides conditions that are manageable by the tunnel ventilation system and suitable for effective self-rescue. These are achieved by way of the following key mechanisms:

- Cooling and oxygen displacement to limit fire growth;
- Water heating and phase change to limit the residual convective heat release;
- Attenuate radiation by screening the radiant heat from the fire.

The FFFS would be required to achieve the following objectives in the event of a vehicle fire in the tunnel:

1) Suppress the fire and limit peak HRR and smoke production;
2) Reduce temperature and radiation in the vicinity of the fire to:
    a) aid self-rescue operations;
    b) reduce likelihood of fire spread;
3) Maintain tenability conditions for fire service intervention.

FFFS are a CMR for Category C tunnels with the primary objective to "*mitigate the impact of fire to improve tenability for tunnel occupants during a fire condition, enhance the ability of first responders to aid in evacuation and engage in manual fire-fighting activities, and/or protect the major structural elements of a tunnel*" (NFPA 502 Clause 9.2.1).

In identifying the type of FFFS to be applied in a tunnel, it is critical to explicitly establish the performance that the system is expected to provide. Four distinct categories are outlined in NFPA 502 Clause 9.2.2, based on the target design performance of a FFFS summarized in Table 9.1.

Table 9.1 - NFPA 502 Performance-Based Categories of FFFSs

| Category | Mechanism |
|---|---|
| Fire Suppression | Sharply reduce the HRR of a fire and prevent its growth by means of direct and sufficient application of extinguishing agent through the fire plume to the burning fuel surface. |
| Fire Control | Limit the size of a fire by distribution of extinguishing agent to decrease the HRR […] while controlling gas temperatures to avoid structural damage. |
| Volume Control | Substantial cooling of products of combustion but is not intended to affect HRR directly |

### 9.1.2. Design Heat Release Rate with FFFS

Non-dangerous-goods fires would be expected to grow slowly initially. During this period an active FFFS would be at its higher efficiency in controlling the development of fire growth, as opposed to later stages where fire becomes uncontrolled as it accelerates to a fast growth period, reducing the efficiency of FFFS.

On the other hand, incidents involving HazMat vehicles such as a fuel tanker, rupture and rapidly developing pool fire, potentially resulting in explosion, is beyond what is considered a reasonable design scenario for Floyd Hill Tunnel. Such incidents are predicted to be very rare (estimated to be of the order of 1 in 473,981 years as per Section 8) and given their speed of development would not be controlled effectively by any practicable FFFS response. However, the system is able to mitigate the fire effects for such cases by screening radiation and providing cooling. In addition, for incidents where the tanker vehicle is secondary to a fire source, the FFFS will act to cool the Hazmat vehicle’s surfaces (e.g. fuel tank outer skin) and reduce the likelihood of involving the dangerous goods vehicle in the fire incident.

The flow chart in Figure 8.2 outlines the benefits of FFFS on the design HRR accommodated by the tunnel ventilation system, with a proposed peak convective HRR of 135MBTU/hr (≈ 40MW) with an active FFFS, noting the reduction from 550MBTU/hr (≈ 160MW) without considering FFFS.

### 9.1.3. Performance Considerations

Although there is a high level of research and testing of active suppression systems in tunnels, there is not yet consensus on what constitutes acceptable minimum performance for FFFSs. However, suitable approaches to performance specification have been developed for recent projects and have proven acceptable to leading suppliers in the FFFS market.

## 9.2. System Overview

### 9.2.1. Overview

Engineering guidance from the scientific research project ‘Safety of Life in Tunnels’ SOLIT$^2$ [14] describes general methods to consider the following aspects as minimum when deciding the type of system to go for:

- Fire risk;
- Level of protection;
- Other safety measures available;
- Geometry;
- Ventilation/wind conditions during a fire and interaction with the selected FFFS;
- Type and performance of the fire detection systems;
- Activation mode of the FFFS;
- Space restrictions for equipment (i.e. nozzles);
- Distance to emergency exits;
- Signage and lighting;
- Thermal conditions;
- Any other specific requirements.

There are various types of fire-fighting systems, distinguished by their droplet size and whether they introduce foam. Water-based systems aim only to suppress fire and not to extinguish it. They typically work by a system of pipes with a fixed water supply (i.e. a dedicated reservoir), which discharges water directly onto the fire and the surrounding area in order to suppress the fire and cool the neighboring areas. The introduction of foam provides additional benefits, particularly where large liquid fuel fires are a significant risk. However, the high discharge of water particles and evaporation due to the effects of fire can compromise visibility in the zones of system discharge.

Two main classes of system are considered for this report, classified according to droplet size and system pressure; this is described in the following sections, with a sketch of a deluge-type mist or sprinkler system shown in Figure 9.1. For both systems, the main pipework is typically fully charged (‘wet’) up to the section valve so that, on fire detection, the nozzles in the activated section may be quickly filled with water. Main distribution pipes are therefore typically fitted with trace heating. The higher flow rate of low pressure deluge systems (compared to mist systems) requires careful consideration of water supply and drainage requirements.

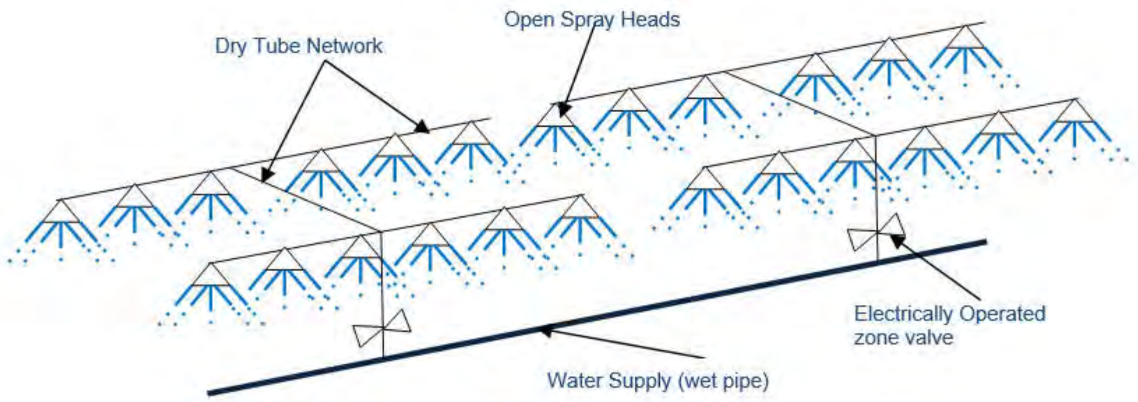


Figure 9.1 - Sample Zoned Fixed Fire-Fighting System

### 9.2.2. Low-Pressure Deluge

The term 'deluge system' is often used to describe a more traditional sprinkler-type system which releases larger droplets at a higher flow rate and lower pressure than 'mist' systems. On confirmation of fire, nozzles in a number of zones (often three zones of about 82ft or ≈ 25m each) are simultaneously activated, discharging spray onto the fire zone and adjacent zones.

### 9.2.3. High Pressure Mist

Water mist systems eject relatively small amounts of water at high pressures from specially designed nozzles. The small droplets provide a mist with a relatively large surface area which results in an improved heat transmission from the fire to the water, allowing the water to be evaporated more rapidly and efficiently than systems that generate larger droplets. The effectiveness of a system can rely heavily on the choice of pressure and droplet size and can vary from tunnel to tunnel depending on the ventilation regime and tunnel geometry. Mist systems also typically operate in a similar fashion to a 'deluge' mode, whereby a number of nozzles are activated simultaneously in a predefined zone. High pressure mist systems have been installed in a number of tunnels worldwide.

### 9.2.4. Operational Concept

Automatic fire detection may be through a combination of automatic incident detection (AID), linear heat detection system and direct operator intervention. Nevertheless, an incident can be detected through direct reporting of an emergency by tunnel users (e.g. through emergency telephone) or by the operator upon visual inspection of the tunnel activity (e.g. through CCTV). In order to instigate a delayed-automatic operation of suppression (a countdown), two separate means of detection may be required to be confirmed; one of which can be the linear heat detection system which identifies the fire location.

The FFFS for Floyd Hill Tunnel would consist of 27 zones of 82ft (≈ 25m) each for the length of the tunnel 2,200ft (≈ 670m). The operating principle is usually to operate three zones (fire location and one zone upstream and downstream of the fire). Details of activation times, running times, etc. should be defined in the further project stages depending upon system design, specifications and agreement on operational matters with the AHJ, key stakeholders and the tunnel operator.

## 9.3. Considerations for MEP Systems

### 9.3.1. Overview

Both water mist and low-pressure deluge suppression systems require water supplies and associated pumps, and control and detection equipment, with some of these elements requiring plant rooms to be located close to the tunnel. The sizes and ratings will depend on the system used and the performance specifications of the system for their operating environment.

An installed FFFS would span along the full length of Floyd Hill Tunnel between portals, i.e. 2,200ft (≈ 670m) with key system components summarized below:

- Water storage tank(s) with division plate at the services buildings;
- Pumps and controls located at the services buildings;
- Wet main distribution pipework from the tank(s), to pump room and routed into and along the walls in the tunnel at high-level;
- Section valves to create suppression zones of appropriate length and with respect to the evacuation strategy and configuration of the tunnel ventilation system;
- Dry secondary distribution pipework at high-level to suppression nozzles above the maintained headroom, 'flap zone' and carriageway;
- Suppression nozzles connected to the secondary distribution network; longitudinal and transverse spacing of nozzles to be as appropriate to provide the required cross-sectional spray coverage and water density in the protected area with a suitable redundancy;
- Water treatment facility subject to the quality of incoming water supply;
- Trace heating of wet distribution system to prevent freezing.

The following sections outline the design requirements that can be considered for the purposes of the feasibility study to establish the impact on in-tunnel services, utilities and associated service buildings.

### 9.3.2. Water Supply

With regards to types of supply water, SOLIT$^2$ guidance [14] defines the supply as a system consisting of a water reservoir, a pump system, pipework and section valves. The water supply and discharge requirements may be specified in terms of a flow per ground surface area of protected space, i.e. gpm/ft$^2$ (lit/min/m$^2$).

Typical water consumption per volume unit could be around 18-38gpm/ft$^3$ (≈ 2-4lit/min/m$^3$) for a large droplet system and between 2-10gpm/ft$^3$ (≈ 0.2-1.0lit/min/m$^3$) for a water mist installation, according to PIARC. The selected flow rate is subject to detail design from the manufacturer. The flow rates together with system pressure and running times would dictate the tank and pipeline sizes.

Floyd Hill Tunnel temperatures below freezing conditions during winter months would need extra consideration, not only to prevent pipe freezing by means of having trace heating but also the potential release of water particles that may freeze upon release.

### 9.3.3. Drainage

Drainage system dimensions should consider the maximum flow rate of the FFFS, in addition to other tunnel specific requirements. In the case of additives, their collection and disposal should be also included. Pump stations should have a drainage capacity equal to the water supply flow rate as a minimum (see Section 11.10).

### 9.3.4. Pumping

Guidance is provided in the SOLIT$^2$ documents; the system should be dimensioned to provide as a minimum 110% of the nominal flow required for the most demanding protected area. This volume flow requirement can be provided by one or more pumps. Building space for housing the pumping station needs to be provided; ideally this space should be located close to the tunnel location due to the pressure drop (and therefore increase in power demand) as water travels through the pipeline system.

### 9.3.5. Power

Power supply for a FFFS should have the same level of reliability as required for ventilation and main control system. The power requirements would depend upon the system design (i.e. flow and pressure requirements for the most demanding section) and can be significant depending on the type of system selected (i.e. greater energy demand for a high-pressure water mist system).

### 9.3.6. Interfaces to Other Systems

Further to the above, the tunnel FFFS will be required to have the following system interfaces:

1. Plant Monitoring and Control System for status monitoring and fault reporting feedback;
2. CCTV to permit monitoring of the tunnel environment and activity both outside and inside the activated zones, subject to visibility due to spraying activity;
3. Tunnel Operations Command and Control System to permit a full remote monitoring, operation and override capability;
4. Fire Detection and Alarm to determine the location of any active fire detections.

## 9.4. Summary of FFFS

Section 13.1.3 outlines that for three out of four selected tunnel options, there is adequate space provision to accommodate the required number of jet fans for a longitudinal ventilation system to provide a tenable environment upstream of the fire location to allow self-rescue and enable fire-fighting services to access the fire location. Although the introduction of a FFFS would benefit the size of the ventilation system assets due to the reduced convective HRR, it will result in an increase in the project whole life costs.

Nevertheless, an assessment on the tunnel ventilation system sizes has been performed for a design HRR of 135MBTU/hr (≈ 40MW), as described in Section 9.1.2; comparative results are shown in Table 9.2 for the same quantity of banks and fans used in the absence of FFFS.

Table 9.2 - Longitudinal Ventilation System with and without FFFS

| Parameter | Without FFFS | With FFFS |
|---|---|---|
| Fan Inlet Diameter | 64” (≈ 1,600mm) | 40” (≈ 1,000mm) |
| Discharge Velocity | 6,200cfm (≈ 31.5m/s) | 6,120cfm (≈ 31.1m/s) |
| Motor Rating | 74HP (≈ 55kW) | 30HP (≈ 22kW) |

Solutions are feasible for the tunnel with or without FFFS. Without FFFS, ventilation capacities are larger but project whole life costs would be expected to be lower. With FFFS, the higher whole life costs may be offset to an extent by reduced damage and disruption that could occur due to a severe fire.

In summary, there are challenges in designing an appropriate FFFS solution for the climatic conditions at Floyd Hill. FFFS is not recommended for Floyd Hill Tunnel due to its unidirectionality and the tenable environment upstream of the fire for self-rescue by virtue of the ventilation system; however, if this option is progressed, the challenges outlined will need careful consideration in the next design stage.

# 10. Tunnel Ventilation

## 10.1. System Objectives

The primary objectives of the tunnel ventilation system for Floyd Hill Tunnel are as follows:

- Prevent the accumulation of vehicle emitted pollutants such as CO, $NO_x$ and particulate matter (PM);
- Prevent back-layering of smoke and hot gases in the event of a fire in the tunnel;
- Facilitate the intervention and rescue operations of the Fire Department.

An engineering analysis has been performed to assess the performance of the proposed system configuration with due consideration of a range of parameters described in this section. The system performance is assessed against a convective peak HRR of 550MBTU/hr (≈ 160MW) in accordance with the recommendations from the fire frequency analysis described in Section 8. The various parameters that affect the system performance are also described in this section including geometrical, topographical, and operational characteristics of the tunnel and the ventilation assets, in order to provide context of the expected conditions that the system will be required to operate in.

## 10.2. Concept Design

The proposed concept for the ventilation of the tunnel is a mechanical longitudinal ventilation system consisting of multiple banks with evenly-spaced jet fans positioned at soffit level at minimum intervals along the tunnel. The sampled system configuration is summarized below and considered for each tunnel option in this report:

- Total 6 banks of jet fans and 4 jet fans per bank (total 24 fans);
- Banks arranged at 10 x tunnel hydraulic diameter intervals with first bank at 33ft (≈ 10m) from entry;
- Each jet fan carrying the following characteristics:
  - 64” (≈ 1,600mm) inlet diameter;
  - 6,200cfm (≈ 31.5m/s) discharge velocity;
  - 74HP (≈ 55kW) motor rating.

## 10.3. Operating Principles

In-tunnel air quality is reduced by vehicle emissions, the primarily pollutants of interest being $NO_x$, CO and PM, the production of which are dependent primarily on traffic conditions, vehicle types and mix and road gradient.

The rate of generated emissions at roadway level was assessed using MOVES 2014b software, with resultant emission levels extrapolated using an industry-standard approach set out in PIARC (2012), to assess the flow of outside air required to dilute emissions to allowable concentrations (outside air demand) based on fleet composition, vehicle speed and traffic density.

In cases of freely-flowing traffic, the ‘piston effect’ will be calculated to determine the conditions under which the tunnel can effectively self-ventilate. For stationary and congested (slow-moving) traffic and due to the loss or reduction of the piston effect, it is expected that mechanical ventilation will be required to induce the same outside air demand which will be assessed using standard PIARC numerical methodologies.

For the control of smoke and hot gases in the event of a fire and due to the unidirectional nature of the tunnel, a longitudinal forced mechanical ventilation system will generate air flow velocity in the direction of traffic in the tunnel that exceeds the minimum critical velocity to control smoke by preventing back-layering as the primary design criterion in accordance with NFPA 502 requirements.

The operating system will be required to overcome resistances in the tunnel, installation efficiencies, vehicle drag and prevailing wind conditions that dictate its performance and configuration.

## 10.4. Alignment

The tunnel length has been considered as 2,200ft (≈ 670m) that includes portal construction or extended canopies at each portal. From the vertical alignment drawings, the longitudinal gradient of the tunnel is constantly positive at +1.3% for the Westbound traffic direction.

## 10.5. Tunnel Cross-Sections

The key geometrical features of each tunnel configuration option are summarized below as parameters that influence the performance of the ventilation system, with an objective of subsequent analysis being to establish a single system configuration that can be accommodated for all options. It is noted that due to typical geometries, Option B and Option C will share the same conclusions, whereas the results will be typical for each bore of Option D.

Table 10.1 - Tunnel Option Geometries

| Variable | Option A | Option B | Option C | Option D (Typical) |
|---|---|---|---|---|
| Area | 1,681ft$^2$ (≈ 156m$^2$) | 1,473ft$^2$ (≈ 137m$^2$) | 1,473ft$^2$ (≈ 137m$^2$) | 1,102ft$^2$ (≈ 102m$^2$) |
| Average height | 30ft (≈ 9m) | 26ft (≈ 8m) | 26ft (≈ 8m) | 27ft (≈ 8m) |
| Internal perimeter | 167ft (≈ 51m) | 158ft (≈ 48m) | 158ft (≈ 48m) | 130ft (≈ 40m) |

## 10.6. Ambient Conditions

Floyd Hill Tunnel stands at approximately 7,217ft (≈ 2,200m) above sea level and is surrounded by mountainous terrain that experiences sub-zero winter temperatures and potential funneling of wind due to the Rocky Mountain line. Due to these adverse conditions, it is important to establish a representative set of weather data that characterize the performance of the tunnel ventilation system under the appropriate operating environment.

The American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE) Handbook 2001: Fundamentals (Chapter 27) [15] contains a range of information on weather data collected from the DATSAV2 satellite records over a range of years for locations around the world. The database considers all parameters at the time of recording, such as: cloud coverage, irradiation, wind speed, and direction – with additional further references to the United States Climate Reference Network (USCRN) and Integrated Surface Database (ISD) from NOAA where information was unavailable. ASHRAE has thereafter translated this data into design days per month for one year to establish annual cumulative frequency of occurrence and annual percentiles, as guidelines on ambient conditions for designing cooling, heating and ventilation systems.

Weather information from each available weather station can be found from a comprehensive map developed by ASHRAE [16], with a summary of weather data for each unique station presented in ASHRAE Handbook 2001: Fundamentals. These data sets will be used for the purposes of this report, with further interpolation of this data to be performed at the detailed design stage for further refinement of the ambient conditions that the tunnel ventilation system is expected to operate in (e.g. direction and speed of prevailing wind). Figure 10.1 presents the available weather stations in the vicinity of Floyd Hill Tunnel as per the ASHRAE Climatic Database.

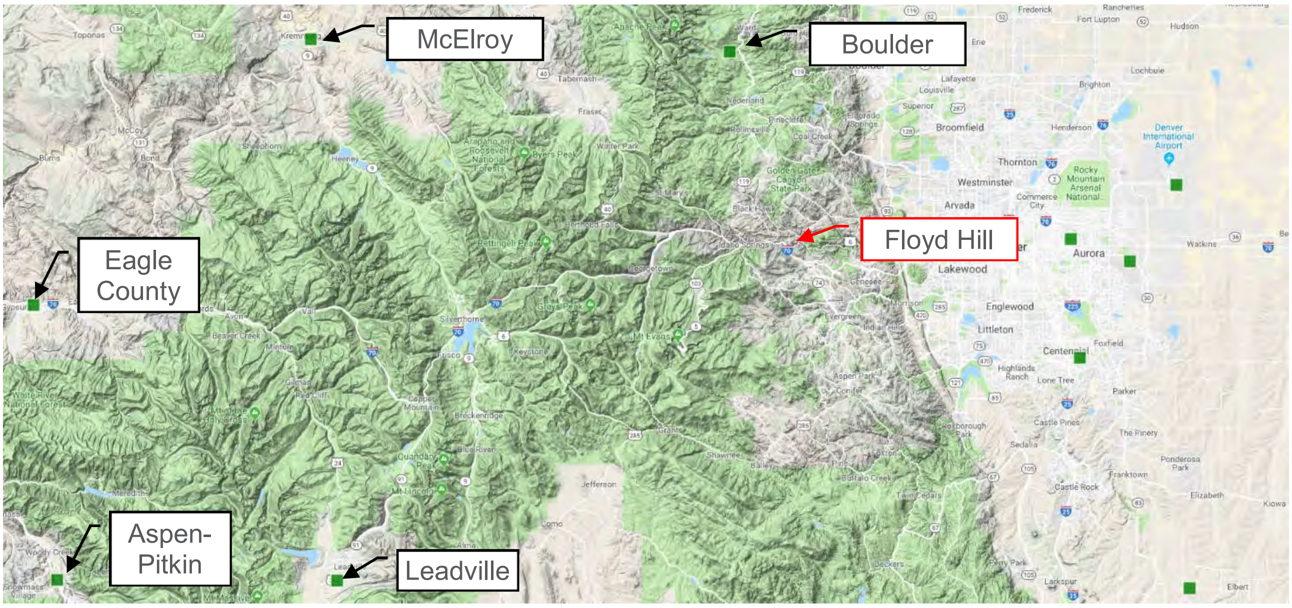


Figure 10.1 – Proximate Weather Stations (ASHRAE)

The elevation of stations around the city of Denver are approximately 5,413ft (≈ 1,650m) above sea level, which is significantly lower than the elevation of the tunnel, resulting in non-representative parameters of atmospheric pressure, air density and dry bulb temperature, in addition to differences in data due to the different environment (urban versus rural). From the other available weather stations in the proximity of the tunnel, the most representative is noted to be McElroy Airfield, with an elevation of 7,411ft (≈ 2,259m) above sea level and surrounded by a terrain similar to Floyd Hill. Following the above, and in the absence of more specific data for Floyd Hill (e.g. Clear Creek County), the weather data for McElroy Airfield will be used for the purposes of this report. The summary from ASHRAE is presented in Appendix C, whilst key ambient conditions for the 99.6% of occurrence are listed in Table 10.2 below. The wind speed selected reflects the 95th percentile as a common industry practice.

Table 10.2 - Ambient Conditions for McElroy Airfield (ASHRAE Fundamentals 2017)

| Variable | | Value (Metric) | Value (Imperial) |
|---|---|---|---|
| Distance to Floyd Hill Tunnel site | | 88km | 55miles |
| Weather data period | | 2004-2014 | 2004-2014 |
| Altitude above sea level | | 2,259m | 7,411ft |
| Atmospheric pressure | | 76.97kPa | 22.73in.Hg |
| Summer conditions | Dry bulb temperature | 31.3°C | 88.3°F |
| | Coincident wet bulb temperature | 11.9°C | 53.4°F |
| | Relative humidity | 9% | 9% |
| | Air density | 0.882kg/m$^3$ | 0.055lb/ft$^3$ |
| Winter conditions | Dry bulb temperature | -27.7°C | -17.8°F |
| | Dew point temperature | -32.2°C | -26.0°F |
| | Relative humidity | 62% | 62% |
| | Air density | 1.093kg/m$^3$ | 0.068lb/ft$^3$ |
| Wind speed | | 7.3m/s | 1,437cfm |

## 10.7. Background Pollution Levels

The background (ambient) pollutants being brought in with the outside air needs to be taken into account when calculating the in-tunnel pollution concentrations. PIARC (2012) suggests CO may reach 5ppm and nitrogen dioxide ($NO_2$) may reach 1.25E-8lb/$ft^3$ (≈ 200μg/$m^3$) for the background levels of urban tunnels. In the absence of similar information for rural tunnels such as Floyd Hill, the same limits will be considered as applicable for the purposes of this report. Due to the absence of significant population densities in the vicinity of the tunnel portals that could be adversely affected by polluted air, portal emissions and environmental impacts due to in-tunnel pollutants exiting the tunnel have not been evaluated in this early feasibility study. However, it is recommended that pollution levels at the tunnel portals is considered in the later design stages of the project.

## 10.8. Maximum In-Tunnel Admissible Pollutant Limits

The potential effects on human health due to inhalation of CO at high altitude are complicated and of medical/physiological nature. Guidance from EPA and FHWA outlines maximum allowable CO concentration limits for highway tunnels as 120ppm for 15min exposure and for altitudes up to 5,000ft (≈ 1,500m) above sea level. For higher altitudes within the troposphere, the same guidance recommends an altitude correction factor to be applied on those limits to account for higher elevation conditions. Nevertheless, uncertainty remains with regards to the potential reduction in allowable limit levels at altitude, for which there is no contemporary guidance in tunnel standards or guidelines. This issue warrants further investigation at a later design stage.

The pollutant level limits are presented in Table 10.3, in accordance with recommendations from PIARC that will be used to assess whether the tunnel achieves acceptable levels with and without an active tunnel ventilation system. It is noted that the analyses do not account for the effect on the tunnel air quality due to external pollution levels due to the rural environment surrounding Floyd Hill Tunnel.

Table 10.3 - In-Tunnel Pollutant Concentration Limits (PIARC)

| Parameter | Value |
|---|---|
| CO | 70ppm |
| $NO_x$ | 1ppm |
| PM | 0.007m-1 extinction coefficient |

## 10.9. Hydraulic Losses

Air flow through Floyd Hill Tunnel will be resisted by the aerodynamic drag generated by the following elements:

- Structural lining (wall friction);
- Protruding MEP elements in the tunnel such as cable containment, hydrant pipes and PA speakers;
- Energy losses at tunnel entrance and exit;
- Wind pressure at exit portal where smoke discharges to ambient;
- Stalled vehicles upstream of the fire (i.e. from tunnel entry until fire location);
- Pressure drop due to fire.

The sum of the above characterizes the total resistance that the tunnel ventilation system must overcome for emergency conditions, with the influence of each parameter on system performance briefly described in the following sections.

### 10.9.1. Portal Effects

Head loss coefficients of the tunnel entry are widely accepted within the industry for representing the abrupt fluid entry and exit loss-effects. In contrast, density and velocity are variables determined by the environmental conditions upstream (ambient) and downstream (smoke-laden) of the fire location; as such, in these areas the effect of portals are assessed through simplified simulation.

### 10.9.2. Adverse Wind Pressure

As the prevailing wind conditions are a key factor affecting the system performance, for conservatism and spaceproofing the tunnel profile, the established adverse wind speed will be used as a direct pressure on the exit (West) portal which the system will need to overcome, with a maximum value of 0.00422psi (≈ 29.1Pa) observed for winter. It is noted that a more refined value at the detailed design stage can be achieved by analysis of the detailed weather data that can associate the orientation of the prevailing wind and its speed to derive a more representative coincident wind pressure at the exit portal.

### 10.9.3. Tunnel Resistance

The friction factors considered in the models are characterizing the various MEP services across the tunnel section that the model is divided into, in addition to the roughness of the concrete. The friction factors are derived by iteration of Darcy-Weisbach and Colebrook equations for turbulent flow for each segment of the tunnel that consists the tunnel sections in the models. The respective roughness lengths of each element and its overall contribution to the weighted tunnel friction factor serving as an input to simulations is derived from the extent of the protrusion of each component from the wall and soffit surfaces.

### 10.9.4. Fire Pressure Change

The tunnel has a constant positive gradient in the direction of flow, therefore the natural buoyancy effect of the smoke will act in the direction of ventilation. For the purposes of this study and for conservatism towards spaceproofing the tunnel profile, this beneficial effect is not taken into account into the assessments and configurations of the longitudinal ventilation system, but may be considered in the subsequent design stages.

### 10.9.5. Vehicle Drag

The aerodynamic drag of vehicles results in the piston effect for free-flowing traffic, but also local pressure gradients when stationary in the tunnel during emergency conditions, as obstructions to the flow induced by the tunnel ventilation system. The pressure is dependent upon the frontal areas and drag coefficients of the constituent vehicle types in the fleet, with assumed values for each vehicle type listed in the table below. The frontal areas of the various vehicle categories are based on the dimensions of the corresponding AASHTO Design Vehicles Handbook, with their drag coefficients taken from PIARC (1995).

Table 10.4 - Vehicle Aerodynamic Parameters

| I-70 Vehicle Type | AASHTO Design Vehicle | Frontal Area, $A$ [ft$^2$ (m$^2$)] | Drag Coefficient, $C_D$ |
|---|---|---|---|
| Motorcycle | - | 11 (1) | 0.7 |
| Passenger Car | Passenger Car | 30 (2.8) | 0.35 |
| Passenger Truck | Single-Unit Truck (min. height) | 88 (8.2) | 0.8 |
| Light Commercial Truck | Single-Unit Truck (max. height) | 108 (10) | 0.8 |
| Intercity Bus | Intercity Bus (Motor Coaches) | 102 (9.5) | 0.8 |
| Transit Bus | City Transit Bus | 90 (8.3) | 0.8 |
| School Bus | Conventional School Bus | 84 (7.8) | 0.8 |
| Refuse Truck | Single-Unit Truck (max. height) | 108 (10) | 0.8 |
| Single Unit Short-haul Truck | Single-Unit Truck (max. height) | 108 (10) | 0.8 |
| Single Unit Long-haul Truck | Single-Unit Truck (max. height) | 108 (10) | 0.8 |
| Combination Short-haul Truck | Interstate Semitrailer | 114 (10.6) | 0.8 |
| Combination Long-haul Truck | Interstate Semitrailer | 114 (10.6) | 0.8 |

## 10.10. Fan Efficiency

The performance of a bank of jet fans, as well as the jet fans individually, depends on a wide range of design parameters. These parameters result in aerodynamic implications on discharge and dispersion profiles of jet streams from the fan outlet, either due to the Coanda effect from a parallel surface, obstruction effects from another element (e.g. public address (PA) speaker, signage) or convoluted flow profiles of closely spaced jet fans. The following sections describe how the assessment considers losses of thrust as percentages of thrust loss from the nominal thrust of the fan at its outlet.

### 10.10.1. Installation Clearances

Research and industry practice on optimization of jet fan installations has resulted in design guidelines in terms of minimum distances of jet fans from all surroundings elements, as summarized in ASHRAE Handbook 2011: HVAC Applications (Chapter 15) [17] and highlighted in Figure 10.2 as recommended clearances.

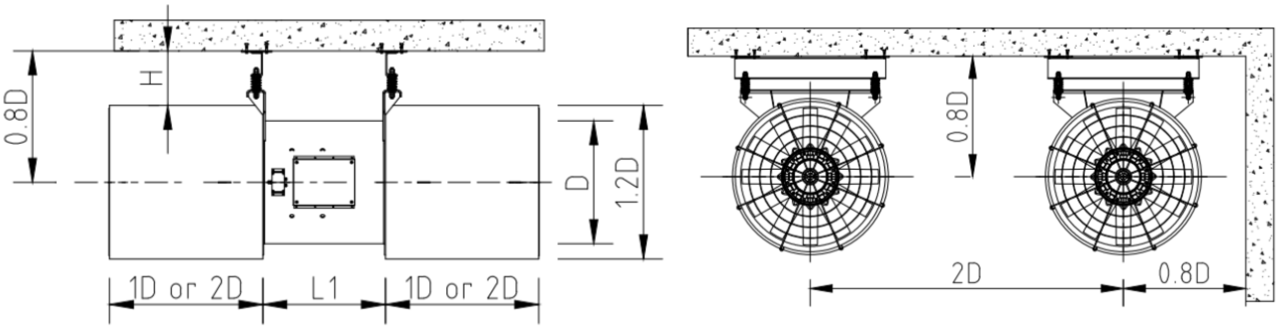


Figure 10.2 - Design Guidelines on Fan Installation Clearances [17]

For system sequences that require active jet fans on neighboring banks, the assessment considers the minimum sufficient longitudinal distance between banks of fans along the longitudinal tunnel length to prevent impact of efficiency of jet fans downstream of each flow stream. Guidance from common industry practices and field research agree on distances between fans (preceding fan outlet to following fan inlet) equal to 8-10 times the tunnel hydraulic diameter or 100 jet fan diameters without deflectors on fans.

### 10.10.2. Proximity to Tunnel Surfaces

Field research suggests that the installation efficiency of a jet fan due to its proximity to a wall and/or ceiling can be estimated as a function of the distance of the fan centerline from the wall surface and the fan external diameter, and characterized by the following equation:

$$n = \left[0.0912\left(\frac{z}{D_0}\right) - 0.144\left(\frac{z}{D_0}\right) + 1.27\right]^{-1}$$

where: n = jet fan efficiency (%)
z = distance between the horizontal centerline axis of the jet
$D_o$ = outlet diameter of jet fan

Resultant reduction of fan efficiency is considered in the assessment following iteration between outlet fan diameters and proximity to tunnel wall and/or soffit, being respectful of space allowances across the bank and against the tunnel width.

### 10.10.3. Proximity to Portals

As the flow stream emerging from the jet fan will be required to transfer the momentum to the bulk of the flow with a fully developed velocity profile, a close proximity of the jet fan outlet to an open portal (i.e. to ambient) will result in total loss of the jet energy to the environment and reduced total delivered thrust to the tunnel. Guidance from common industry practices and field research agree on longitudinal spacing between fans equal to 10 times tunnel hydraulic diameters, which has been used for the purposes of this report.

For the case of inlet (suction) of unidirectional fans only, it is common industry practice for fans to be physically close to the portal, noting that this location requires coordination at the design stage with the high density of light fixtures and signage expected in entrances to the tunnel.

## 10.11. Smoke Control

### 10.11.1. Redundancy and Availability

Both industry practice and NFPA 502 considers that fans located in close proximity to the fire incident must be considered as unavailable (i.e. destroyed), with the remaining fans in the system required to achieve the critical velocity. Table 10.5 reproduces the guidance contained in UK Design Manual for Roads and Bridges BD78/99 [18] for road tunnels on the distance upstream and downstream of the fire location within which any fan is to be considered as destroyed. It is noted that in the absence of relevant information on similar distances for design fires greater than 1,341MBTU/hr (≈ 100MW), these distances might be greater than the ones listed in the table below, which will need to be further addressed at the future design stages.

Table 10.5 - Fans Considered as Destroyed at a Distance from Fire (BD78/99)

| **Fire Size [MBTU/hr (MW)]** | **Distance Upstream of the Fire Location** | **Distance Downstream of the Fire Location** |
|---|---|---|
| 17 (5) | - | - |
| 68 (20) | 33ft (≈ 10m) | 131ft (≈ 40m) |
| 171 (50) | 66ft (≈ 20m) | 263ft (≈ 80m) |
| 1,341 (100) | 98ft (≈ 30m) | 394ft (≈ 120m) |

Fan operational security must be maintained at all times. The system is designed to operate to its design requirement even when fans are unavailable, either due to routine maintenance or occasional failure. NFPA 502 Clause 11.1.5 prescribes that the emergency ventilation for the tunnel "*[…] shall be sized to meet minimum ventilation requirements with one fan out of service*", complimented by NFPA 502 Clause A.11.5.3 and industry practice for entire banks of fans located above or in the proximity of the fire incident to be considered as destroyed.

Following the above, the assessment for the tunnel ventilation system considers the following parameters for the selected peak HRR with respect to the distances from Table 10.5:

- Fans that are further upstream of the minimum distance are normally operating for ambient density conditions and upstream (critical) velocity;
- Fans that are further downstream of the maximum distance are de-rated with regards to the lower smoke density and downstream tunnel air velocity;
- Fans that are within the minimum and maximum distance are considered destroyed and unavailable for the respective fire scenario.

In addition to the above, the assessment considers an additional fan being taken out of service to comply with NFPA 502 redundancy requirements. Since the thrust delivered by fans upstream of the fire location is considered more critical, one fan in the first bank is placed out of service with similar unavailability of downstream fans being a less stringent scenario of loss of thrust.

### 10.11.2. Fire Locations

Unavailability of all fans in the incident tunnel is possible for a scenario where a fire occurs up to 164ft (≈ 50m) in the tunnel from the entry (East) portal. Nevertheless, such an event is not considered a credible scenario for smoke control due to the following considerations:

- Dissipation of smoke to ambient, assisted by a positive wind pressure effect coming from exit portals or naturally due to buoyancy effects, being mindful of the significant upward gradient at both portals;

- A blockage due to an incident (especially if fire is visible) is expected to bring traffic to a standstill upstream of the portal with a very low number of vehicles being physically in the tunnel (entry to fire location), resulting in negligible resistance to the flow;
- Passengers that are already close to the tunnel portal will be able to immediately evacuate outside before the fire becomes life threatening;
- Fire fighters will be able to intervene externally from the approach ramp using fire hydrants located accordingly without compromise of fire-fighting procedures or safety due to untenable conditions or restricted access.

Following the above, an engineering judgement has been made for the performance of the ventilation system to be assessed for fire locations originating from 164ft (≈ 50m) from the tunnel entrance portal onwards.

### 10.11.3. Assessment Results

Each of the design criteria described in this report have been factored into an assessment to determine the number of banks and fans per bank, by sampling the overall system performance to achieve critical velocity in the tunnel. The assessment is performed with the following steps:

- Determine the **minimum in-tunnel target thrust** required at the bulk of airflow to achieve critical velocity for the estimated tunnel resistances, the peak HRR and environmental conditions;
- Appropriately space banks of fans across the tunnel length and sample fan performance to obtain the **minimum in-tunnel delivered thrust** from the operating system (sum of thrust from all banks), by varying fan diameter, discharge velocity and quantity of duty fans per bank (typical for all). This criterion encompasses de-rating factors described earlier in the report and system performance under upstream ambient conditions of fire, downstream smoke-laden conditions or unavailability (i.e. no thrust) due to proximity to the fire location (every 33ft or ≈ 10m);
- From the selected quantities of fans per bank, assign one duty fan out of service from a bank located upstream of the fire locations as a more stringent case for available overall system thrust;
- Iterate the above sequence for each geometry of the tunnel profile options (cross-sectional area, perimeter and average height) and for selected peak summer and winter conditions.

The acceptance criterion for all assessments and iterations is when the **minimum in-tunnel delivered thrust** exceeds the **minimum in-tunnel target thrust** that concludes the adequacy of number of fans and number of banks in the tunnel for the sampled fan performance, tunnel geometries and operating environment. It is noted that the assessment does not consider the effects of a FFFS that can result in a lower peak HRR to be accommodated by the tunnel ventilation system. The influence of a FFFS in the design of the tunnel ventilation system is discussed further in a sensitivity analysis presented in Section 9.4.

Appendix D presents the results of the assessment for the delivered and minimum target thrust thresholds (as above) for a single fire every 33ft (≈ 10m) across the tunnel and for both summer (red color) and winter (blue color) conditions of each tunnel option. The same graphs highlight the unavailability of more than one bank (destroyed by fire), indicated by the significant local loss of total system thrust for specific fire locations, with the most stringent case being:

- Fans upstream (first bank) operating at ambient conditions with one fan unavailable;
- Second and third bank destroyed by fire with all fans unavailable;
- All remaining fans downstream operating de-rated smoke-laden conditions.

Drawings provided in Appendix E provide an indicative arrangement of fans for each tunnel option, considering the aforementioned longitudinal and lateral clearances. From the sections, it can be noted that for Tunnel Configuration Options A, B and C, the proposed fans can be accommodated within the tunnel profile with adequate clearance from the traffic envelope and flexibility on coordination with in-tunnel systems.

On the other hand, due to the smaller geometry of Option D, the same fans can be accommodated in the tunnel soffit if fans are spaced more closely at approximately one fan diameter clearance. The effect of the proximity of fans within the same bank requires consultation with manufacturer data at a future design stage, following refinement of equipment sizes and tunnel operational concepts.

## 10.12. Pollution Control

### 10.12.1. Vehicle Exhaust Pollutant Emissions

The total emissions produced by traffic in Floyd Hill Tunnel have been assessed using a MOVES 2014b model, to determine the vehicle exhaust pollutant concentration within an order of magnitude for the design year 2040 and for varying traffic speeds. The results from the analysis are conditioned further with standard PIARC methodologies, to assess the conditions under which free-flowing traffic can naturally achieve the minimum outside air demand (self-ventilating tunnel) or the conditions where the tunnel ventilation system would need to be activated to achieve the same objective.

The analysis is based on the default database of MOVES for Clear Creek County as location specific to Floyd Hill Tunnel. The database has been adapted with previous information obtained from CDOT (2015) on vehicle age distribution and source type distribution of vehicles for Denver, which would require refinement for more specific information for I-70 Mountain Corridor at a future design stage. A more detailed summary of the input data and assumptions used is given in Appendix F.

The effects of high temperature or high humidity on internal combustion engines is known to result in higher concentrations of carbon monoxide and nitrous oxides respectively, that are not necessarily coincidental. A preliminary sensitivity analysis has been performed to determine reference months that present the highest emissions of CO and $NO_2$ pollutants. The analysis is performed with traffic flow and traffic speed as arbitrary constants for the reference year, to allow variations in emitted pollutants to be influenced by the changes of temperature and humidity across the year. From the results presented in the figure below, the months with the highest emissions have been identified as July for CO and January for $NO_2$, as shown in Figure 10.3.

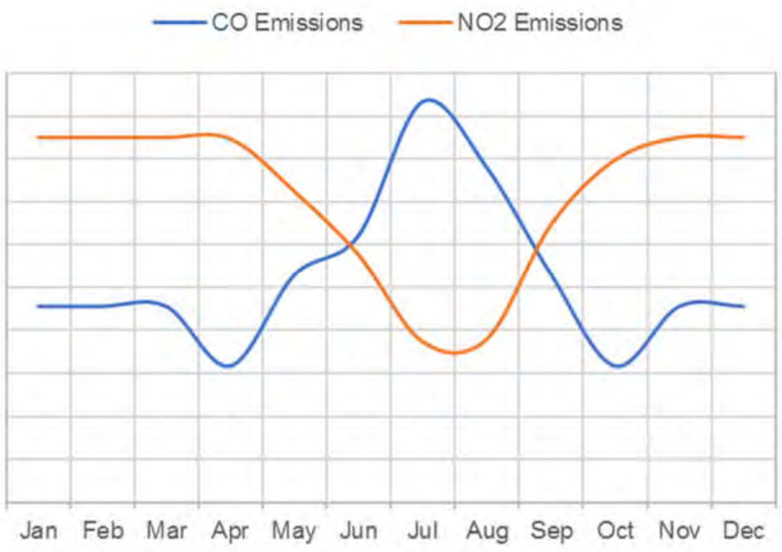


Figure 10.3 - Results for Peak Months for CO and $NO_2$ Emissions

### 10.12.2. Outside Air Demand

When traffic moves through the tunnel it induces a quantity of outside air (OA) through the tunnel due to the piston effect of the vehicles. At high enough vehicle speeds, the amount of outside air induced by moving vehicles could prove to be sufficient to meet the minimum outside air demand (OAD) required to dilute in-tunnel pollution for CO and $NO_2$ to acceptable levels discussed in Section 10.8.

The MOVES model was further refined for the selected reference months from the above analysis for a range of traffic speeds, to observe the vehicle emissions as a function of ambient conditions for each reference month and varying traffic speed. The profile of CO (Figure 10.4) and $NO_2$ (Figure 10.5) are presented in the graphs separately for Option A, B & C and Option D, due to their different lane configurations of three-lane and two-lane per bore respectively.

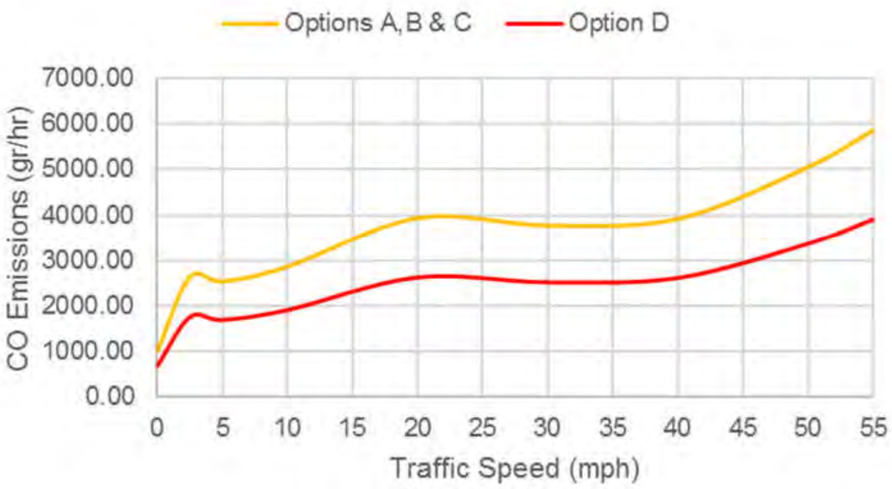


Figure 10.4 - CO Emissions for January

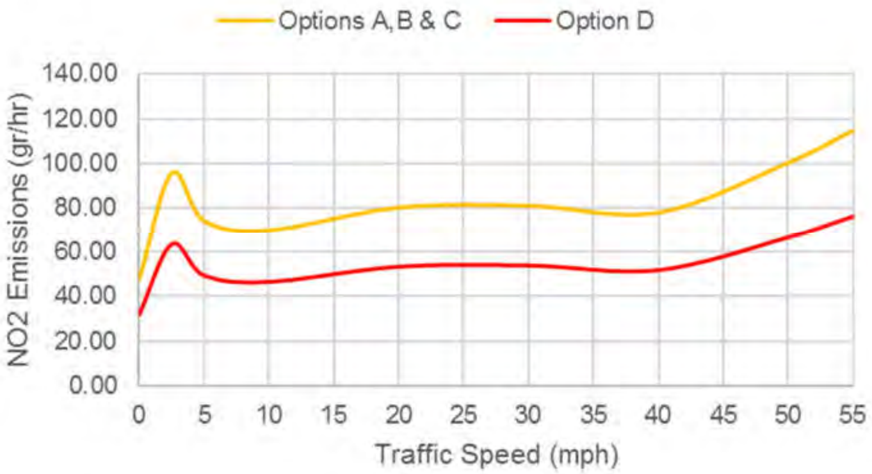


Figure 10.5 - $NO_2$ Emissions for July

The piston effect for the various traffic speeds is assessed numerically with both no-wind and adverse wind conditions, with the latter being 0.00422psi (≈ 29.1Pa) applied as a boundary condition at the exit portal, i.e. opposing the traffic and airflow direction. With the minimum outside air demand to dilute the in-tunnel pollutants to acceptable obtained through standard PIARC methodology, the objective of the analysis was to establish the point where the piston effect from travelling vehicles overcomes the tunnel resistances (including adverse wind effects at exit portal) and achieves the minimum established OAD. The procedure followed to establish this objective is outlined below:

- Establish the point where the induced OA from free-flowing traffic intersects the minimum OAD threshold, i.e. introduced OA due to piston effect is equal or greater than the minimum OAD;
- Obtain the corresponding speed at the above intersection point as a minimum traffic speed limit above which the tunnel can self-ventilate;
- Assess whether standstill and slow-moving traffic would require activation of the tunnel ventilation system to control pollution;
- Reiterate above steps for each investigated pollutant and each tunnel option.

Appendix G presents the results for CO and $NO_2$ for various traffic speeds and for each tunnel option, with the intersection of the OA lines and corresponding traffic speeds summarized in Table 10.6.

Table 10.6 - Naturally Induced Outside Air for Pollution Control

| **Parameter** | **Carbon Monoxide (CO)** | | **Nitrogen Dioxide ($NO_2$)** | |
|---|---|---|---|---|
| | No wind | With wind | No wind | With wind |
| **Option A** | | | | |
| Induced OA ≥ Min OA demand [cfm ($m^3/s$)] | 19,756 (9.3) | 27,665 (13.1) | 28,914 (13.6) | 32,224 (15.2) |
| Minimum Corresponding Traffic Speed (mph) | 0.2 | 10.3 | 0.3 | 10.4 |
| **Option B and C** | | | | |
| Induced OA when ≥ Min OA demand [cfm ($m^3/s$)] | 19,793 (9.3) | 27,009 (12.7) | 28,928 (13.7) | 31,960 (15.1) |
| Minimum Corresponding Traffic Speed (mph) | 0.2 | 9.4 | 0.4 | 9.4 |
| **Option D** | | | | |
| Induced OA when ≥ Min OA demand [cfm ($m^3/s$)] | 13,171 (6.2) | 18,215 (8.6) | 19,277 (9.1) | 21,394 (10.1) |
| Minimum Corresponding Traffic Speed (mph) | 0.2 | 9.8 | 0.3 | 9.8 |

From the results it can be concluded that for a traffic speed in excess of 10mph, the tunnel can self-ventilate for both adverse wind and no-wind conditions; whereas for standstill or slow-moving traffic up to 10mph, the tunnel ventilation system would need to be engaged to provide the required OAD. It is highlighted that the above results are for unidirectional traffic only, and based on current available information that will need to be refined at a future design stage.

# PART E – Tunnel Services

# 11. Mechanical, Electrical & Plumbing Systems

## 11.1. Basis of Design

Colorado have not adopted the NFPA codes of practice; however, CDOT have stated that it is their desire that this tunnel is designed to be compliant with NFPA 502.

## 11.2. Operational Control Center

As discussed in Section 4.5, it is believed that a primary and a backup Operational Control Center is required to manage Floyd Hill Tunnel. The Operational Control Center should be equipped with control and monitoring equipment and staffed to provide safe normal and emergency operations. This control and monitoring equipment will normally consist of a Supervisory Control and Data Acquisition (SCADA) based system monitoring and controlling plant connected to Programmable Logic Controllers (PLCs) local to the tunnel. The locations of these Operational Control Centers are currently undecided, but a study should be undertaken to determine the best strategy for this and other CDOT tunnels in the area.

## 11.3. Fire Alarm and Detection

NFPA 502 Clause 7.4 requires at least one manual means of identifying and locating a fire by the use of manual fire alarm boxes or emergency telephones. Discussions with Denver Fire Department on the Central 70 Project have revealed that manual fire alarm boxes are not favored due to false alarms experienced in other projects. It is also understood that these boxes are likely to be removed from the next update of NFPA 502. It is therefore proposed that the tunnel be equipped with emergency telephones only, with recommendation to discuss with the AHJ. Clause 7.4 requires that if the tunnel is not supervised 24 hours a day or if a FFFS is installed, then an automatic fire detection system shall be installed. NFPA 502 Clause 7.4.3 states that CCTV can be used to identify and locate fires if the tunnel is supervised 24 hours a day. Both automatic fire detection and CCTV are CMRs (as defined in NFPA 502 Table A.7.2), therefore an engineering analysis is required to determine whether these facilities shall be provided.

The safety functions that these systems contribute to are:

- To prevent additional road users entering the tunnel during an incident. They achieve this by providing an alarm to tunnel operators at the earliest opportunity and prompting them to close the tunnel immediately after they confirm that an incident is ongoing. Taking definitive actions early in this context will reduce the number of road users exposed to danger and thus reduce the number of people involved in the incident;
- To permit the effective and safe management of an ongoing incident. CCTV systems achieve this by allowing the tunnel operator to determine the extent and growth rate of an incident, determine the number and disposition of road users affected by the incident, and determine the effectiveness of incident management actions taking place. This permits a deployment strategy for emergency response crews that maximizes their effectiveness whilst minimizing their exposure to risk.

Due to the relatively low cost of these systems and the likely benefits, they are recommended to be included. A fire alarm panel control panel is required by NFPA 502 Clause 7.4.8. It is recommended that the basis of design for these systems are that a fire detection system is provided with 100% coverage at a resolution of 50ft in the tunnel space and in accordance with NFPA 72, and that the CCTV coverage covers 100% of the tunnel, portal areas, immediate approaches and escape routes.

## 11.4. Emergency Communications Systems

Emergency communications systems are CMRs (as defined in NFPA 502 Table A.7.2) and, subject to compliance with other laws, codes or standards, the decision to equip is delegated to the AHJ. This section presents information for AHJ to inform their judgement in this matter.

In this context, emergency communications systems consist of two independent systems: a two-way radio rebroadcast system to allow emergency services personnel and maintainers to communicate between themselves and with the Operational Control Center from within the tunnel; and a system to allow the control center to communicate with road users by means of an interruption to commercial radio transmissions and/or Highway Advisory Radio (HAR).

In terms of the two-way radio systems, the ability of the Operational Control Center to communicate their situational awareness to emergency response teams on the ground will play a major role in ensuring that the emergency plans are enacted in the most effective and safe manner, protecting the first responders and the road users caught up in any incident. An emergency services and maintainer two-way radio system is a relatively low-cost item and the benefits of the provision of such a system would be significant; it is therefore recommended that a two-way radio system be provided.

In terms of the provision, or otherwise, of a public radio break-in system, the effectiveness of the system comes from the ability to influence road users in their vehicle to leave the perceived safety of their vehicle and evacuate immediately in the event of a fire. A voice alarm/public address system provides the additional benefit of being able to communicate with users once they have left their vehicle and provide instructions and assurance until they are in a place of relative safety. This is a common feature in many tunnels and is part of the Central 70 Tunnel. It is therefore recommended that consideration should be given to the provision of a voice alarm/public address system.

It is recommended that the basis of design for emergency communications systems are that the tunnel will be provided with an emergency two-way radiocommunication system for emergency service and maintainer use. The tunnel will also be provided with public radio rebroadcast with local override from the voice alarm/public address system and that the radio broadcast system will also carry the HAR transmissions.

## 11.5. Tunnel Closure and Traffic Control

A method to close the tunnel to traffic entering the tunnel and its immediate approaches is a mandatory requirement of NFPA 502 Clauses 7.6.1 and 7.6.2 (sub-clauses (1) and (2)). Clause 7.6.2 (3) requires a means to be able to clear the traffic downstream of the incident.

To effect Tunnel Closure and Traffic Control, it is recommended that the basis of design provides for the tunnel portal to be equipped with lane-use signs that are capable of displaying a red cross and a green arrow. Within the tunnel bore, similar lane-use signs will be provided to coincide with the emergency exits. In the case of the requirements contained in Clause 7.6.2 (3), it is considered unlikely that the tunnel will suffer from exit blocking and therefore, apart from displaying a green arrow (lane open) sign downstream of the incident, no special measures are required. The operation and control of the lane-use sign settings will be from the Operational Control Centre.

## 11.6. Standpipe, Fire Hydrants and Water Supply

It is recommended that the basis of design provides for the design and installation of a class I stand pipe system to comply with NFPA 14 and NFPA 502. The standpipe system will be a wet system, except if the Colorado Fire Department accept a dry system or a combination of a wet and dry system. The stand pipe system will be protected from freezing by insulation and electric trace heating. The wet standpipe system will be connected to the nearest municipal or private water mains system provided the system is capable to provide the system demand for 1 hour. If there are no suitable water supply mains available, water storage tanks and pumps will be provided to comply with NFPA 22 and NFPA 20. The standpipe system will comprise of Fire Department connection points, hose connection points and interconnecting pipework. The Fire Department hose connection point locations will be collocated with emergency access points and will be approved by the Colorado Fire Department. The hose connection points will be provided to ensure that no part of the tunnel/road will be more than 150ft (≈ 45m) from the hose connection. The Fire Department hose connection points and the standpipe hose connection points will be protected from mechanical damage from vehicles and vandals.

## 11.7. Portable Fire Extinguishers

Portable fire extinguishers are required by NFPA 502 Clause 7.9. It is recommended that these are installed in emergency cabinets to keep them clean and that alarms are provided to advise the operator of their use.

## 11.8. Fixed Fire-Fighting Systems

NFPA 502 describes the implementation of a FFFS as a CMR. Although there is a high level of research and testing of active suppression systems in tunnels, there is not yet consensus on what constitutes acceptable minimum performance for FFFSs. The suitability of a FFFS in Floyd Hill Tunnel lends itself to an assessment exercise that captures the operational and diverse environmental conditions of the system, to allow informed decisions to be taken before implementation.

The analysis in Section 8.5 describes the potential of limiting the peak HRR from the recommended 550MBTU/hr (≈ 160MW) peak HRR to 135MBTU/hr (≈ 40MW) in the presence of a FFFS in the tunnel, with a further potential of limitation to the residual convective HRR seen by the ventilation system. The aforementioned assessment offers a view of the feasibility of this key criterion for a FFFS and other in-tunnel systems that renders advance consultation with the AHJ and stakeholders prior to integration.

If a FFFS is required, it is recommended that the basis of design incorporates a deluge system which will be designed, installed and tested to comply with NFPA 11, NFPA 13, NFPA 15, NFPA 16. NFPA 18, NFPA 18A, NFPA 25 and NFPA 750. The system will be designed to integrate with the FFFS to reduce the growth rate of fire and to reduce the fire HRR. The system will comprise of open nozzles which will be connected to a water supply system via a network of pipework. The deluge system will be zoned with each zone being controlled by its own control valve. This will ensure that the discharge will concentrate on the area/zone of the fire incidence rather than the whole tunnel. The zone lengths will be dependent on vehicle lengths and hydraulics. The nozzles will be spaced to cover the entire tunnel and the coverage will extend to roadway shoulders. To prevent accidental discharge, the system will be manually activated with an automatic release after a time delay. To prevent the development of a major fire, the time delay will not exceed three minutes. The deluge system will be activated by the fire alarm system. A manned control room will be required, the system activation will be relayed to the fire control room which will allow for human interaction for incidence verification and assessment before activation to reduce the chances of false alarm and accidental discharge. The selection of the nozzle will depend on: ventilation regime; tunnel height; nozzle installation height; expected fire load; water application rate; and environmental conditions. Fire Department connection will be provided to boost the deluge system, whilst the pipework will be installed to be self-draining.

## 11.9. Tunnel Ventilation

A mechanical longitudinal ventilation system is proposed for Floyd Hill Tunnel due to its unidirectional nature in controlling smoke in the direction of the traffic. Based on analyses performed in in Section 10, the system comprises of 24 jet fans of 64" (≈ 1,600mm) diameter across 6 banks, spaced at equal intervals of 10 times the tunnel hydraulic diameter, with the first bank at 33ft (≈ 10m) from tunnel entry.

The system fulfills the key objective of achieving critical velocity by overcoming all resistances the flow encounters in the tunnel, in addition to adverse wind and portal effects and in the absence of a FFFS. NFPA 502 specifies critical velocity as the primary criterion to be achieved by the system upstream of the fire location, whilst also considering unavailability of one fan due to potential malfunction during the system operation. Available tunnel cross sections have shown that each bank can accommodate four unidirectional jet fans to control smoke at a fire location in the tunnel, with due consideration of banks destroyed relative to the fire location or de-rated due to their exposure to smoke downstream in the tunnel. As the benefits of stack effect on the size or quantities of fans for conservatism have not been considered, there is an opportunity for optimization at the detailed design stage.

The same system can adequately control in-tunnel pollution levels for standstill or slow-moving traffic in the tunnel in addition to free-flowing traffic at posted speed, in the event of insufficient influx of OA due to a piston effect to permit self-ventilation of the tunnel. It is recommended that the ventilation system will be activated manually by the tunnel operator as part of the tunnel emergency protocols, suitable to the nature and location of a confirmed fire in the tunnel following appropriate feedback from monitoring field equipment. The activation and control of the tunnel ventilation system as part of a holistic emergency response plan requires the collaboration between the tunnel operator and fire rescue services to establish a sequence of actions appropriate to the operation of the ventilation system, which must be explored at the detailed design stage.

## 11.10. Tunnel Drainage System

The drainage system will be designed to ensure that there is no flooding. The drainage will be designed to collect any surface water spill and the discharge from the FFFS on the road and convey it into the nearest utility drainage system or water way by gravity if possible. Where a gravity system is not possible, the system will incorporate storage tank(s) with pumps to discharge the effluent. This could be achieved by grading the tunnel road transversely to low points, which will be provided with drainage collection goods such as channels and gullies as appropriate. The drainage collection goods will be connected to a buried surface water drainage system incorporating pipes and culverts, and will incorporate silt traps and baskets to capture dirt and any highway rubbish and prevent them from entering the drainage system.

The system will also be designed to capture any spill of hazardous or flammable fluid to prevent them from spreading the length of the tunnel. The drainage system will be designed to cater for the discharge rates of the standpipe, FFFS, tunnel washing, fuel spill rate and any other road catchments contributing to the drainage system. Oil interceptors will be provided to the tunnel drainage system to capture any hazardous liquids and prevent them from entering the natural water course. The tunnel drainage storage tanks and pumping chambers will be classified for hazardous locations in accordance with NFPA 70 and NFPA 820. The storage tank and pump chambers will be monitored for hydrocarbon content and will incorporate local and remote alarm systems. Gas or foam fire suppression systems may be required. The pumps will be supplied from a life safety system and will be designed for resilience and reliability. The tunnel drainage system will be installed with non-combustible materials such as steel, ductile iron or concrete.

## 11.11. Electrical Systems

### 11.11.1. Overview

The tunnel shall have an electrical power system that complies with the following requirements:

- NFPA 502 Section 12;
- NFPA 503 Clauses 12.2.1.3 and 12.2.1.4;
- NFPA 70 National Electrical Code;
- NFPA 110 Standard for Emergency and Standby Power Systems;
- NFPA 111 Standard on Stored Electrical Energy Emergency and Standby Power Systems.

It shall use diversity of routing, dual supply and fire protection measures to ensure that all life safety equipment as listed in NFPA Clause 12.1.4 remains operational for a minimum of one hour. All cables in the tunnel shall be resistant to the spread of fire and shall be of a low smoke zero halogen construction, in accordance with NFPA 503 Clauses 12.2.1.3 and 12.2.1.4. The cables shall also be suitable for use in a wet environment. The tunnel shall be provided with Emergency Power in accordance with NFPA 502 Clause 12.4 and NFPA 70 Article 700.

### 11.11.2. General

It has been assumed that as there is a ‘low likelihood’ of significant seismic activity (Section 12.1.3) and hence a highly damaging earthquake, the design process may continue without including the mandatory requirements of NFPA 502 Chapter 12 Electrical Systems Clause 12.1.4 into the design.

The electrical systems shall be designed to support life safety operations, fire emergency operations, and normal operations; they should also be designed to allow for routine maintenance without disruption of traffic operation. The main electrical distribution shall be configured, interconnected and controlled to allow all services to remain operational in the event of a single power supply transformer failure in the substation at either end of the tunnel. Main low voltage (LV) switchboards shall also be configured with interlocking switchgear to allow for emergency standby generator installation to be connected to serve all essential services supplies to the tunnel. Provision shall also be made for a temporary generator connection point in the services building at each end of the tunnel.

## 11.11.3. Emergency Power

### 11.11.3.1. Emergency Standby Generator

Emergency power shall be provided by an emergency standby generator in accordance with NFPA 70 Article 700 (For emergency and standby power systems as NFPA 110). The following systems shall be connected to the emergency power system:

- Emergency lighting;
- Covered section closure and traffic control;
- Exit signs;
- Emergency communication;
- Covered section drainage;
- Emergency ventilation;
- Fire alarm and detection;
- CCTV or video;
- FFFS.

### 11.11.3.2. Emergency Power Circuits

Emergency circuits installed in the tunnel and ancillary areas shall remain functional for a period of not less than one hour, for the anticipated fire condition. Emergency circuits shall comprise one of the following:

- Fire-resistive cables;
- Circuits embedded in concrete that are protected by a two-hour fire barrier system;
- By the routing of the cable system external to the roadway using diversity in system routing as approved, such as separate redundant or multiple circuits separated by a one-hour fire barrier, so that a single fire or emergency event will not lead to a failure of the system.

### 11.11.3.3. Emergency Power UPS System

Two separate UPS systems shall be provided within services buildings located towards each end of the tunnel; one of these will feed the lighting system, whilst the other will feed the remaining safety critical plant. The UPS will be rated in accordance with Clause 7106 of the Manual of Contract Documents for Highway Works Section 2 Part 2. The new UPS specification shall be developed based on the following:

- Three-phase, on-line, double-conversion, static-type, UPS units with 240-minute battery autonomy;
- 20% spare capacity;
- N+1 parallel redundant configuration;
- External wraparound bypass unit.

## 11.11.4. Containment

New containment shall be provided throughout the tunnel for all cabling services. Separate containment systems shall be provided for LV and extra-low voltage (ELV) systems, segregated in line with industry good practice. Armored cables shall be run on cable trays with non-armored cables run in trunking or conduit to suit the required routing; twisted pair cables shall only be run in conduit.

### 11.11.5. Cabling

All cables and associated materials shall be insulated or clad using low sulphur, zero halogen (LSOH) materials, and where required, certain cables will be fire survivable cables.

## 11.12. Tunnel Service Buildings

Various equipment needs to be installed in service buildings near the tunnel. This equipment is typically switchboards, battery backup systems, and control and communications equipment racks. In addition, space will need to be found for transformers and generator(s). Normally two buildings are provided in order to provide resilience from fire or failures.

## 11.13. Tunnel Lighting

### 11.13.1. Basis of Design

It is recommended that the tunnel lighting is based on the guidance given in ANSI/IES RP 22-11 (2011); the data in Table 11.1 is based on the tunnel being unidirectional, with three lanes running East to West.

Table 11.1 - Key Criteria for Tunnel Lighting

| **Description** | **Value** |
|---|---|
| Tunnel length | 2300ft |
| Portal height | 24.6ft |
| Tunnel direction | East-West |
| Traffic flow AADT | >1500 |
| Traffic arrangement | Single direction |
| Tunnel approaches environment | Mountain scene 8 |
| Traffic speed limit | 55 mph |
| Stopping sight distance (assumed level road surface) | 495ft |
| Cyclists | No |
| Exit visibility | Exit not visible |
| Daylight penetration | Good |
| Threshold zone length | To be calculated |
| Threshold zone luminance | To be calculated |

It is clear from a brief analysis of the data above that the tunnel will include a threshold zone, transition zone, interior zone and exit zone lighting. The luminaires should be installed in continuous lines increasing in number from the interior zone up to the maximum required in the threshold zone. Continuous rows reduce flicker which is a major cause of driver fatigue.

### 11.13.2. Lighting Technologies

Lighting technology has developed significantly in the last ten years. There have been significant advances in light sources, luminaire design and lighting control systems; however, these technologies are currently not covered adequately in the latest best practice advice and codes. The following recommendations are based upon best practice design across previous international tunnel projects Atkins has worked on, as well as those within the U.S.

#### 11.13.2.1. Light Sources

Modern durable and energy efficient light-emitting diode (LED) lighting technology is recommended, consistent with widespread best practice. The benefits are particularly pertinent to a tunnel environment in terms of photometric performance, energy efficiency, reduced maintenance, color rendering, controllability of output, flicker, and vibration resistance.

#### 11.13.2.2. Luminaires

Luminaires can now be sealed luminaires that have no internal maintenance requirements. Luminaire drivers and lighting control system interfaces can be grouped into common locations that reduce electrical distribution and can have ground level maintenance access. The luminaire distributions could use pro-beam or counter-beam distributions, as these may be more appropriate than normal symmetrical beam distributions as the tunnel is unidirectional.

#### 11.13.2.3. Lighting Control

Lighting control through a digitally addressable control system offers considerable advantages over conventional analogue control such as switching or phase dimming.

#### 11.13.2.4. Emergency Lighting

Emergency lighting requirements can be divided into three sections:

- Essential lighting to allow the emergency evacuation of the tunnel bore by vehicle;
- Emergency lighting allowing vehicle occupants to evacuate the tunnel bore on foot using the designated escape route/passage system;
- Out of bore emergency lighting covering escape routes, plant rooms, and service accommodation.

The emergency lighting for the tunnel bore will be guided by the requirements of ANSI/IES RP-22-11 with reference to additional requirements of NFPA 502. The ANSI/IES RP22-11 requirements are:

- Average minimum maintained illuminance of 1fc;
- Minimum illuminance of 0.1fc.

The emergency lighting outside the tunnel bore will need to be detailed in accordance with NFPA 101 Chapters 8 and 11, and relevant cross-referenced documents.

### 11.13.3. Lighting Maintenance

Lighting is a major maintenance item within a tunnel environment. Tunnel closures for high level maintenance tend to be ventilation and lighting related. The use of up-to-date lighting equipment will reduce the requirements for maintenance. The advantages of this include:

- Health and safety by reducing the frequency and need for high level maintenance within the bore;
- Minimal tools required to change luminaires;
- Ground level maintenance of luminaire drivers and lighting control systems;
- Increase of service life of luminaires based upon the longevity of LEDs L80 B10 at 100,000 hours;
- Use of vehicle mounted photometric surveys to measure luminance performance through operational life of lighting installation;
- Longer maintenance intervals and shorter on-site maintenance periods increasing tunnel availability and reducing costs.

### 11.13.4. Lighting Recommendations

There are key advantages in the use of a tunnel lighting system that adopts modern lighting technologies. There may be an increase in capital costs; however, the ongoing operational savings in maintenance, service life, consumed energy and tunnel availability substantially are likely to outweigh this. Advantages of LED systems include:

- Less consumed electricity;
- Increased service life of LEDs versus conventional lamp and control gear technologies;
- Better controllability than lamp and control gear technologies;
- Increased diagnostic feedback;

- Ability to site control gear remotely reducing maintenance at height, rationalizing wiring, cable management systems and allowing the installation of sealed LED only luminaires with minimal maintenance;
- The use of continuous diming of light sources in lieu of conventional staged switching, thus extending service life;
- Sealed luminaires are much more robust and do not have seals and failure points as there is no on-site maintenance to be carried out;
- Advanced optical control and luminaire photometry;
- Reduction in luminaire weight;
- Use of standard open digital control protocols to optimize sharing of data allowing smart maintenance through the tunnel management system;
- Reduction in tunnel support systems due to reduction in weight of lighting system;
- Luminaires become ‘plug and play’ with sealed connection units and easy release luminaire fixings;
- Drop-in electrical load reduces the size of the electrical distribution system and size of generator/no break (UPS) electrical systems, thus reducing size plant and plant accommodation.

The lighting within the tunnel should be considered from a whole life operational standpoint. The operational regime is inevitably linked with the lighting strategy and the fire/power failure strategy. The use of modern lighting equipment with an appropriate track record should be considered. This approach needs to be adopted in a wider context than just the lighting systems to ensure the best value for CDOT over the life of the tunnel.

# PART F – Tunnel and Portals Design Development

# 12. Geology and Ground Conditions

## 12.1. Geology

### 12.1.1. Regional Geology

A wide range of geological conditions are represented and exposed along the I-70 corridor due to the large period of time represented in the multiple rock formations. The geological time reflected along the corridor ranges from recent river, debris and mudflow deposits to Precambrian rocks between one and two billion years old. The Precambrian age metamorphic and igneous rocks are intruded by Precambrian, Tertiary and Cretaceous age stocks and numerous porphyritic dikes. The regional rock type of most relevance to the tunnel is biotite gneiss, identified in Figure 12.1 – which shows the geological map of the area as well as an approximate tunnel alignment. The most common porphyries range in composition from Precambrian pegmatite and lamprophyres to Cretaceous quartz monzonite and granodiorite.

Most of the present configuration of the area is characterized by moderately rugged topographic relief. The mountains to the South and North are deeply incised by Clear Creek Canyon and its tributaries. The maximum local relief is about 3,000ft. The elevation in the project area ranges from slightly over 7,000ft along Clear Creek to more than 10,000ft at Santa Fe Mountain to the south. Slopes are typically steep, averaging approximately 35° on the proposed East Portal rock face. Topographic forms are generally influenced by minor faulting, fractures, and zones of weakness in rock. In addition, rain, snowmelt, freeze-thaw, wind and Clear Creek have created deposits of alluvium (stream deposits), talus (rockfall deposits) and alluvial fans.

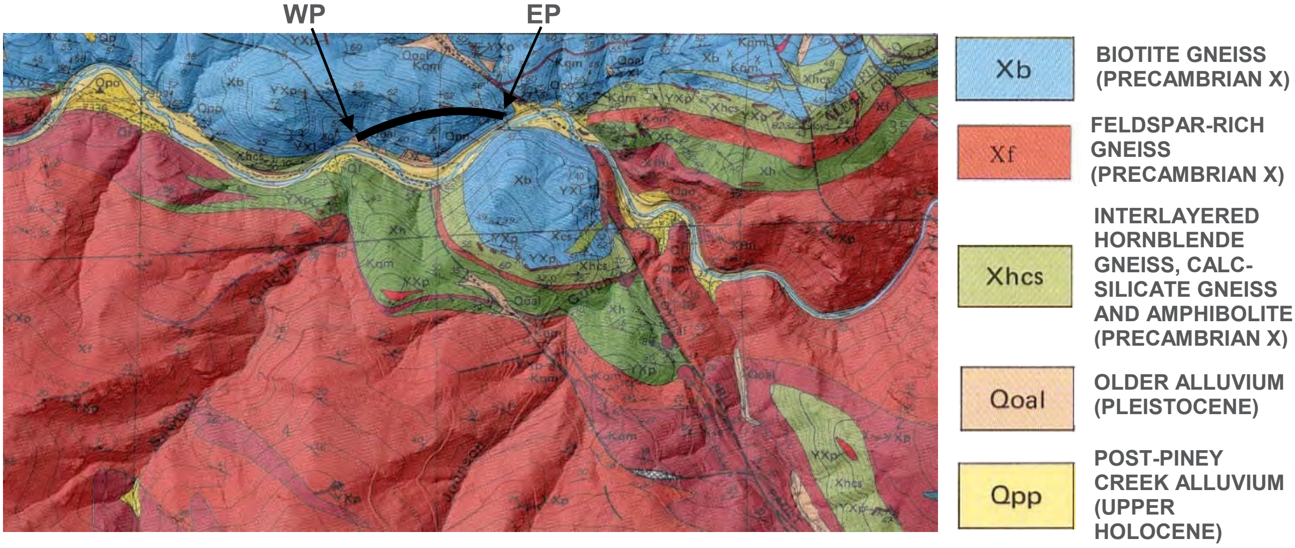


Figure 12.1 - Modified Geological Map of the Squaw Pass Quadrangle, Clear Creek, Jefferson and Gilpin Counties, Colorado [19]

### 12.1.2. Local Geology

Bedrock in the project area is primarily Precambrian metamorphic gneiss and migmatite. Biotite gneiss is the predominant mapped bedrock along the proposed tunnel alignment, however, the geological map includes feldspar gneiss, hornblende gneiss, calc-silicate gneiss, and amphibolite, as shown in Figure 12.1. Locally the bedrock is also known as migmatite, a composite rock consisting of igneous and metamorphic portions. In Colorado, migmatite is generally a blend of quartz pegmatite or granite intruded into a metamorphic host rock and intensely deformed. Biotite gneiss in the immediate area of the proposed tunnel is generally light gray to medium gray in color and fine to medium grained. Precambrian pegmatite dikes, lamprophyre dikes, and Cretaceous quartz monzonite porphyry or granodiorite porphyry dikes are also mapped in the project area.

Some of the rock exposed in the existing cut along I-70 shows up to 1ft thick parallel layering while some of the rock is more schistose, well foliated or banded. Foliation generally trends North-Northeast, and lineations generally plunge 45-55° to the North-Northwest based on the Squaw Pass Quadrangle geological map. In many places, the rock is highly deformed by folding and faulting. Chaotic folding is common in the project area and is typical of migmatite. Measurements of these folds is typically not feasible or indicative of regional tectonic movement. Pegmatite is common, generally concordant in the banded gneiss and cross cutting intrusively in the migmatite (Figure 12.2). The density and orientation of fracturing is highly variable in the proposed tunnel area.

Figure 12.2 - Pegmatite Intrusion into Migmatite With Biotite Inclusions

The bedrock at the West Portal is covered by Pleistocene alluvium. The alluvium ranges from gray to brown in color and consists of sand and silt with some gravel and larger material. This alluvium deposit is probably age equivalent to the Broadway and Louviers Alluvium formations found elsewhere in the Front Range. The larger grains tend to be well rounded.

The tunnel is planned between two mapped limbs of the Floyd Hill Fault. The Floyd Hill Fault generally trends North-Northwest. Few fractures exposed along the existing rock cut are continuous. Those that are continuous show very weak hydrothermal alteration or oxidation staining, as shown in Figure 12.3. Some breccia zones are also present.

Figure 12.3 - Example of Minor Oxidation and Hydrothermal Alteration Along a Fracture

The tunnel project site is located East of historical metal mining activity known as the Idaho Springs Mining District. Bedrock in the Floyd Hill Tunnel area has few sulfide veins and displays only very weak sulfide mineralization or alteration along select fractures. At least three prospect pits have been dug in the vicinity of the proposed tunnel as shown on the Squaw Pass Quadrangle map. Other possible sites of exploration activity or borrow pits have been observed at the West Portal area in the alluvium, and only minor evidence of rock excavation was observed in a breccia zone.

### 12.1.3. Seismicity

The site is classified as Site Class B in accordance with AASHTO LRFD Bridge Design Specifications Table 3.10.3.1-1. The Peak Ground Acceleration (PGA), and the short- and long-period spectral acceleration coefficients ($S_S$ and $S_1$ respectively) for the Twin Tunnel site were obtained using the USGS/AASHTO 2007 Seismic Parameters for an event with a 7% Probability of Exceedance (PE) in 75 years and a Site Class B (reference site). An event with the above probability of exceedance has a return period of about 1,033 years. Since the site classification (B) is the same as the reference site (B), no value adjustments were necessary. The seismic parameters for this site are shown in Table 12.1 and Table 12.2.

Table 12.1 - Seismic Design Parameters

| PGA (0.0 sec) | $S_S$ (0.2 sec) | $S_1$ (1.0 sec) |
|---|---|---|
| 0.068 | 0.141 | 0.036 |

Table 12.2 - Seismic Design Parameters for Site Class B

| $A_S$ (0.0 sec) | $S_{DS}$ (0.2 sec) | $S_{D1}$ (1.0 sec) | Seismic Zone |
|---|---|---|---|
| 0.068g | 0.141 g | 0.036 g | 1 |

The American Society for Civil Engineers (ASCE) classifies seismic activity using the $S_{DS}$ and $S_{D1}$ values, as shown in Table 12.3 below.

Table 12.3 - Seismicity Definitions (ASCE41-13 Table 2-5)

| Level of Seismicity | $S_{DS}$ | $S_{D1}$ |
|---|---|---|
| Very low | < 0.167g | < 0.067g |
| Low | > 0.167g | > 0.067g |
|  | < 0.33g | < 0.133g |
| Moderate | > 0.33g | > 0.133g |
|  | < 0.50g | < 0.20g |
| High | > 0.50g | > 0.20g |

Using the ASCE classification above, the level of seismicity in the area local to the proposed Floyd Hill Tunnel area is considered to be ‘very low’. On this basis, it is unlikely that any special provisions for seismicity will need to be incorporated in the design of the Floyd Hill Tunnel.

## 12.2. Ground Investigation

To continue on from the desk-study approach which provided background on the regional and local geology, a preliminary Ground Investigation (GI) exercise was carried out for the tunnel. This consisted of the drilling of five boreholes; two were located at the East Portal location (EP-1, EP-2), whilst three were drilled at the West Portal location (WP-1, WP-2, WP-3) – this is displayed in Figure 12.4, which shows the boreholes location in relation to the alignment of the tunnel.

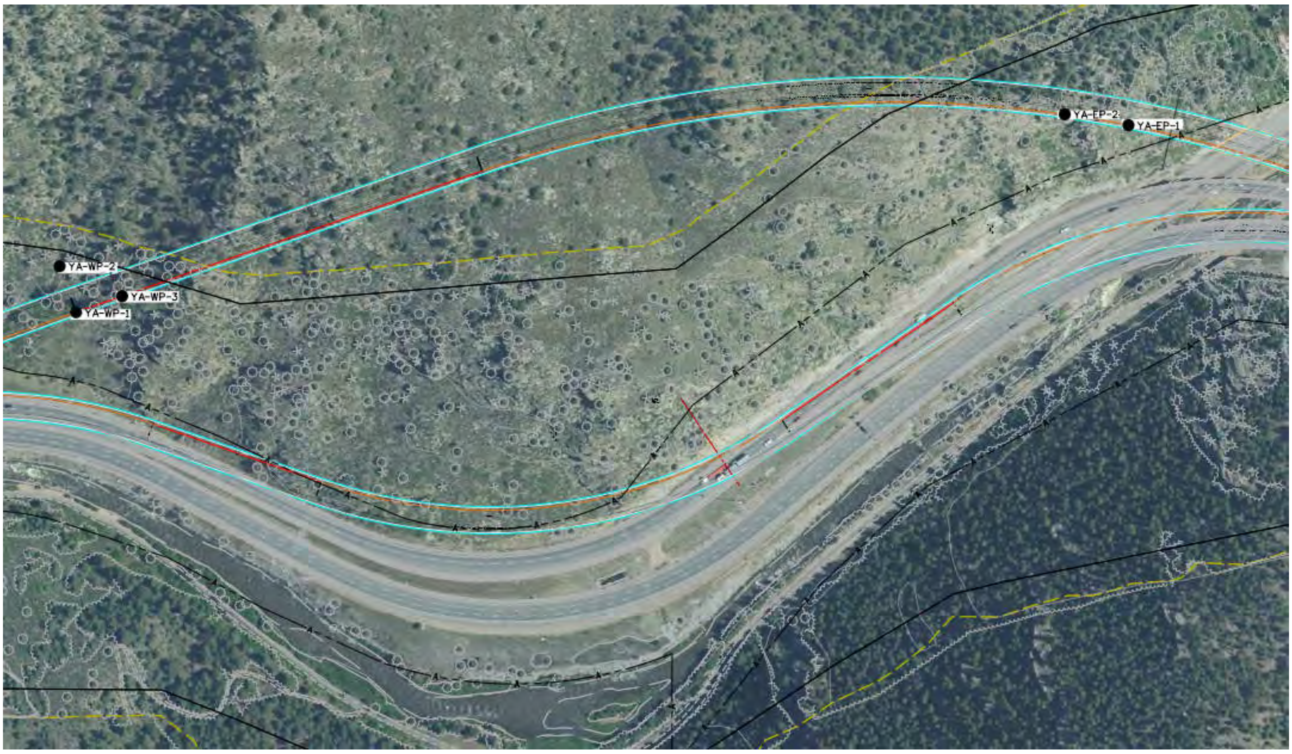


Figure 12.4 - Location of Five Boreholes Drilled for the Ground Investigation

The following information was provided by the GI for each of the five boreholes:

- Borehole logs;
- Core box photos;
- Optical televiewer plots and summary tables;
- Stereonet diagrams (Schmidt projection);
- Rose diagrams (dip directions and dip angles).

The borehole logs contain key bits of information, such as: a detailed description of the material; the depth below ground level at which that material was located; and the depth below ground level at which groundwater was struck (if at all). In addition to this, an RQD (%) value was provided, which was subsequently used as an input for classifying the quality of the rock. The final piece of information provided as part of the GI package was a preliminary classification of the rock mass across the length of the tunnel, based on three different systems (two of which use the RQD value). The inputs used as part of these classifications, as well as the different methods used are discussed further in Section 12.4.

## 12.3. Groundwater

The borehole logs produced as part of the GI were used to gain an understanding of the groundwater profile along the proposed alignment of the tunnel. Two of the boreholes (EP-1 and WP-2) struck water at a depth of 17ft and 34ft (below ground level) respectively (Figure 12.5), whilst the three remaining boreholes have no log for groundwater level. Given the elevation and local topography of the Floyd Hill Valley, it is likely that the source of this water is discrete and unconnected bodies of water located in joints, principally collecting above the tunnel from rain water and snow met with the subsequent infiltration of water from these events into the rock mass. In terms of tunnel construction, joints that are large could contain volumes of water that may be deemed significant, due to the high pressures present during construction. Once joints (if any) filled with water have been drained during the construction phase, it is likely that the ground around the proposed Floyd Hill Tunnel will be relatively dry with low flows of water expected, thus having limited effect on the permanent design of the tunnel.

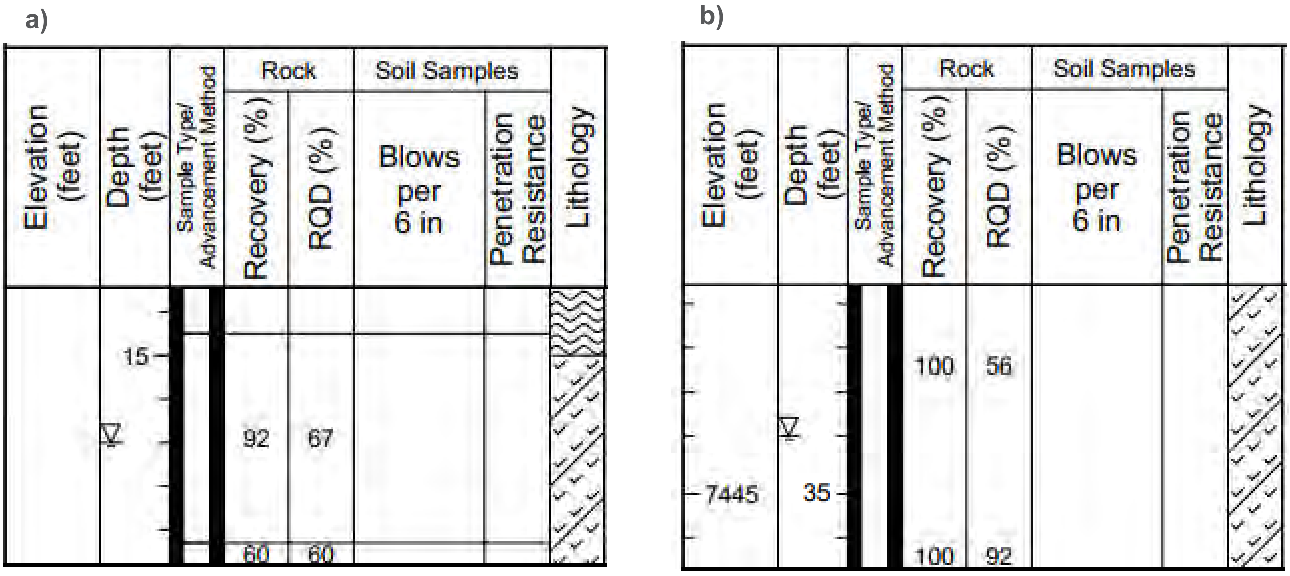


Figure 12.5 - Groundwater Strikes Taken from Boreholes: a) EP-1; b) WP-2

## 12.4. Rock Design Parameters

The assessment of ground conditions carried out is a preliminary assessment to estimate the quality of rock, understand the likely ground support required and approximate the type of tunnel structure(s) required, in order to inform the cost and construction programme aspects of this feasibility study. This should be extended to an observationally-selected approach at the subsequent stages of design and construction as increased geological information becomes available.

### 12.4.1. Rock Mass Classification

As mentioned in Section 12.2, a key output from the GI was the classification of the rock mass based on three different methods: Q-system; rock mass rating (RMR) system; and rock mass index (RMi) system. For each borehole, the geological information pack included the inputs/ratings used for each classification system, and subsequently the output values calculated from each system. Of the five boreholes, three boreholes were drilled to a depth below the tunnel invert level (WP-1, WP-2, WP-3), one borehole was drilled to a depth between tunnel axis level and tunnel invert level (EP-1), and one borehole was drilled to a depth above the tunnel crown level (EP-2). The main outputs were values for three types of rock mass classification systems – these are displayed in Table 12.4.

Table 12.4 - Rock Mass Classification Ratings for Five Borehole Logs

| Borehole No. | RMR | Q | RMi |
|---|---|---|---|
| WP-1 | 57 | 2.24 | 1.15 |
| WP-2 | 56 | 3.92 | 1.78 |
| WP-3 | 64 | 3.31 | 1.50 |
| EP-1 | 50 | 3.36 | 1.51 |
| EP-2 | 50 | 3.01 | 1.38 |

The Q, RMR and RMi systems all use different geological and hydrological characteristics of the rock, based on both GI data and some judgement, in order to provide an overall classification for the rock. The Q and RMR systems are the most alike, in terms of the input values required and the classification scale (very poor to very good) used for the rock mass. The rock obtained from the four boreholes at tunnel axis level was categorized as ‘poor’ rock (Q = 1-4) according to the Q-system, whilst the RMR system classifies it as ‘fair’ (RMR = 41-60).

The Q-system was used previously to determine the rock classification and hence the primary ground support required for the Veterans Memorial Tunnels (VMT) Project, thus the same approach was applied in order to recommend the primary ground support for Floyd Hill Tunnel (Section 14.2.1). The Q ratings provided for each borehole were based on the updated Q-system [20], which considers: degree of jointing (through rock quality designation (RQD)), jointing pattern, joint characteristics, groundwater, and rock stresses (through stress reduction factor (SRF)).

The rock mass classification results have been used as the main criteria for determining the primary ground support required for the tunnel – this procedure has been outlined in Section 14.3.1. In addition, the Q-value for the rock mass, as well as an ensemble of other methods/approaches, were used to determine additional parameters, which have subsequently informed the design of the secondary lining (e.g. used to determine the ground loads acting on the lining), and are discussed in Section 12.4.3.

### 12.4.2. Assessment of General Rock Mass Quality

Following classification of the rock mass, a further assessment of the general rock quality and distribution has been made. This has primarily been undertaken using RQD and rock descriptions from the borehole logs and provides an additional understanding of the rock support required. In all cases the various parameters for a Q assessment of the borehole have been taken from the description of the rock mass. A detailed assessment of the Unconfined Compressive Strength (UCS) has not been undertaken due to the lack of information.

Assessing the general rock quality, the predominant description of the rock and an estimate of the rock quality indicates that the rock is generally of a similar type and of a similar quality. A typical description of the rock might be:

**Migmatite** – greenish gray to white-black, slightly weathered to fresh, medium hard to hard, smooth surface joint, close spacing (2in to 1 ft), open fractures with iron oxide staining

The Q value for this type of material varies along a borehole string but tends of reasonable quality. However, locally in all boreholes there are indications of zones of weakness where there is a distinct local drop in the quality of rock. Generally, all weakness zones are indicative of rock that is not expected to cause a problem for the design or installation of the rock support. Most zones are indicative of regions where the Q value is greater than 1.0. The single weakness zone where the Q is below 1.0 is of significant concern. It is located towards the end of borehole EP-2 and is positioned just over the crown of the expected tunnel excavation, with the distance between the crown of the tunnel and the weakness zone varying depending on the option being considered. The orientation and shape of the weakness zone is unknown as it does not trace through to other boreholes; this would need to be understood to understand the implications of the weakness zones on the tunnel rock support. The thickness of this weak region is 23ft at the borehole and the borehole log includes little description of the material in the borehole at this depth. Currently the orientation of the weakness zones is not definitively known;

Based on the available information this feature does pose a significant risk to the construction of both the tunnel and the rock walls for the portals. If it is assumed to be a planar feature, approximately at the level where it has been identified in the borehole, then the crown of the tunnel can be expected to be unstable whilst it is unsupported. This will restrict blast lengths during excavation of the tunnel where this feature is over the crown, resulting in a slower advance rate. It will also require additional support, requiring rock bolts to both penetrate the weak zone (requiring an increased length) and a higher strength from the bolts if the weak region does not contribute to a stabilizing arch within the rock mass. The weak zone could also significantly influence the rock support required in any rock faces at the portal that it intersects.

A detailed assessment of this would need to be undertaken once more complete information on the nature and geometry of this particular weakness zone becomes available. As such it is essential that further investigation is undertaken of this weakness zone to clarify the nature, extent and orientation of this feature relative to the proposed tunnel alignment and portal location.

### 12.4.3. Additional Parameters

A number of other design parameters specific to the rock mass have been calculated in order to provide suitable information and data required to design and analyze the secondary lining, and also determine the size/location of the portals. The different parameters calculated as part of this exercise are summarized below:

- Worst case dip of the main joint sets (used for roof/wall wedge block failure loads);
- Elastic modulus of the rock, $E_r$ (used to determine spring stiffnesses for numerical model);
- Friction angle of the joints (used as an input to calculate wall load);
- Rock load depth (used to determine generalized rock load);
- Factored arch depth (used to determine minimum rock cover and location of portals);
- Pillar widths (used to determine minimum distance between main highway tunnel(s) and/or egress tunnels for Options C and D respectively)

# 13. Spaceproofing

## 13.1. Main Highway Tunnel

### 13.1.1. Three-Lane Highway (Single Bore Tunnel)

A total of three highway lanes will run through Floyd Hill Tunnel (Options A, B and C). During normal operational conditions traffic flow will be unidirectional. Figure 13.1 demonstrates the space requirements for the three-lane highway during normal operational conditions, as specified by CDOT. A road thickness of 2ft was assumed for the highway, based on the pavement design for VMT; the road thickness is subject to change.

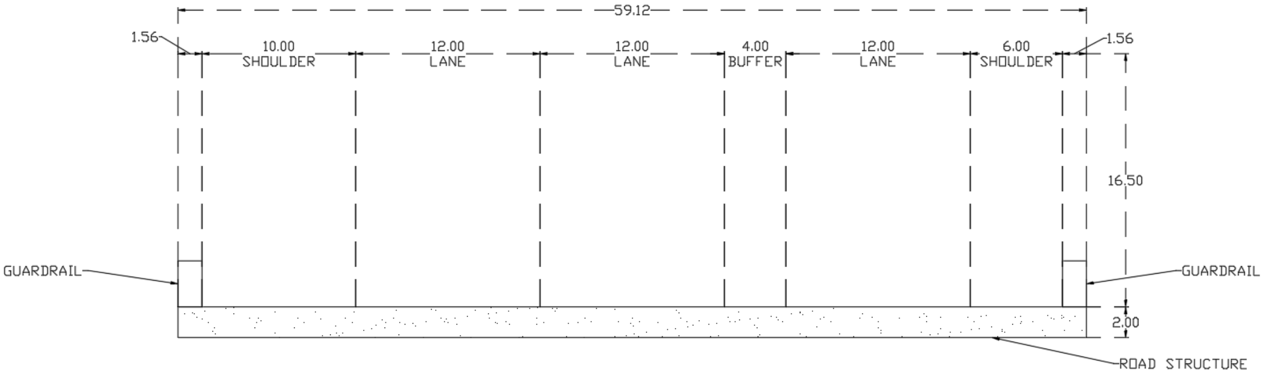


Figure 13.1 - Three-Lane Unidirectional Traffic Flow Representing Normal Operating Condition for Options A, B and C

In exceptional circumstances the tunnel may be required to accommodate bidirectional traffic flow. Events which could require this flow mode include: planned construction works on the Eastbound carriage; maintenance on the Eastbound carriage; and emergency incidents on the Eastbound carriage that require it to be temporarily closed. In these circumstances, the tunnel will accommodate a total of four lanes of traffic, with two lanes in either direction. Figure 13.2 demonstrates how the road will be reconfigured to facilitate this.

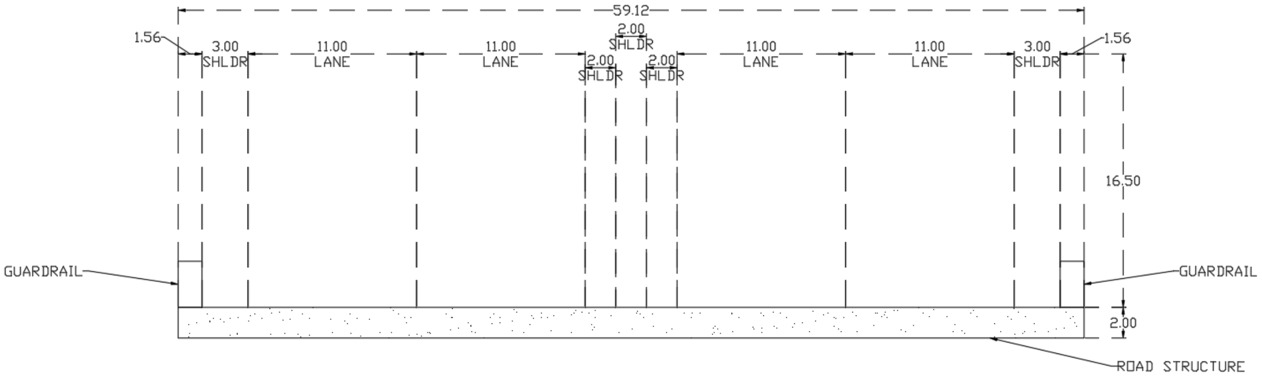


Figure 13.2 - Four-Lane Bidirectional Traffic Flow Representing the Special Operating Condition for Option A, B and C

The tunnel intrados, including any mechanical and electrical equipment fixed to the intrados, must not encroach upon the highway envelope. The geometry for the tunnel layout options has thus been developed with this as a key constraint. For all tunnel layout options, a minimum separation of 1.56ft (typical width of the permanent guardrails) from the top corners of the highway envelope to the tunnel intrados was initially provided to allow for fixings of mechanical and electrical equipment in this area, as well as fire boards (see Figure 13.3).

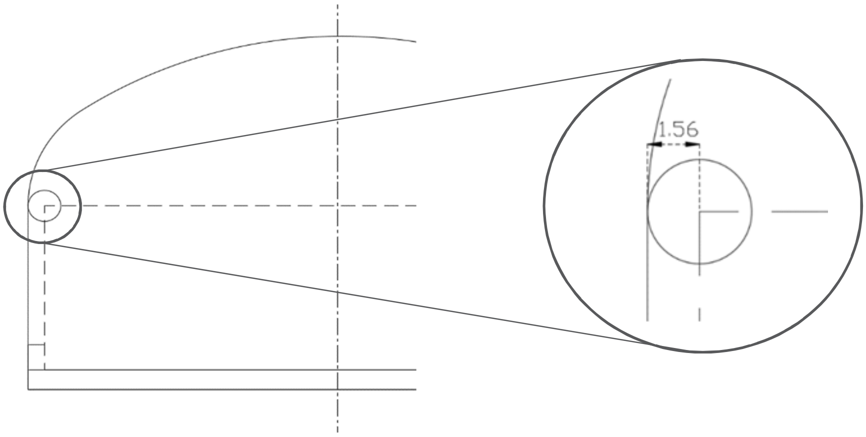


Figure 13.3 - Initial Minimum Separation Detail Between Vehicle Envelope and Tunnel Intrados

The cross fall of the road surface through the tunnel varies between -2% to 4.4%, with a minimum cross fall of 0% located within the super elevation transitions. Any cross fall on the road surface will cause rotation of the highway envelope. For simplicity and ease of construction, the tunnel will retain vertical sides throughout the excavation. The tunnel layouts were thus sized to allow for the increased space requirements of the highway envelope (using the maximum cross fall) rotated by 2.52° (4.4%). Considering the requirements and constraints, the envelope in Figure 13.4 was derived which the intrados of the tunnel must not encroach up, highlighting the difference in tunnel widths when the maximum cross fall is applied. The adapted envelope (to take into account of cross fall) has been taken as a maximum profile to suit all cross falls; therefore, the tunnel profile has been assumed to stay consistent throughout the length of the tunnel. Consequently, the excavated volume of the tunnel and hence any aspects dependent on the geometry of the tunnel (e.g. travelling shutter for construction of the secondary lining) will also be the same.

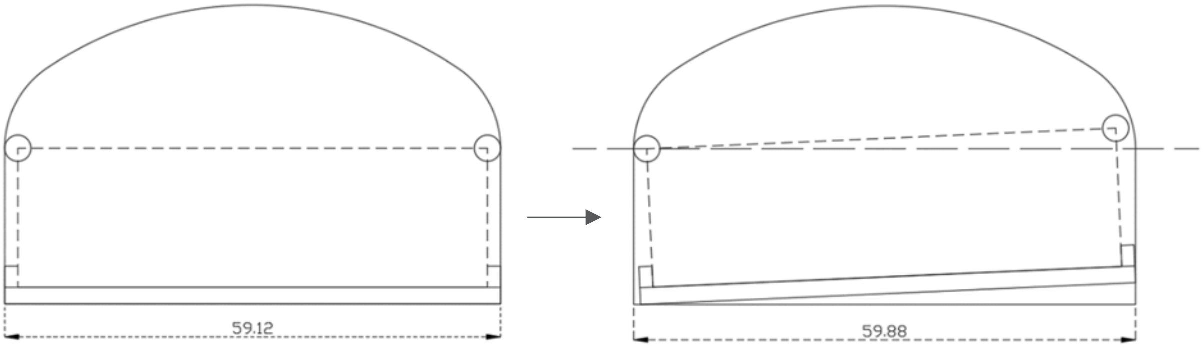


Figure 13.4 - Width Increase for Three-Lane Highway Tunnel Intrados Profile due to Consideration of Crossfall

A typical curved soffit profile was initially adopted for the cross sections, similar to the form used for the VMT. The roof has two different curvatures, the first is a transitional curve from the vertical side walls to the main curved roof profile (shoulders), the second creates the main (flatter) curved roof profile (crown). The profile of the tunnel soffit has been shown as indicative only; it is therefore subject to change in the later design stages, in order to meet the structural requirements of the tunnel lining design and the spaceproofing requirements for M&E equipment.

### 13.1.2. Two-Lane Highway Variant

Tunnel Layout Option D utilizes two separate highway spaces – achieved through the use of twin bore tunnels. This option presents a potential operational benefit over the base case of one tunnel bore by allowing traffic flow to continue in the event of closure of one highway (albeit with two lanes only). In the two-lane envelope option, each highway would only be required to carry two lanes of traffic, with traffic flow in both envelopes providing the required three lanes of traffic. Figure 13.5 demonstrates the spaceproofing requirements for the two-lane envelope.

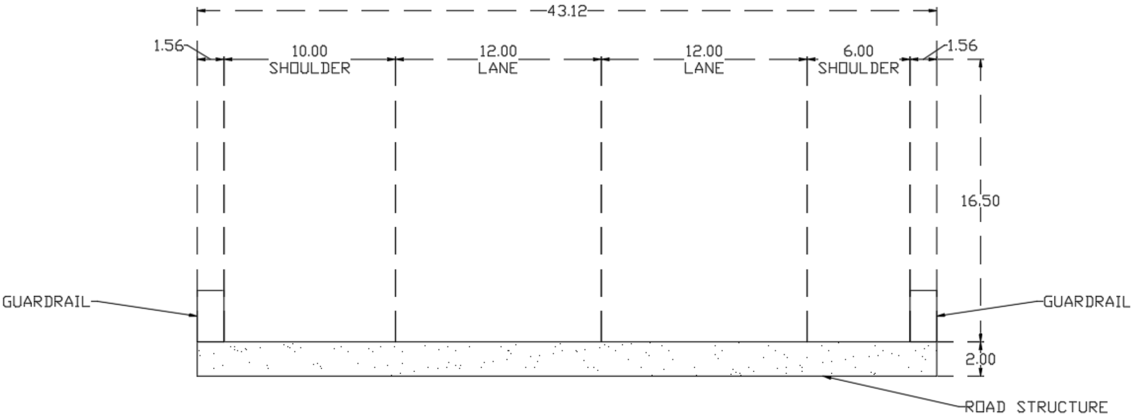


Figure 13.5 - Two-Lane Unidirectional Traffic Flow Representing Normal Operating Condition for Option D

As with the three-lane highway, the two-lane highway also initially maintained a minimum separation of 1.56ft between the tunnel intrados and the top corners of the highway envelope. The maximum cross fall gradient of 4.4% also applies to the two-lane variation and was taken account of in the spaceproofing of the tunnel. Considering the requirements and constraints outlined above, an envelope shown in Figure 13.6 was derived which the intrados of the tunnel must not encroach upon.

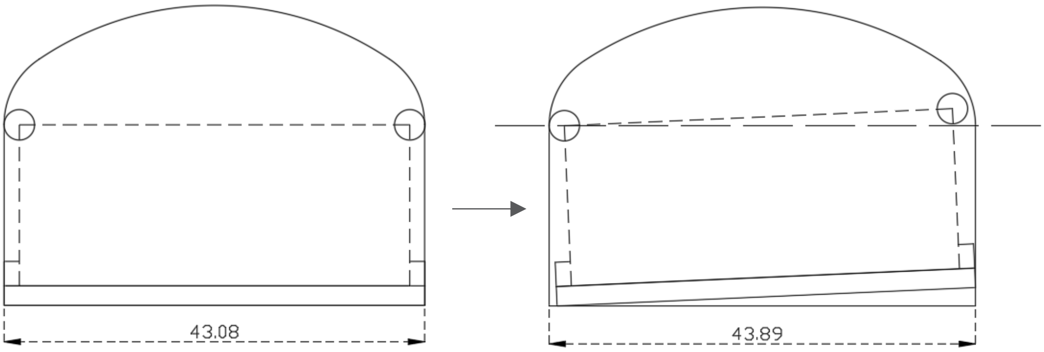


Figure 13.6 - Width Increase for Two-Lane Highway Tunnel Intrados Profile due to Consideration of Crossfall

A full drawing showing the three different highway lane configurations and traffic envelopes can be found in Appendix H.

### 13.1.3. Revision of Tunnel Profile

Following the initial spaceproofing of the tunnel, an exercise was undertaken to reduce the cross-sectional area of the main highway tunnel for the different Tunnel Configuration Options. The aim of this exercise was to reduce the amount of rock excavation required, by lowering the roof (thus creating a flatter roof profile), in order to provide more accurate cost estimates for construction of the tunnel. For Option B and C, the amount of space in the tunnel roof, and similarly between the vehicle envelope and the shoulders of the tunnel, to allow for M&E equipment, was deemed excessive. As a result, this space was reduced, thus reducing the cross-sectional area of the tunnel. However, a trade-off had to be made to ensure the finalized tunnel shape would be efficient under loading – therefore, engineering judgement had to be used to determine the point at which the tunnel profile should not be reduced further. Option A also experienced a reduction in the cross-sectional area of its main highway tunnel; however, the reduction in the roof space was less due to the large span of the tunnel and therefore the need to maintain a structurally efficient profile. For Option D, the main highway tunnel cross-section was not reduced further. The main reason for this was due to the large space required by the ventilation fans, as the same number and size of fans have been specified for Option D as for Options A-C. Due to the limited space within the tunnel soffit, the fans for Option D have been given a clear spacing of 3.0ft (≈ 0.9m), compared to the 5.9ft (≈ 1.8m) clear spacing used for Options A-C.

Another revision to the tunnel profile following the initial spaceproofing exercise carried out was the detailing of the invert. The invert had initially been drawn as flat; however, this has been modified so that it follows a sloped profile, with the same gradient as the crossfall of the road. Further details regarding the invert, particularly the profile that can be achieved using the drill-and-blast method, as well as the interface between the vertical tunnel walls and the invert will need to be considered in the following design stages.

The drawings showing the profile of Tunnel Configuration Options A-D, including details of the tunnel lining, can be found in Appendix E. Following recommendations for primary and secondary lining thicknesses outlined in Section 14.3, the extrados (external face of the tunnel) dimensions have been shown in the drawings; however, construction tolerances (e.g. rock excavation, lining placement, etc.) have not been included but must be considered in the subsequent design stages.

## 13.2. Egress Routes and Cross Passages

NFPA 502 Clause 7.16.1.1 states '*the means of egress requirements for all road tunnels […] shall be in accordance with NFPA 101, Chapter 7, except as modified by this standard*'. Spaceproofing requirements for the provision of egress have been sourced from NFPA 502, and where information is not provided, NFPA 101 has been used:

- '*The egress pathway shall have a minimum clear width of 1.12 m (3.7 ft), lead directly to an emergency exit, and be protected from traffic*' (NFPA 502 Clause 7.16.6.3.2);
- '*Headroom shall not be less than 7ft 6in (2285mm). Projections from the ceiling shall provide headroom of not less than 6ft 8in (2030mm), with a tolerance of -¾in (-19mm) above the finished floor*' (NFPA 101 Clause 7.1.5.1);
- '*Headroom on stairs and stair landings shall not be less than 6ft 8in, and shall be measured vertically above a plane parallel to, and tangent with, the most forward projection of the stair tread*' (NFPA 101 Clause 7.1.5.3).

There are no particular provisions in NFPA 502 which define the required length of the passages connecting the main highway tunnels with any dedicated egress tunnels or other highway tunnel bores (for egress purposes). Furthermore, there is no particular guidance on the number of doors required to separate the highway tunnel from the egress tunnels. In the absence of guidance, it was assumed that a single door would be required to separate the highway tunnel from the egress tunnels, and that a minimum separation of 10ft should be provided between the tunnels to provide sufficient space to allow for full inwards opening of doors into the connecting passages. The height and width of cross passages and egress tunnels are shown in Figure 13.7. The respective dimensions of each have been based on engineering judgement and are therefore subject to change; however, the current provision for egress routes and cross passages pass the requirements set out in NFPA 101 and NFPA 502. It is likely that constructability requirements (e.g. space needed to evacuate operatives with a stretcher) for the egress tunnels will result in sizes significantly higher than the minimum sizes required to meet the conditions of NFPA 502 and must be taken into consideration in the later design stages of the project.

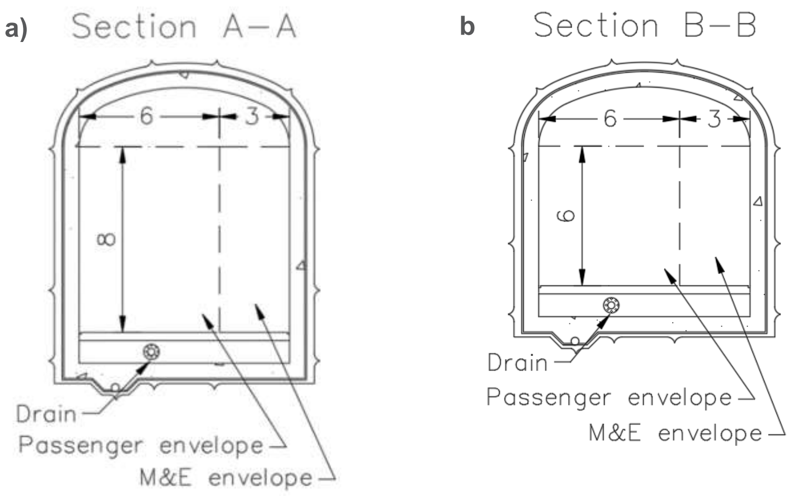


Figure 13.7 - Cross-Section Detail of: a) Cross Passage; and b) Egress Tunnel

# 14. Tunnel Lining Development

## 14.1. Tunnel Lining Concept

A tunnel lining concept has been developed, appropriate for all configuration options considered in this feasibility study. The lining concept is suitable to demonstrate technical feasibility, and provides a basis for estimation of cost and construction durations for the four tunnel configuration options currently under consideration.

The key features of the tunnel lining concept are outlined below, with a typical section of the tunnel lining shown in Figure 14.1. Rationale for selection of these features are provided in Section 14.2.

- Initial rock support provided by sprayed concrete lining (SCL) and rock bolts. The level of primary support has been assumed to be consistent throughout the main highway tunnel's excavation length;
- Secondary lining formed from *in situ* concrete, reinforced with conventional bar reinforcement;
- Lining will be drained, utilizing a dimpled drainage membrane and drainage channel which will be placed between the primary and secondary linings;
- Fireproofing will be provided by insulating fireboards, which will limit temperature increases within the tunnel lining in accordance with the requirements of NFPA 502;
- Design life is assumed to be a minimum of 100 years. Maintenance of the lining structure across its operational life to be minimized as much as practicable.

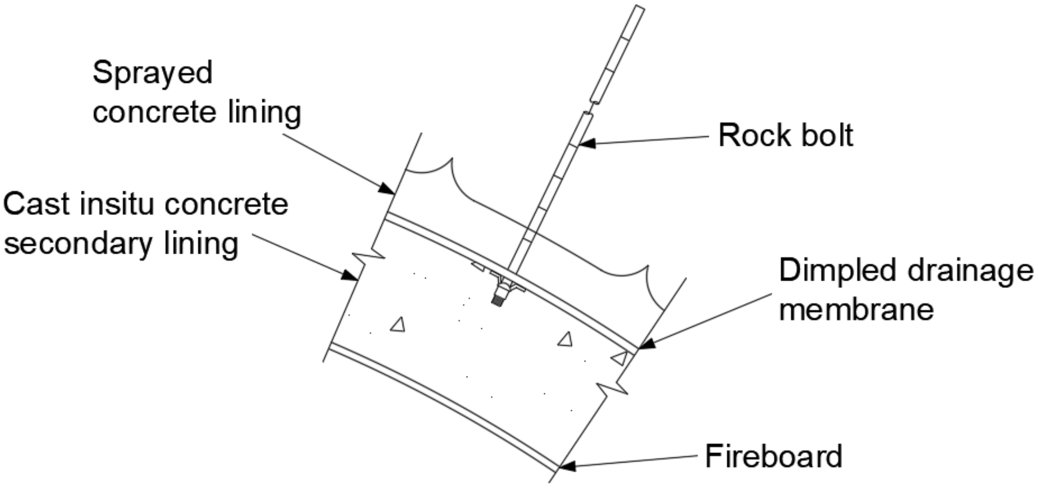


Figure 14.1 - Example Detail of the Tunnel Lining

## 14.2. Rationale for Development of Tunnel Lining Concept

### 14.2.1. Primary Ground Support

In most circumstances, tunnels excavated in rock require some form of primary ground support before the permanent lining is installed. The purpose of the primary ground support is to stabilize the excavation, preventing rock falls from occurring during the construction process. The form of primary support provided is largely dependent upon the quality of the rock through which the tunnel is excavated.

Initial geological information from boreholes taken at the East and West Portals suggests that the rock mass is of 'fair' (according to RMR classification) and 'poor' (according to Q classification). It must be noted however that the RMR and Q classifications are not synonymous, as their use originates in different locations, for different rock types and for different purposes. A Q rating of 'poor' (Q = 1-4) indicates that the rock will require both sprayed concrete and rock bolts to provide support to the excavation in the temporary condition, as highlighted in Figure 14.2.

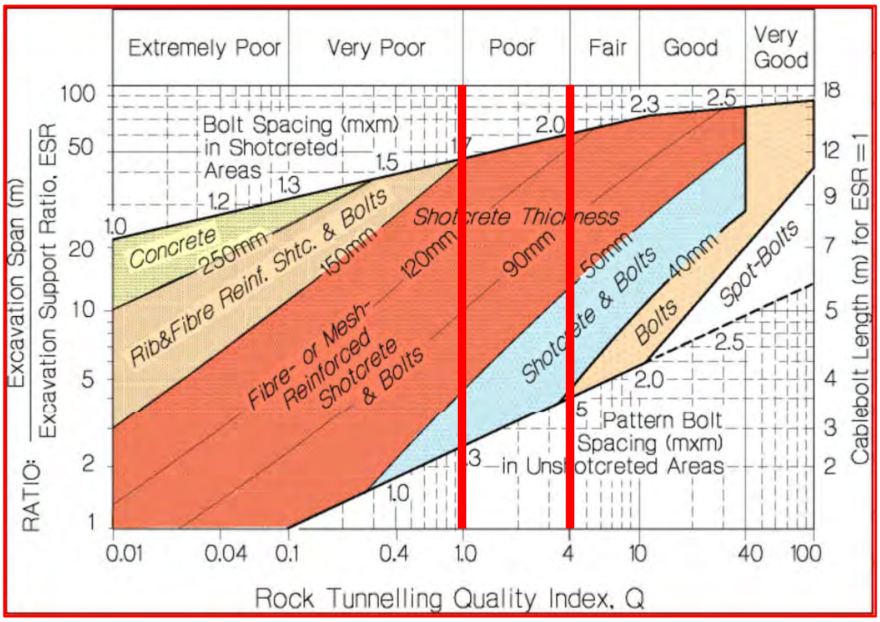


Figure 14.2 - Simplified Rock Support Chart [21]

Water strikes were only encountered in two of the five boreholes taken at the tunnel portals. Given the elevation and local topography of the Floyd Hill Valley, it is likely the source of this water is discrete and unconnected bodies of water located in joints, principally collecting above the tunnel from rain water and snow melt. It is likely the ground around the proposed Floyd Hill Tunnel will be relatively dry with low flows of water expected – conditions which are suitable for the SCL and rock bolts provisions recommended.

The rationale behind choosing SCL and rock bolts to provide the primary ground support is discussed further in Section 14.3.1. This section also explains the choice of Q value used to classify the rock mass, along with the recommended thickness of the SCL and size and spacing of rock bolts, based on empirical data.

### 14.2.2. Secondary Tunnel Lining

Tunnels in rock often utilize a primary SCL and a secondary cast *in situ* concrete lining. While the primary lining is used for initial temporary ground support during construction (as discussed in Section 14.2.1), the secondary lining is used to support permanent loading conditions. The initial classification of the rock through the tunnel lining indicated a secondary lining would be the most suitable option to provide the permanent load support. The use of a primary lining to provide permanent support to the rock is an option that may be explored at the next design stage of the project; however, for the purposes of the feasibility study, a secondary lining in addition to the primary lining has been recommended, with the primary lining assumed to be non-durable and therefore form no part of the permanent works structure.

In addition to providing permanent ground support, the secondary lining has a number of other functions, as detailed below:

- Additional support for ground loads developing over time;
- Support for waterproofing system (drainage layer located between SCL and secondary lining);
- Withstand temporary hydrostatic loads due to some degree of blockage in the drainage system (drained tunnel assumed);
- Anchorage for M&E equipment and fireboards;
- Provide a more aesthetic environment for road users (smooth secondary lining rather than rough and irregular primary SCL);
- Be sufficiently durable to meet criteria for design life (achieved through design requirements);
- Provide a suitable amount of reflection/lighting (in addition to artificial lighting) to create a well-lit environment for road users.

In certain circumstances, where the geology and tunnel span are favorable, a single primary lining can be used as the permanent ground support. However, for Floyd Hill Tunnel a secondary lining has been specified in addition to a primary SCL, due to the nature of the rock and the assumption a standalone SCL would not be sufficiently reliable to carry any long-term ground loads. In addition to this, the use of a secondary lining has been deemed to have a number of additional benefits, linked to its functions described above. For example, a secondary lining will provide support for the dimpled drainage membrane recommended, thus achieving a drained tunnel, and one with a high level of watertightness. It will also provide a smooth surface to anchor fireboards from (in addition to the aesthetics), as fixing fireboards to a SCL has proved problematic on previous projects.

The standard presumption for cast *in situ* secondary tunnel linings is that wherever possible, they should be constructed in plain concrete (bar reinforcement avoided), for the most durable lining solution. Where this is not possible due to imposed loadings and the resultant bending moments, reinforcement, whether provided through steel bars or steel fibers, is adopted to increase the moment capacity of the concrete (allowing it to act better in tension). Therefore, the use of reinforcement is often required when the loads acting on the lining or the profile of the lining are not favorable. An initial assessment of the geometry of the tunnel (spans of approximately 48-74ft) and the loading conditions suggested plain concrete was not possible for the secondary lining. Following analysis of the secondary lining, the use of conventional bar reinforcement was deemed necessary. The procedure for undertaking a numerical analysis for the secondary lining, along with the subsequent recommendations for lining thickness and reinforcement, is outlined in Section 14.3.2.

### 14.2.3. Tunnel Lining Drainage

As discussed in Section 14.2.1, the borehole logs were used to gain an understanding of the groundwater profile along the proposed alignment of the tunnel. Two of the boreholes (EP-1 and WP-2) struck water at a depth of 17ft and 34ft (below ground level) respectively, while the three remaining boreholes have no log for groundwater level. It is likely any water in the ground around the tunnel is likely to come from discrete occurrences of rain and/or snow, and the subsequent infiltration of water from these events into the rock mass. As such, it is unlikely there will be a significant volume of water in the near vicinity of the tunnel in the permanent case.

Notwithstanding the above, the tunnel must be able to accommodate the presence of water against the lining. A standard concrete lining can be considered essentially permeable to a degree, and thus it is assumed water will be able to enter the tunnel through cracks in the lining, unless a method of waterproofing is used.

Preventing water ingress directly through the secondary lining is important for the following reasons:

- Prevent interaction between water and M&E equipment;
- Improve user experience;
- Improve driving safety (does not allow water to pool/freeze on road surface and result in dangerous driving conditions);
- Prevent corrosion of steel reinforcement in concrete lining (for tunnel structures utilizing a secondary lining).

Undrained tunnels aim to provide a tunnel with a very high level of watertightness through utilizing water proofing materials, such as: water-tight concrete, injections, sprayed water proof membranes and sheet water proof membranes. Undrained systems allow water to collect around the tunnel and as a result water pressures can grow significantly, which often provides an onerous load case for the tunnel structure.

Drained tunnels utilize drainage channels, typically installed between the primary and secondary lining, to intercept water passing through the outer primary lining. Once in the tunnel the groundwater will be collected in a drainage system and removed from the tunnel either passively, through flow of the water under gravity, or actively with pumping. Drained tunnels relieve the pressure exerted on them by groundwater against the tunnel by allowing it to flow outside the secondary lining.

A drained lining is recommended to avoid high pressures resulting from water pressure building up against the lining over time. This has been assumed for the basis of this feasibility study. A dimpled drainage membrane layer has been suggested to be placed between the SCL and secondary lining – an example image of a typical drainage channel product in use is shown in Figure 14.3.

Figure 14.3 - Application of a Typical Dimpled Drainage Membrane Layer

### 14.2.4. Fireproofing

Concrete experiences a reduction in mechanical strength when subject to temperature increases above a certain threshold. Explosive spalling, in which water held within pores inside the concrete vaporize and ‘blow off’ the surface layers of the concrete, can also occur when concrete is exposed to high energy fires. Steel reinforcement within the concrete will also lose mechanical strength when it experiences increases in temperature above a certain threshold. For these reasons, fireproofing will be required in order to protect the tunnel from fire loading – primarily from vehicles and/or goods fires.

Section 4.4 discusses the guidance in NFPA 502 concerning the protection of structural elements, notably Clause 7.3.4, which specifies ‘*tunnel structural elements shall be protected to achieve the following for concrete*’:

- ‘*The concrete is protected such that fire-induced spalling is prevented*’;
- ‘*The temperature of the concrete surface does not exceed 380°C (716°F)*’;
- ‘*The temperature of the steel reinforcement within the concrete [assuming a minimum cover of 25mm (1in.)] does not exceed 250°C (482°F)*’.

In addition to this, Clause 7.3.4 states ‘*structural fire protection material, where provided, shall have a minimum melting temperature of 1350°C (2462°F)*’. This will have to be a key requirement of the fire protection method chosen during the later design stages of the project.

NFPA 502 Clause 7.3.2 outlines ‘*the structure shall be capable of withstanding the temperature exposure represented by the Rijkswaterstaat (RWS) time-temperature curve or other recognized standard time-temperature curve that is acceptable to the AHJ*’. NFPA 502 Clause 7.3.3 also states ‘*during a 120-minute period of fire exposure, irreversible damage and deformation leading to progressive structural collapse shall be prevented*’. Requirements related to both Clauses 7.3.2 and 7.3.3 are outside the scope of this feasibility study and have not been considered further but must be considered at the following design stages.

Fireproofing method(s) should be employed to ensure the conditions above are satisfied in the event of a high energy fire (e.g. a hydrocarbon fire). Section 4.4 also highlights that if no system is incorporated into the design of the tunnel to either put out or control the heat energy of the fire (active fireproofing methods), then measures would have to be included in the structural design of the tunnel lining (passive fireproofing methods). For the purpose of this report, only passive methods of fire protection will be discussed further (i.e. active methods such as fire-fighting and sprinklers have been disregarded). The two primary methods of passive fire protection considered for Floyd Hill Tunnel are: fireboards (cladding), and sacrificial concrete layers. Other methods include the use of sprayed fireproofing lining (intumescent paint), and the addition of polypropylene fibers (PPF) as a concrete admixture to control explosive spalling – the latter may be used in addition to the two primary methods to increase fire protection.

Fireboards are the most common form of passive fire protection used for road tunnels in the U.S. as they carry a number of operational benefits. In the event of a fire, fireboards ensure the concrete lining is not directly exposed to the heat source, reducing the potential for damage being sustained by the lining. In certain large-scale fires, damage may be sustained by the fireboards and they may need replacing. For low energy fires this is likely to be a quick process; however, repair times may be more significant for high energy fires in which the anchor points connecting the fireboards to the intrados of the tunnel are also heavily damaged. In addition, fireboards are designed to be waterproof, both from preventing water leakage from external conditions, and also in the instance of exposure to water during tunnel cleaning. Typically, the design life of fireboards is approximately 20-25 years, and therefore may need to be replaced more than once over the duration of the tunnel life. This may require closure of one lane in the tunnel to allow for inspection, maintenance and possibly replacement, or in certain circumstances total tunnel closure – although this would occur very infrequently (in the event of a fire). A typical section showing a fireboard fixed to an *in situ* concrete lining is displayed in Figure 14.4.

Figure 14.4 - Fireboards Fixed onto an *in situ* Concrete Lining

Sacrificial concrete layers are an alternative method of passive protection, whereby an additional layer of concrete cover is provided, increasing the size of the section (as shown in Figure 14.5). This additional concrete layer is usually non-reinforced, and acts as a barrier between the fire and the main reinforced concrete section of the lining. It is assumed this additional cover of concrete does not contribute to the structural thickness of the section; therefore in the event of a fire, the sacrificial layer is assumed to be redundant, with subsequent explosive spalling of this layer not affecting the structural stability of the lining. A benefit of this method is the layer does not need to be replaced during the operational lifetime of the tunnel (unless a fire event occurs), as it is assumed to have the same design life as the main tunnel lining. However, in the event of a serious fire, the sacrificial concrete may experience explosive spalling, which may provide a serious hazard to people exiting the tunnel or firefighters tackling the fire. In addition to this, the sacrificial concrete layer, although not included as additional concrete capacity for structural design of the concrete lining, will likely experience temperatures above 250°C (482°F). Therefore, it is unlikely this layer will meet the fire requirements for concrete as specified in NFPA 502 – it would thus require special approval from the AHJ.

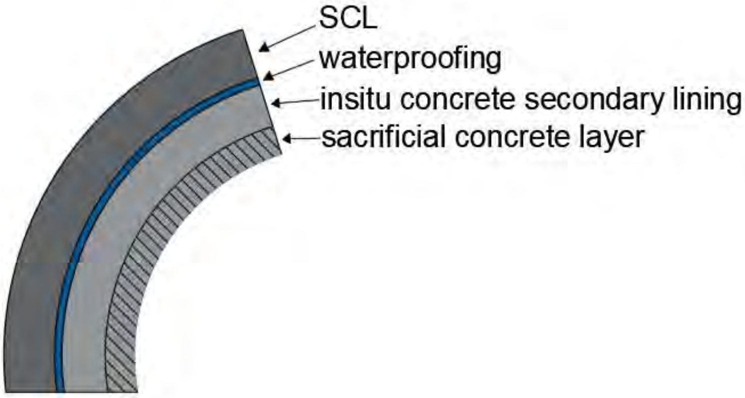


Figure 14.5 - Detail of Sacrificial Concrete Layer within Tunnel Lining

Fireboards have been chosen to provide the main fireproofing element of the tunnel lining due to a number of benefits identified over other methods. The decision on which fireproofing method to use, including ones not discussed in detail in this report, would be subject to further assessment at subsequent stages of design.

### 14.2.5. Design Life and Durability

A nominal 100-year design life has been assumed for the tunnel structure. The tunnel lining should be designed to meet the requirements of a 100-year design life. The design should aim to minimize any future maintenance works to reduce ongoing costs for operating the structure.

Durability issues to be addressed not only depend on the location of the tunnel and hence the geological environment, but also the use of the tunnel – these factors will affect the design life of the tunnel and therefore the durability requirements set. To ensure the structural integrity of the tunnel lining in the long-term, it is important the materials used (SCL, *in situ* concrete, steel reinforcement) meet requirements for durability. Factors tending to influence the durability of concrete and should be considered during the later design stages include:

- Operational environment;
- Shape and bulk of concrete;
- Cover to reinforcement;
- Type of cement;
- Type of aggregate;
- Type and dosage of admixture;
- Cement content and free water/cement ratio;
- Workmanship;
- Permeability, porosity and diffusivity of final concrete.

As mentioned previously, the waterproofing and fire-resistance of the lining must also be sufficiently durable to ensure the tunnel can function as it should over its whole operational lifetime. Not only are these aspects critical from a structural perspective, but it will likely result in a reduction in the maintenance conducted for the tunnel, which will significantly reduce the disruption to traffic along the I-70 Corridor.

## 14.3. Tunnel Lining Structural Assessment

For the purposes of the structural assessment of the tunnel lining, all calculations were carried out in metric units, with key outputs converted to the equivalent imperial units. Where lining thicknesses and reinforcement sizes have been specified, the closest (as reasonable) upper bound thicknesses/sizes have been used.

### 14.3.1. Primary Ground Support

As outlined in Section 12.4.1, the rock mass classification is one of the key inputs for determining the primary ground support required for the tunnel. Although the Q values summarized in Table 12.4 ranged from 2.24 to 3.92, a value of Q = 1 was selected, as this was considered a conservative lower bound value based on the geological data obtained to date.

The NGI Q-System Handbook [22] was used to identify the type of temporary rock support required for the tunnel. This required an excavation support ratio (ESR) value to be defined – a value of ESR = 1.0 was used as Floyd Hill Tunnel fell under the ‘major road and railway tunnels’ category. Given the span of the tunnel, the ESR value and the Q value, the type of support was specified using Figure 14.6.

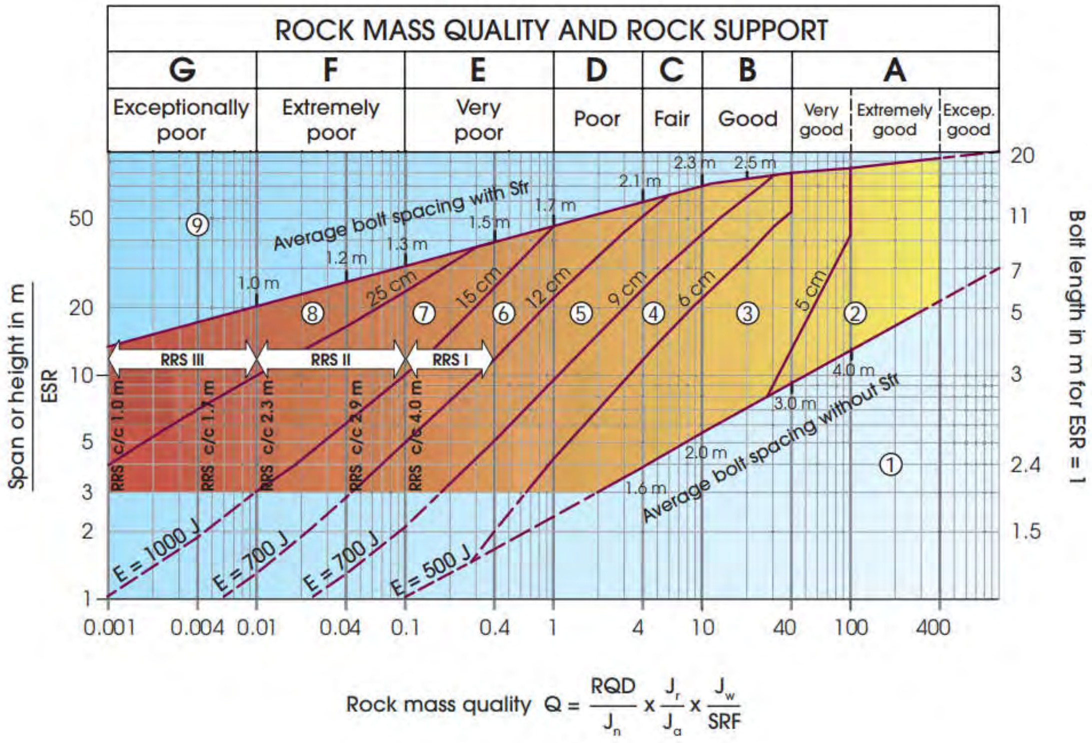


Figure 14.6 - Support Recommendations based on Q-values and Span/ESR [22]

At this stage of the project it is not appropriate to define a large number of primary support classes. As such, one primary rock support class has been defined for each of the four configuration options; as the main tunnel cross-sections (and hence spans) of Options B and C are the same, an identical rock support has been suggested for each. The different rock support requirements for the configuration options are outlined in Table 14.1. It must be noted that this is based on Q = 1.0; therefore, as the design maturity increases, different rock support classes will need to be specified depending on the variation in rock quality (i.e. Q = 10+ to Q = 0.1)

Table 14.1 - Recommended Primary Ground Support Requirements for Tunnel Layout Options A-D based on Q = 1.0

| Tunnel Configuration Option [Span] | Fibre Reinforced SCL [Minimum Thickness] | Rock Bolts for Crown/Shoulders [Length, Spacing] | Steel Ribs |
|---|---|---|---|
| Option A [73.8ft (≈ 22.5m)] | Yes [0.4ft (≈ 0.12m)] | Yes [18.0ft, 5.6ft (≈ 5,5m, ≈ 1.7m)] | No |
| Option B/C [64.0ft (≈ 19.5m)] | Yes [0.4ft (≈ 0.12m)] | Yes [16.4ft, 5.6ft (≈ 5.0m, ≈ 1.7m)] | No |
| Option D [47.6ft (≈ 14.5m)] | Yes [0.3ft (≈ 0.10m)] | Yes [13.1ft, 5.6ft (≈ 4.0m, ≈ 1.7m)] | No |

The rock bolts have been assumed to be 1in (≈ 25mm) diameter, acting in a radial direction and positioned in a pattern array. Additional bolts may be required, based on the local geometry of the joints, and should be explored during the detailed design stage. The longitudinal spacing of the rock bolts has been assumed to be a similar distance as the transverse spacing – thus a longitudinal spacing of 6.0ft (≈ 1.8m) has been recommended. The rock bolts specified in Table 14.1 were determined for the crown and shoulders of the excavated tunnel, which is typically where the most unstable parts of the excavation can be found and thus greater support is required.

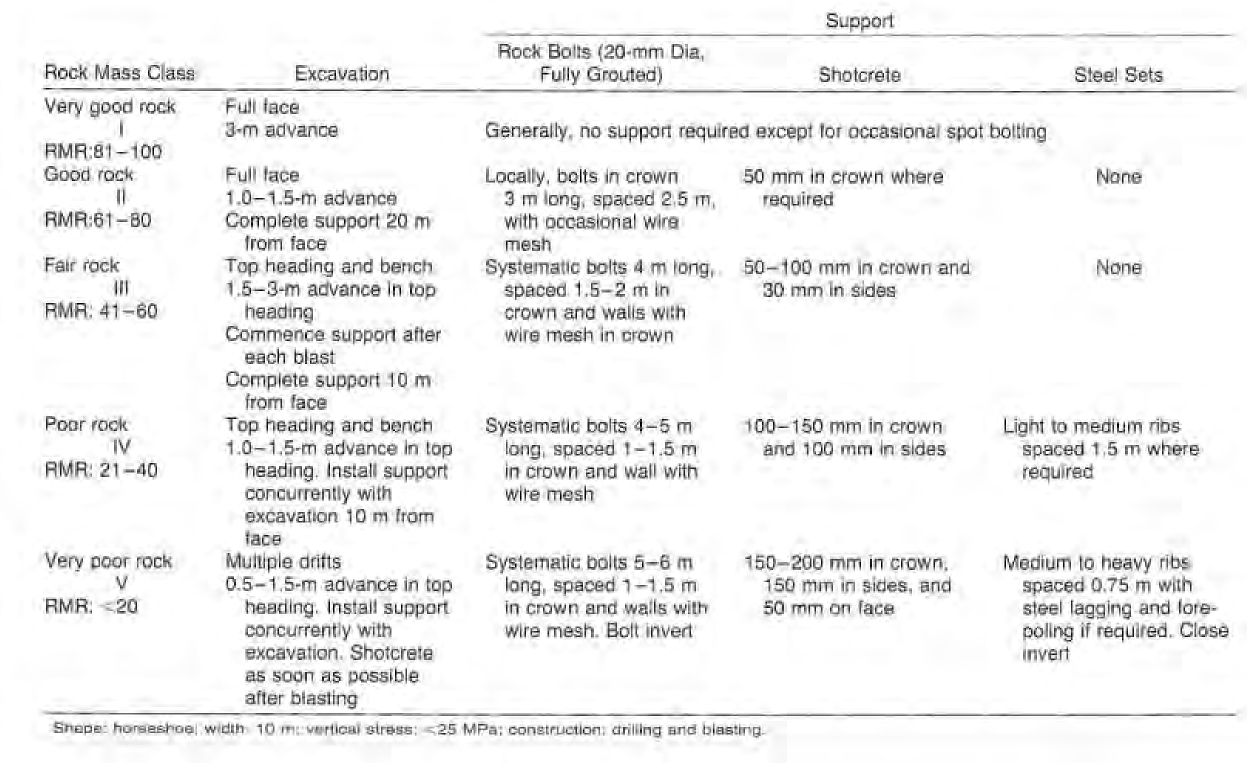
For the side walls of the excavated tunnel, the following rock bolts have been recommended:

- 9.8ft (≈ 3m) long rock bolts at 5.6ft (≈ 1.7m) center-to-center (c/c) spacing, starting at approximately 5ft (≈ 1.5m) above the tunnel invert level up to the top of the side walls.

For options with cross passages and/or an egress tunnel, the following rock bolts have been recommended:

- 3 no. 9.8ft (≈ 3m) long rock bolts positioned in the crown/shoulders (side walls of cross passages/egress tunnel assumed to not require rock bolts due to more stable excavation).

The RMR system for determining rock support was utilized to validate the rock support specified from the Q-system. Using a lower bound RMR value of 45 (rock mass class III, RMR: 41-60) and Figure 14.7, similar recommendations to those provided by the Q-system can be identified. It must be noted the guidelines are for a tunnel with a horseshoe-shape profile and of 33ft (≈ 10m span). This contrasts the vertical side walls and curved soffit profile of Floyd Hill Tunnel, and is substantially smaller in width (33ft compared to 48-74ft). Therefore, the requirements set out in Table 14.1, which specify longer rock bolts and a thicker shotcrete layer, have been deemed as suitable for this tunnel based on the geological data available, pending further data being retrieved in the following design stages.



| | | Support | | |
|---|---|---|---|---|
| Rock Mass Class | Excavation | Rock Bolts (20-mm Dia, Fully Grouted) | Shotcrete | Steel Sets |
| Very good rock<br>I<br>RMR:81–100 | Full face<br>3-m advance | Generally, no support required except for occasional spot bolting | | |
| Good rock<br>II<br>RMR:61–80 | Full face<br>1.0–1.5-m advance<br>Complete support 20 m from face | Locally, bolts in crown 3 m long, spaced 2.5 m, with occasional wire mesh | 50 mm in crown where required | None |
| Fair rock<br>III<br>RMR: 41–60 | Top heading and bench<br>1.5–3-m advance in top heading<br>Commence support after each blast<br>Complete support 10 m from face | Systematic bolts 4 m long, spaced 1.5–2 m in crown and walls with wire mesh in crown | 50–100 mm in crown and 30 mm in sides | None |
| Poor rock<br>IV<br>RMR: 21–40 | Top heading and bench<br>1.0–1.5-m advance in top heading. Install support concurrently with excavation 10 m from face | Systematic bolts 4–5 m long, spaced 1–1.5 m in crown and wall with wire mesh | 100–150 mm in crown and 100 mm in sides | Light to medium ribs spaced 1.5 m where required |
| Very poor rock<br>V<br>RMR: <20 | Multiple drifts<br>0.5–1.5-m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting | Systematic bolts 5–6 m long, spaced 1–1.5 m in crown and walls with wire mesh. Bolt invert | 150–200 mm in crown, 150 mm in sides, and 50 mm on face | Medium to heavy ribs spaced 0.75 m with steel lagging and fore-poling if required. Close invert |

Shape: horseshoe; width: 10 m; vertical stress: <25 MPa; construction: drilling and blasting.

Figure 14.7 - Guidelines for Support of Rock Tunnels based on RMR System [23]

## 14.3.2. Secondary Lining

As discussed in Section 14.2.2, an *in situ* secondary lining was deemed to be required for Floyd Hill Tunnel to provide a number of functions, such as: support for loads; surface for internal fixings; and a smooth tunnel intrados for aesthetics. An analysis of the secondary lining was undertaken to determine the size (thickness) of the lining and the amount of reinforcement required (if any).

### 14.3.2.1. Secondary Lining Analysis

To carry out a structural assessment of the secondary lining in order to determine the moments/forces induced as a result of various load combinations, a two-dimensional beam-spring numerical model was created using STAAD.Pro (structural design software). The model was created using the revised tunnel profiles discussed in Section 13.1.3. The tunnel lining was approximated in the STAAD.Pro software using nodes and straight-lined beams (which connect the nodes). The interaction between the rock and the tunnel lining was idealized using ‘compression-only’ spring supports; this allowed the different load cases to be correctly analyzed.
The following key design assumptions were made for the analysis of the secondary lining:

- The primary support was assumed to not provide any long-term ground support – therefore all loads were assumed to be taken by the secondary lining;
- A preliminary concrete mix with a 28-day compressive strength of 5000psi was used for the design of the secondary lining, which is analogous to a C35/45 concrete grade – different strengths of concrete may be explored in future design stages if deemed necessary;
- Two main structural (ULS) checks have been carried out and are assumed to provide the worst case – a crack width (SLS) check has not been carried out. However, it must be considered at the next design stage, as crack widths can sometimes be the dominant case for design.

Several loads were considered to be acting on the tunnel lining as part of the analysis – a description of each load is shown in Table 14.2.

Table 14.2 - Summary of Loads Used for the Secondary Lining Analysis

| Load | Symbol | Load Description | Option A | Option B/C | Option D |
|---|---|---|---|---|---|
| Self-weight | SW | Varied factor for tunnel roof and walls | $(F_{roof})$ = 1.42<br>$(F_{wall})$ = 1.17 | $(F_{roof})$ = 1.42<br>$(F_{wall})$ = 1.17 | $(F_{roof})$ = 1.63<br>$(F_{wall})$ = 1.25 |
| Generalized ground load | G1 | UDL acting vertically on the tunnel crown/shoulders | 34.4psi<br>(≈ 237.5kN/$m^2$) | 27.9psi<br>(≈ 192.5kN/$m^2$) | 16.7psi<br>(≈ 115.0kN/$m^2$) |
| Tensile failure of the joints | G2 | VDL acting vertically on roof, with varied max. load and 0kN (≈ 0lbf) load where wedge intercepts lining | 37588.1lbf<br>(≈ 167.2kN) | 33451.6lbf<br>(≈ 148.8kN) | 22750.7lbf<br>(≈ 101.2kN) |
| Wall load | G3 | UDL acting horizontally on one side wall over a 1-meter strip of lining | 877.1lbf/ft<br>(≈ 12.8kN/m) | 904.5lbf/ft<br>(≈ 13.2kN/m) | 1055.2lbf/ft<br>(≈ 15.4kN/m) |
| Water pressure | W | VDL acting horizontally on lining, with 0kN/$m^2$ (≈ 0psi) at invert, increasing by 5kN/$m^2$ (≈ 0.7psi) per meter height, with varied max. pressure | 7.6psi<br>(≈ 52.7kN/$m^2$) | 7.2psi<br>(≈ 49.5kN/$m^2$) | 6.9psi<br>(≈ 47.8kN/$m^2$) |
| Equipment + cladding | EC | Equipment: 4no. 38kN (≈ 8542.7lbf) point loads at varied centers, acting symmetrically about tunnel centerline | 11.8ft (≈ 3.6m) centres | 11.8ft (≈ 3.6m) centres | 8.9ft (≈ 2.7m) centres |
| | | Cladding: UDL acting perpendicular to lining | 0.4psi (≈ 3kN/$m^2$) | | |
| Air pressure | A | UDL acting perpendicular to lining | +/-1.5psi (≈ +/-10kN/$m^2$) | | |
| Shrinkage | S | Change in beam length applied as a load | 0.00039ft (≈ 0.00012m) | | |

The following loads were not deemed appropriate to be assessed as part of this analysis, but may need to be considered in the later design stages of the project:

- Long-term failure of the joints (ground load) – deemed unnecessary due to high rock load provided by G1;
- Pillar loads (ground load) – assumed rock pillars between adjacent structures will be sized to avoid failure of the pillar;
- Vehicle impact – assumed guardrails will provide sufficient protection, but will likely need to be assessed as a dynamic load;
- Early-age shrinkage/creep – deemed to be a more detailed design problem and therefore falls outside of the scope of the feasibility study, although must be considered in the next design stage.

The loads outlined in Table 14.2 were multiplied by a partial safety factor of 1.35, based on the factors recommended in the British Standards Institution (BSI) Eurocode [24], in order to provide a conservative approach in the form of a 'safety buffer' for the design. The loads were combined in order to produce the worst case loading arrangements (highest bending moments, highest stresses/forces) for the tunnel lining. An overview of the different load combinations considered for the analysis is shown in Table 14.3.

Table 14.3 - Overview of Load Combinations Used for the Secondary Lining Analysis

| **Load Combination** | **Load** | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | SW | G1 | G2 | G3 | W | EC | A (+/-) | S |
| 1 | 🟦 | 🟦 ` | | | 🟦 | 🟦 | 🟦 - | |
| 2 | 🟦 | 🟦 | | | | | 🟦 + | |
| 3 | 🟦 | 🟦 | | 🟦 | | | | |
| 4 | 🟦 | 🟦 | | 🟦 | 🟦 | 🟦 | 🟦 - | |
| 5 | 🟦 | 🟦 | | 🟦 | | | 🟦 + | |
| 6 | 🟦 | | 🟦 | | 🟦 | 🟦 | 🟦 - | |
| 7 | 🟦 | | 🟦 | | | | 🟦 + | 🟦 |
| 8 | 🟦 | | 🟦 | 🟦 | | | | |
| 9 | 🟦 | | 🟦 | 🟦 | 🟦 | 🟦 | 🟦 - | |
| 10 | 🟦 | | 🟦 | 🟦 | | | 🟦 + | 🟦 |

Once the load combinations were identified, the STAAD.Pro model was analyzed in order to find the maximum bending moments and shear forces within the secondary lining, as shown in Table 14.4 (Option A), Table 14.5 (Option B/C) and Table 14.6 (Option D). These results were for a secondary lining thickness of 2ft (≈ 0.6m) (outputs varied depending on lining thickness). Two main checks were carried out in accordance with Eurocode 2:1-1 [25], and are detailed below:

- Bending moment and axial force capacity (M-N capacity) – using the max. bending moment ($M_z$) and its corresponding axial force ($F_x$) for each load combination to ensure the lining is within its M-N capacity;
- Shear force capacity – using the max. shear force ($F_y$) of all the load combinations and its corresponding axial force ($F_x$) to ensure the shear force acting on the lining is less than the lining's shear resistance.

Table 14.4 - Summary of Maximum Moments/Forces for Option A for Each Load Combination

| **Load Combination** | **M-N Capacity Check** | | | | **Shear Capacity Check** | | | |
|---|---|---|---|---|---|---|---|---|
| | Max. $M_z$ | | $F_x$ | | Max. $F_y$ | | $F_x$ | |
| | [lbf·ft/ft] | [kNm/m] | [lbf/ft] | [kN/m] | [lbf/ft] | [kN/m] | [lbf/ft] | [kN/m] |
| 1 | 301401 | 1340.7 | 558781 | 8154.8 | 51241 | 747.8 | 563749 | 8227.3 |
| 2 | 238275 | 1059.9 | 436073 | 6364.0 | 40873 | 596.5 | 442575 | 6458.9 |
| 3 | 239377 | 1064.8 | 439512 | 6414.2 | 41038 | 598.9 | 446022 | 6509.2 |
| 4 | 301469 | 1341.0 | 558960 | 8157.4 | 51241 | 747.8 | 563921 | 8229.8 |
| 5 | 238320 | 1060.1 | 436244 | 6366.5 | 40873 | 596.5 | 442747 | 6461.4 |
| 6 | 211478 | 940.7 | 341519 | 4984.1 | 32842 | 479.3 | 338278 | 4936.8 |
| 7 | 165549 | 736.4 | 191080 | 2788.6 | 20625 | 301.0 | 204784 | 2988.6 |
| 8 | 149453 | 664.8 | 222244 | 3243.4 | 22825 | 333.1 | 220544 | 3218.6 |
| 9 | 203452 | 905.0 | 325533 | 4750.8 | 31499 | 459.7 | 322477 | 4706.2 |
| 10 | 165549 | 736.4 | 191251 | 2791.1 | 20625 | 301.0 | 204955 | 2991.1 |

Table 14.5 - Summary of Maximum Moments/Forces for Tunnel Option B/C for Each Load Combination

| **Load Combination** | **M-N Capacity Check** | | | | **Shear Capacity Check** | | | |
|---|---|---|---|---|---|---|---|---|
| | Max. $M_z$ | | $F_x$ | | Max. $F_y$ | | $F_x$ | |
| | [lbf·ft/ft] | [kNm/m] | [lbf/ft] | [kN/m] | [lbf/ft] | [kN/m] | [lbf/ft] | [kN/m] |
| 1 | 280472 | 1247.6 | 418593 | 6108.9 | 48637 | 709.8 | 421662 | 6153.7 |
| 2 | 212130 | 943.6 | 312028 | 4553.7 | 37255 | 543.7 | 316749 | 4622.6 |
| 3 | 280472 | 1247.6 | 418593 | 6108.9 | 48637 | 709.8 | 421662 | 6153.7 |
| 4 | 280517 | 1247.8 | 418737 | 6111.0 | 48637 | 709.8 | 421800 | 6155.7 |
| 5 | 212175 | 943.8 | 312172 | 4555.8 | 37255 | 543.7 | 316886 | 4624.6 |
| 6 | 207386 | 922.5 | 278959 | 4071.1 | 33343 | 486.6 | 275841 | 4025.6 |
| 7 | 154803 | 688.6 | 149268 | 2178.4 | 20995 | 306.4 | 160780 | 2346.4 |
| 8 | 140191 | 623.6 | 175669 | 2563.7 | 22242 | 324.6 | 174203 | 2542.3 |
| 9 | 198641 | 883.6 | 264768 | 3864.0 | 31849 | 464.8 | 261856 | 3821.5 |
| 10 | 154803 | 688.6 | 149412 | 2180.5 | 20995 | 306.4 | 160917 | 2348.4 |

Table 14.6 - Summary of Maximum Moments/Forces for Option D for Each Load Combination

| **Load Combination** | **M-N Capacity Check** | | | | **Shear Capacity Check** | | | |
|---|---|---|---|---|---|---|---|---|
| | Max. $M_z$ | | $F_x$ | | Max. $F_y$ | | $F_x$ | |
| | [lbf·ft/ft] | [kNm/m] | [lbf/ft] | [kN/m] | [lbf/ft] | [kN/m] | [lbf/ft] | [kN/m] |
| 1 | 57574 | 256.1 | 197452 | 2881.6 | 17528 | 255.8 | 199494 | 2911.4 |
| 2 | 38307 | 170.4 | 128581 | 1876.5 | 12807 | 186.9 | 131158 | 1914.1 |
| 3 | 38622 | 171.8 | 130630 | 1906.4 | 12300 | 179.5 | 133364 | 1946.3 |
| 4 | 57596 | 256.2 | 197555 | 2883.1 | 17514 | 255.6 | 199597 | 2912.9 |
| 5 | 38330 | 170.5 | 128656 | 1877.6 | 12800 | 186.8 | 131260 | 1915.6 |
| 6 | 48581 | 216.1 | 148082 | 2161.1 | 13238 | 193.2 | 146828 | 2142.8 |
| 7 | 44220 | 196.7 | 64657 | 943.6 | 8764 | 127.9 | 78800 | 1150.0 |
| 8 | 32912 | 146.4 | 68816 | 1004.3 | 7996 | 116.7 | 80712 | 1177.9 |
| 9 | 46670 | 207.6 | 124737 | 1820.4 | 12519 | 182.7 | 137921 | 2012.8 |
| 10 | 44220 | 196.7 | 64760 | 945.1 | 8709 | 127.1 | 78896 | 1151.4 |

Figure 14.8 (Option A), Figure 14.9 (Option B/C) and Figure 14.10 (Option D) shows that the M-N capacity of the secondary lining was not exceeded by any of the load combinations assessed for all the Options.

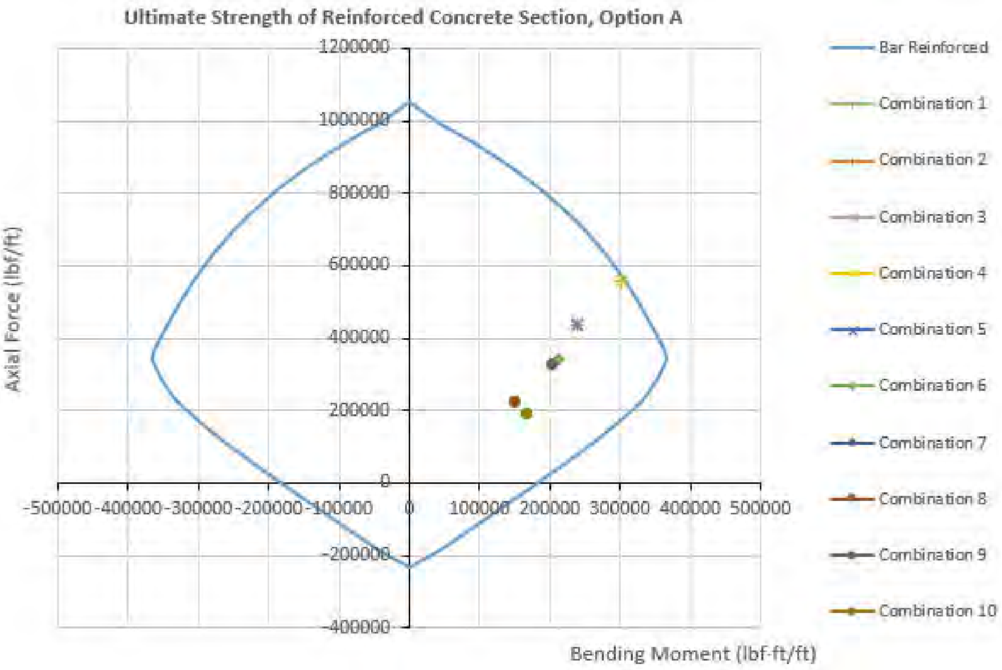


Figure 14.8 - M-N Capacity Diagram for Option A for all Load Combinations

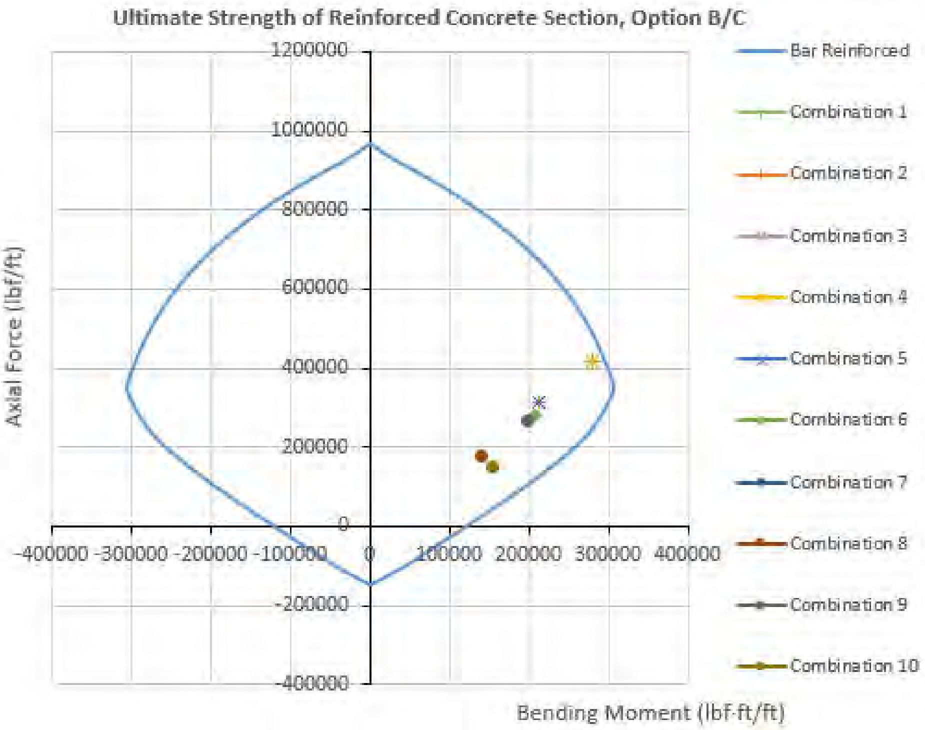


Figure 14.9 - M-N Capacity Diagram for Option B/C for all Load Combinations

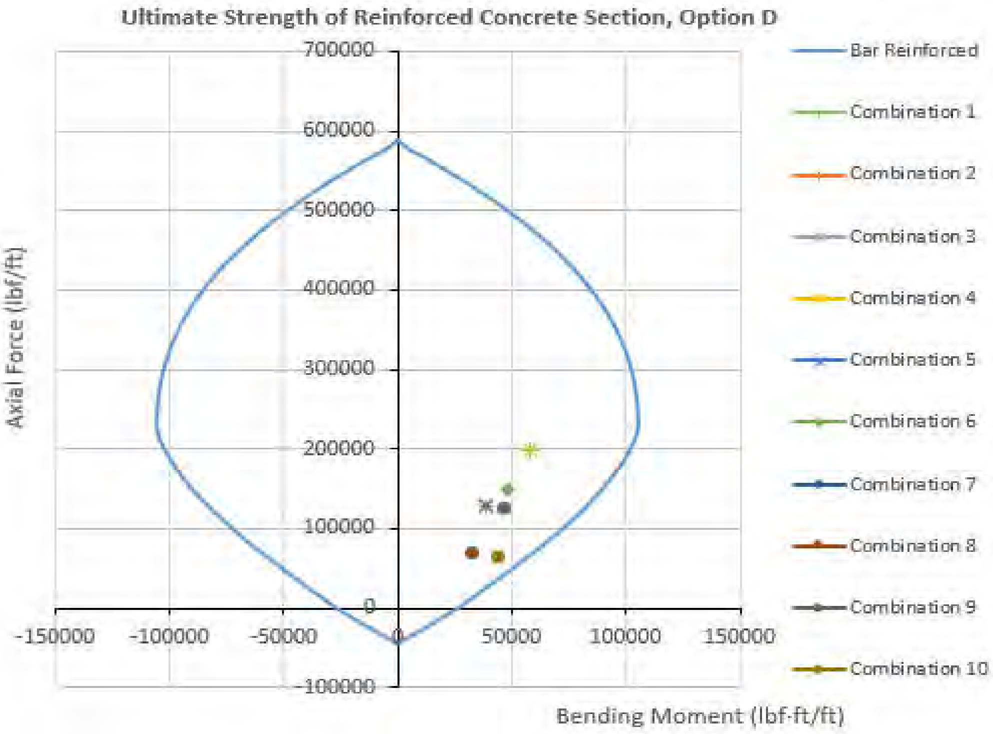


Figure 14.10 - M-N Capacity Diagram for Option D for all Load Combinations

In addition to this, the secondary lining passed the check for shear force capacity for all Tunnel Configuration Options.

### 14.3.2.2. Secondary Lining Design

The conceptual design and hence thickness of the secondary lining was initially based on a rough estimate of 1.1ft (≈ 0.35m). Due to the significant ground loads (particularly G1 which governed the analysis) calculated, the thickness of the lining was increased to 1.3ft (≈ 0.4m) for Option A and 2.0ft (≈ 0.6m) for Option B/C and D. The recommended secondary lining thickness and reinforcement required in order for the lining to pass both design checks (greatest utilization was M-N capacity) is displayed in Figure 14.11 (Option A), Figure 14.12 (Option B/C) and Figure 14.13 (Option D).

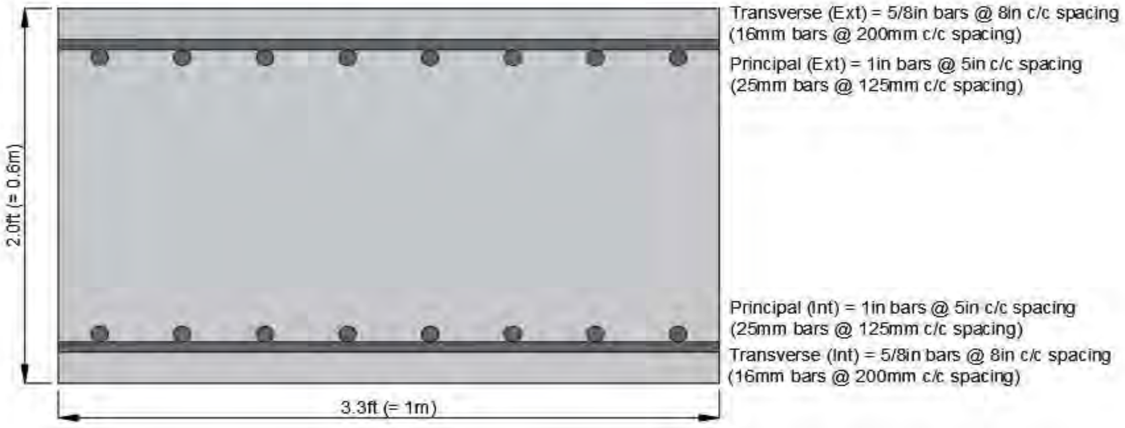


Figure 14.11 - Diagram of Recommended Secondary Lining Thickness and Reinforcement Requirements for Option A

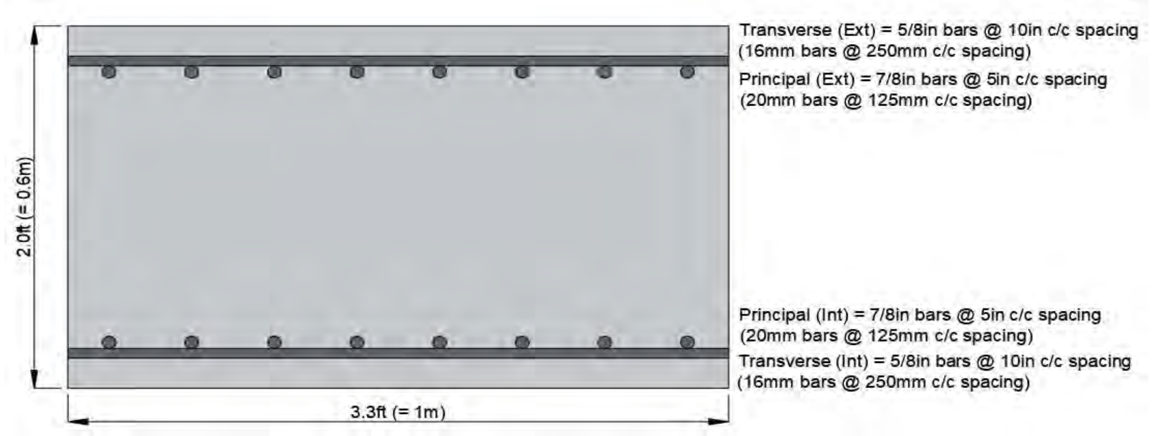


Figure 14.12 - Diagram of Recommended Secondary Lining Thickness and Reinforcement Requirements for Option B/C

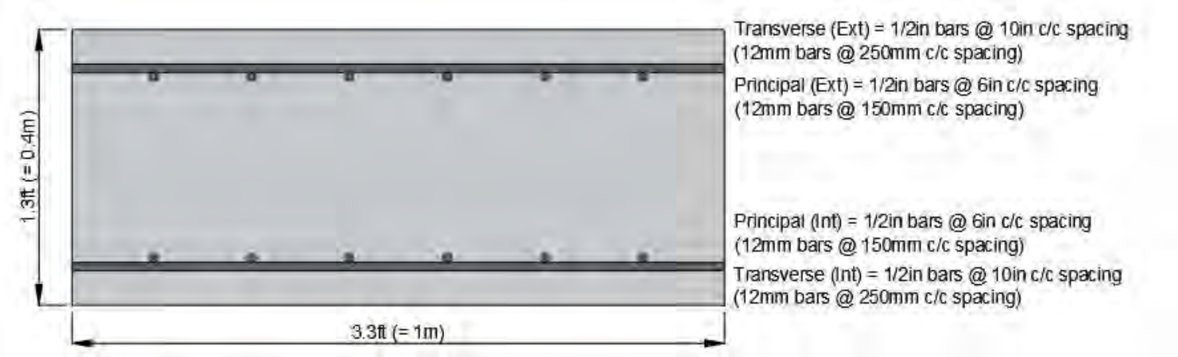


Figure 14.13 - Diagram of Recommended Secondary Lining Thickness and Reinforcement Requirements for Option D

## 14.4. Tunnel Lining Summary

This section has provided a logical process for the development of the tunnel lining, with key features identified. Different aspects of the tunnel lining for the main highway tunnel are summarized in Table 14.7 below for Tunnel Configuration Options A, B/C and D.

Table 14.7 - Summary of the Recommended Tunnel Lining Features

| **Tunnel Lining Feature** | | **Selection** | | |
|---|---|---|---|---|
| | | Option A | Option B/C | Option D |
| Design life | | Assumed to be a minimum of 100 years | | |
| Structural form | | Primary SCL with a cast *in situ* concrete secondary lining | | |
| Waterproofing | | Drained lining. Dimpled drainage membrane layer (drained system) located between primary SCL and *in situ* secondary lining | | |
| Fireproofing | | Fireboards fixed to intrados of *in situ* secondary lining | | |
| Primary ground support | Fibre reinforced SCL: min. thickness | 0.4ft (≈ 0.12m) | 0.4ft (≈ 0.12m) | 0.3ft (≈ 0.10m) |
| | Rock bolts: length; spacing | 18.0ft (≈ 5.5m);<br>5.6ft (≈ 1.7m) | 16.4ft (≈ 5.0m);<br>5.6ft (≈ 1.7m) | 13.1ft (≈ 4.0m);<br>5.6ft (≈ 1.7m) |
| *In situ* concrete secondary lining | Secondary lining min. thickness | 2.0ft (≈ 0.6m) | 2.0ft (≈ 0.6m) | 1.3ft (≈ 0.4m) |
| | Principal reinforcement bars: diameter; spacing (c/c); cover | 1in (≈ 25mm);<br>5in (≈ 125mm);<br>2in (≈ 50mm) | 7/8in (≈ 20mm);<br>5in (≈ 125mm);<br>2in (≈ 50mm) | 1/2in (≈ 12mm);<br>6in (≈ 150mm);<br>2in (≈ 50mm) |
| | Transverse reinforcement bars: diameter; spacing (c/c) | 5/8in (≈ 16mm);<br>8in (≈ 200mm) | 5/8in (≈ 16mm);<br>10in (≈ 250mm) | 1/2in (≈ 12mm);<br>10in (≈ 250mm) |

Drawings showing the different Tunnel Configuration Options, including details of the tunnel lining, can be found in Appendix E.

The key features of the tunnel lining at this stage, particularly the more detailed structural design, are based on the current level of data available. Different structural forms and features of the tunnel lining may be explored in the following design stages of the project if this is felt necessary, in order to satisfy cost, time and/or constructability issues. The reinforcement specified for each option is a function of the geometry of the lining and the loading acting on the lining. There are a number of advantages of using steel fibre reinforced concrete (SFRC) over conventional bar reinforced concrete, such as some cost and constructability benefits. Therefore, an exercise of modifying the tunnel lining geometry should be undertaken at the next design stage to determine whether the use of SFRC would be suitable based on the loads calculated (or to reduce the amount of bar reinforcement) – as typically the flatter the roof profile, the greater the bending moments experienced, and hence the more reinforcement required.

The whole design of the tunnel lining (primary ground support and secondary lining) will need to be updated in light of new information, with further detailed assessment required to ensure the tunnel lining has been suitably designed.

# 15. Portal Design Development

## 15.1. Portal Functional Requirements

In the context of Floyd Hill Tunnel, the portals have a number of functions both during the construction and operational phase, including:

- Creating a platform from which drill and blasting of the main highway tunnel can begin;
- Allowing machinery, materials and workers to enter/exit the tunnel during its construction;
- Providing permanent fall protection at the entrance/exit of the tunnel;
- Providing an entrance/exit for vehicles into/out of the main highway tunnel;
- Providing an entrance/exit for people accessing the egress tunnel/compartment;
- Creating an assembly point for passengers exiting the tunnel in an emergency event.

## 15.2. Portal Locations

The tunnel portal location represents the location where the at grade/in cutting highway transitions to a tunneled highway. This location is typically determined by the particulars of the geology and the topography at the portal locations. If a tunnel is constructed in a location where there is insufficient intact rock over the tunnel, then only limited load carrying capacity in the rock can be assumed and the rock support for the tunnel has to be significantly increased. Therefore, there is a minimum amount of rock required over the tunnel for an optimum portal location. Decreasing the amount of rock cover lower than this minimum could significantly increase the cost and risk associated with the rock support at the start of the tunnel.

The longitudinal profile of the highway and the ground level along the centerline of the alignment is known. On this basis, the location of the tunnel portals can be estimated by determining the required rock cover over the crown of the tunnel, comparing it with the longitudinal profile, and then identifying the location along the profile where the minimum rock cover is first satisfied.

The required rock cover over the tunnel for an optimum cover at the start of the excavation is primarily determined by the quality of rock over the tunnel and the excavation span. Different options, which require different excavation spans, therefore have different required cover depths and so different optimum portal locations. The determination of the required cover depths at the portals is covered in Section 15.3.2. It is based on a model of a stable rock arch at each portal. It should be noted that this model is not subject to local variations in the rock; therefore, additional estimates of the required cover should be undertaken once results of additional rock investigation have been provided.

## 15.3. Portal Design

### 15.3.1. Portal Rock Quality

The available information at the portal locations consists primarily of a geological desk study and 5 no. boreholes, all positioned at approximately the proposed portal locations. The portal boreholes are the same boreholes that have been used to interpret the ground for the rest of the tunnel alignment, for which no additional boreholes exist. The interpretation of the ground for the portals is therefore the same as the interpretation for the rest of the alignment, as outlined in Section 12.4.

Joint orientations (dip and dip direction) have been estimated for each of the boreholes using a stereonet (graph for plotting geological data). In all cases only a single joint set was identified, all with similar dip and dip direction. On this basis the jointing is assumed to be broadly the same at both portals and along the alignment. The dip and dip direction of each joint set identified in each borehole is summarized in Table 15.1.

Table 15.1 - Estimated Joint Orientations for Each Borehole

| Borehole | Dip | Dip Direction |
|---|---|---|
| YA-EP-1 | 47° | 3° |
| YA-EP-2 | 36° | 348° |
| YA-WP-1 | 52° | 16° |
| YA-WP-1 | 46° | 11° |
| YA-WP-2 | 50° | 16° |

Based on the available information there is an indication of joints sets dipping into some of the portal faces with potential implications for the stability of the rock wall. However, additional information will be required to determine the actual rock support required, in particular the fracture pattern within the rock mass. Mapping of the exposed rock faces in the area will permit estimates of parameters such as joint persistence, which could be critical for determining the required support, particularly for portal faces where joints are dipping into the face of the excavation.

A critical parameter for determining the final geometry of the rock faces at the portal will be the depth of weathered material from ground level down to the rock head. This weathered material will have to be excavated with a much flatter face than the intact rock, and therefore a significant depth of weathered material can result in a large increase in the width of the required portal excavation.

In all boreholes there is a region where the intact rock is weathered towards the top of the borehole. In these regions, there is a reduction in the rock quality; however, the material will behave as a rock and a significant change to the rock support approach will not be required – although a shallower slope angle and additional support may be required. At the West Portal, there is an additional region at the top of the borehole where the material could not be considered to behave as a rock, and as such a much shallower slope will be required. From the three boreholes located at the West Portal, this weathered region is between 3.5ft and 32ft thick, indicating that the depth of the weathered material will vary significantly around the top of the portal cut faces.

It is likely that there will be a significant soil overburden at the top of the portal cuts, which could be addressed by two ways: remove the soil (if feasible) or grade the soil back to a stable angle; or construct a retaining structure to contain the soil (e.g. a soil nail wall). Additional investigation will be required to determine the profile of change from soil to rock in order to determine a detailed profile for the portal excavation at the West Portal. This could significantly influence issues such as volume of material excavated to form the portal.

### 15.3.2. Rock Brow Thickness

A rock arch model has been used to estimate the required cover, based on a thrust line calculation method. This approach assumes that a self-supporting arch forms in the rock across the width of the tunnel excavation. The thrust line approach gives a calculation for the shape of the arch based on the weight of rock supported by the arch. The following assumptions were made in order to determine the rock brow thickness for each option:

- The thickness of the line of thrust was defined by the axial force in the line of thrust and the uniaxial compressive strength of the rock mass;
- The stress distribution within the line of thrust is triangular, with maximum compression at the extrados of the line of thrust;
- The arch calculation was undertaken for multiple different lateral reactions at the springing of the arch;
- The design arch has been taken as the arch with the lowest overall height, taking into account the shape of the arch and the width of the thrust line;
- The weight of ground supported by the arch was based on the projected angle of the dip of the joints over the width of the tunnel, giving a trapezoidal block thicker on one side of the tunnel than the other.

The thrust line calculations also include a check on the required rock support to avoid slipping along any defined joint sets. Typically, the amount of rock cover required for the portal increases non-linearly as the span of the excavation increases, as shown in Figure 15.1.

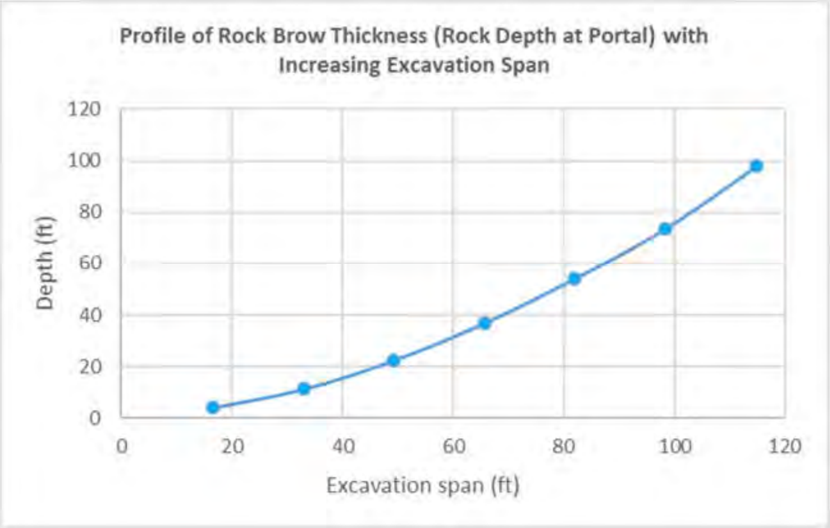


Figure 15.1 - Variation of Rock Brow Thickness with Excavation Span

Using the graph above, the rock brow thicknesses which determine the size of the portals for the different tunnel configuration options have been found and are provided in Table 15.2.

Table 15.2 - Summary of the Portal Rock Brow Thicknesses

| Configuration Option | Rock Brow Thickness |
|---|---|
| A | 47.3ft (≈ 14.4m) |
| B | 36.7ft (≈ 11.2m) |
| C | 36.7ft (≈ 11.2m) |
| D | 22.3ft (≈ 6.8m) |

Quality assurance checks on the results above using the 'Voussoir' approach (a similar method) were deemed unnecessary for this model because the thrust line maintains a reasonable curvature when deformations are calculated. Nevertheless, the deformations were checked in all cases to ensure this was a valid assumption.

### 15.3.3. Portal Size

A number of factors influence the size of the portal openings (height/width), including:

- Size of main highway tunnel excavation;
- Size of egress tunnel (for Option C);
- Required length of cross passage between main highway tunnel and egress tunnel (for Option C) or other main bore (for Option D) – i.e. minimum pillar width;
- Required rock brow thickness for portal;
- Additional buffer to account for weathered material (weak zone);
- Emergency egress provision – area required for passengers within tunnel to evacuate to.

The factors outlined above have been used as a basis for specifying the size of the portal openings. However, the requirement for emergency egress – in terms of the provision of an assembly point located at the tunnel portals – has not been considered as part this feasibility study. This requires discussions with the AHJ to take place in order to determine the emergency plans in the event that the tunnel must be evacuated. It must be noted that the size of the portal openings may have to be increased if it is deemed necessary to provide a certain amount of space next to the portal for an assembly point in case of evacuation. In addition to the space required for both the road and any emergency evacuation (of both vehicles and passengers from the tunnel portal), there are a number of other features that will need to sit within the portal cutting. This would include any tunnel services buildings, sumps, pumps and storage tanks required, as well as any additional space required for emergency or maintenance vehicles to park whilst the road remains operational. At the East Portal, sufficient space will also need to be provided to construct the new viaduct abutment; this space requirement will need to be specified at the next design stage.

A three-dimensional topographical model of the portal areas was created, which when combined with the alignment of the tunnel (from Microstation) allowed solid modeling of the portals using Google SketchUp. As a result, the portal locations were determined for each option, based on the general method outlined in Section 15.2. In addition to the rock brow thickness specified, a buffer of 20ft was used in the models to account for any weathered rock/soil (i.e. weakness zone) which may impact on the stability of the portal construction (primarily impacting the height of the portal excavation), as discussed in Section 15.3.1. At this stage in the design the quality of rock is not entirely known, and therefore this buffer has been provided to ensure conservative excavation volumes are determined – this buffer may be reduced/eliminated if/when more detailed geological information becomes available.

The portal opening sizes as well as the portal excavation volumes and subsequent main highway tunnel length for each option is provided in Table 15.3. The models used to produce the excavation volumes for each option can be found in Appendix I.

Table 15.3 - Summary of the Portal Opening Sizes, Excavation Volumes and Tunnel Lengths

| **Configuration Option** | **Main Highway Tunnel** | | **Egress Tunnel Width** | **Pillar Min. Width** | **Portal Cut Excavation Volumes** | | **Main Tunnel Length** |
|---|---|---|---|---|---|---|---|
| | Width | Height | | | East | West | |
| A | 74.4ft<br>(≈ 22.7m) | 39.9ft<br>(≈ 12.2m) | - | - | 49,000yd$^3$<br>(≈ 37,000m$^3$) | 101,000yd$^3$<br>(≈ 77,000m$^3$) | 2,260ft<br>(≈ 668.8m) |
| B | 64.1ft<br>(≈ 19.5m) | 36.5ft<br>(≈ 11.1m) | - | - | 25,000yd$^3$<br>(≈ 19,000m$^3$) | 68,000yd$^3$<br>(≈ 52,000m$^3$) | 2,340ft<br>(≈ 713.2m) |
| C | 64.1ft<br>(≈ 19.5m) | 36.5ft<br>(≈ 11.1m) | 11.1ft<br>(≈ 3.4m) | 31.2ft<br>(≈ 9.5m) | 54,000yd$^3$<br>(≈ 41,000m$^3$) | 70,000yd$^3$<br>(≈ 54,000m$^3$) | 2,285ft<br>(≈ 696.5m) |
| D | 96.8ft<br>(≈ 29.5m) | 36.2ft<br>(≈ 11.0m) | - | 36.1ft<br>(≈ 11.0m) | 88,000yd$^3$<br>(≈ 67,000m$^3$) | 117,000yd$^3$<br>(≈ 89,000m$^3$) | 2,260ft<br>(≈ 688.8m) |

### 15.3.4. Portal Structural Form

The portal will take the form of a cutting cut into the side of the ridge that the tunnel passes through. Although detailed information is not available on the local topography, it can be presumed that the cutting on the West Portal will have a substantial rock wall on the North side and a similar wall on the East side where the tunnel excavation will take place. On the South side it is expected that the excavation will take place roughly to the grade level of the I-70 highway because of the dip of the topography in the area from North to South. At the East Portal a similar arrangement might be expected with substantial rock walls to the North and West of the portal. For the purposes of modeling, the South face rock has been taken as the same as the other rock walls for the East Portal. However, it is not clear if an excavation of the South wall to road grade level (level of the new I-70 viaduct) will be necessary; the optimum solution is a function of the local topography as well as the exact portal location, which will vary depending on the option considered.

For the purposes of this feasibility report and the portal modeling (Section 15.3.3), the following assumptions have been made:

- A gradient of 1:¼ has been proposed for the headwalls at the tunnel portals. Near the top of the rock walls where the rock is weathered then shallower angles are likely. Subject to the results of detailed mapping of the rock there may be a requirement to flatten the slope of the rock walls where, for example, joints are dipping into the rock wall;
- The side walls of the portal cuts have been specified as 1:¼ (same as headwall) for both sides of the East Portal and just the North side of the West Portal, whilst the South face of the West portal has been given a 1:3 rock face to take into account the topography of the ground on this side of the rock face;

- Side ditches (slope 1:4) of 20ft width have been proposed at either side of the portal opening to provide rock protection for the I-70 highway – this also provides a minimum 20ft wide side wall pillar (larger than 20ft in most cases) which has assumed to be sufficient at this stage of the design. In the case of Option C, 25ft wide and 40ft wide side ditches have been recommended at the South side of the West and East Portals respectively, to account for the egress tunnel opening and ensure the minimum pillar width outlined above is maintained.

On completion of tunneling, in combination with the construction of the tunnel secondary lining, it is expected that a reinforced concrete portal structure will need to be constructed, which will primarily provide permanent rockfall protection at the portal. At this stage it is assumed that the significant back filling of the portal structure will not be necessary.

### 15.3.5. Portal Rock Support

Generally, it is expected that the rock support for the portal rock wall faces would be through shotcrete and bolts, similar to the primary rock support recommended for the main tunnel excavation. For the purposes of costing the bolting, it has been assumed #10 threaded bars of 25ft length will be used, 100no. for the East Portal and 200no. for the West Portal. In the more weathered material higher up the rock walls the support is likely to be shotcrete and rock bolts, although soil nails are likely to be substituted for rock bolts. The tunnel excavation into the headwall of the portal is expected to be supported by a canopy tube array (grouted tubes approx. 2.5” in diameter positioned at approx. 1.5ft c/c) approx. 36ft long. This will provide additional support to the initial part of the tunnel drive before the ground starts fully arching over the excavation and to support the potential for failure due to interaction with the portal wall and the tunnel profile. In addition to the primary rock support, additional drain holes may also need to be installed whilst constructing rock walls.

Depending on construction requirements and rock support requirements at the portals, the ends of the canopy tubes may be supported by a large steel frame at the portal to enhance the initial support to the canopy tubes. In addition, it is likely that a substantial steel framed fall protection structure will be constructed at the portals. This will provide protection to miners and machines from rocks or loose shotcrete falling from the rock face. The length of the temporary fall protection structure would be subject to detailed design but might be expected to be in the region of 15ft.

## 15.4. Portal Construction

### 15.4.1. Portal Constructability

Construction of the portals has been identified as a potentially challenging aspect of the project and must be investigated in greater detail in the next design stage. Inspection of the proposed portal locations indicates that there is potentially a significant difference in the constructability of the East and West Portals. The West Portal is roughly at grade with the adjacent I-70 highway and is constructed on a relatively shallow slope, as shown in Figure 15.2. The East Portal is constructed above the adjacent road in a relatively steep rock face (Figure 15.3) with the revised alignment of the I-70 to connect to the cutting directly from a new viaduct. It is expected that the portals will generally be constructed using drill and blast excavation although as noted in Section 15.3.1, there is the potential for a significant region of the West Portal to be in soil and so conventional bulk excavation would be expected in this area. Slopes in poorer material will also become flatter.

Figure 15.2 - Local Topography of the West Portal Region

Figure 15.3 - Local Topography of the East Portal Region

It is expected that muck from the excavation will be transported away from site by rock trucks (or similar) along the I-70 highway. Due to the difficulty accessing the East Portal, there will be limited excavation options. To avoid significant disruption to traffic flow on the I-70, the excavation rate at the portals may therefore be limited by the amount of muck that can be removed from site per day. Until the portal excavation is well under way there will likely be limited opportunity to stock pile muck on site, without impacting portal excavations. A temporary haul road could be constructed up to the portal; however, based on available information this could require significant additional excavation at the portal to allow temporary access.

Alternatively, access to construct the portal could either be through a pilot heading for the main tunnel construction or via the new viaduct for the I-70. Both of these would impose significant programme constraints on construction of the East Portal and therefore would require assessment at a more detailed stage in the development of the tunnel design. Due to the proximity of the I-70 highway to the proposed portal locations, consideration for blast protection will have to be given when assessing issues such as the construction programme.

### 15.4.2. Rockfall Protection

The nature of the portal cuts, in terms of their height and slope, mean that rockfall protection will be necessary at the tunnel portals. Whilst this section does not go into detailing the requirements for rockfall protection, consideration has been given to the issues surrounding its implementation and therefore what will need to be addressed at the next design stage.

In general, 200ft high slopes are the upper bound limit of what can be mitigated by conventionalrockfall protection measures – usually the preferred limit is 160ft. Whilst it is unlikely the portal faces will exceed this limit, it must be kept in mind when the portal geometry is finalized during the detailed design of the portals. Although many methods of rockfall protection exist, it is very likely that tunnel portal extensions (canopy structures) will be required to provide protection to traffic entering and exiting the tunnel. This will almost certainly be required for the headwalls, whilst some sort of protective structure is also likely to be required for the higher side cuts (¼:1 slope) – a similar structure used for the VMT is shown in Figure 15.4.

Figure 15.4 - Canopy Structure Used for the Veterans Memorial Tunnels Project for Rockfall Protection

# PART G – Tunnel Configuration Final Evaluation

# 16. Tunnel Configuration Options Assessment

## 16.1. Assessment Criteria and Constraints

### 16.1.1. Cost

Construction cost estimates (priced at 2019, Quarter 1) have been produced for each tunnel configuration option and are all inclusive, portal to portal. These cost estimates have been produced using ‘bottom-up’ estimating method combining labor, materials, equipment and indirect costs for each item. Wherever possible, construction sequencing, production, rates means and methods and other costs were used from similar previous projects (e.g. the Veterans Memorial Tunnel enlargement project).

The cost estimates have been developed according to the following criteria:

- A 9% profit markup fee has been included in all bid items;
- Indirects (accounts for: contractors staff, planning, setup, minor construction design, etc.) are included in the 12.5% range for all bid items;
- Design and mobilization are not included in the cost estimates – mobilization will be a line item in the entire project cost estimate;
- A 2.5% risk allowance has been included (contractor’s risk, not design risk) in all bid items;
- Some minor risk items have been included in the cost estimates (e.g. overbreak, potential steel sets).

These cost estimates are indicative only and have primarily been produced to allow comparison of costs between the four proposed tunnel configuration options. Cost estimates are accurate to approximately plus or minus 25%.

### 16.1.2. Operational Benefit

Each option has been considered for its operational benefit and has been assessed according to: normal operation; emergency operation; and maintenance. Each option meets the minimum operational requirements (outlined in Section 2); however, some options provide greater flexibility during operation and maintenance that may be of benefit to CDOT. Operational benefit will be measured qualitatively and will be assessed in relative terms between options.

For each tunnel configuration, operational benefit will be rated as either low, moderate or high (considered in relative terms in the context of the four proposed options).

### 16.1.3. Construction Duration

Options will be evaluated based on predicted construction duration. Shorter construction durations are considered to be favorable for the Floyd Hill Tunnel. Estimation of construction durations have been based on the metrics shown in Table 16.1.

Construction duration estimates are indicative and have been produced to allow for high level comparative studies between options only. Metrics have been based on a limited amount of information from similar previous projects. Future studies should review and develop these duration estimates.

Table 16.1 - Estimated Duration of Construction Activities

| **Activity** | **Duration** |
|---|---|
| **All Options** | |
| Site set up – (site preparation, establishment of site offices and welfare facilities etc.) | 2 months |
| Site set up – Connection of key services (electricity and water) to portal sites | 2 months |
| Portals – slope cutting | 6 months |
| Portals – portal structure construction | 3 months |
| Tunneling – highway tunnel excavation and primary support | 4 ft/day |
| Civils – secondary lining | 20 ft/day |
| Civils - drainage and structural invert | 20 ft/day |
| Highway construction – paving and barriers | 1 month |
| Fit-out – mechanical, electrical and plumbing systems installation | 4 months |
| Fit-out – testing and commissioning | 2 months |
| **Option A – Internal Egress Compartment** | |
| Egress compartment installation | 300 ft/month |
| **Option B – Perpendicular Egress Tunnels** | |
| Egress tunnel excavation and primary support | 8 ft/day |
| Egress tunnel secondary lining | 20 ft/day |
| **Option C – Parallel Egress Tunnels** | |
| Egress tunnel excavation and primary support | 8 ft/day |
| Egress tunnel secondary lining | 20 ft/day |
| **Option D – Twin Tunnels** | |
| Cross passage excavation and primary support | 2 months |
| Cross passage secondary lining | 1 month |

### 16.1.4. Construction Risk

Options have been assessed in terms of construction risk, i.e. the degree of inherent risk and difficulty associated with each tunnel configuration. Constructability considers issues such as: health and safety hazards; and the amount of resources (plant, materials and personnel) required to construct something.

This factor will be measured qualitatively and will be assessed in relative terms between options. It will consider the following items:

- Size of excavation for the main highway tunnel;
- Portal sizes required;
- Additional tunneling and civil works (works required in addition to the construction of the main highway tunnel, i.e. mining of egress tunnels and construction of internal egress compartments);
- Inherent health and safety hazards (over and above excavation and construction of the main highway tunnel and the tunnel portals).

For each tunnel configuration, construction risk will be rated as either low, moderate or high (considered in relative terms in the context of the four proposed options).

## 16.2. Options Review

### 16.2.1. Option A

#### 16.2.1.1. Cost

The estimated capital cost for Option A is $177,279,000.

A cost breakdown for Option A is included in Appendix J.

### 16.2.1.2. Operational Benefit

#### 16.2.1.2.1. Normal Operation

During normal operation, the emergency escape doors into the escape cell (compartmentalized egress passageway) will be closed and the escape cell will be ventilated to avoid the risk of traffic fumes getting into the escape cell.

#### 16.2.1.2.2. Emergency Operation

During emergency operation, the escape cell will be available for evacuation through the escape doors and will provide a safe route to both portals. The cell will be equipped with lighting, CCTV and signage to direct the users to the nearest portal. The first responders will be able to access the tunnel on foot by walking through the escape cell from either portal and through any of the escape doors.

#### 16.2.1.2.3. Maintenance

The escape cells provide a convenient location to install much of the M&E equipment. By installing this in a protected area away from the traffic cell, it allows maintenance staff safe access for inspections and repairs with less need to shut the tunnel. Cabling for services can be easily passed through the internal diving wall. This allows the vast majority of M&E equipment to be housed within the egress compartment, with discrete 'modules' of cabling and equipment then passed into the tunnel as required. This provides a very robust tunnel services system which can be repaired easily with minimal disruption to the operation of the highway, and one that for most part is intrinsically protected in the event of a fire within the main bore.

#### 16.2.1.2.4. Summary

The provision of a compartmentalized passage through which M&E equipment can be provided throughout the entire length of the tunnel provides a benefit for the operations of the tunnel. It allows the majority of M&E equipment to be housed separately from the main highway bore, and as such any maintenance, repair or replacement works can be undertaken with no disruption to the ongoing operation of the highway. With Option A, services can be easily routed through the internal dividing wall to allow cabling to run from the egress passage to the main highway bore.

In emergency scenarios, the compartmentalized passage allows passengers to egress to both portals. It also provides emergency services with access to the egress passage from the East and West Portals.

In summary, Option A is considered to provide a high level of operational benefit when compared to the other shortlisted options. This is primarily based on the ease through which cabling for services can be passed through the internal dividing wall.

### 16.2.1.3. Construction Duration

The estimated construction duration for Option A is 46 months (3 years, 10 months).

A high-level program for the construction of Option A is included in Appendix K.

### 16.2.1.4. Construction Risk

#### 16.2.1.4.1. Size of Excavation for the Main Highway Tunnel

The size of the excavation for the main highway tunnel for Option A is provided in Table 16.2.

Table 16.2 - Size of Main Highway Tunnel Excavation for Each Option

| **Tunnel Configuration Option** | **Main Highway Tunnel** | | |
|---|---|---|---|
| | **Height** | **Width** | **Cross-Sectional Area** |
| Option A | 39.9ft (≈ 12.2m) | 74.4ft (≈ 22.7m) | 1681ft$^2$ (≈ 156m$^2$) |
| Option B | 36.5ft (≈ 11.1m) | 64.1ft (≈ 19.5m) | 1473ft$^2$ (≈ 137m$^2$) |
| Option C | 36.5ft (≈ 11.1m) | 64.1ft (≈ 19.5m) | 1473ft$^2$ (≈ 137m$^2$) |
| Option D | 36.2ft (≈ 11.0m) | 48.4ft (≈ 14.8m) | 2204ft$^2$ (≈ 205m$^2$) [1102ft$^2$ x 2] |

### 16.2.1.4.2. Portal Sizes Required

The size of the portal cut excavation volumes (East/West Portals and total) for Option A is provided in Table 16.3.

Table 16.3 - Portal Cut Excavation Volumes for Each Option

| **Tunnel Configuration Option** | **Portal Cut Excavation Volumes** | | |
|---|---|---|---|
| | **East** | **West** | **Total** |
| Option A | 49,000yd$^3$ (≈ 37,000m$^3$) | 101,000yd$^3$ (≈ 77,000m$^3$) | 150,000yd$^3$ (≈ 110,000m$^3$) |
| Option B | 25,000yd$^3$ (≈ 19,000m$^3$) | 68,000yd$^3$ (≈ 52,000m$^3$) | 93,000yd$^3$ (≈ 71,000m$^3$) |
| Option C | 54,000yd$^3$ (≈ 41,000m$^3$) | 70,000yd$^3$ (≈ 54,000m$^3$) | 124,000yd$^3$ (≈ 95,000m$^3$) |
| Option D | 88,000yd$^3$ (≈ 67,000m$^3$) | 117,000yd$^3$ (≈ 89,000m$^3$) | 205,000yd$^3$ (≈ 156,000m$^3$) |

### 16.2.1.4.3. Additional Tunneling and Civil Works

For the Option A tunnel configuration, an internal dividing wall will be used to create an insulated compartment through which passengers can evacuate the tunnel in emergency situations. This internal dividing wall could be formed using either precast concrete/steel sections or with *in situ* concrete, and is likely to be carried out in series with construction of the secondary lining and tunnel fit-out activities.

In the case of precast elements being used, mobile plant, cranes and other lifting devices will be required to bring these pieces into the tunnel and to install them in their permanent positions. Further work may then be required to seal joints between the precast elements and between the tunnel soffit and tops of the precast elements.

With *in situ* concrete, the wall will be created in sections using some form of travelling shutter. Given the height of the internal dividing wall and the loads it is likely to be subject to (self-weight, air pressure for the tunnel piston effect and potentially impact loads from vehicles), this wall will most likely need bar reinforcement.

From a constructability perspective, precast elements are preferred over *in situ* concrete. Precast elements can be quickly lifted into place, do not require on site concrete batching plants (and/or large numbers of concrete deliveries) and do not suffer from the same time constraints inherent when working with wet concrete. The quality of concrete produced in a manufacturing factory is likely to be higher than that achieved with an *in situ* pour and with precast elements the Contractor will be able to reject any elements that do not meet minimum quality standards (in contrast to *in situ* pours where any quality deficiencies must be repaired *in situ*).

It is important to note that the connection to the internal dividing wall must not carry any of the roof arch load (due to breaking down of the arching effect); therefore, careful detailing would be required, both to ensure a horizontal shear connection is provided and also that the internal egress compartment is suitably fire-protected.

### 16.2.1.4.4. Inherent Health and Safety Hazards

Option A introduces the construction of an internal compartmentalized egress passageway within the main highway tunnel. The main dividing wall for this compartment will be constructed using either precast concrete/steel units or with *in situ* concrete.

Use of precast concrete introduces additional health and safety hazards associated with lifting and erection of the precast sections and transportation of the precast units to the sight (significant amount of additional lorry movements on the local road network required to supply the project).

Use of *in situ* concrete introduces additional health and safety hazards associated with erection of rebar cages, erection of formwork and working at height.

### 16.2.1.4.5. Summary

The construction risk for Option A has been assessed as moderate.

Option A requires a larger main highway bore than any of the other shortlisted options, resulting in reasonably large portal cut excavation volumes for each portal. The internal dividing wall also requires a significant amount of post tunneling work, with a significant amount of lifting work and/or a significant amount of *in situ* concrete pours and rebar erection activities.

Option A involves no mining works other than for the construction of the main highway bore. This removes some of the hazards inherent to working in confined spaces and reduces the overall number of tunneling activities, an inherently hazardous activity, required to build the tunnel.

## 16.2.2. Option B

### 16.2.2.1. Cost

The estimated capital cost for Option B is $164,552,000.

The cost shown for Option B includes an approximate cost estimate of $15,000,000 for construction of the egress tunnel portals. The cost estimate for this item is not included in the cost breakdown for Option B, due to the uncertainty around the rock face at the possible portal locations. A cost breakdown for Option B can be found in Appendix J.

### 16.2.2.2. Operational Benefit

#### 16.2.2.2.1. Normal Operation

During normal operation, the emergency escape doors into the escape route will be closed and the escape route will not be in use.

#### 16.2.2.2.2. Emergency Operation

During emergency operation, the escape route will be available for evacuation through the escape doors and will provide a safe route to the eastbound carriageway. The route will be equipped with lighting, CCTV and signage to encourage users to exit. The first responders will be able to access the tunnel on foot by walking through the one of the escape routes and through the escape door.

#### 16.2.2.2.3. Maintenance

The escape route will not provide any added benefits for maintenance, as most of the M&E equipment will have to be located in the soffit of the main highway tunnel (i.e. not separate from the traffic cell).

#### 16.2.2.2.4. Summary

Option B is not considered to provide any significant operational benefits when compared to the other shortlisted options and has thus been assessed as low.

### 16.2.2.3. Construction Duration

The estimated construction duration for Option B is 46 months (3 years, 10 months).

A high-level program for the construction of Option B is included in Appendix K.

### 16.2.2.4. Construction Risk

#### 16.2.2.4.1. Size of Excavation for the Main Highway Tunnel

The size of the excavation for the main highway tunnel for Option B is provided in Table 16.2.

Table 16.2 - Size of Main Highway Tunnel Excavation for Each Option

| Tunnel Configuration Option | Main Highway Tunnel | | |
|---|---|---|---|
| | Height | Width | Cross-Sectional Area |
| Option A | 39.9ft (≈ 12.2m) | 74.4ft (≈ 22.7m) | 1681ft$^2$ (≈ 156m$^2$) |
| Option B | 36.5ft (≈ 11.1m) | 64.1ft (≈ 19.5m) | 1473ft$^2$ (≈ 137m$^2$) |
| Option C | 36.5ft (≈ 11.1m) | 64.1ft (≈ 19.5m) | 1473ft$^2$ (≈ 137m$^2$) |
| Option D | 36.2ft (≈ 11.0m) | 48.4ft (≈ 14.8m) | 2204ft$^2$ (≈ 205m$^2$) [1102ft$^2$ x 2] |

### 16.2.2.4.2. Portal Sizes Required

The size of the portal cut excavation volumes (East/West Portals and total) for Option B is provided in Table 16.3. The total volume does not take into account the excavated volume of the egress tunnel portals, which has not been calculated at this stage of the design.

Table 16.3 - Portal Cut Excavation Volumes for Each Option

| Tunnel Configuration Option | Portal Cut Excavation Volumes | | |
|---|---|---|---|
| | East | West | Total |
| Option A | 49,000yd$^3$ (≈ 37,000m$^3$) | 101,000yd$^3$ (≈ 77,000m$^3$) | 150,000yd$^3$ (≈ 110,000m$^3$) |
| Option B | 25,000yd$^3$ (≈ 19,000m$^3$) | 68,000yd$^3$ (≈ 52,000m$^3$) | 93,000yd$^3$ (≈ 71,000m$^3$) |
| Option C | 54,000yd$^3$ (≈ 41,000m$^3$) | 70,000yd$^3$ (≈ 54,000m$^3$) | 124,000yd$^3$ (≈ 95,000m$^3$) |
| Option D | 88,000yd$^3$ (≈ 67,000m$^3$) | 117,000yd$^3$ (≈ 89,000m$^3$) | 205,000yd$^3$ (≈ 156,000m$^3$) |

### 16.2.2.4.3. Additional Tunneling and Civil Works

For the Option B tunnel configuration, perpendicular egress tunnels will be used to create egress passages that will terminate at positions on the valley side (see Figure 6.1, Section 6), through which passengers can evacuate the tunnel in emergency situations.

These egress passages will be excavated using the drill and blast mining technique. A combination of sprayed concrete and rock bolts is envisaged for the excavation primary support. The secondary lining could be formed using either sprayed concrete or *in situ* concrete formed with a travelling shutter. With egress tunnels terminating in the valley side, portals will need to be constructed to form openings at the ends of the tunnels and to support an area large enough for evacuating passengers to marshal.

This option is generally unfavorable from a constructability perspective due to requirement for additional tunnel portals at the end of the egress tunnels. These portals would require a significant amount of construction work and this work would have to be carried out adjacent to the current I-70 Westbound highway, which would still be operational at that point. Working adjacent to the road will introduce significant restrictions in terms of available working area; in addition, supply of plant, materials and labor to the portal sites could cause significant disruptions to the operation of the existing I-70 Westbound.

### 16.2.2.4.4. Inherent Health and Safety Hazards

Option B introduces the construction of two mined egress tunnels. These tunnels will be relatively small in size and as such introduce hazards associated with working in confined spaces (deoxygenation, buildup of noxious gases, frustrated access and egress, increase in manual handling due to lack of room to utilize some plant and equipment, etc.).

Option B also requires the construction of a portals on the valley side adjacent to the current Westbound I-70 highway. This introduces hazards both to operatives, who will have to work next to a busy four lane highway, and to the public, who might be put at risk from rockfalls and a number of other hazards associated with excavation and construction of the portals.

### 16.2.2.4.5. Summary

The construction risk for Option B has been assessed as high.

This has been based on two factors: additional tunneling works required for construction of the dedicated egress tunnels and requirement for additional tunnel portals at the ends of the egress passages. Despite Option B having the lowest amount of excavated material (both main highway tunnel and East and West Portals) associated with it, the main issue lies with the egress tunnel portals - construction of which, directly adjacent to the live I-70 highway, presents a significant amount of logistical and health and safety challenges.

## 16.2.3. Option C

### 16.2.3.1. Cost

The estimated capital cost for Option C is $160,754,000.

A cost breakdown for Option C is included in Appendix J.

### 16.2.3.2. Operational Benefit

#### 16.2.3.2.1. Normal Operation

During normal operation, the emergency escape doors into the escape passageway will be closed and the escape passageway will be ventilated to avoid the risk of traffic fumes getting into the escape passageway.

#### 16.2.3.2.2. Emergency Operation

During emergency operation, the escape passageway will be available for evacuation through the escape doors and will provide a safe route to both portals. The cell will be equipped with lighting, CCTV and signage to direct the users to the nearest portal. The first responders will be able to access the incident bore on foot by walking through the escape passageway from either portal and through any of the escape doors.

#### 16.2.3.2.3. Maintenance

The escape passageway provides a convenient location to install much of the M&E equipment. By installing this in a protected area away from the traffic cell, it allows maintenance staff safe access for inspections and repairs with less need to shut the tunnel.

#### 16.2.3.2.4. Summary

The provision of a mined egress passage through which M&E equipment can be provided throughout the entire length of the tunnel provides a benefit for the operations of the tunnel. It allows the majority of M&E equipment to be housed separately from the main highway bore, and as such any maintenance, repair or replacement works can be undertaken with no disruption to the ongoing operation of the highway.

In Option C, cabling for the M&E systems can only pass from the egress passage to the highway tunnel at the two cross passages. This is less advantageous than with Option A, for which cabling can be passed through the internal dividing wall with ease.

In emergency scenarios, the compartmentalized passage allows passengers to egress to both portals. It also provides emergency services with access to the egress passage from the East and West Portals.

In summary, Option C is considered to provide moderate operational benefits, when compared to the other shortlisted options.

### 16.2.3.3. Construction Duration

The estimated construction duration for Option C is 44 months (3 years, 8 months).

A high-level program for the construction of Option C is included in Appendix K.

### 16.2.3.4. Construction Risk

#### 16.2.3.4.1. Size of Excavation for the Main Highway Tunnel

The size of the excavation for the main highway tunnel for Option C is provided in Table 16.2.

Table 16.2 - Size of Main Highway Tunnel Excavation for Each Option

| Tunnel Configuration Option | Main Highway Tunnel | | |
|---|---|---|---|
| | Height | Width | Cross-Sectional Area |
| Option A | 39.9ft (≈ 12.2m) | 74.4ft (≈ 22.7m) | 1681ft$^2$ (≈ 156m$^2$) |
| Option B | 36.5ft (≈ 11.1m) | 64.1ft (≈ 19.5m) | 1473ft$^2$ (≈ 137m$^2$) |
| Option C | 36.5ft (≈ 11.1m) | 64.1ft (≈ 19.5m) | 1473ft$^2$ (≈ 137m$^2$) |
| Option D | 36.2ft (≈ 11.0m) | 48.4ft (≈ 14.8m) | 2204ft$^2$ (≈ 205m$^2$) [1102ft$^2$ x 2] |

### 16.2.3.4.2. Portal Sizes Required

The size of the portal cut excavation volumes (East/West Portals and total) for Option C is provided in Table 16.3.

Table 16.3 - Portal Cut Excavation Volumes for Each Option

| Tunnel Configuration Option | Portal Cut Excavation Volumes | | |
|---|---|---|---|
| | East | West | Total |
| Option A | 49,000yd$^3$ (≈ 37,000m$^3$) | 101,000yd$^3$ (≈ 77,000m$^3$) | 150,000yd$^3$ (≈ 110,000m$^3$) |
| Option B | 25,000yd$^3$ (≈ 19,000m$^3$) | 68,000yd$^3$ (≈ 52,000m$^3$) | 93,000yd$^3$ (≈ 71,000m$^3$) |
| Option C | 54,000yd$^3$ (≈ 41,000m$^3$) | 70,000yd$^3$ (≈ 54,000m$^3$) | 124,000yd$^3$ (≈ 95,000m$^3$) |
| Option D | 88,000yd$^3$ (≈ 67,000m$^3$) | 117,000yd$^3$ (≈ 89,000m$^3$) | 205,000yd$^3$ (≈ 156,000m$^3$) |

### 16.2.3.4.3. Additional Tunneling and Civil Works

For the Option C tunnel configuration, a parallel tunnel will be used to create an egress passage that will terminate at the highway portal positions. This tunnel will be used by passengers to evacuate in emergency situations (see Figure 6.1, Section 6), through which passengers can evacuate the highway tunnel in emergency situations.

These egress passage will be excavated using the drill and blast mining technique. A combination of sprayed concrete and rock bolts is envisaged for the excavation primary support. The secondary lining could be formed using either sprayed concrete or *in situ* concrete formed with a travelling shutter.

Although additional tunneling work is required to construct the egress tunnel, it is envisaged that construction of the egress tunnel will take place before the main highway tunnel is excavated. This will allow the egress tunnel to act as a ‘pilot’ tunnel for the main highway tunnel, allowing feedback on the quality of the rock and informing (to a certain degree) the level of primary rock support required for the main tunnel excavation. Although the rock excavated for the egress tunnel may differ from that of the main highway tunnel (i.e. no guarantee that any weakness zone will be identified), Option C is the only option that carries this additional benefit.

### 16.2.3.4.4. Inherent Health and Safety Hazards

Option C introduces the construction of a mined egress tunnel. This tunnel will be relatively small in size and as such introduces hazards associated with working in confined spaces (deoxygenation, buildup of noxious gases, frustrated access and egress, increase in manual handling due to lack of room to utilize some plant and equipment etc.).

### 16.2.3.4.5. Summary

The construction risk for Option C has been assessed as moderate.

While the construction of a dedicated mined egress tunnel across the entire length of the main highway tunnel does introduce some additional risks (primarily associated with working in confined spaces), it also provides an opportunity to verify the condition of the rock local to the proposed route of the main highway bore, and as such could help to de-risk the construction of the main tunnel. In addition to this, the excavation volumes (for both the main highway tunnel and portals) are relatively low, particularly when the excavated volume required for the egress tunnel portals for Option B is considered.

## 16.2.4. Option D

### 16.2.4.1. Cost

The estimated capital cost for Option D is $193,134,000.

A cost breakdown for Option D is included in Appendix J.

### 16.2.4.2. Operational Benefit

#### 16.2.4.2.1. Normal Operation

During normal operation, traffic will be using both bores in the same direction, the emergency escape doors into the other bore will be closed and the cross passages will be ventilated to avoid the risk of traffic fumes getting into the cross passages.

#### 16.2.4.2.2. Emergency Operation

During emergency operation, the cross passages will be available for evacuation through the escape doors and will provide a safe route to the other bore. Procedures will need to be in place to close both bores during any evacuation to avoid escapees running into a live bore. The cross passages will be equipped with lighting, CCTV and signage to direct the users to take care exiting into the other bore. The first responders will be able to access the incident bore by driving into the non-incident bore and walking through any of the cross passages. Should a situation occur where the Eastbound carriageway is unavailable for a significant time, the tunnel bores could be configured to take Eastbound traffic in one bore and Westbound in the other.

#### 16.2.4.2.3. Maintenance

Having two separate bores means that maintenance can be carried out in one bore whist the other is still open to traffic. Clearly the traffic capacity would be restricted during this time, so maintenance is likely to take place overnight when demand is lowest.

#### 16.2.4.2.4. Summary

Option D provides a high level of operational benefit when compared to the other shortlisted options, primarily because it enables evacuation of people into the other bore if there is a significant emergency incident in one of the highway bores. It also provides an additional lane of traffic over the required minimum three lanes, which can be utilized to ensure continued flow of Westbound traffic if one of the highway bores is closed for maintenance.

### 16.2.4.3. Construction Duration

The estimated construction duration for Option D is 43 months (3 years, 7 months).

A high-level program for the construction of Option D is included in Appendix K.

### 16.2.4.4. Construction Risk

#### 16.2.4.4.1. Size of Excavation for the Main Highway Tunnel

The size of the excavation for the main highway tunnel for Option D is provided in Table 16.2.

Table 16.2 - Size of Main Highway Tunnel Excavation for Each Option

| Tunnel Configuration Option | Main Highway Tunnel | | |
|---|---|---|---|
| | Height | Width | Cross-Sectional Area |
| Option A | 39.9ft (≈ 12.2m) | 74.4ft (≈ 22.7m) | 1681ft$^2$ (≈ 156m$^2$) |
| Option B | 36.5ft (≈ 11.1m) | 64.1ft (≈ 19.5m) | 1473ft$^2$ (≈ 137m$^2$) |
| Option C | 36.5ft (≈ 11.1m) | 64.1ft (≈ 19.5m) | 1473ft$^2$ (≈ 137m$^2$) |
| Option D | 36.2ft (≈ 11.0m) | 48.4ft (≈ 14.8m) | 2204ft$^2$ (≈ 205m$^2$) [1102ft$^2$ x 2] |

### 16.2.4.4.2. Portal Sizes Required

The size of the portal cut excavation volumes (East/West Portals and total) for Option D is provided in Table 16.3.

Table 16.3 - Portal Cut Excavation Volumes for Each Option

| **Tunnel Configuration Option** | **Portal Cut Excavation Volumes** | | |
|---|---|---|---|
| | **East** | **West** | **Total** |
| Option A | 49,000yd$^3$ (≈ 37,000m$^3$) | 101,000yd$^3$ (≈ 77,000m$^3$) | 150,000yd$^3$ (≈ 110,000m$^3$) |
| Option B | 25,000yd$^3$ (≈ 19,000m$^3$) | 68,000yd$^3$ (≈ 52,000m$^3$) | 93,000yd$^3$ (≈ 71,000m$^3$) |
| Option C | 54,000yd$^3$ (≈ 41,000m$^3$) | 70,000yd$^3$ (≈ 54,000m$^3$) | 124,000yd$^3$ (≈ 95,000m$^3$) |
| Option D | 88,000yd$^3$ (≈ 67,000m$^3$) | 117,000yd$^3$ (≈ 89,000m$^3$) | 205,000yd$^3$ (≈ 156,000m$^3$) |

### 16.2.4.4.3. Additional Tunneling and Civil Works

Option D utilizes two main highway bores, both of which carry two lanes of traffic. The work required to excavate a second highway bore is significantly more onerous from a constructability point of view, as it will require a much higher tunneling effort than with a single main bore, will require a lot more material to form the tunnel and fit it out, and will generate a higher volume of spoil.

Use of two bores also means that the Westbound I-70 highway will need to split before it reaches the Floyd Hill Tunnel portals. Splitting the road could introduce a significant amount of construction works outside of the tunnel extents and this could cause disruption to the delivery of the wider I-70 upgrade program.

### 16.2.4.4.4. Inherent Health and Safety Hazards

Option D involves a greater amount of tunneling that the other options, and as such increases the likelihood of occurrence of the standard hazards associated with the construction of the main highway tunnel.

### 16.2.4.4.5. Summary

The construction risk for Option D has been assessed as high.

This has been based on two factors: the requirement to construct two main highway bores, which significantly increases the amount of tunneling works; and the requirement to split the highway into two sets of lanes on the approach to the East Portal. Due to the size of the portals (particularly the width of the East Portal), a larger viaduct structure would need to be constructed to connect the tunnel entrance to the existing I-70 highway, which may create further challenges from a construction risk perspective.

## 16.3. Options Assessment Summary

Following the evaluation of Tunnel Configuration Options A, B, C and D based on the assessment criteria, a summary of the options assessment can be found below (Table 16.4).

Table 16.4 - Summary of Options Assessment

| Tunnel Configuration Option | Operational Benefit | Construction Risk | Cost (Nearest $1,000) | Construction Duration |
|---|---|---|---|---|
| Option A | High | Moderate | $177,279,000 | 46 months<br>(3 years, 10 months) |
| Option B | Low | High | $164,552,000* | 46 months<br>(3 years, 10 months) |
| Option C | Moderate | Moderate | $160,754,000 | 44 months<br>(3 years, 8 months) |
| Option D | High | High | $193,134,000 | 43 months<br>(3 years, 7 months) |

* *The cost shown for Option B includes an approximate cost estimate of $15,000,000 for construction of the egress tunnel portals (not included in breakdown of cost estimate in Appendix J)*

## 16.4. Conclusions

### 16.4.1. Options Discussion

Four tunnel configuration options were evaluated according to: operational benefit; construction risk; estimated capital cost; and estimated duration of construction. This allowed direct comparison of the options, by using the assessment criteria as a basis for the evaluation.

The estimated costs and construction durations are fairly similar across all the options, as shown in Table 16.4. At this stage of the design process, where uncertainty around capital costs and construction programmes is high, the values stated were not used as primary criterion for differentiating between the options. Although initial estimates suggest Option D will be slightly more expensive than the other options but will constructed quicker, the potential for these values to fluctuate as the design maturity increases may result in Option D being ranked differently compared to the other options than it is now. As a result, operational benefit and construction risk were used as the main criteria for the final options selection.

### 16.4.1.1. Option A

Option A provides a high level of operational benefit due to the ease in routing the cabling through the internal dividing wall (where necessary) into the compartmentalized egress passageway. This will reduce the number/severity of disruptions to traffic as a result of maintenance activities, as the majority of services can be accessed from the separate egress route. In addition to this, the risks associated with constructing Option A are lower compared to other options. Although some challenges may arise due to the large excavation sizes (main highway tunnel and portals) and construction of the internal dividing wall, eliminating the need to construct an egress tunnel (confined space) negates the increasing difficulty of construction of the main bore. As a result, Option A has been deemed a viable option for Floyd Hill Tunnel and has thus been recommended for further consideration at the next stage of the project.

### 16.4.1.2. Option B

Option B provides negligible additional operational benefit compared to the other options, primarily because all M&E equipment would need to be located within the soffit of the main highway tunnel (increasing difficulty for maintenance access). The construction risk for this option has been assessed as high. This is primarily because construction of the portals for the egress tunnels would be next to the live I-70 highway, which would provide significant health and safety hazards, for both workers and the public. Although the excavation span for the highway tunnel and volumes for the portals seems lower, a significant volume of material would need to be excavated for the egress tunnel portals, which has not been accounted for. Therefore, Option B has been deemed to provide no additional advantages compared with Option C. Following the reasons outlined, Option B has not been recommended as a viable option for Floyd Hill Tunnel and has therefore not been considered further. This being said, if CDOT wish to develop this option, opportunities may be explored to reduce the number of egress tunnel portals from two to one by bringing each egress tunnel out to the same location, as outlined in Section 6.2.3.

#### 16.4.1.3. Option C

Option C has been deemed to provide a moderate level of operational benefit, due to the ability of running M&E equipment through the egress tunnel – although routing of equipment would be limited to through cross passages, providing less of an advantage than Option A. The use of a parallel egress tunnel does provide health and safety risks associated with working in confined spaces; however, treating the egress tunnel as a 'pilot' excavation and constructing it before the main highway tunnel will allow information on rock quality to be obtained, which will help inform the excavation and subsequently the support of the main bore. Option C is deemed to be significantly better than Option B, from both an operational and constructability perspective. As a result, Option C has been deemed a viable option for Floyd Hill Tunnel and has thus been recommended for further consideration at the next stage of the project.

#### 16.4.1.4. Option D

Option D provides a fundamentally different solution to Options A to C, through construction of two highway tunnels (each with two lanes of traffic) as opposed to a single tunnel. This provides significant operational benefits compared to the other options, both in the emergency mode whereby passengers can evacuate through cross passages into the other bore, and also the maintenance mode whereby traffic can be moved to a single bore in the event of closure of one of the bores. Contrastingly, Option D has been deemed high in terms of construction risk. This is due to the large excavation volumes associated with constructing a second highway tunnel, the larger portal cut excavation volumes compared to the other options, and the difficulties in splitting the existing road network to tie it in with the tunnel. It is expected this option will be more expensive than the other options, which has been corroborated through the cost estimates produced. Following the reasons outlined, Option D has not been recommended as a viable option for Floyd Hill Tunnel and has therefore not been considered further.

### 16.4.2. Final Options Selection

Evaluation of the shortlisted options identified two preferred configurations for the I-70 Floyd Hill Tunnel:

- **Option A** – single bore, dedicated egress route provided by a compartmentalized corridor within the main highway tunnel;
- **Option C** – single bore, dedicated egress route provided by tunnels mined parallel and on the same level as the main highway tunnel.

Both of these options meet the requirements of NFPA 502, whilst both satisfy the operational requirements outlined by CDOT. Therefore, it is likely a decision will need to be made by CDOT in terms of how they value the potential operational benefits of the remaining options. Each option has its own advantages and disadvantages in terms of construction risk, and therefore may require the input of the Contractor appointed to build the tunnel, on the basis of preferences on construction methodologies. Although Option C is slightly cheaper and has a quicker construction duration than Option A, these estimates are indicative only and were based on limited information; thus, these estimates are subject to change and should not solely be used to select a preferred option until more detailed information becomes available.

Ultimately, both Option A and Option C are viable solutions for Floyd Hill Tunnel, with further discussions required between CDOT and the Contractor in order to decide on a single preferred option.

# 17. Recommendations & Further Work

## 17.1. Recommendations

This report shows that Floyd Hill Tunnel is feasible and can be designed and built to comply with the relevant codes, in particular NFPA 502. There are a number of matters within NFPA 502 (and additional design issues) that require the opinion of the AHJ, and it is recommended that these are sought in a face to face meeting prior to progressing the next stage of design, as these opinions will influence the design significantly. Table 17.1 summarizes the topics for discussion with the AHJ. The details are in the body of the report as shown in the paragraph references.

Table 17.1 - Topics for Discussion with the AHJ

| Report Reference | Discussion Point | |
|---|---|---|
| | **Topic** | **Decision** |
| 2.4.1 | Emergency protocol | To implement automatically, semi automatically or manually. |
| 2.4.4 | Evacuation | The harsh winter ambient conditions of Floyd Hill may also suggest the need for some sheltering for external assembly points. |
| 3.1.2 | Tunnel strategies for contraflow operation | Contraflow traffic through the tunnel in situations when the eastbound carriageway is closed. |
| 4.6 & 11.3 | Fire alarm and detection | Manual fire alarm boxes or emergency telephones. |
| 4.6 | Fire alarm and detection | 24/7 CCTV monitoring. |
| 4.7 & 11.4 | Emergency communications | Two-way radio communication enhancement systems. |
| 4.7 | Emergency communications | HAR and re-broadcasting of AM/FM commercial radio with overrides. |
| 4.9 | Fire apparatus | Water should be made available to supplement the Colorado Fire Department’s water. This can be either by the Fire Department or the tunnel facility. |
| 4.10 | Standpipes | Fire hydrant hose connections should be provided such that no location on the protected roadway is more than 150ft from the connection and a maximum spacing of 275ft, or less if required by the AHJ. |
| 4.16 | Means of egress | Tunnel roadway surface upstream of fire as means of escape with longitudinal ventilation system, exit doors serving as alternative access for fire rescue services. |
| 4.17 | FFFS | Provision of fixed fire-fighting system. |
| 4.20 | Emergency response | Emergency response plan. |
| 13.2 | Emergency exits | Emergency exit provision. |
| 8.4 & 11.8 | Fire likelihood analysis | Likelihood of fire events. |
| 9.2.4 | FFFS | Details of activation times and running times. |
| 14.2.4 | Passive fire protection to the structure | Acceptance of RWS time-temperature curve. |
| 15.3.3 | Evacuation | Provision of assembly points and portal configuration. |

## 17.2. Further Work

Areas for optimization and further investigation exist within the design of Floyd Hill Tunnel, and should be explored during the next design stage following closure of this feasibility study. Requirements for further work have been detailed in this section for different aspects of the tunnel.

### 17.2.1. Tunnel Operations

CDOT need to develop a control strategy for Floyd Hill Tunnel and their other tunnels so that they can be controlled in a safe and efficient way. This strategy needs to be developed before or in the early stages of the next design stage so that the tunnel facilities can be designed, and space allocated for local equipment. Without this strategy in place, the tunnel will have to be equipped with its own control room.

### 17.2.2. Tunnel Services

The requirements for drainage and utilities supplies for water, electricity, gas and telecoms need to be established and consultations need to take place with the utility companies to determine what work may be required to facilitate Floyd Hill Tunnel. This also includes the provision of a tunnel services building, which will likely be located at the West Portal, subject to further discussions with the AHJ.

### 17.2.3. Tunnel Ventilation

The likelihood of fires in the tunnel, the recommendation of the peak HRR and the conclusions on the ventilation strategy require analysis of latest information sourced from local authorities and state departments to inform decisions to be taken on fire life safety aspects appropriate to the operating conditions of the tunnel. System concepts and design strategies will require an extensive range of analysis and simulations to be performed to demonstrate the ability of the adopted systems to fulfil the required objectives.

As noted in this report, NFPA 502 is not the governing standard for the State of Colorado and referred to as a good practice. It is therefore prudent that early consultation is sought from underwriters of applicable codes to ensure the selected systems and tunnel operations comply with local regulations.

### 17.2.4. Tunnel Lining and Portals

To date, only a preliminary GI has been carried out, consisting of five boreholes at the portal locations. As there is currently limited information regarding the ground conditions across the proposed alignment of the tunnel, a more detailed GI must be undertaken across the entire alignment. This will provide more clarity on the slope of the rock face and the degree of jointing within the rock, as well as the depth of weathered material across the portal sections, thus unearthing any significant issues regarding the design and construction of the portals. A possible weakness zone was identified just above crown level at the East Portal, potentially impacting on the level of temporary support both for construction of the portals and the main tunnel, and therefore must be studied in greater detail through further GI testing.

In terms of the tunnel lining design, an exercise should be undertaken to see if it is feasible to remove the reinforcement bars recommended for the secondary lining, instead using SFRC if possible. This should be undertaken by changing the arc profile (a more curved profile will reduce the amount of bending moment and shear force acting on the secondary lining), with a subsequent cost assessment being carried out to determine whether optimization of the roof profile changes the overall cost of constructing the secondary lining and/or carries any long-term benefits in terms of durability of the lining.

The construction of the portals will also require consideration at the next design stage, particularly at the East Portal where there may be significant impacts on the existing I-70 highway, which will remain open during construction of Floyd Hill Tunnel. In terms of the portal size, it may be deemed necessary to provide additional space for emergency assembly points, the size requirements of which will have to be determined. As the design maturity increases, the amount and type of support for the construction of the portals will have to be revised. To provide sufficient rockfall protection, a canopy structure protruding out from the headwall may be required, as well as additional structures to provide protection to traffic due to the steep side slopes – this will require further investigation.

# References

[1] NFPA (2017). Standard for Road Tunnels, Bridges and Other Limited Access Highways.

[2] The Office of Hazardous Materials Safety Research and Special Programs Administration, U.S. Department of Transportation (1998). Hazardous Materials Shipments.

[3] Ingason, H. & Lonnermark, A. (2005). Heat release rates from heavy goods vehicle trailer fires in tunnels. Fire Safety Journal, Vol 40, no.7, pp. 646-668.

[4] PIARC: Technical Committee C4, Road Tunnels Operation (Unpublished, 2011). Design fire characteristics for road tunnels.

[5] UPTUN (2008). Work Package 2: Fire development and mitigation measures D213 - Runehamar Tunnel fire tests.

[6] FHWA, U.S. Department of Transportation (2018). Highway Statistics 2016. https://www.fhwa.dot.gov/policyinformation/statistics/2016/vm1.cfm

[7] Karter, M. (2014). Fire Loss in the United States During 2013. NFPA Fire Analysis and Research Division.

[8] NFPA (2016). Highway vehicle fires. https://www.nfpa.org/News-and-Research/Data-research-and-tools/ARCHIVED/Fire-statistics/Vehicle-fires/Highway-vehicle-fires

[9] DARTS (2004). Work Package 4 Report 4.4: Identification and quantification of hazards - Fire and smoke.

[10] TRL (2006). PPR140: Ventilation During Road Tunnel Emergencies.

[11] U.S.NRC (2011). Analysis of Severe Roadway Accidents Involving Long Duration Fires.

[12] Tarada, F. & Bertwistle, J. (2010). Performance-Based Design Using Tunnel Fire Suppression. North American Tunneling Conference, Portland, Oregon, USA, 2010, pp. 375-386.

[13] Cheong, M.K, Cheong, W.O., Leong K.W. & Tarada, F. (2013). Energy Budget in Suppressed Tunnel Fires. BHRG ISAVFT 2013 Proceedings, Barcelona, Spain, 18-20 September 2013, pp. 649-662.

[14] SOLIT$^2$ (2012). Engineering Guidance for a Comprehensive Evaluation of Tunnels with Fixed Fire Fighting Systems.

[15] ASHRAE (2001). Handbook 2001: Fundamentals, Chapter 27 – Climatic design information.

[16] ASHRAE (2017). Climatic Design Conditions – Station Finder. https://klimaat.github.io/StationFinder/

[17] ASHRAE (2011). Handbook 2011: HVAC Applications, Chapter 15 – Enclosed Vehicular Facilities.

[18] The Stationery Office Ltd. (2017). Design Manual for Roads and Bridges – Volume 2, Section 2, Part 9, BD 78/99: Design of Road Tunnels.

[19] Sheridan D.M. & Marsh, S.P. (1976). Geologic map of the Squaw Pass Quadrangle, Clear Creek, Jefferson, and Gilpin Counties, Colorado. https://ngmdb.usgs.gov/Prodesc/proddesc_10872.htm

[20] Grimstad, E. & Barton, N. (1993). Updating of the Q-System for NMT. Proceedings of the International Symposium on Sprayed Concrete, Fagernes, 22-26 October 1993, pp. 46-66.

[21] Kaiser, P.K., Diederichs, M.S., Martin, C.D., Sharp, J. & Steiner, W. (2000). Underground Works In Hard Rock Tunnelling And Mining. International Society for Rock Mechanics and Rock Engineering International Symposium, Melbourne, Australia, 19-24 November 2000, pp. 841-926.

[22] NGI (2013). Using the Q-System – Rock Mass Classification and Support Design. NGI Publication.

[23] Lowson, A.R. & Bieniawski, Z.T. (2013). Critical assessment of RMR-based tunnel design practices: A practical engineer's approach. pp. 180-198.

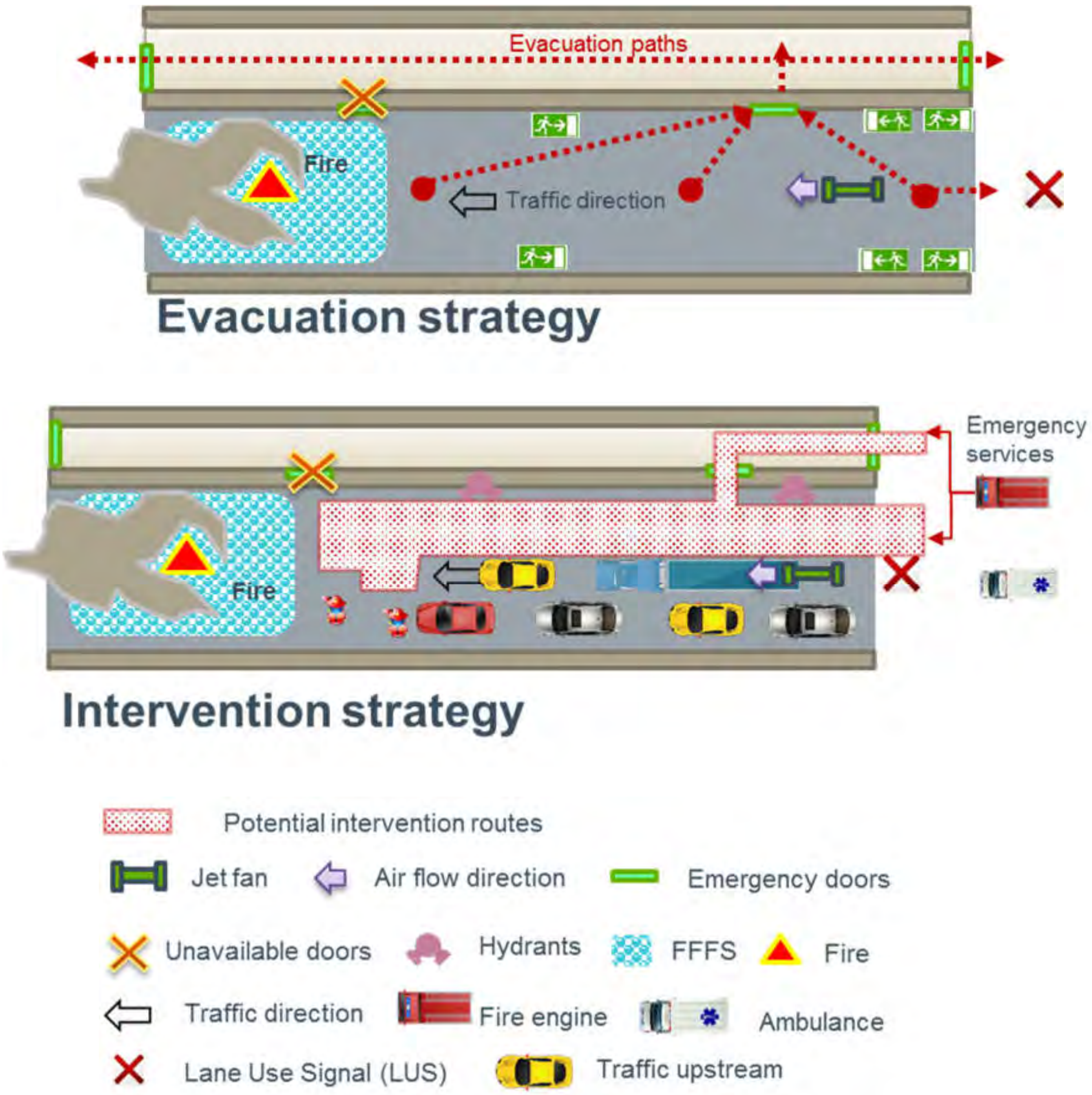
[24] BSI (2002). BS EN 1990:2002 Eurocode 0: Basis of structural design.

[25] BSI (2004). BS EN 1992-1-1:2004 Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings.

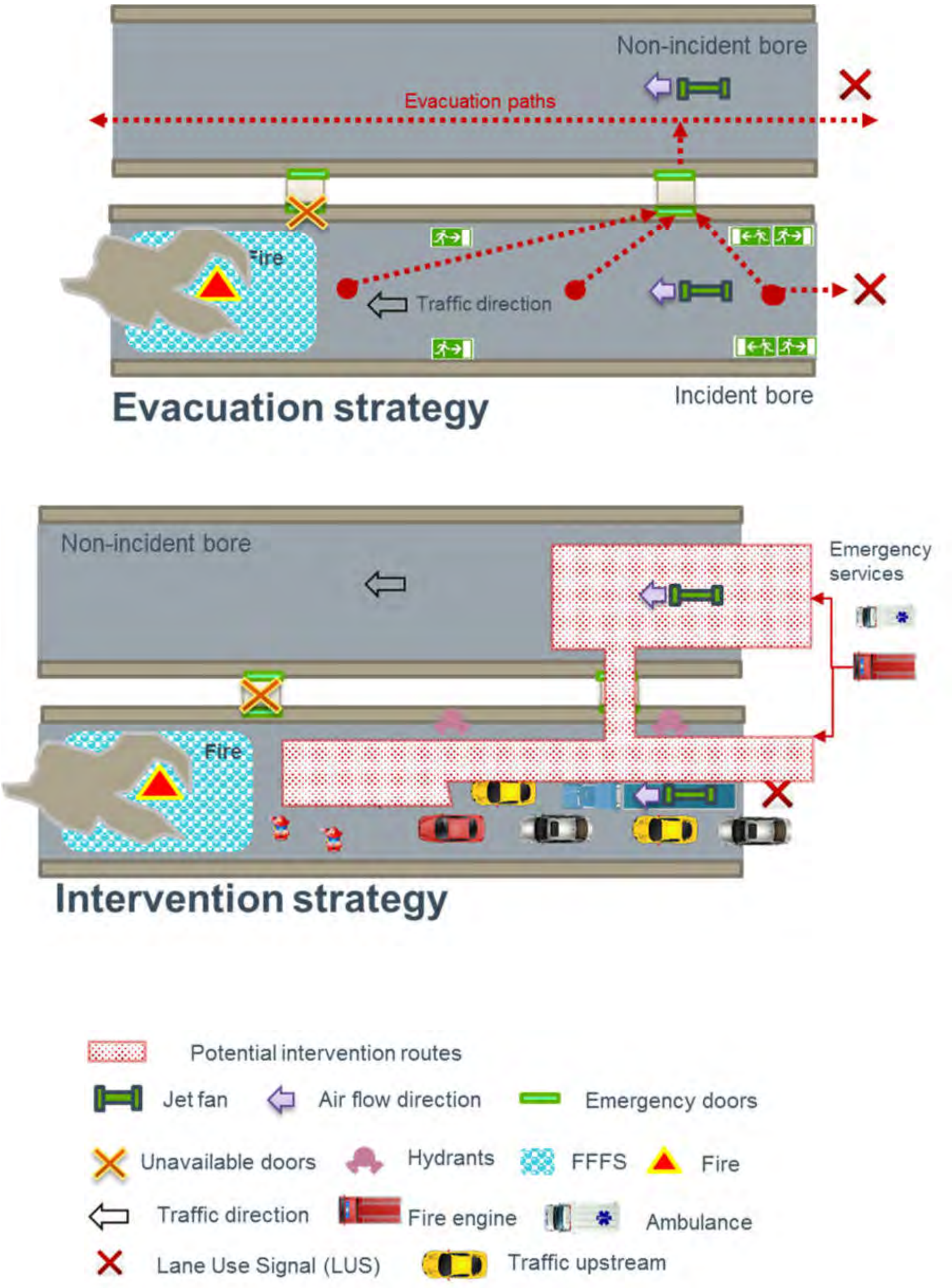
# Appendices

# Appendix A. Sample Emergency Strategies

## A.1. Tunnel Configuration Options A, B or C



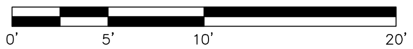
## A.2. Tunnel Configuration Option D

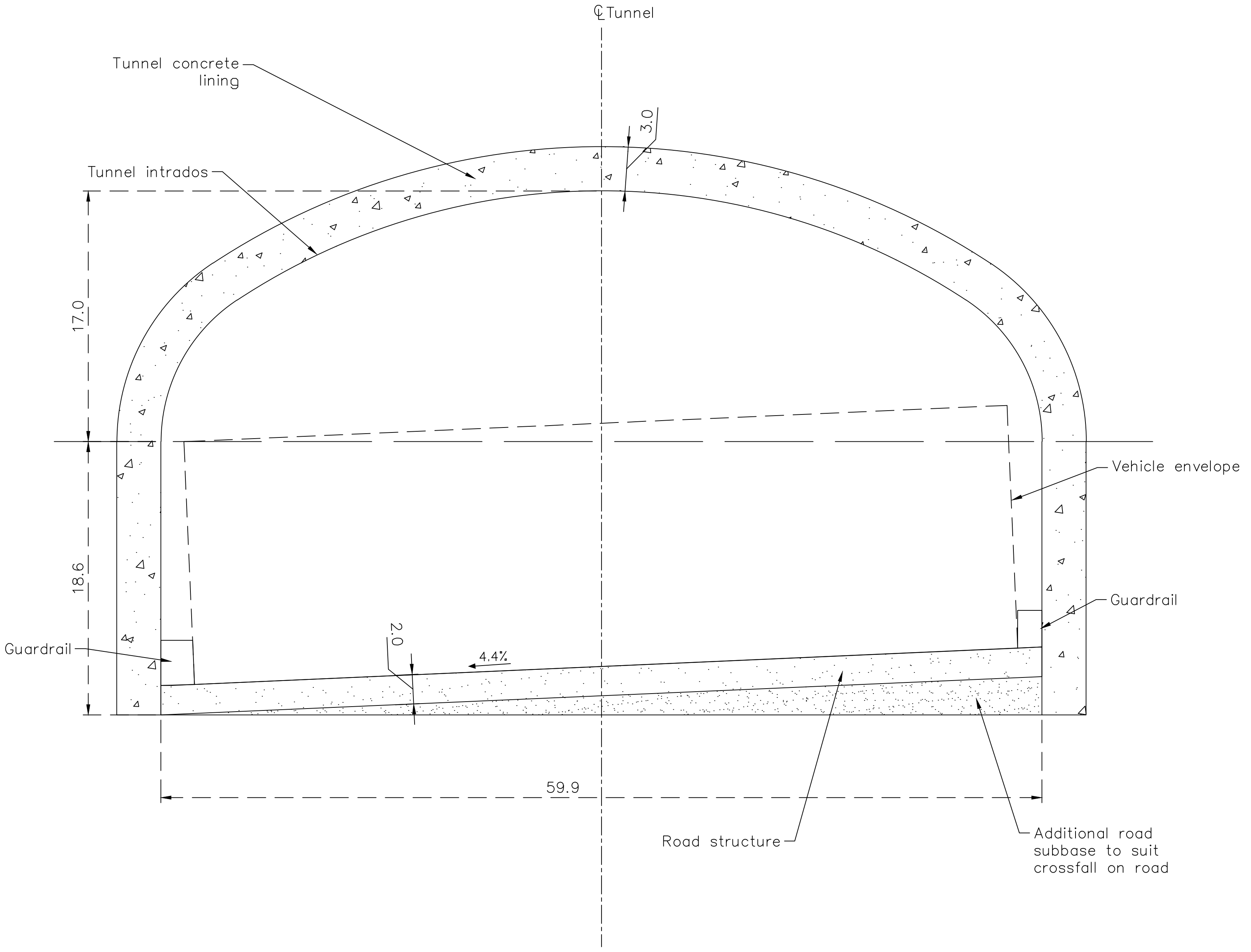


**Evacuation strategy**

**Intervention strategy**

# Appendix B. Tunnel Configuration Options 1-9 Drawings





Option 1: Single bore, no specific refuge or egress provisions

Notes:

1. All dimensions are in feet unless otherwise shown.
2. The section is taken as Westbound through the tunnel.
3. Road thickness and tunnel concrete lining thickness are indicative and subject to change.
4. This drawing is illustrative and does not represent a final schematic for the tunnel.

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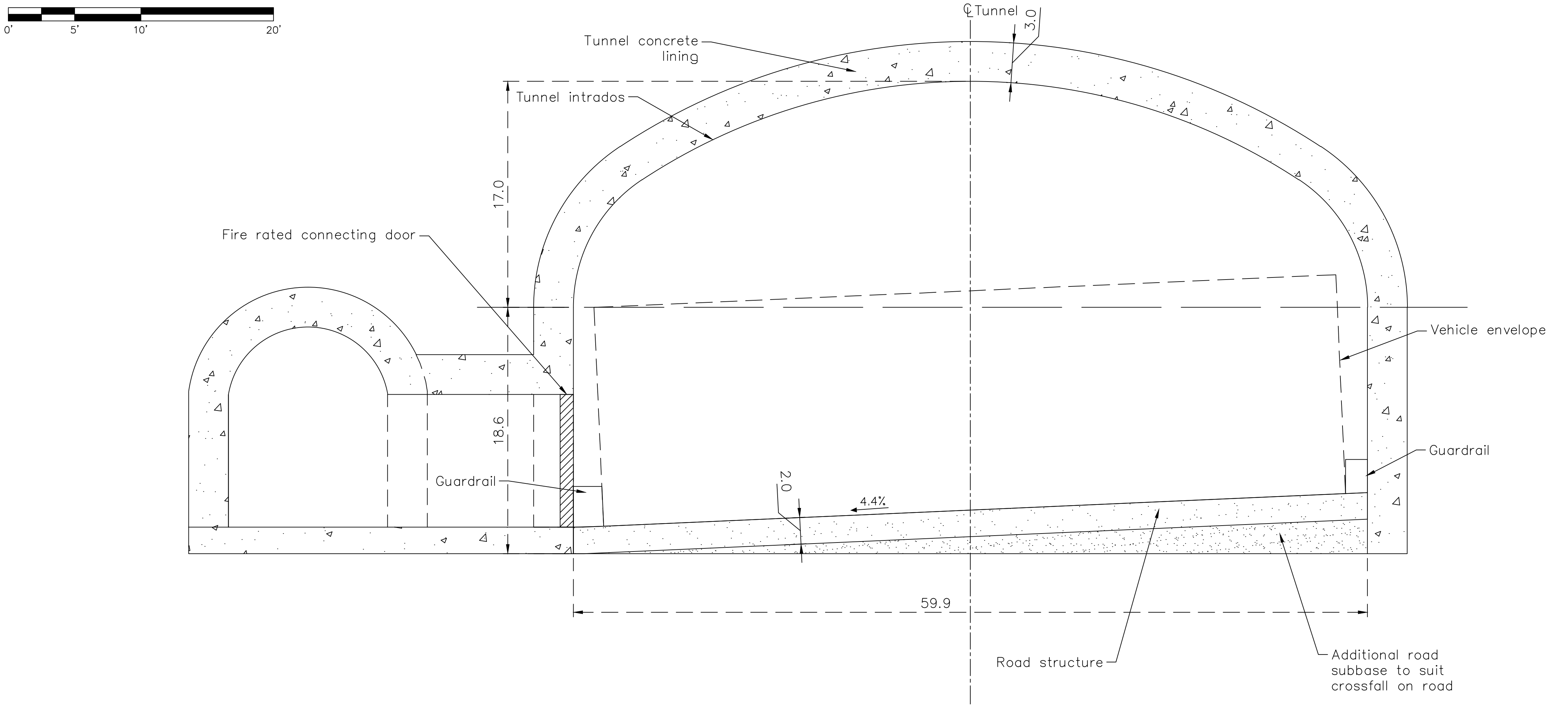
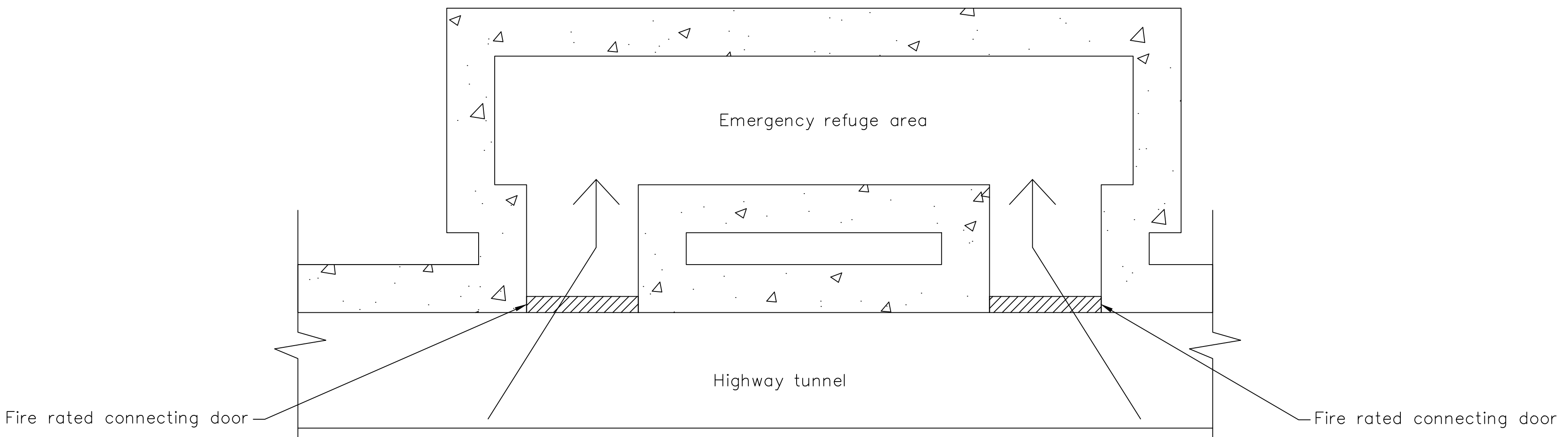


Option 2: Single bore, dedicated refuge chambers, no specific egress provisions



Plan on in–tunnel refuge area

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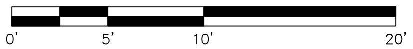

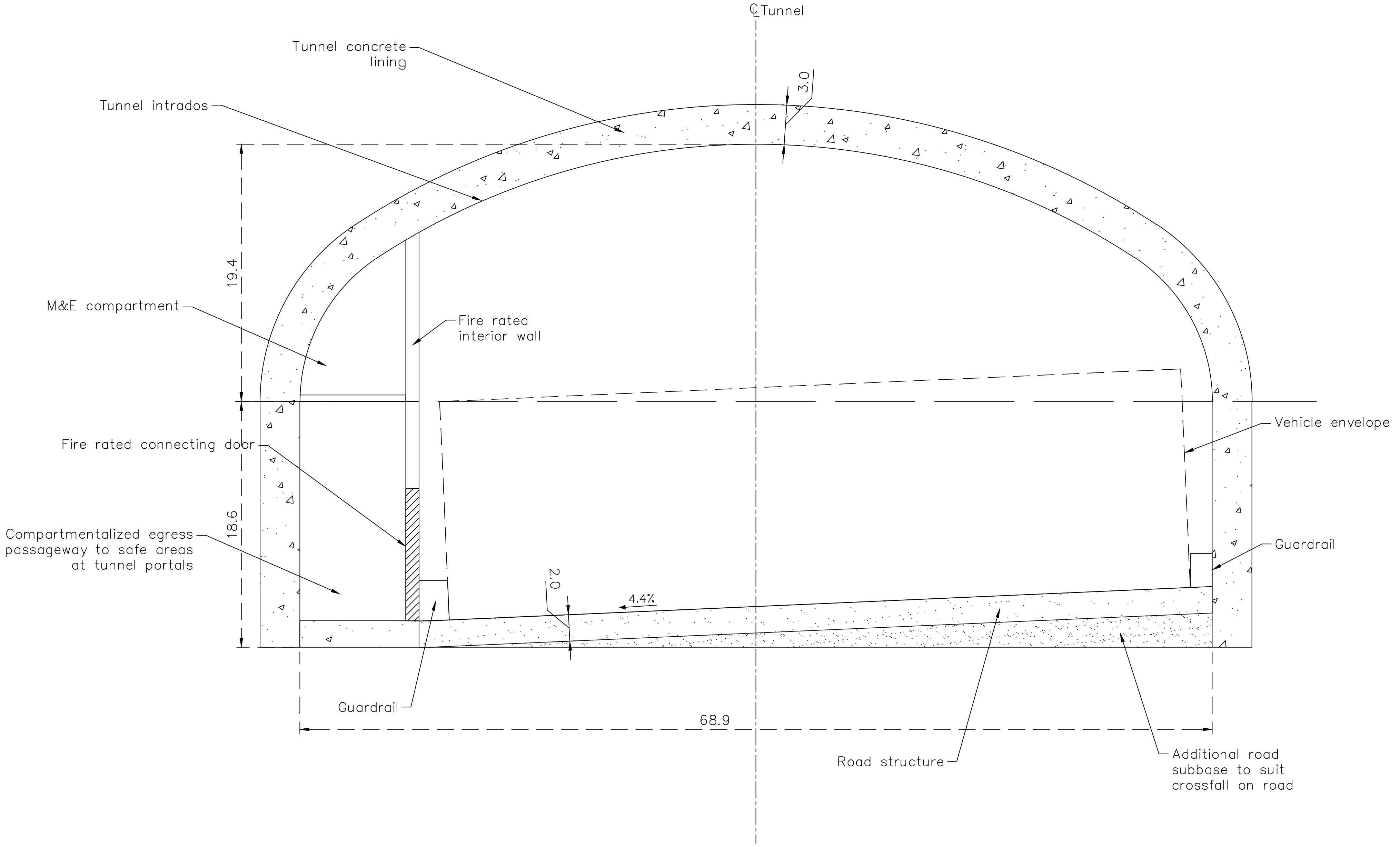

Option 3: Single bore, dedicated egress route provided by a compartmentalized corridor within the main highway tunnel

Notes:

1. All dimensions are in feet unless otherwise shown.
2. The section is taken as Westbound through the tunnel.
3. Road thickness and tunnel concrete lining thickness are indicative and subject to change.
4. This drawing is illustrative and does not represent a final schematic for the tunnel.

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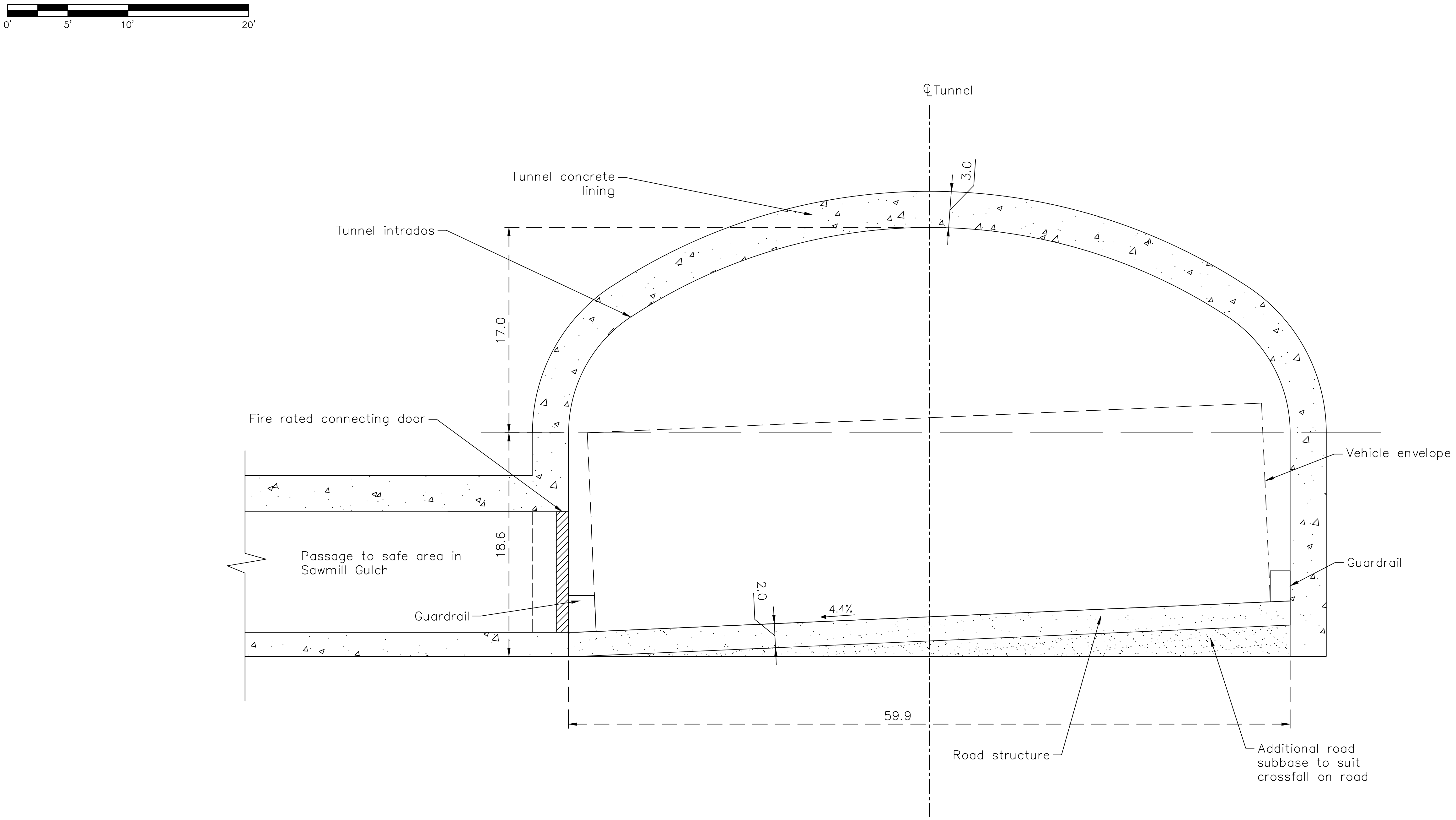

Option 4: Single bore, dedicated egress route provided by tunnels mined perpendicular to the main highway tunnel

Notes:

1. All dimensions are in feet unless otherwise shown.
2. The section is taken as Westbound through the tunnel.
3. Road thickness and tunnel concrete lining thickness are indicative and subject to change.
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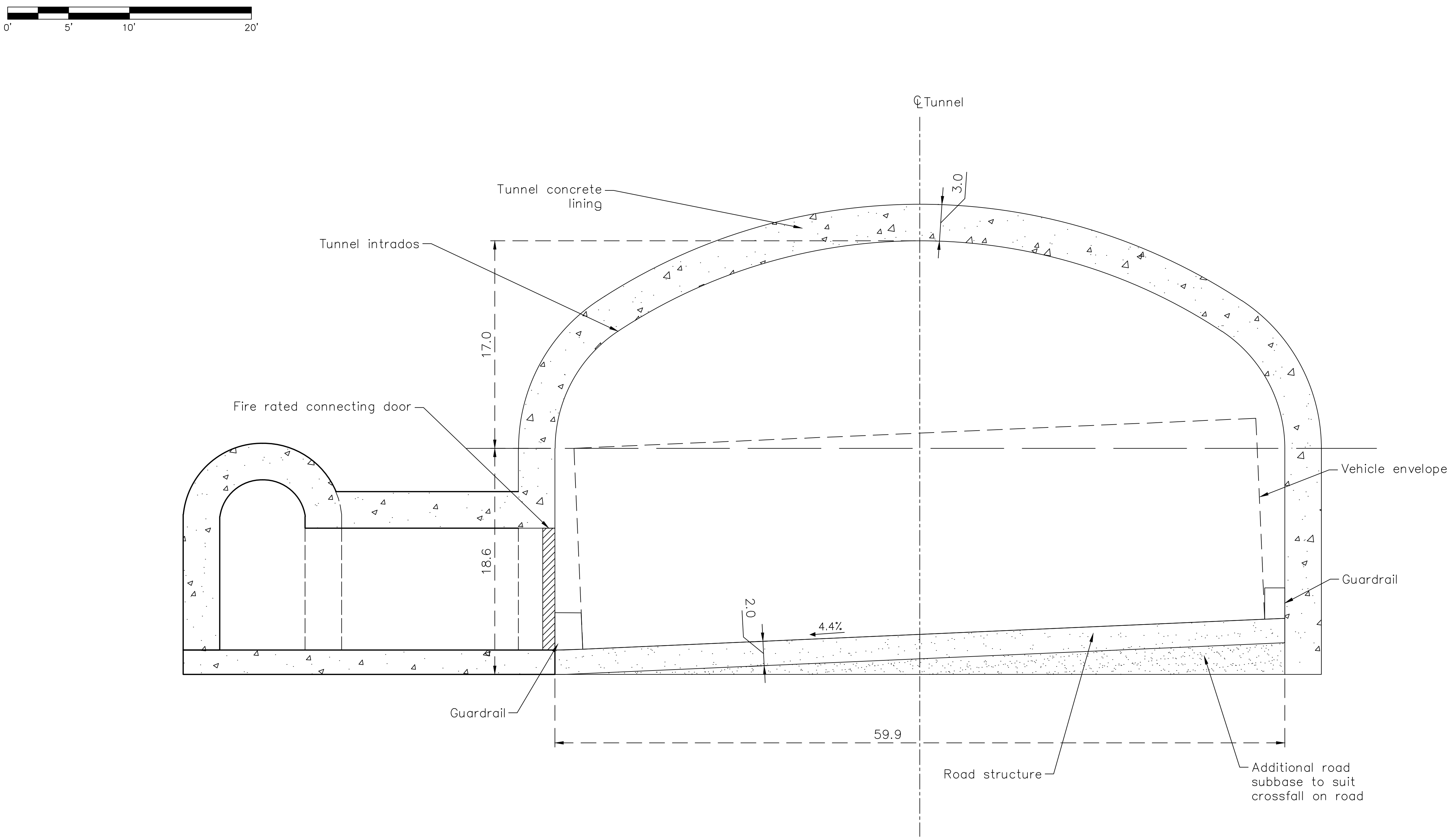

Option 5: Single bore, dedicated egress route provided by tunnels mined parallel and on the same level as the main highway tunnel

Notes:
1. All dimensions are in feet unless otherwise shown.
2. The section is taken as Westbound through the tunnel.
3. Road thickness and tunnel concrete lining thickness are indicative and subject to change.
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I-70 Floyd Hill Tunnel
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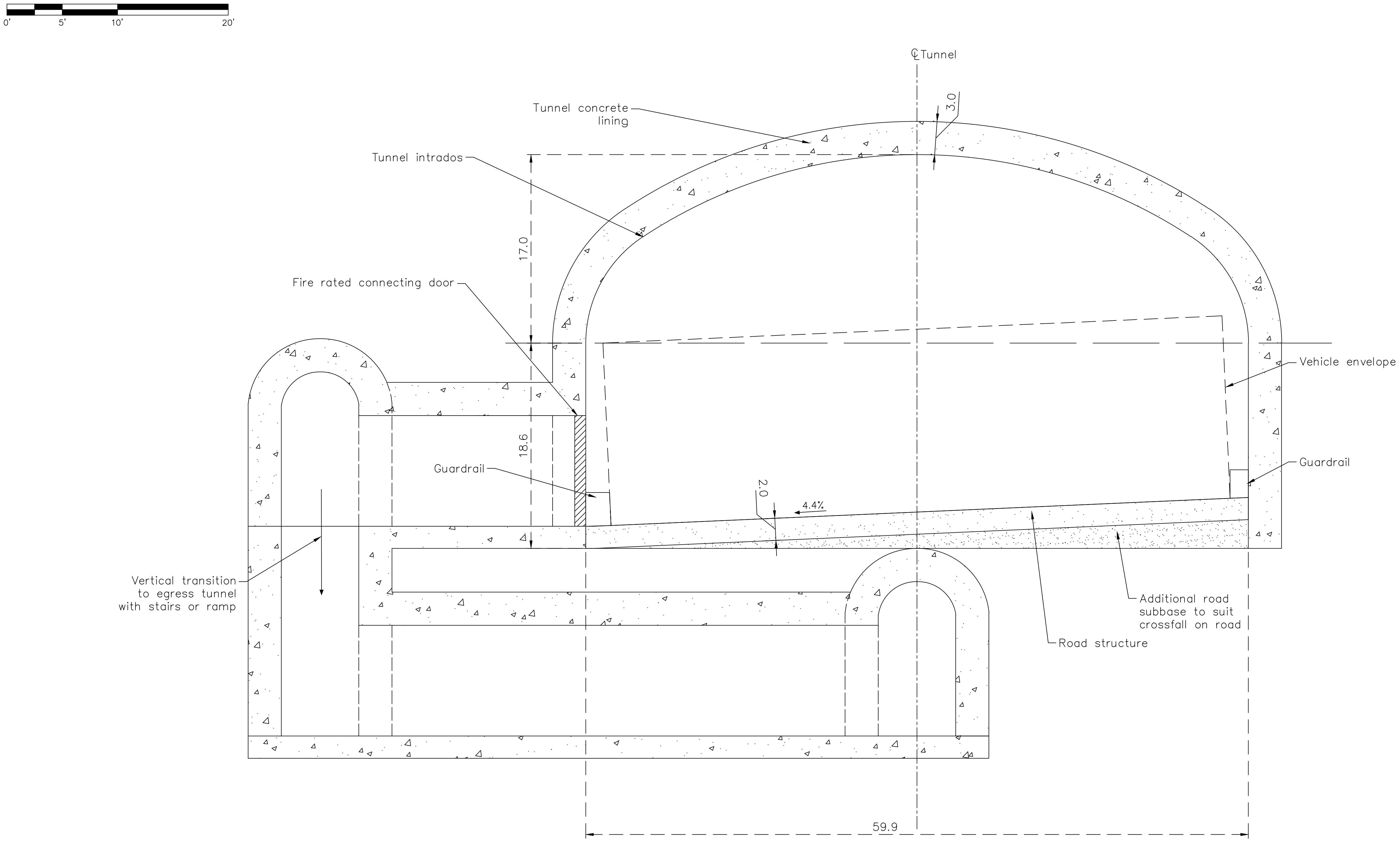

Option 6: Single bore, dedicated egress route provided by tunnels mined parallel and beneath the main highway tunnel

Notes:
1. All dimensions are in feet unless otherwise shown.
2. The section is taken as Westbound through the tunnel.
3. Road thickness and tunnel concrete lining thickness are indicative and subject to change.
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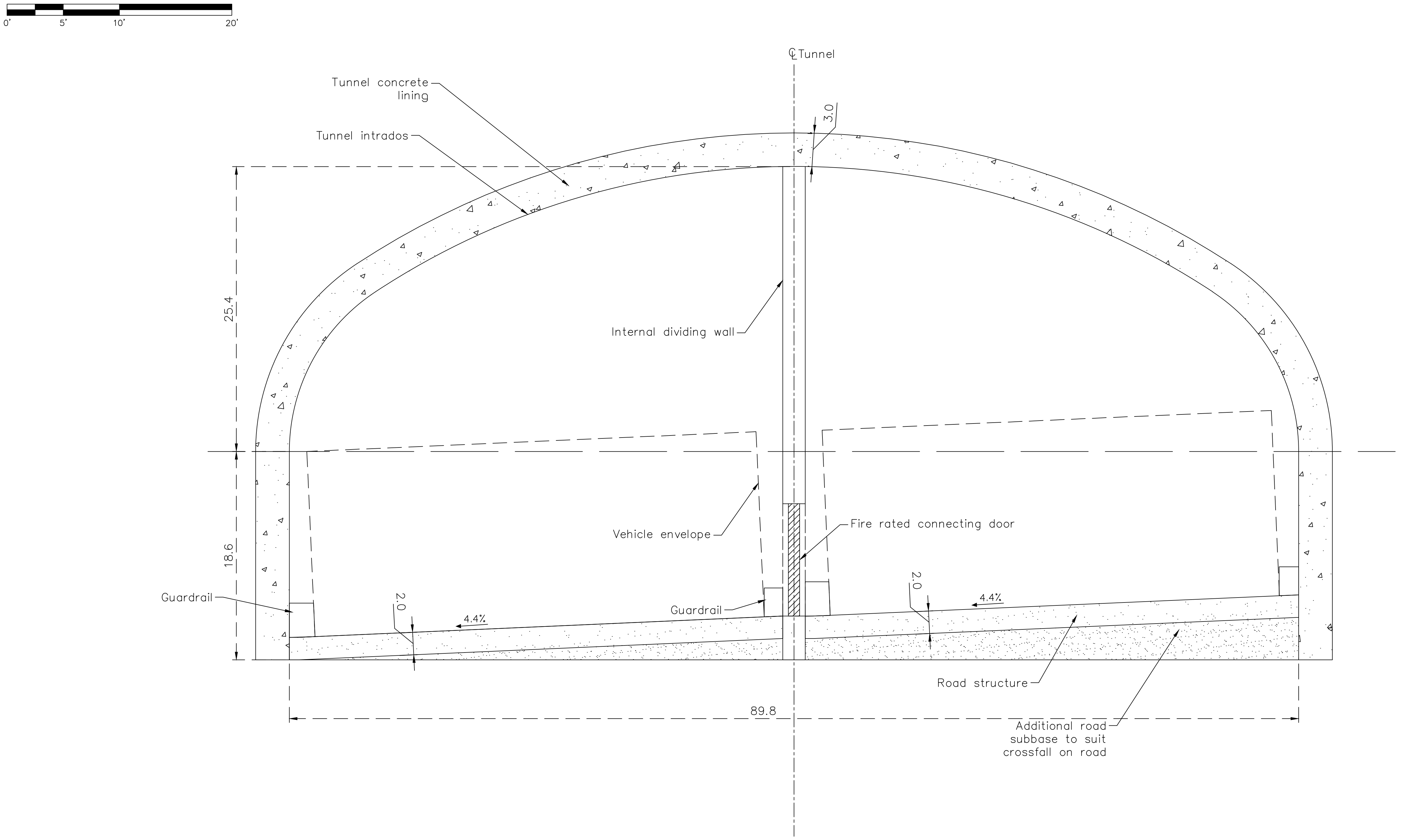

Option 7: Single bore with two segregated highway compartments provided by an internal dividing wall (egress is through evacuation into the other highway compartment)

Notes:
1. All dimensions are in feet unless otherwise shown.
2. The section is taken as Westbound through the tunnel.
3. Road thickness and tunnel concrete lining thickness are indicative and subject to change.
4. This drawing is illustrative and does not represent a final schematic for the tunnel.

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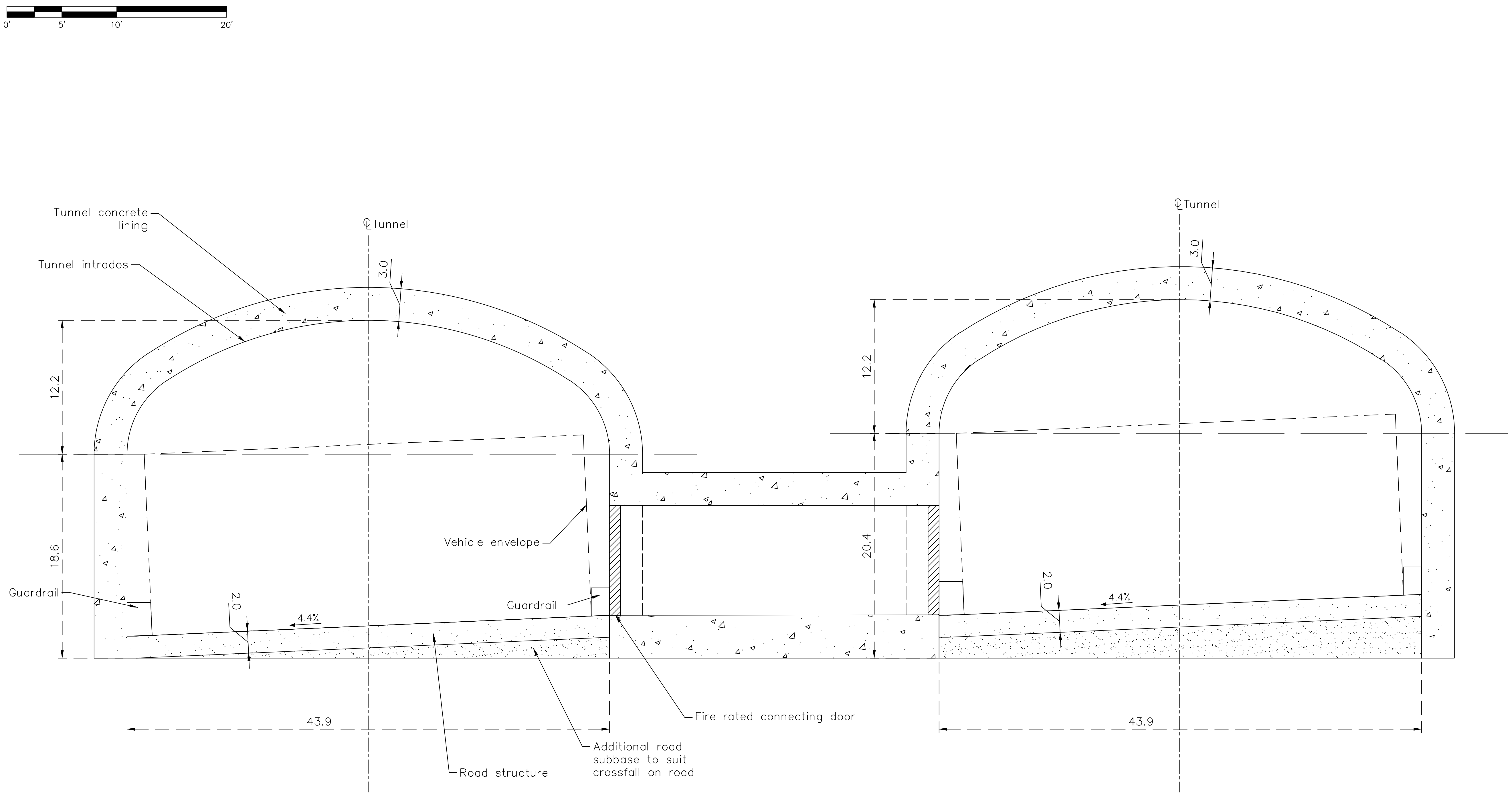

Option 8: Twin bore, with interconnecting cross passages
(egress is through evacuation into the other bore)

Notes:
1. All dimensions are in feet unless otherwise shown.
2. The section is taken as Westbound through the tunnel.
3. Road thickness and tunnel concrete lining thickness are indicative and subject to change.
4. This drawing is illustrative and does not represent a final schematic for the tunnel.

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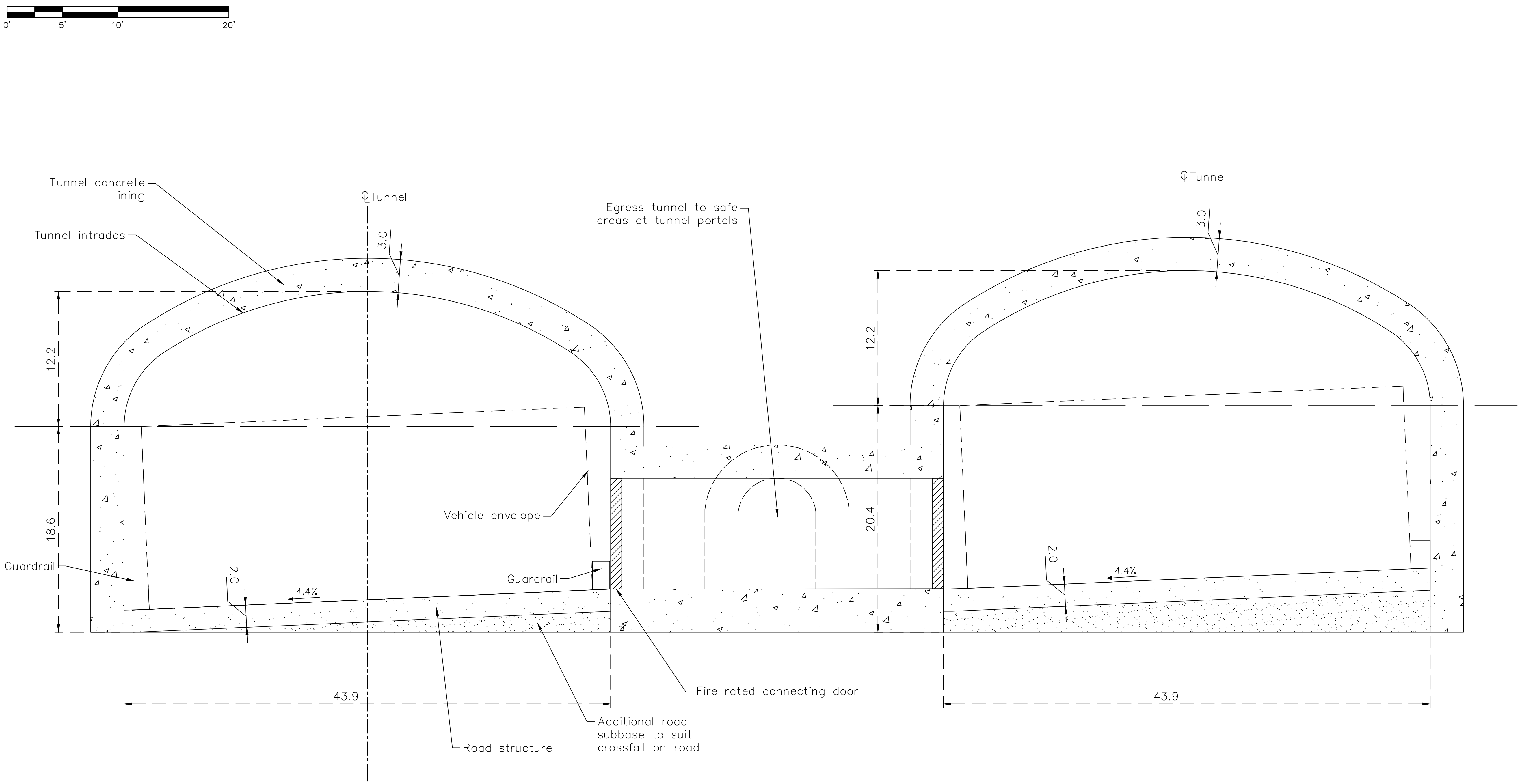

Option 9: Twin bore, with interconnecting cross passages and a dedicated egress tunnel located between the two bores

Notes:

1. All dimensions are in feet unless otherwise shown.
2. The section is taken as Westbound through the tunnel.
3. Road thickness and tunnel concrete lining thickness are indicative and subject to change.
4. This drawing is illustrative and does not represent a final schematic for the tunnel.

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Colorado Department of Transportation
425 A Corporate Circle
Golden, CO 80401
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Region 1

For Information
No Revisions:
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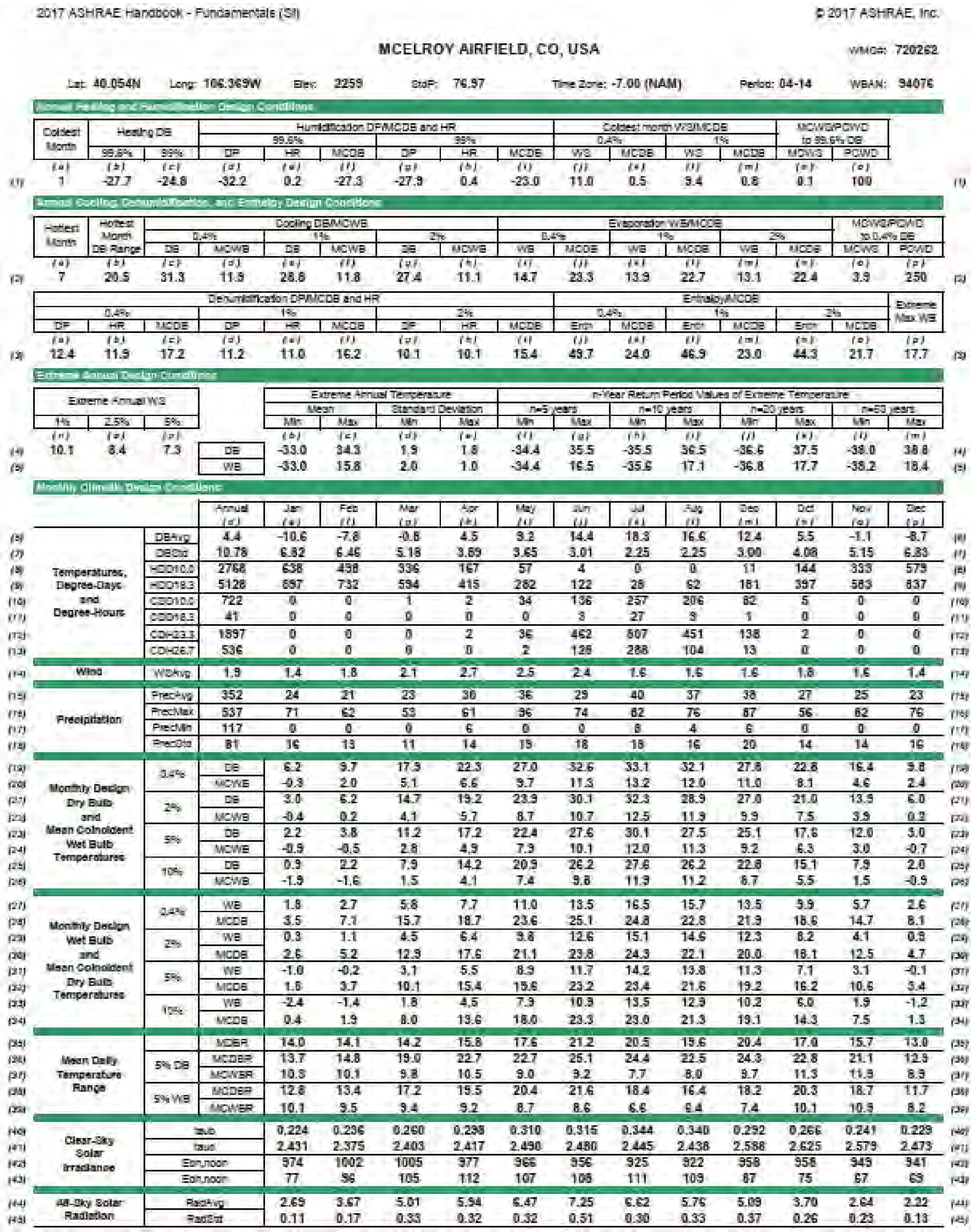
Void:

I-70 Floyd Hill Tunnel
Tunnel Configuration Option 9

Designer: LM
Detailer: HK
Subset:
Structure Numbers
Subset Sheets: of

Project No./Code
21912
Sheet Number

# Appendix C. Ambient Weather Data



2017 ASHRAE Handbook - Fundamentals (SI)

## MCELROY AIRFIELD, CO, USA

WMO#: 720262

Lat: 40.054N Long: 106.369W Elev: 2259 StdP: 76.97 Time Zone: -7.00 (NAM) Period: 04-14 WBAN: 94076

### Annual Heating and Humidification Design Conditions

| | Coldest Month | Heating DB | | Humidification DP/MCDB and HR | | | | | | Coldest month WS/MCDB | | | | MCWS/PCWD to 99.6% DB | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 99.6% | | | 99% | | | 0.4% | | 1% | | | | |
| | | 99.6% | 99% | DP | HR | MCDB | DP | HR | MCDB | WS | MCDB | WS | MCDB | MCWS | PCWD | |
| | (a) | (b) | (c) | (d) | (e) | (f) | (g) | (h) | (i) | (j) | (k) | (l) | (m) | (n) | (o) | |
| (1) | 1 | -27.7 | -24.8 | -32.2 | 0.2 | -27.3 | -27.9 | 0.4 | -23.0 | 11.0 | 0.5 | 9.4 | 0.8 | 0.1 | 100 | (1) |

### Annual Cooling, Dehumidification, and Enthalpy Design Conditions

| | Hottest Month | Hottest Month DB Range | Cooling DB/MCWB | | | | | | Evaporation WB/MCDB | | | | | | MCWS/PCWD to 0.4% DB | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 0.4% | | 1% | | 2% | | 0.4% | | 1% | | 2% | | | | |
| | | | DB | MCWB | DB | MCWB | DB | MCWB | WB | MCDB | WB | MCDB | WB | MCDB | MCWS | PCWD | |
| | (a) | (b) | (c) | (d) | (e) | (f) | (g) | (h) | (i) | (j) | (k) | (l) | (m) | (n) | (o) | (p) | |
| (2) | 7 | 20.5 | 31.3 | 11.9 | 28.8 | 11.8 | 27.4 | 11.1 | 14.7 | 23.3 | 13.9 | 22.7 | 13.1 | 22.4 | 3.9 | 250 | (2) |

| | Dehumidification DP/MCDB and HR | | | | | | | | | Enthalpy/MCDB | | | | | | Extreme Max WB | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0.4% | | | 1% | | | 2% | | | 0.4% | | 1% | | 2% | | | |
| | DP | HR | MCDB | DP | HR | MCDB | DP | HR | MCDB | Enth | MCDB | Enth | MCDB | Enth | MCDB | | |
| | (a) | (b) | (c) | (d) | (e) | (f) | (g) | (h) | (i) | (j) | (k) | (l) | (m) | (n) | (o) | (p) | |
| (3) | 12.4 | 11.9 | 17.2 | 11.2 | 11.0 | 16.2 | 10.1 | 10.1 | 15.4 | 49.7 | 24.0 | 46.9 | 23.0 | 44.3 | 21.7 | 17.7 | (3) |

### Extreme Annual Design Conditions

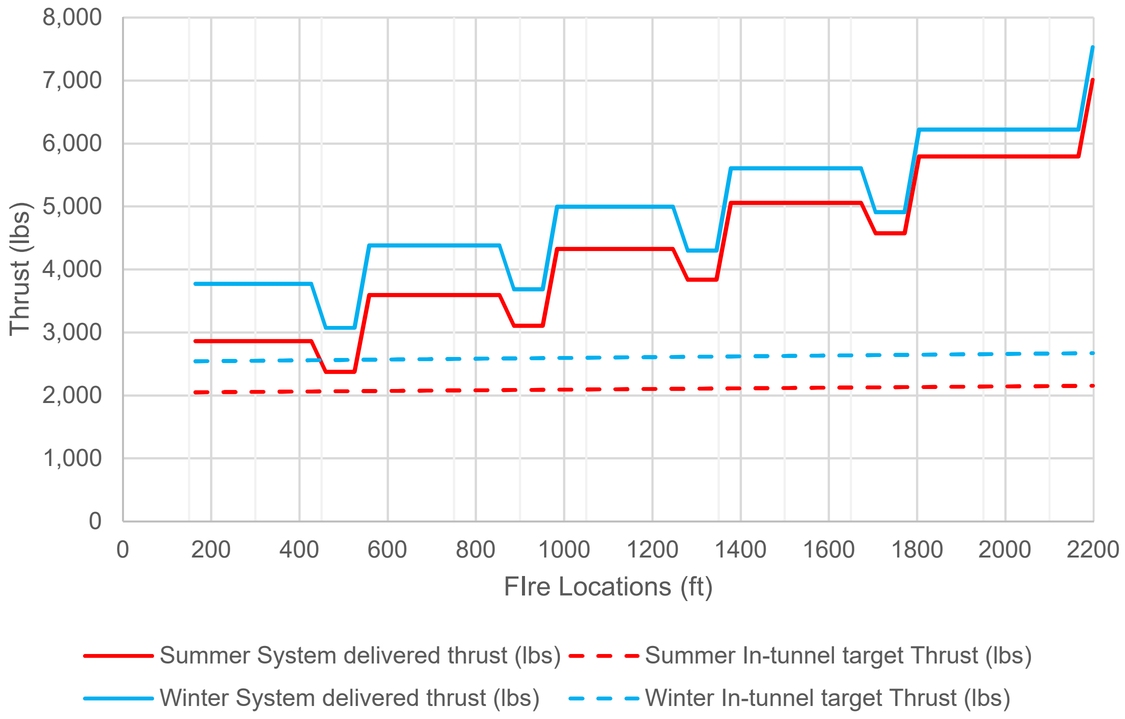
| | Extreme Annual WS | | | | Extreme Annual Temperature | | | | n-Year Return Period Values of Extreme Temperature | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Mean | | Standard Deviation | | n=5 years | | n=10 years | | n=20 years | | n=50 years | | |
| | 1% | 2.5% | 5% | | Min | Max | Min | Max | Min | Max | Min | Max | Min | Max | Min | Max | |
| | (a) | (b) | (c) | | (b) | (c) | (d) | (e) | (f) | (g) | (h) | (i) | (j) | (k) | (l) | (m) | |
| (4) | 10.1 | 8.4 | 7.3 | DB | -33.0 | 34.3 | 1.9 | 1.8 | -34.4 | 35.5 | -35.5 | 36.5 | -36.6 | 37.5 | -38.0 | 38.8 | (4) |
| (5) | | | | WB | -33.0 | 15.8 | 2.0 | 1.0 | -34.4 | 16.5 | -35.6 | 17.1 | -36.8 | 17.7 | -38.2 | 18.4 | (5) |

### Monthly Climatic Design Conditions

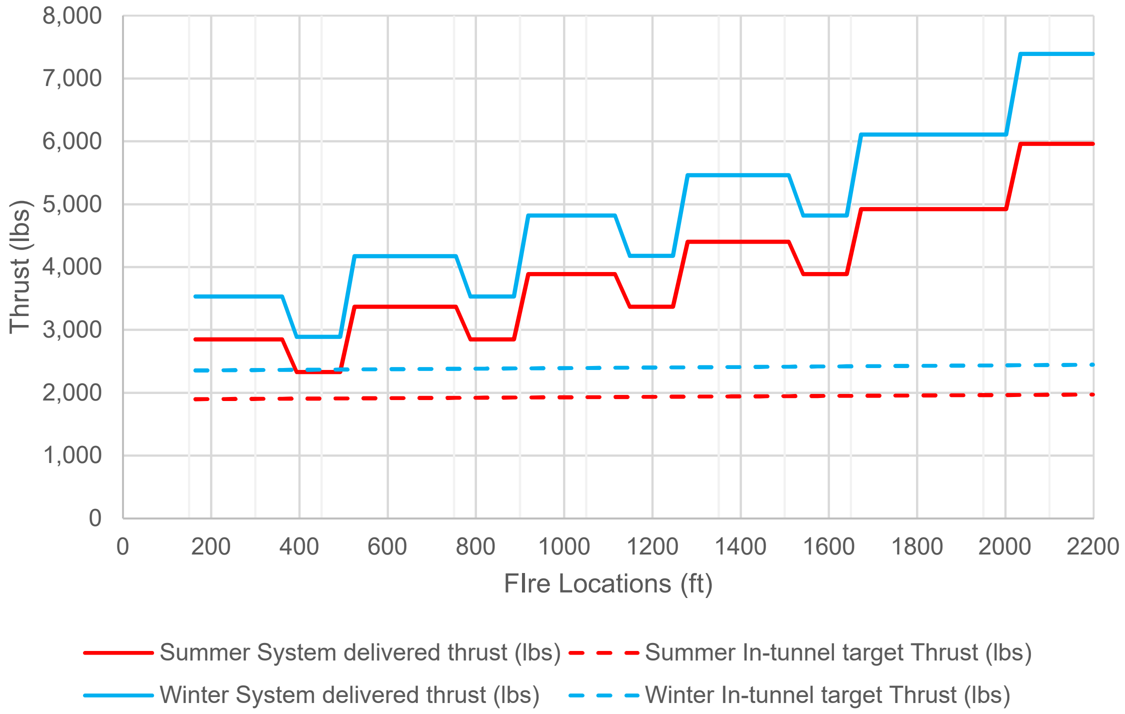
| | | | | Annual | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | (d) | (e) | (f) | (g) | (h) | (i) | (j) | (k) | (l) | (m) | (n) | (o) | (p) | |
| (6) | Temperatures, Degree-Days and Degree-Hours | | DBAvg | 4.4 | -10.6 | -7.8 | -0.8 | 4.5 | 9.2 | 14.4 | 18.3 | 16.6 | 12.4 | 5.5 | -1.1 | -8.7 | (6) |
| (7) | | | DBStd | 10.78 | 6.82 | 6.46 | 5.18 | 3.89 | 3.65 | 3.01 | 2.25 | 2.25 | 3.00 | 4.08 | 5.15 | 6.83 | (7) |
| (8) | | | HDD10.0 | 2768 | 638 | 458 | 336 | 167 | 57 | 4 | 0 | 0 | 11 | 144 | 333 | 579 | (8) |
| (9) | | | HDD18.3 | 5128 | 897 | 732 | 594 | 415 | 282 | 122 | 28 | 62 | 181 | 397 | 583 | 837 | (9) |
| (10) | | | CDD10.0 | 722 | 0 | 0 | 1 | 2 | 34 | 136 | 257 | 206 | 82 | 5 | 0 | 0 | (10) |
| (11) | | | CDD18.3 | 41 | 0 | 0 | 0 | 0 | 0 | 3 | 27 | 9 | 1 | 0 | 0 | 0 | (11) |
| (12) | | | CDH23.3 | 1897 | 0 | 0 | 0 | 2 | 36 | 462 | 807 | 451 | 138 | 2 | 0 | 0 | (12) |
| (13) | | | CDH26.7 | 536 | 0 | 0 | 0 | 0 | 2 | 128 | 288 | 104 | 13 | 0 | 0 | 0 | (13) |
| (14) | Wind | | WSAvg | 1.9 | 1.4 | 1.8 | 2.1 | 2.7 | 2.5 | 2.4 | 1.6 | 1.6 | 1.6 | 1.8 | 1.6 | 1.4 | (14) |
| (15) | Precipitation | | PrecAvg | 352 | 24 | 21 | 23 | 30 | 36 | 29 | 40 | 37 | 38 | 27 | 25 | 23 | (15) |
| (16) | | | PrecMax | 537 | 71 | 62 | 53 | 61 | 96 | 74 | 82 | 76 | 87 | 56 | 82 | 76 | (16) |
| (17) | | | PrecMin | 117 | 0 | 0 | 0 | 6 | 0 | 0 | 8 | 4 | 6 | 0 | 0 | 0 | (17) |
| (18) | | | PrecStd | 81 | 16 | 13 | 11 | 14 | 19 | 18 | 18 | 16 | 20 | 14 | 14 | 16 | (18) |
| (19) | Monthly Design Dry Bulb and Mean Coincident Wet Bulb Temperatures | 0.4% | DB | | 6.2 | 9.7 | 17.9 | 22.3 | 27.0 | 32.6 | 33.1 | 32.1 | 27.8 | 22.8 | 16.4 | 9.8 | (19) |
| (20) | | | MCWB | | -0.3 | 2.0 | 5.1 | 6.6 | 9.7 | 11.3 | 13.2 | 12.0 | 11.0 | 8.1 | 4.6 | 2.4 | (20) |
| (21) | | 2% | DB | | 3.0 | 6.2 | 14.7 | 19.2 | 23.9 | 30.1 | 32.3 | 28.9 | 27.0 | 21.0 | 13.9 | 6.0 | (21) |
| (22) | | | MCWB | | -0.4 | 0.2 | 4.1 | 5.7 | 8.7 | 10.7 | 12.5 | 11.9 | 9.9 | 7.5 | 3.9 | 0.2 | (22) |
| (23) | | 5% | DB | | 2.2 | 3.8 | 11.2 | 17.2 | 22.4 | 27.6 | 30.1 | 27.5 | 25.1 | 17.6 | 12.0 | 3.0 | (23) |
| (24) | | | MCWB | | -0.9 | -0.5 | 2.8 | 4.9 | 7.9 | 10.1 | 12.0 | 11.3 | 9.2 | 6.3 | 3.0 | -0.7 | (24) |
| (25) | | 10% | DB | | 0.9 | 2.2 | 7.9 | 14.2 | 20.9 | 26.2 | 27.6 | 26.2 | 22.8 | 15.1 | 7.9 | 2.0 | (25) |
| (26) | | | MCWB | | -1.9 | -1.6 | 1.5 | 4.1 | 7.4 | 9.8 | 11.9 | 11.2 | 8.7 | 5.5 | 1.5 | -0.9 | (26) |
| (27) | Monthly Design Wet Bulb and Mean Coincident Dry Bulb Temperatures | 0.4% | WB | | 1.8 | 2.7 | 5.8 | 7.7 | 11.0 | 13.5 | 16.5 | 15.7 | 13.5 | 9.9 | 5.7 | 2.6 | (27) |
| (28) | | | MCDB | | 3.5 | 7.1 | 15.7 | 18.7 | 23.6 | 25.1 | 24.8 | 22.8 | 21.9 | 16.6 | 14.7 | 8.1 | (28) |
| (29) | | 2% | WB | | 0.3 | 1.1 | 4.5 | 6.4 | 9.8 | 12.6 | 15.1 | 14.6 | 12.3 | 8.2 | 4.1 | 0.9 | (29) |
| (30) | | | MCDB | | 2.6 | 5.2 | 12.9 | 17.6 | 21.1 | 23.8 | 24.3 | 22.1 | 20.0 | 18.1 | 12.5 | 4.7 | (30) |
| (31) | | 5% | WB | | -1.0 | -0.2 | 3.1 | 5.5 | 8.9 | 11.7 | 14.2 | 13.8 | 11.3 | 7.1 | 3.1 | -0.1 | (31) |
| (32) | | | MCDB | | 1.6 | 3.7 | 10.1 | 15.4 | 19.6 | 23.2 | 23.4 | 21.6 | 19.2 | 16.2 | 10.6 | 3.4 | (32) |
| (33) | | 10% | WB | | -2.4 | -1.4 | 1.8 | 4.5 | 7.3 | 10.3 | 13.5 | 12.9 | 10.2 | 6.0 | 1.9 | -1.2 | (33) |
| (34) | | | MCDB | | 0.4 | 1.9 | 8.0 | 13.6 | 18.0 | 23.3 | 23.0 | 21.3 | 19.1 | 14.3 | 7.5 | 1.3 | (34) |
| (35) | Mean Daily Temperature Range | | MDBR | | 14.0 | 14.1 | 14.2 | 15.8 | 17.6 | 21.2 | 20.5 | 19.6 | 20.4 | 17.0 | 15.7 | 13.0 | (35) |
| (36) | | 5% DB | MCDBR | | 13.7 | 14.8 | 19.0 | 22.7 | 22.7 | 25.1 | 24.4 | 22.5 | 24.3 | 22.8 | 21.1 | 12.9 | (36) |
| (37) | | | MCWBR | | 10.3 | 10.1 | 9.8 | 10.5 | 9.0 | 9.2 | 7.7 | 8.0 | 9.7 | 11.3 | 11.9 | 8.9 | (37) |
| (38) | | 5% WB | MCDBR | | 12.8 | 13.4 | 17.2 | 19.5 | 20.4 | 21.6 | 18.4 | 16.4 | 18.2 | 20.3 | 18.7 | 11.7 | (38) |
| (39) | | | MCWBR | | 10.1 | 9.5 | 9.4 | 9.2 | 8.7 | 8.6 | 6.6 | 6.4 | 7.4 | 10.1 | 10.9 | 8.2 | (39) |
| (40) | Clear-Sky Solar Irradiance | | taub | | 0,224 | 0.236 | 0.260 | 0.298 | 0.310 | 0.315 | 0.344 | 0.340 | 0.292 | 0.266 | 0.241 | 0.229 | (40) |
| (41) | | | taud | | 2.431 | 2.375 | 2.403 | 2.417 | 2.490 | 2.480 | 2.445 | 2.438 | 2.586 | 2.625 | 2.579 | 2.473 | (41) |
| (42) | | | Ebn,noon | | 974 | 1002 | 1005 | 977 | 966 | 956 | 925 | 922 | 958 | 958 | 949 | 941 | (42) |
| (43) | | | Edh,noon | | 77 | 96 | 105 | 112 | 107 | 108 | 111 | 109 | 87 | 75 | 67 | 63 | (43) |
| (44) | All-Sky Solar Radiation | | RadAvg | | 2.69 | 3.67 | 5.01 | 5.94 | 6.47 | 7.25 | 6.62 | 5.76 | 5.09 | 3.70 | 2.64 | 2.22 | (44) |
| (45) | | | RadStd | | 0.11 | 0.17 | 0.33 | 0.32 | 0.32 | 0.51 | 0.30 | 0.33 | 0.37 | 0.26 | 0.23 | 0.13 | (45) |

# Appendix D. Emergency System Performance

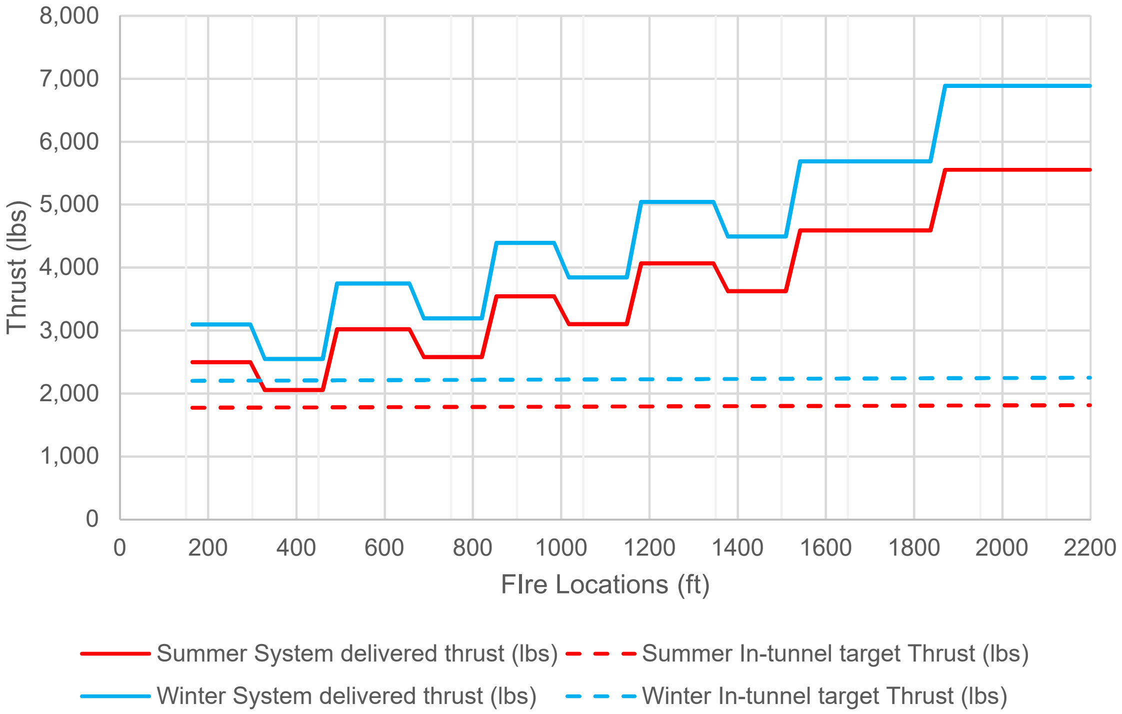
## D.1. Option A



## D.2. Option B/C



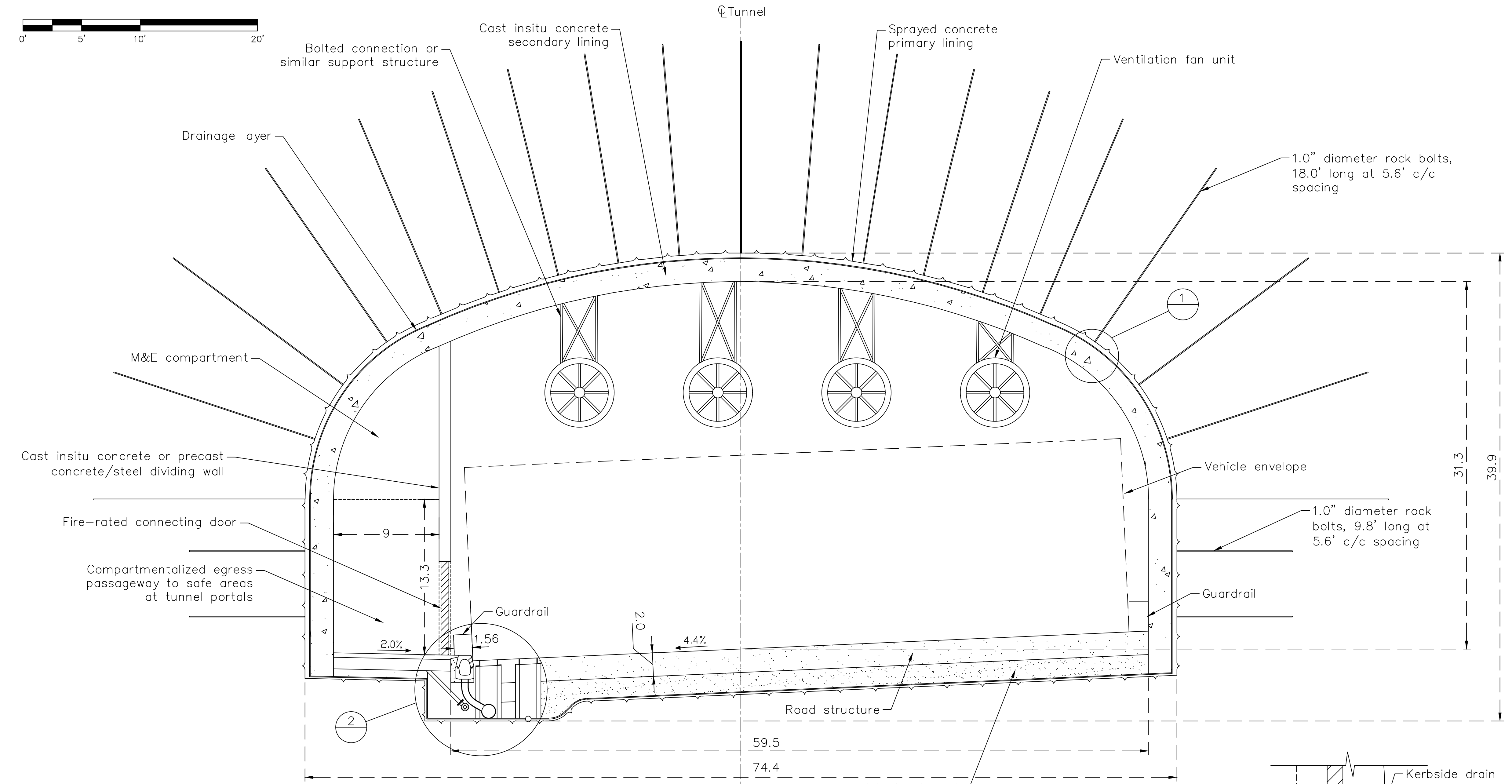
## D.3. Option D



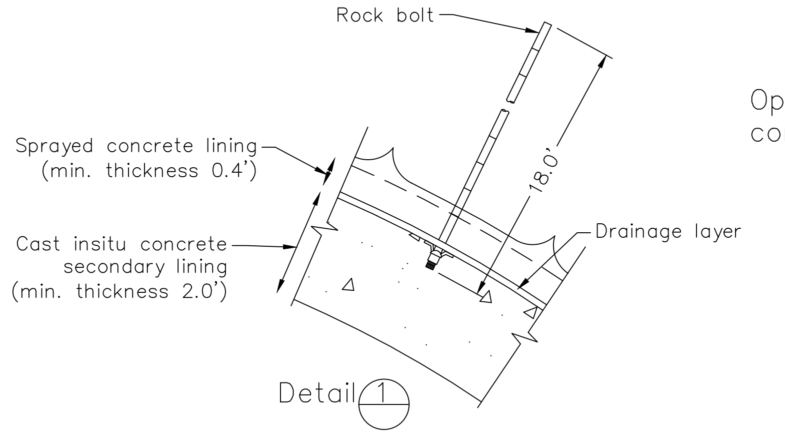
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
Thrust (lbs)
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
FIre Locations (ft)
Summer System delivered thrust (lbs)
Summer In-tunnel target Thrust (lbs)
Winter System delivered thrust (lbs)
Winter In-tunnel target Thrust (lbs)

# Appendix E. Tunnel Configuration Options A-D Drawings

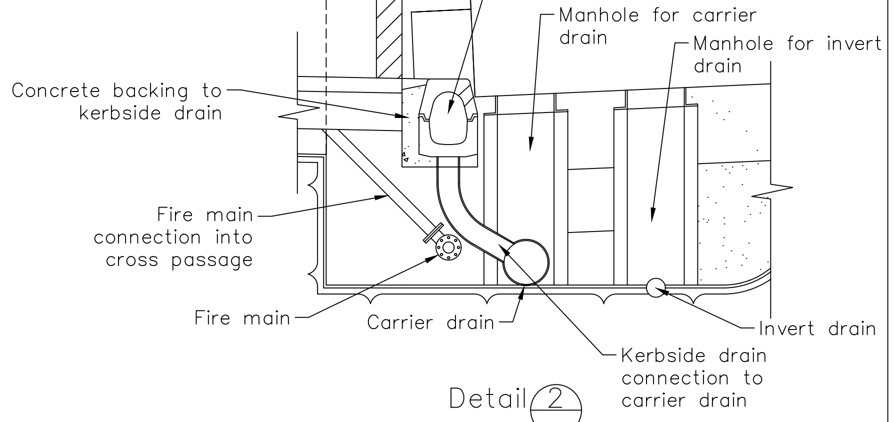
DO NOT SCALE



Option A: Single bore, dedicated egress route provided by a compartmentalized corridor within the main highway tunnel



Detail ①



Detail ②

Notes:

1. All dimensions are in feet unless otherwise shown.
2. The section is taken as Westbound through the tunnel.
3. A crossfall of 4.4% is shown for the main highway tunnel, as this represents the maximum crossfall for the road, and corresponds to the maximum cross-sectional area of the tunnel.
4. A longitudinal spacing of 6ft has been recommended for the rock bolts in the main highway tunnel.
5. This drawing and the details shown (including: road thickness; structural lining thickness; reinforcement arrangements; rock bolt size/spacing; and drainage) are indicative of final requirements and likely to change as the level of design maturity increases.

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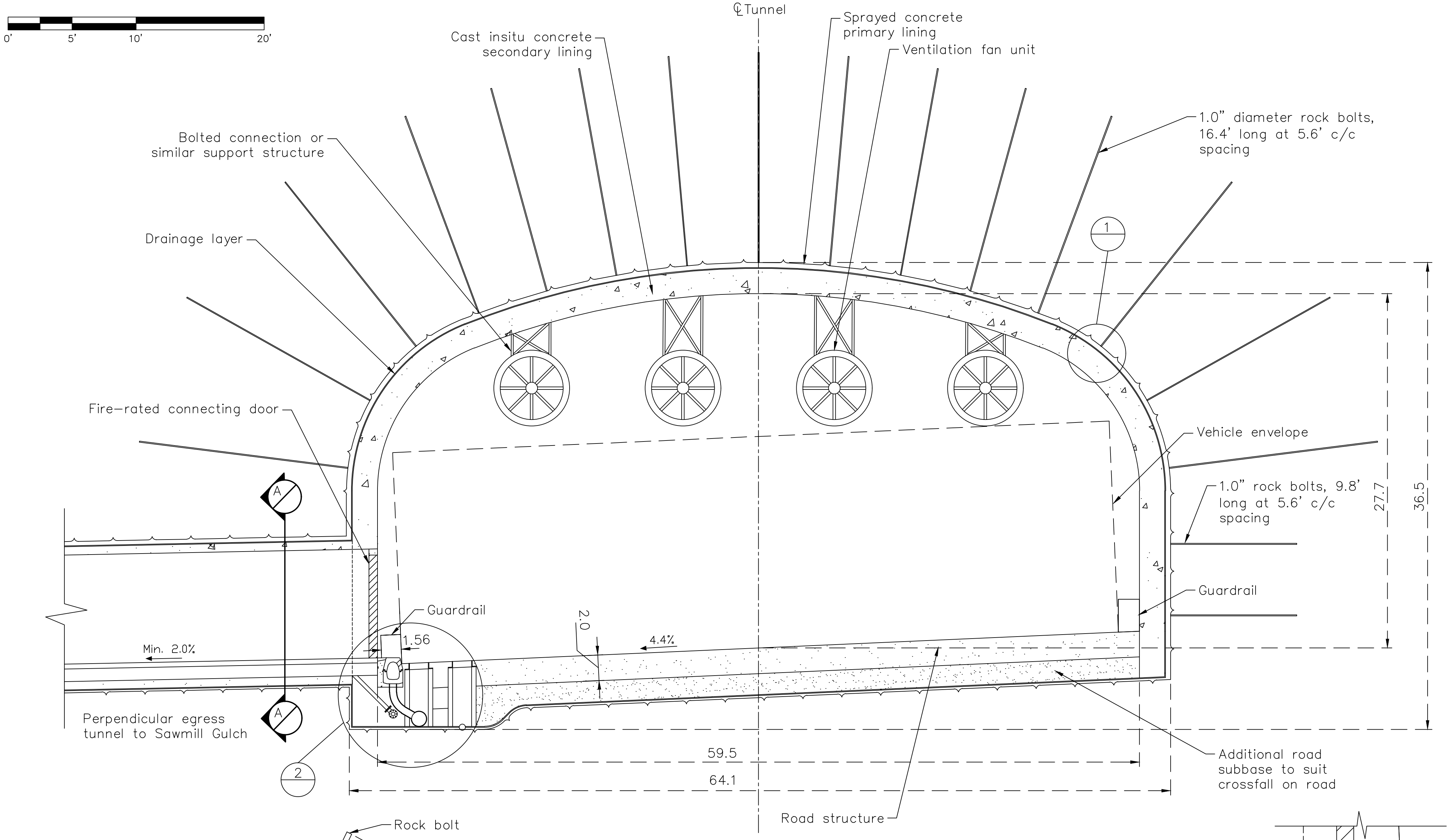
I-70 Floyd Hill Tunnel
Tunnel Configuration Option A

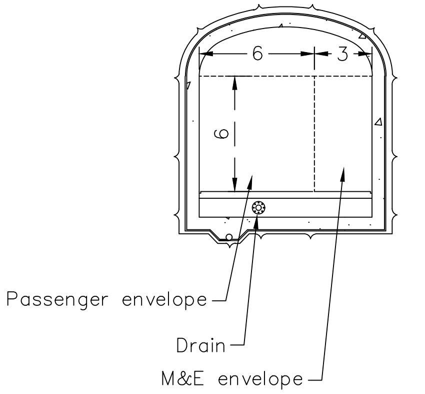
Designer: LM
Detailer: HK
Subset:
Structure Numbers
Subset Sheets: of

Project No./Code
21912
Sheet Number

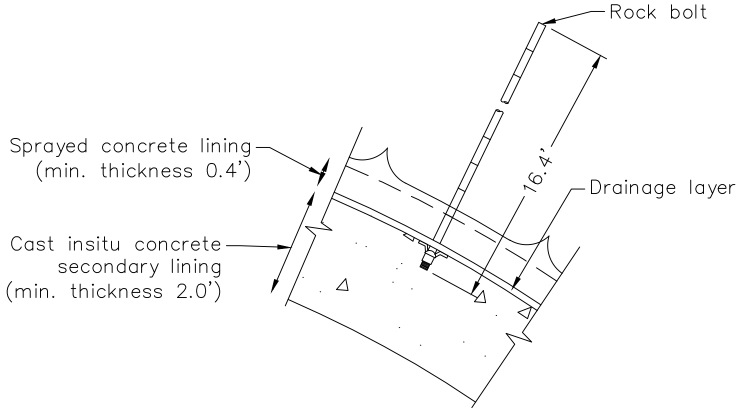
DO NOT SCALE



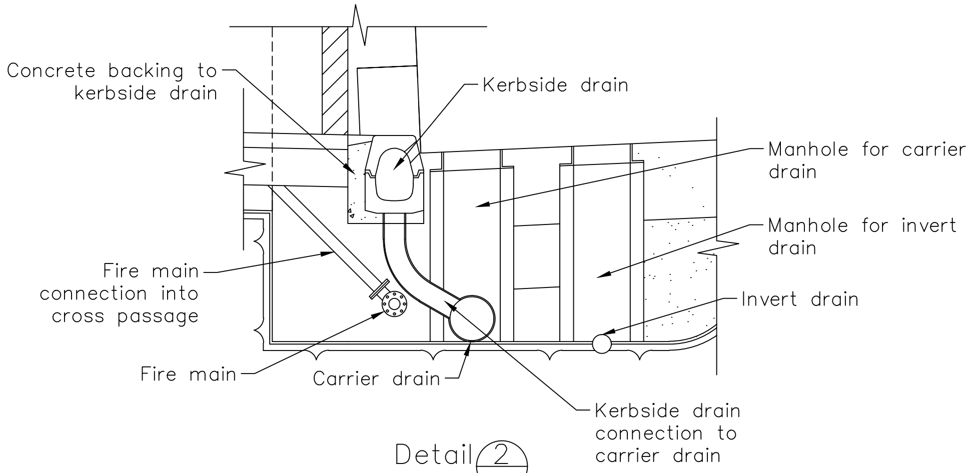
## Section A–A



Option B: Single bore, dedicated egress route provided by tunnels mined perpendicular to the main highway tunnel



Detail 1



Detail 2

Notes:

1. All dimensions are in feet unless otherwise shown.
2. The section is taken as Westbound through the tunnel.
3. A crossfall of 4.4% is shown for the main highway tunnel, as this represents the maximum crossfall for the road, and corresponds to the maximum cross-sectional area of the tunnel.
4. A longitudinal spacing of 6ft has been recommended for the rock bolts in the main highway tunnel and egress tunnels.
5. Three 9.8ft long rock bolts positioned in the crown/shoulders have been recommended for the egress tunnels. Rock bolts for the egress tunnels have been omitted for clarity.
6. This drawing and the details shown (including: road thickness; structural lining thickness; reinforcement arrangements; rock bolt size/spacing; and drainage) are indicative of final requirements and likely to change as the level of design maturity increases.

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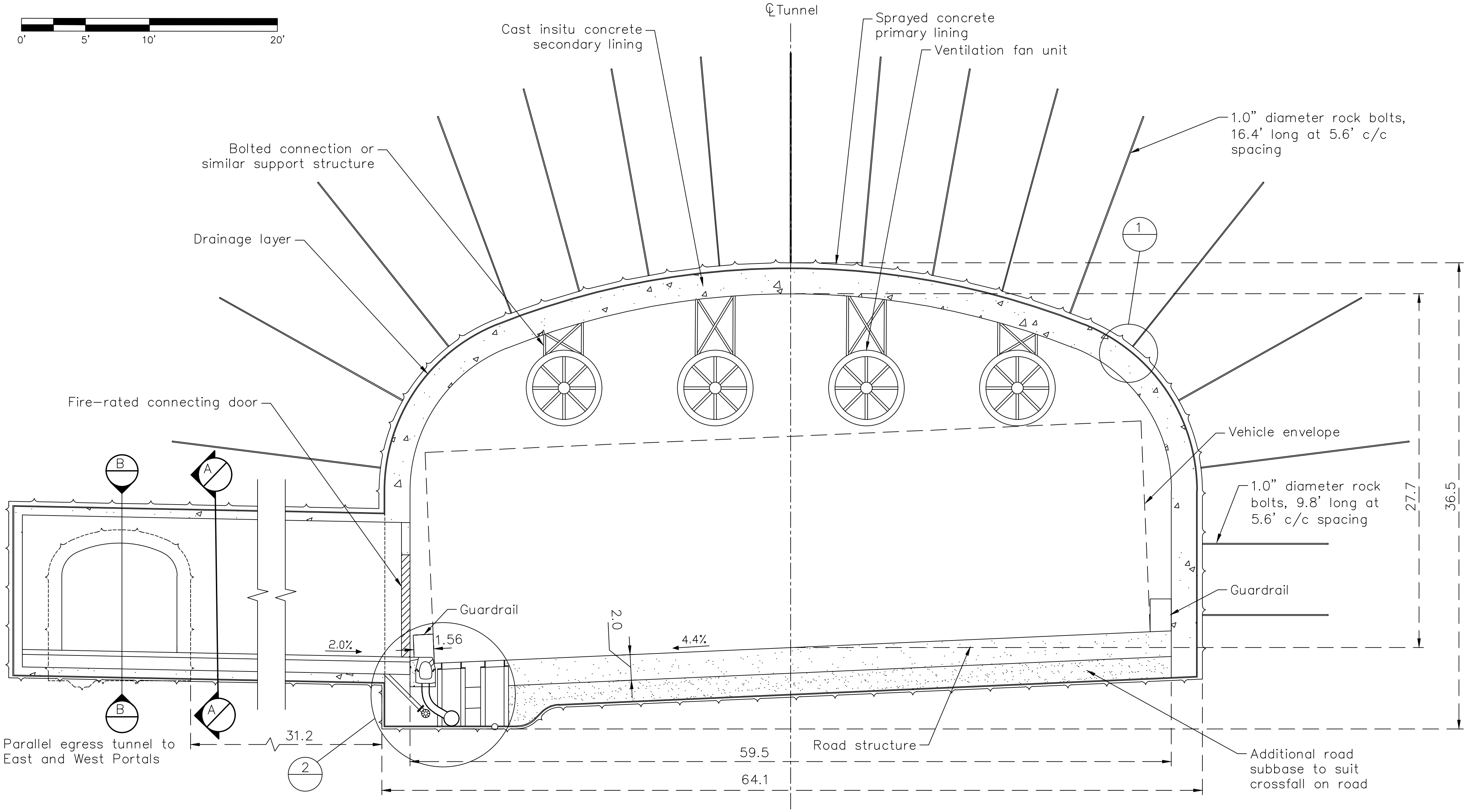
I-70 Floyd Hill Tunnel
Tunnel Configuration Option B

Designer: LM
Detailer: HK
Subset:
Structure Numbers
Subset Sheets: of

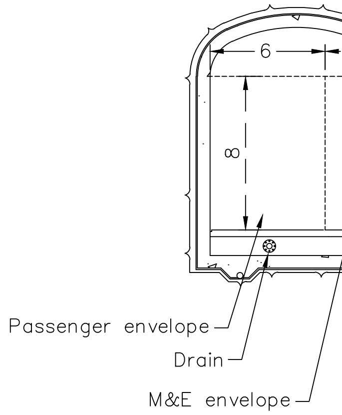
Project No./Code
21912
Sheet Number

DO NOT SCALE

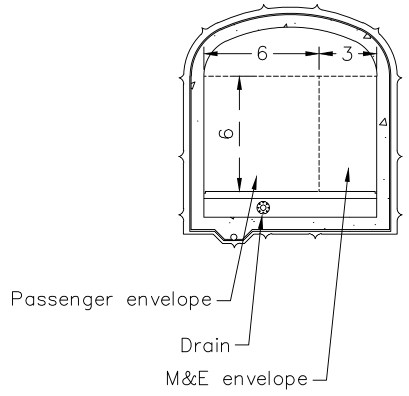


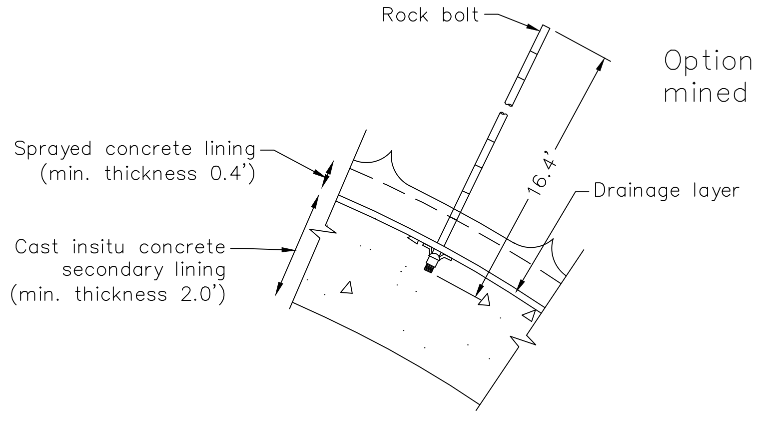
Option C: Single bore, dedicated egress route provided by tunnel mined parallel and on the same level as the main highway tunnel

## Section A–A

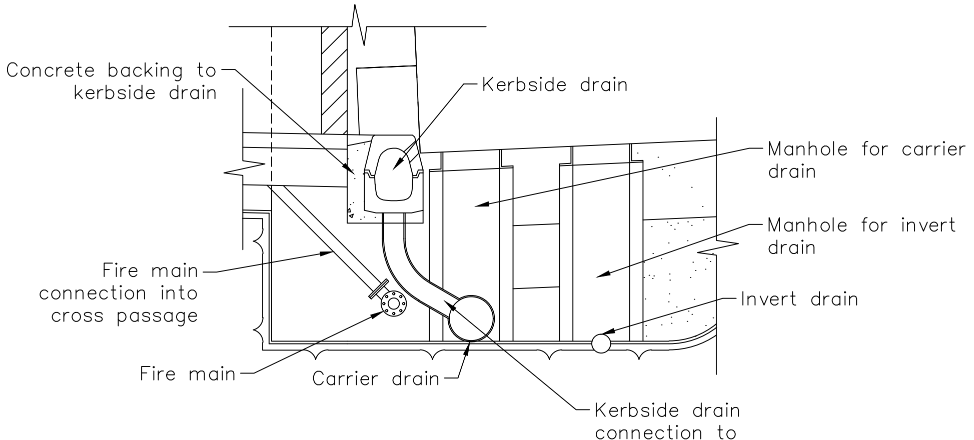


## Section B–B





Detail ①



Detail ②

Notes:
1. All dimensions are in feet unless otherwise shown.
2. The section is taken as Westbound through the tunnel.
3. A crossfall of 4.4% is shown for the main highway tunnel, as this represents the maximum crossfall for the road, and corresponds to the maximum cross-sectional area of the tunnel.
4. A longitudinal spacing of 6ft has been recommended for the rock bolts in the main highway tunnel and egress tunnel.
5. Three 9.8ft long rock bolts positioned in the crown/shoulders have been recommended for the cross passages and egress tunnel. Rock bolts for the cross passages and egress tunnel have been omitted for clarity.
6. This drawing and the details shown (including: road thickness; structural lining thickness; reinforcement arrangements; rock bolt size/spacing; and drainage) are indicative of final requirements and likely to change as the level of design maturity increases.

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I-70 Floyd Hill Tunnel
Tunnel Configuration Option C

Designer: LM
Detailer: HK
Subset:
Structure Numbers
Subset Sheets: of

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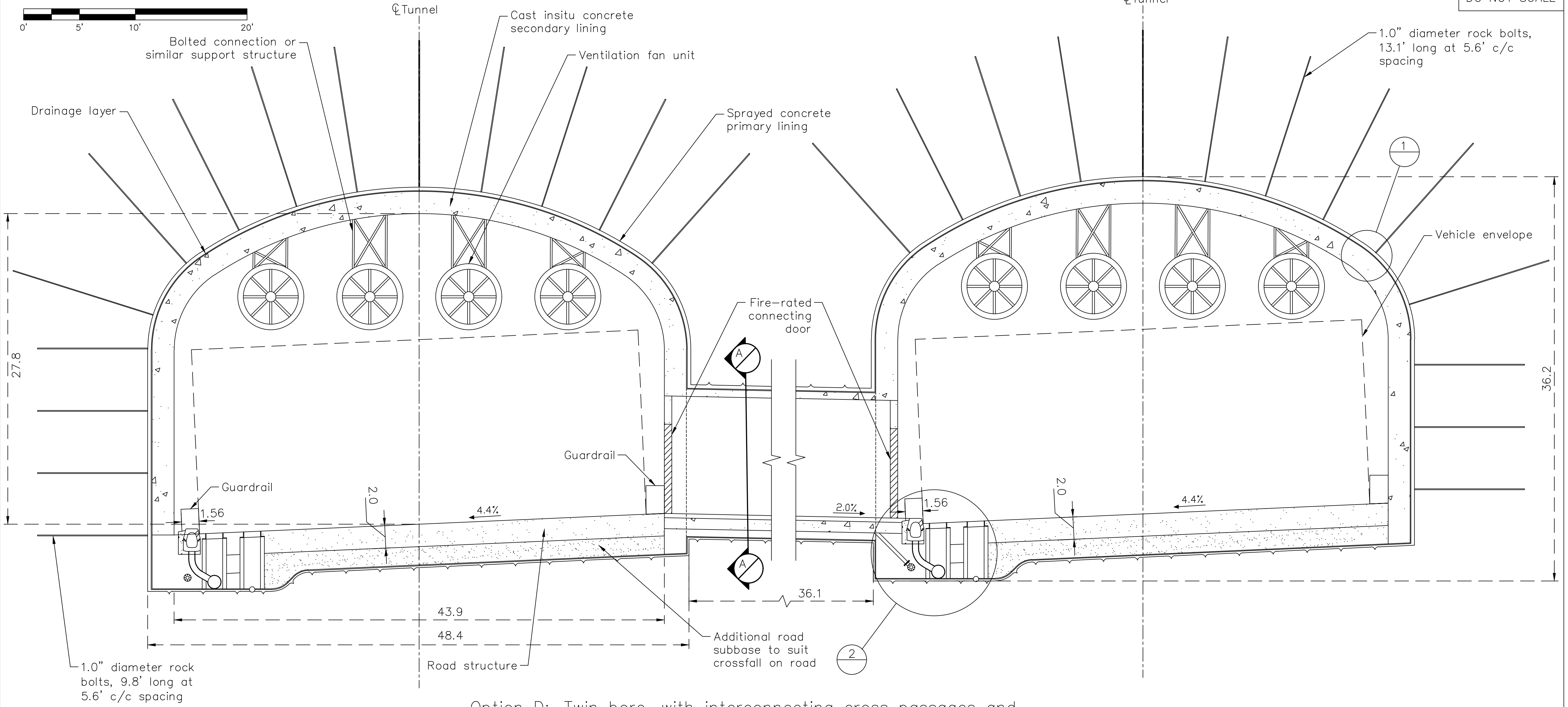
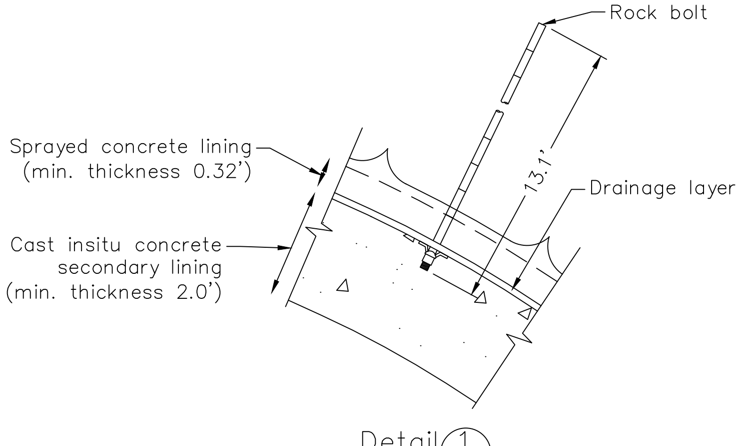
21912
Sheet Number

DO NOT SCALE

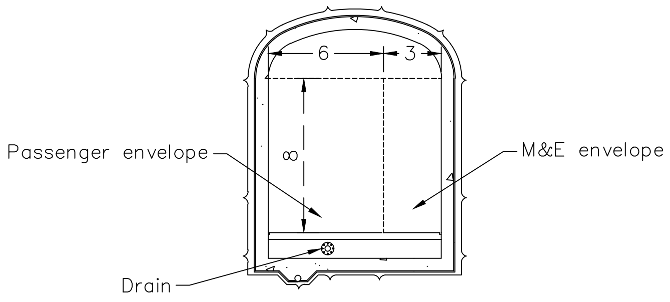


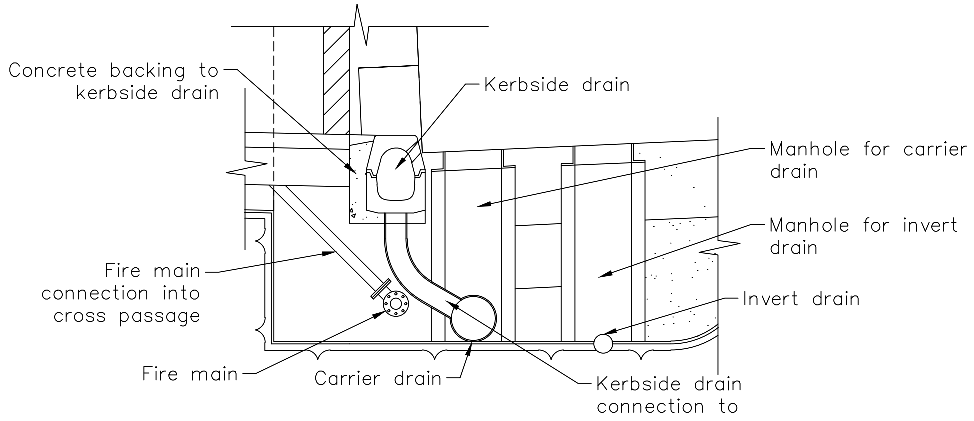
Option D: Twin bore, with interconnecting cross passages and a dedicated egress tunnel located between the two bores



Detail ①

Section A–A





Detail ②

Notes:
1. All dimensions are in feet unless otherwise shown.
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3. A crossfall of 4.4% is shown for the main highway tunnels, as this represents the maximum crossfall for the road, and corresponds to the maximum cross–sectional area of the tunnels.
4. A longitudinal spacing of 6ft has been recommended for the rock bolts in the main highway tunnels.
5. Three 9.8ft long rock bolts positioned in the crown/shoulders have been recommended for the cross passages. Rock bolts for the cross passages have been omitted for clarity.
6. This drawing and the details shown (including: road thickness; structural lining thickness; reinforcement arrangements; rock bolt size/spacing; and drainage) are indicative of final requirements and likely to change as the level of design maturity increases.

Print Date: 18th January 2019
File Name: 21912–AUK–DR–014–P01
Horiz. Scale: 1:10 Vert. Scale:

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Sheet Revisions

| Date: | Comments | Init. |
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Colorado Department of Transportation
425 A Corporate Circle
Golden, CO 80401
Phone: 720–497–6918 FAX: 720–497–6901
Region 1

For Information
No Revisions:
Revised:
Void:

I–70 Floyd Hill Tunnel
Tunnel Configuration Option D

Designer: LM
Detailer: HK
Subset:
Structure Numbers
Subset Sheets: of

Project No./Code
21912
Sheet Number

# Appendix F. MOVES Input Information

## F.1. Navigation Panel Inputs

The following inputs provide a general description of the model and are all input to the Navigation Panel of the MOVES software.

- A 'Project' scale model is used in order to model Floyd Hill Tunnel as an individual link.
- Clear Creek County within Colorado is used as the Geographic bound. Floyd Hill Tunnel falls within this county and the use of which provides significant county specific default data not available if a Custom Domain is created. The default average barometric pressure for this county is 22.756 inHg which is considered adequately close to the barometric pressure for the project specific elevation of approximately 22.7 inHg (based on McElroy Airfield weather station);
- A sensitivity analysis was done to find the worst-case weather condition from the default annual weather data provided by MOVES for Clear Creek County, for $NO_2$ and CO. It is considered at this stage of the study that these will be the dominant pollutants that need to be designed for. This study resulted in a worst-case weather condition for $NO_2$ at 14:00 in July with a corresponding temperature and humidity of 79.1°F and 24.4% respectively and for CO at 06:00 in May with a corresponding temperature and humidity of 33.5°F and 67.7% respectively. These worst-case weather conditions are then used as inputs for the model;
- Only gasoline and diesel fuels are included in the model;
- The road type is specified as Rural Un-restricted Access. Restricted access refers to the road being accessible at predetermined locations such as at on-ramps, with un-restricted accounting for all other roads not included in Restricted;
- No on-road retrofit or rate of progress strategies are implemented in the model.

## F.2. Project Scale Specific Inputs

The following points summarize the detailed data sets that are used as inputs to the model. Any assumptions made are explained here.

- Fuel supply and formulation and meteorological data is taken directly from default MOVES data for Clear Creek County.
- Previous records made available from ANA/CDOT (2015), giving the vehicle age distribution and source type distribution of vehicles for Denver have been used. Considering that the majority of vehicles will be originating from the city of Denver, it is reasonable to assume that a similar distribution is applicable for Floyd Hill Tunnel and any such error induced is acceptable context of assessing the need for a conceptual ventilation system and its capacity estimation at this stage.
- Default fuel type and engine technology data are used.

### F.2.1. Link Definition

The link length is considered as 670m (2200ft) with a constant gradient of 1.3% positive to the flow direction (uphill). The worst-case traffic assumption is that all 3 lanes are at peak density for each of the speeds considered from 0mph to 55mph (tunnel speed limit). The link traffic definition data are redefined for each simulation for the range of average vehicle speeds. The vehicle volume is determined by interpolation of this worst-case road capacity traffic defined by PIARC (2012), shown in Table F.1, the results of which can be found in the link definition Table F.2.

Table F.1 - PIARC Average Peak Traffic Densities for a Rural Uni-Directional Tunnel

| Traffic Description | Traffic Speed (km/h) | Traffic Density/Lane | |
|---|---|---|---|
| | | Passenger car units per km (pcu/km) | Passenger car units per hour (pcu/h) |
| Fluid traffic | 60 | 30 | 1800 |
| Congested traffic | 10 | 70 | 700 |
| Standstill | 0 | 150 | - |

PIARC accepts that for converting passenger car units into number of vehicles, a truck/bus may be assumed to occupy the space of 2 passenger cars in free-flowing traffic and up to 3 passenger cars in slow moving traffic.

Table F.2 - Link Traffic Volume and Speeds

| Link Average Speed (mph) | Truck/Bus pcu Definition (PIARC) | Lane Traffic Volume (veh/h/lane) | Link Traffic Volume (veh/h) |
|---|---|---|---|
| 0 | 3 | 92* | 276* |
| 2.5 | 3 | 434 | 1301 |
| 5 | 3 | 632 | 1897 |
| 10 | 3 | 957 | 2872 |
| 20 | 2 | 1536 | 4609 |
| 30 | 2 | 1736 | 5208 |
| 40 | 2 | 1764 | 5293 |
| 50 | 2 | 2205 | 6616 |
| 55 | 2 | 2426 | 7278 |

** At 0mph Lane and Link traffic volume is a total for the given road length*

## F.2.2. Link Source Types

The Link Source Types defines the vehicle type split on the modelled stretch of road. Previous records made available from ANA/CDOT (2015), giving source type distribution of vehicles for Denver, have been used. This is given below in Table F.3.

Table F.3 - Vehicle Type Proportions

| Vehicle Type | Proportion |
|---|---|
| Motorcycle | 3.45% |
| Passenger car | 48.75% |
| Passenger truck | 31.93% |
| Light commercial truck | 10.29% |
| Intercity bus | 0.39% |
| Transit bus | 0.26% |
| School bus | 1.68% |
| Refuse truck | 0.04% |
| Single unit short haul truck | 1.60% |
| Single unit long haul truck | 0.10% |
| Motor home | 0.16% |
| Combination short haul truck | 0.77% |
| Combination long haul truck | 0.59% |

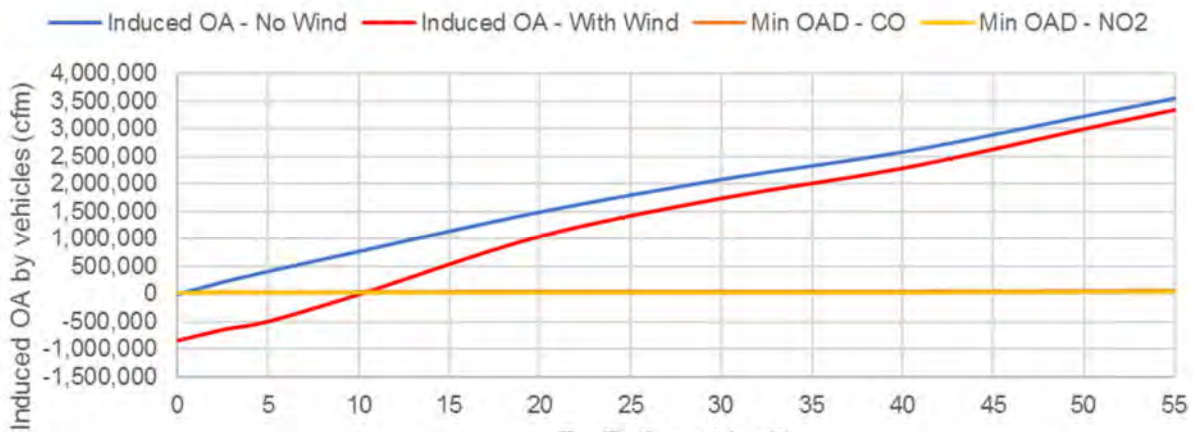
## F.2.3. Vehicle Operating Modes

No link drive schedule or operating mode distribution has been specified and therefore MOVES uses the average speed specified and road type to automatically define vehicle operating modes. Where the average vehicle speed is lower than the default data range the nearest speed is used (no extrapolation occurs, only interpolation). This is considered a reasonable approximation for this level of modelling however more appropriate vehicle operating modes for congested traffic could be specified manually if the required data are available.

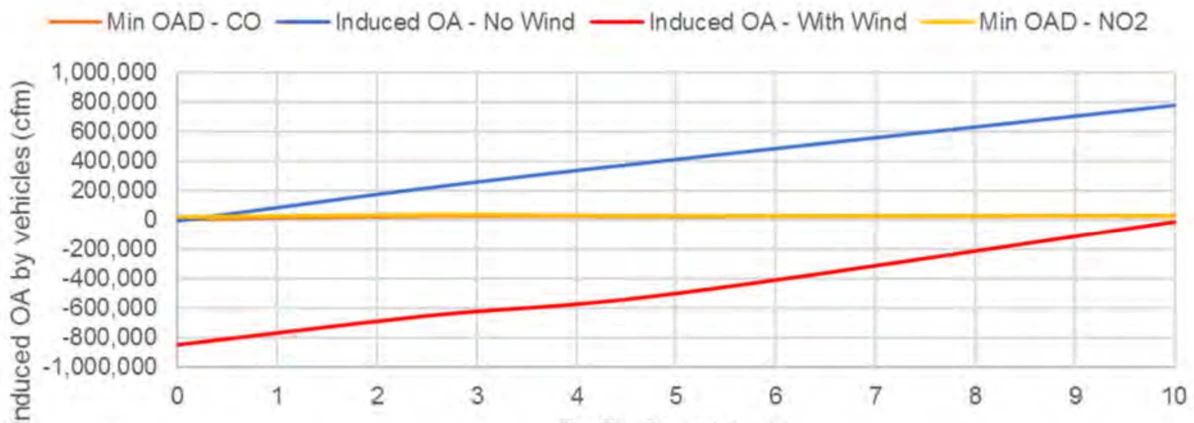
# Appendix G. Pollution Control Results

## G.1. Option A

### G.1.1. Free-Flowing Traffic

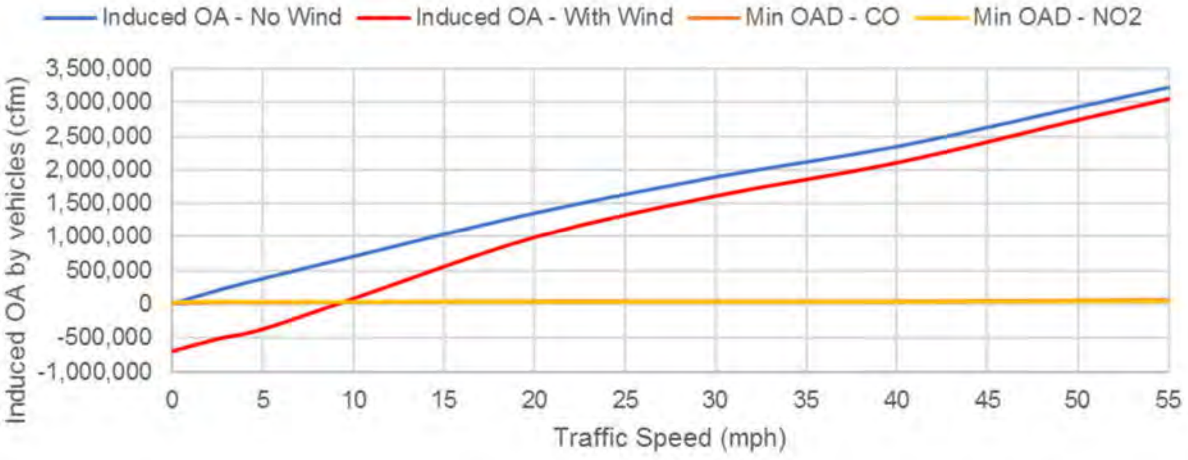


### G.1.2. Standstill and Slow-Moving Traffic

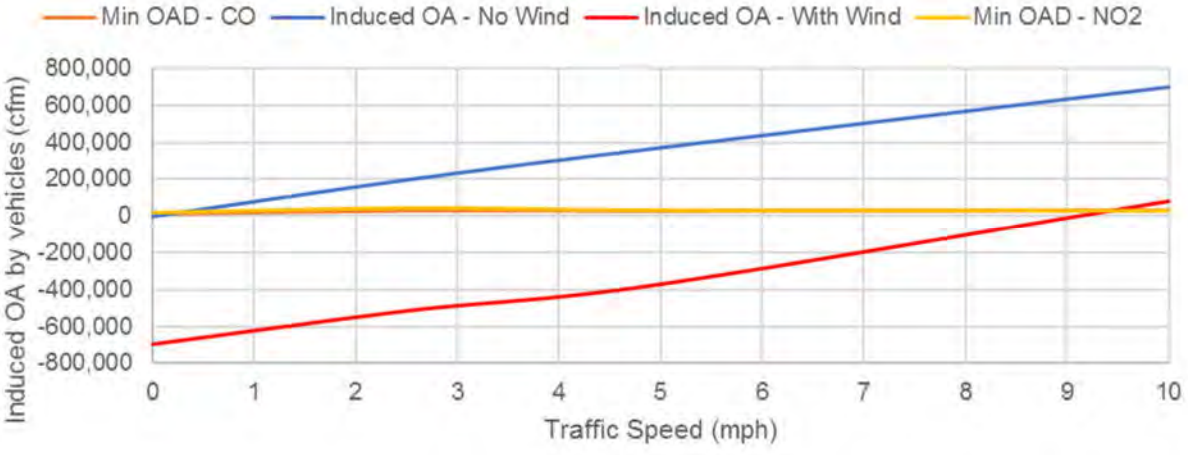


## G.2. Options B and C

### G.2.1. Free-Flowing Traffic

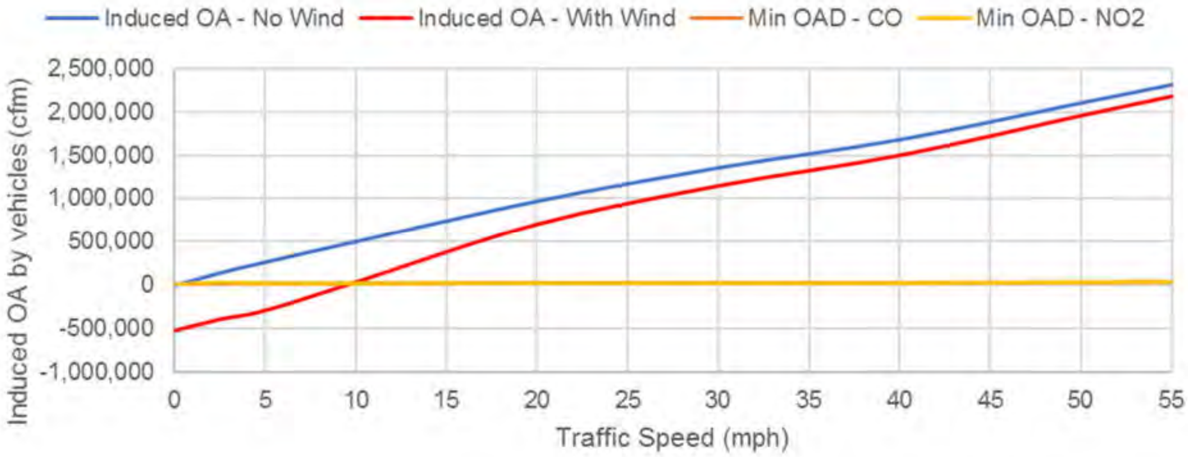


### G.2.2. Standstill and Slow-Moving Traffic

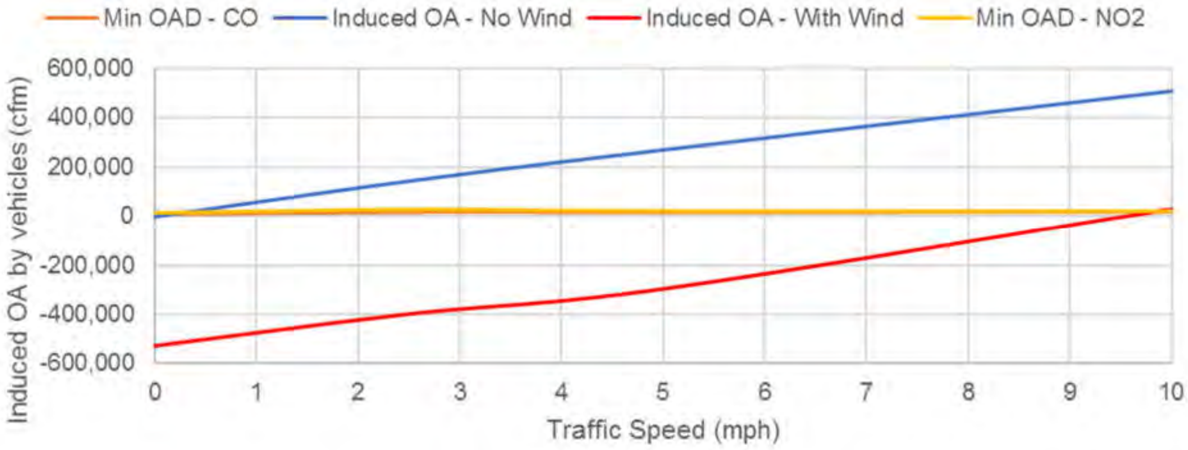


# G.3. Option D

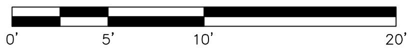
## G.3.1. Free-Flowing Traffic

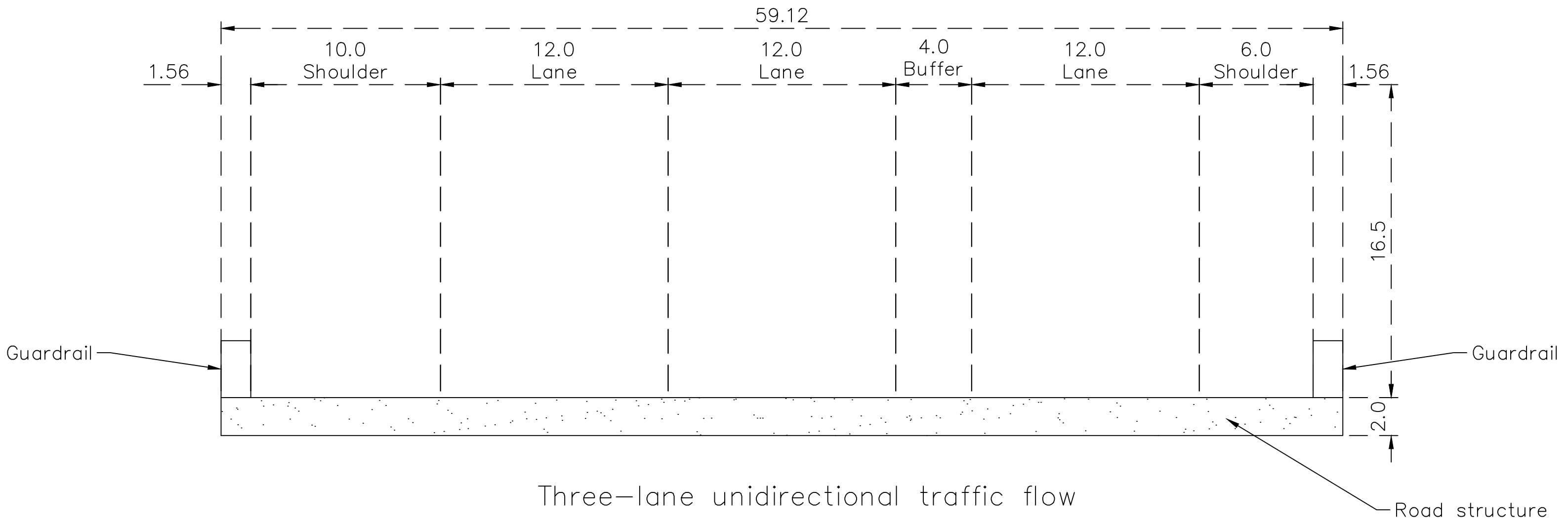


## G.3.2. Standstill and Slow-Moving Traffic

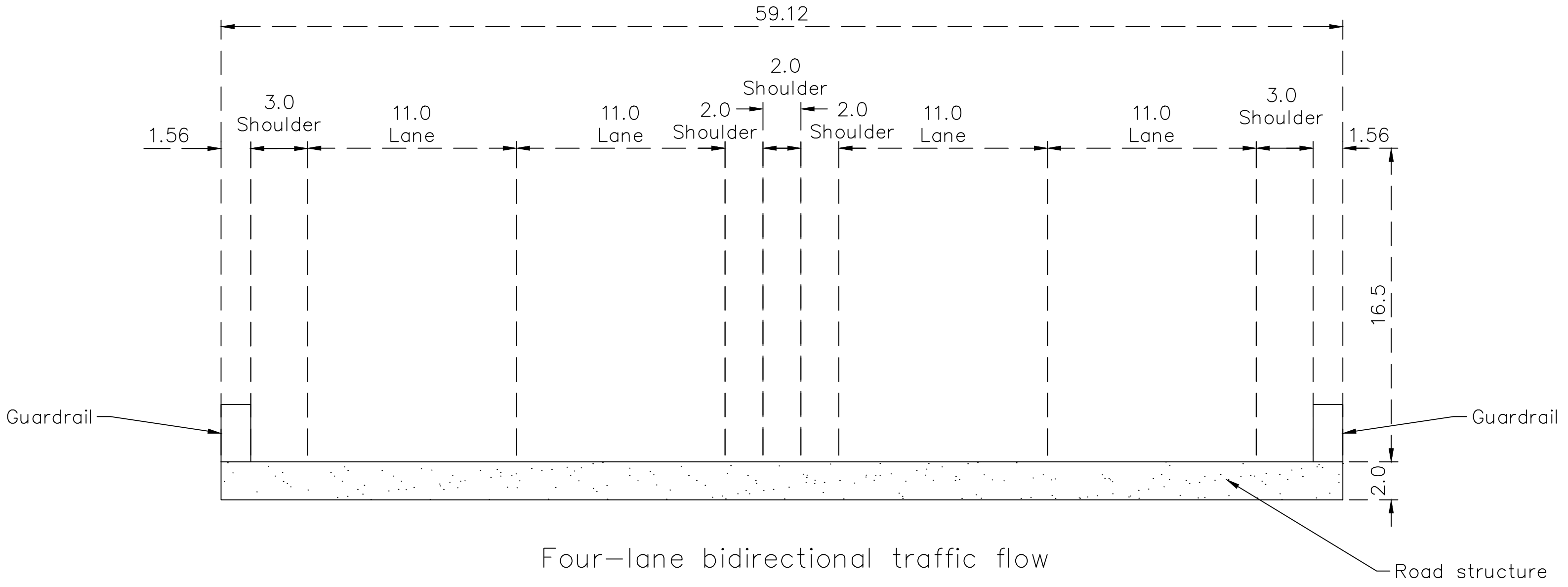


# Appendix H. Highway Lane Configurations Drawing

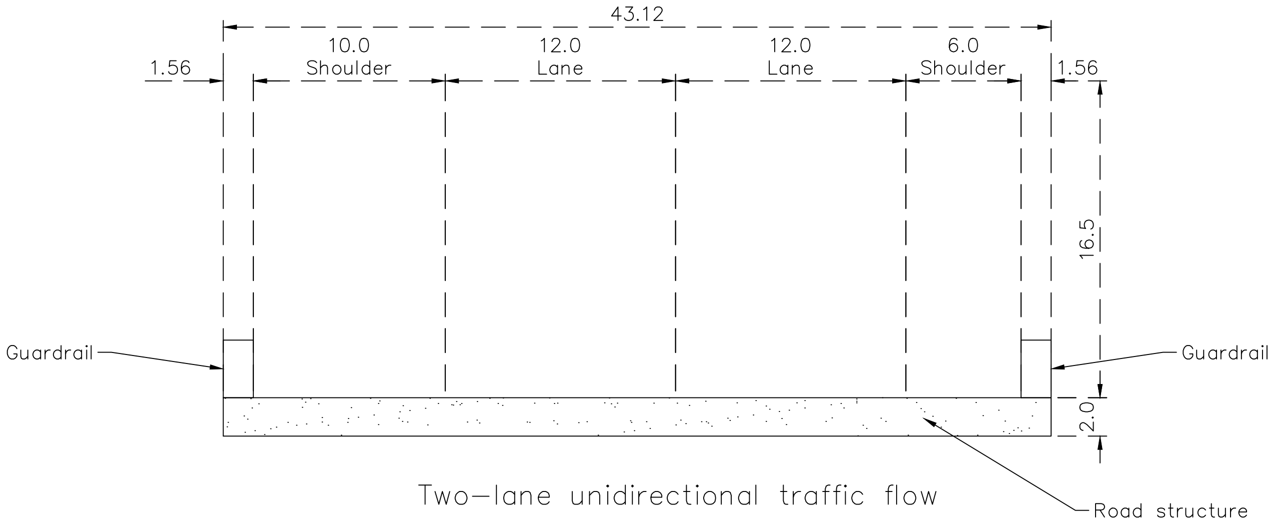




Three-lane unidirectional traffic flow



Four-lane bidirectional traffic flow



Two-lane unidirectional traffic flow

Notes:

1. All dimensions are in feet unless otherwise shown.
2. The section is taken as Westbound through the tunnel.
3. Road thickness is indicative and subject to change.
4. Three–lane unidirectional traffic flow represents the normal operational condition for Options A, B and C (Options 1–6), with all traffic heading Westbound.
5. Four–lane bidirectional traffic flow represents the special operating condition for Options A, B and C (Options 1–6), whereby there is closure of the Eastbound traffic lanes due to the Eastbound being under construction/ maintenance or in the event of an emergency (Eastbound traffic flow must stop).
6. Two–lane unidirectional traffic flow represents the normal operating conditions for Option D (Options 7–9), whereby both bores can be utilized to provide a total of four Westbound traffic lanes or provide two lanes in each direction if the surface Eastbound highway is closed.

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For Information
No Revisions:
Revised:
Void:

## I70 Floyd Hill Tunnel Highway Lane Configurations and Traffic Envelopes

Designer: LM
Detailer: HK
Subset:
Structure Numbers
Subset Sheets: of

Project No./Code
21912
Sheet Number

# Appendix I. Portal Models

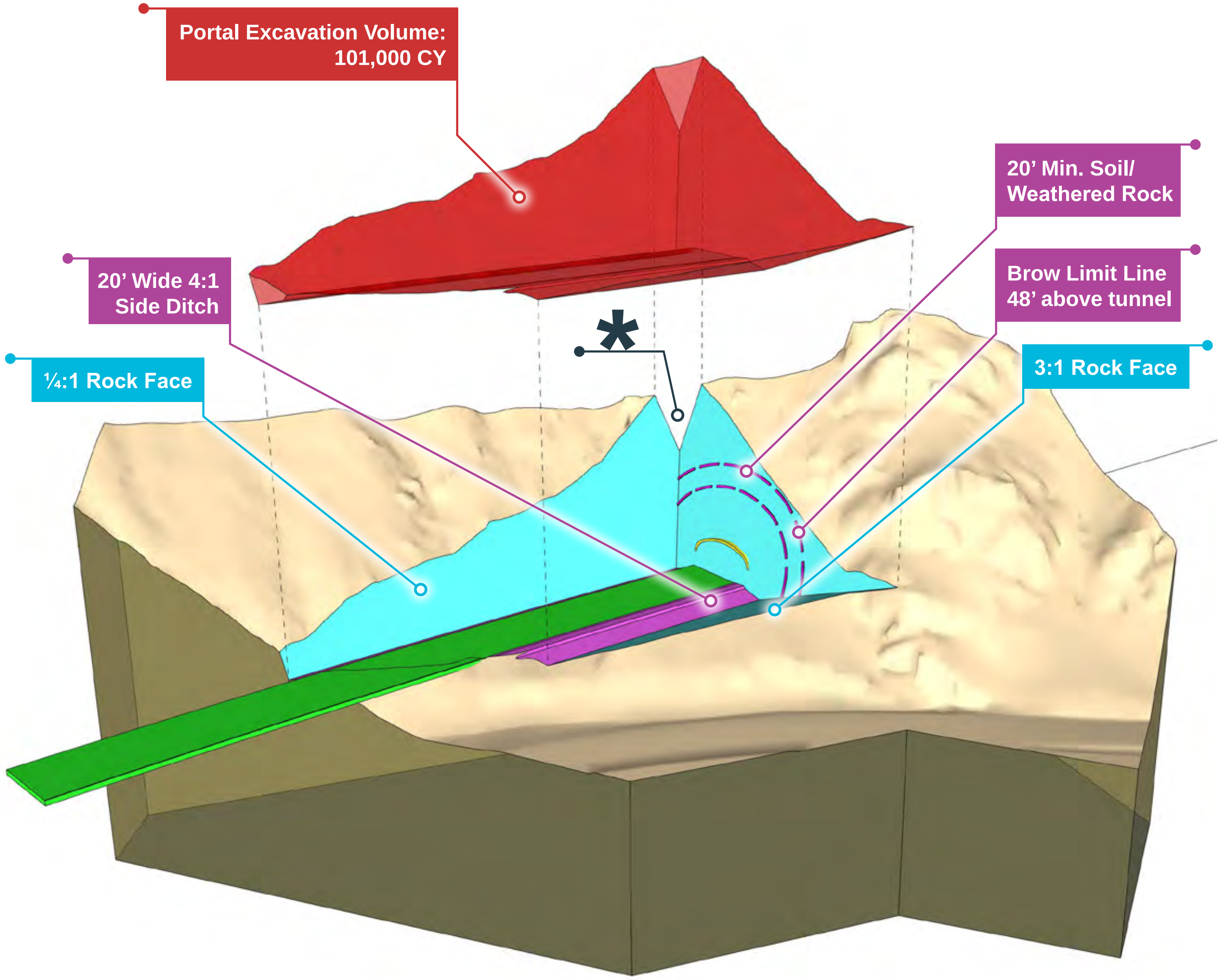
# Tunnel Layout 'A'

¼:1 Headwalls (at tunnel portals)

Total Tunnel Length: 2,260LF

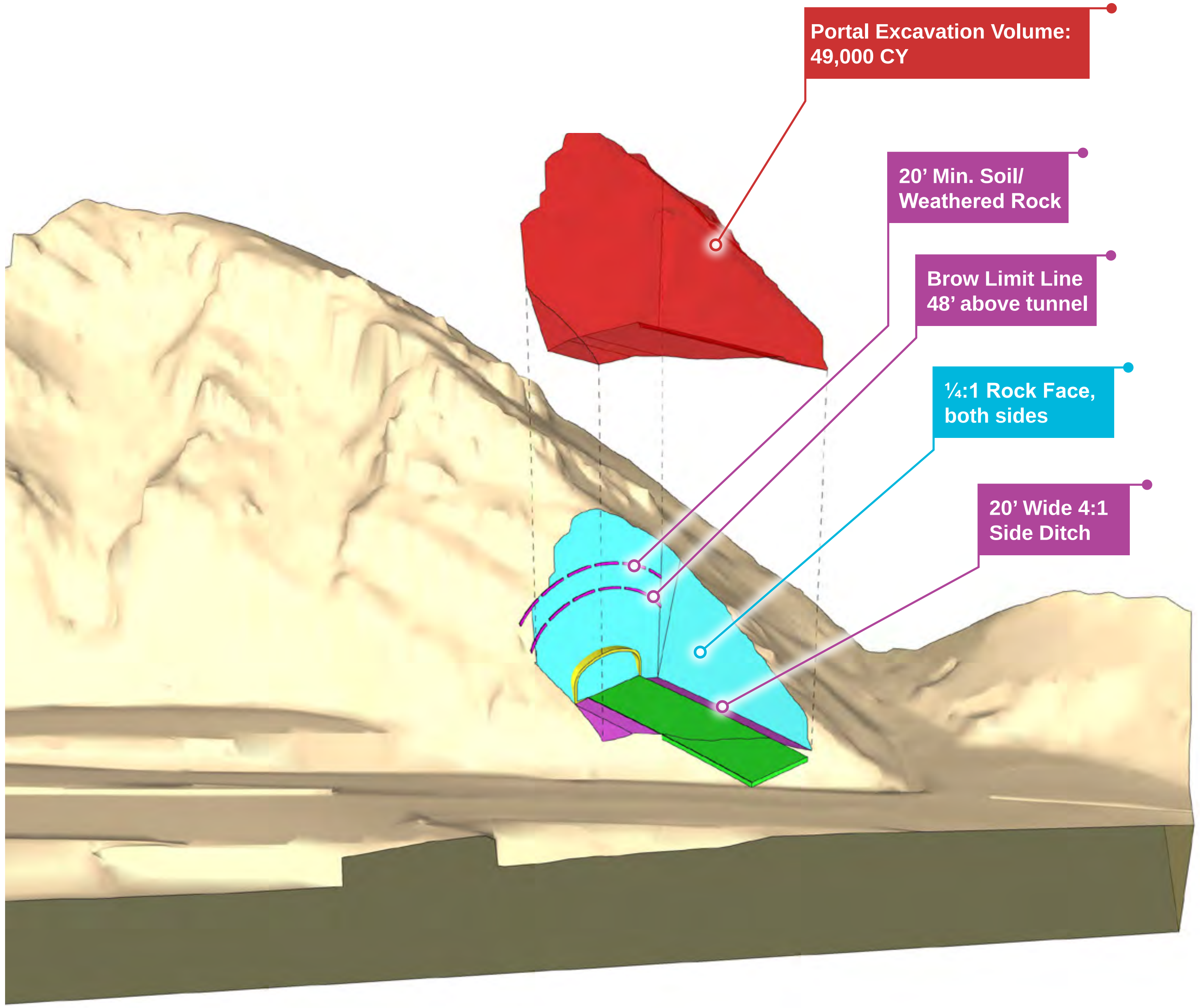
## West Portal

101,000 CY



## East Portal

49,000 CY



* Missing 'triangle' due to extents of existing terrain data.

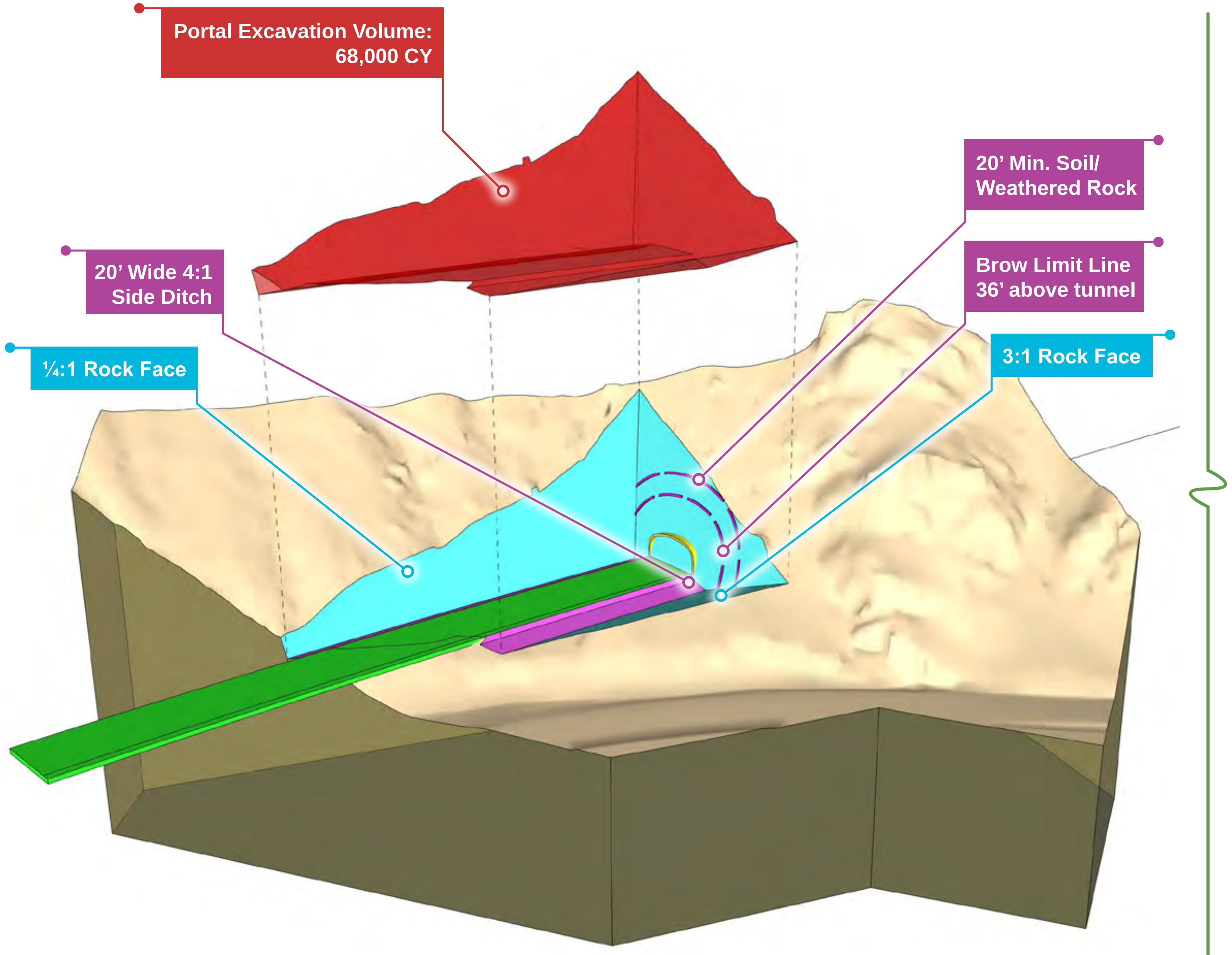
# Tunnel Layout ‘B’

¼:1 Headwalls (at tunnel portals)

Total Tunnel Length: 2,340LF

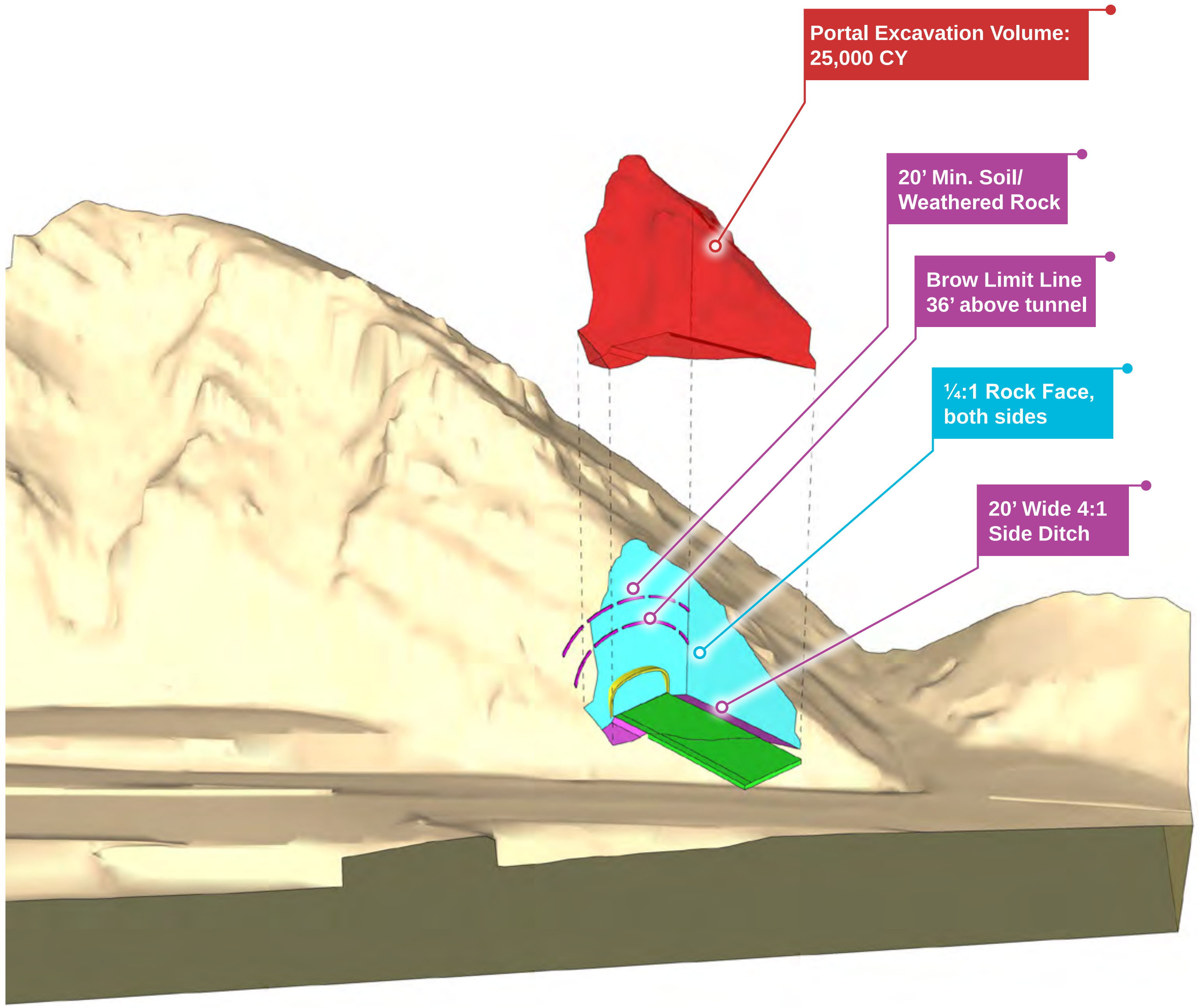
## West Portal

68,000 CY



## East Portal

25,000 CY



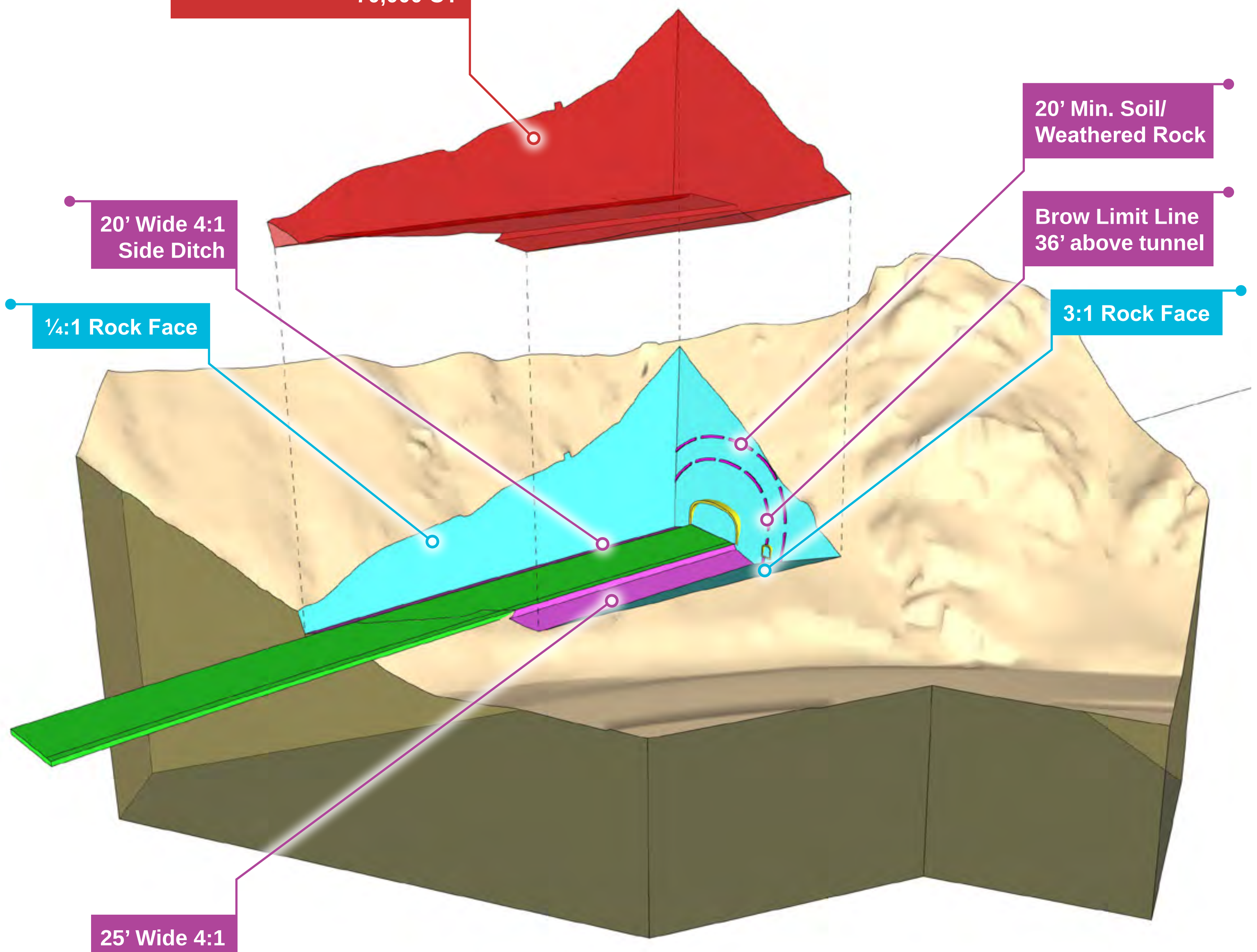
# Tunnel Layout ‘C’

¼:1 Headwalls (at tunnel portals)

Total Tunnel Length: 2,285LF

## West Portal

70,000 CY



Portal Excavation Volume:
70,000 CY

20’ Wide 4:1
Side Ditch

¼:1 Rock Face

20’ Min. Soil/
Weathered Rock

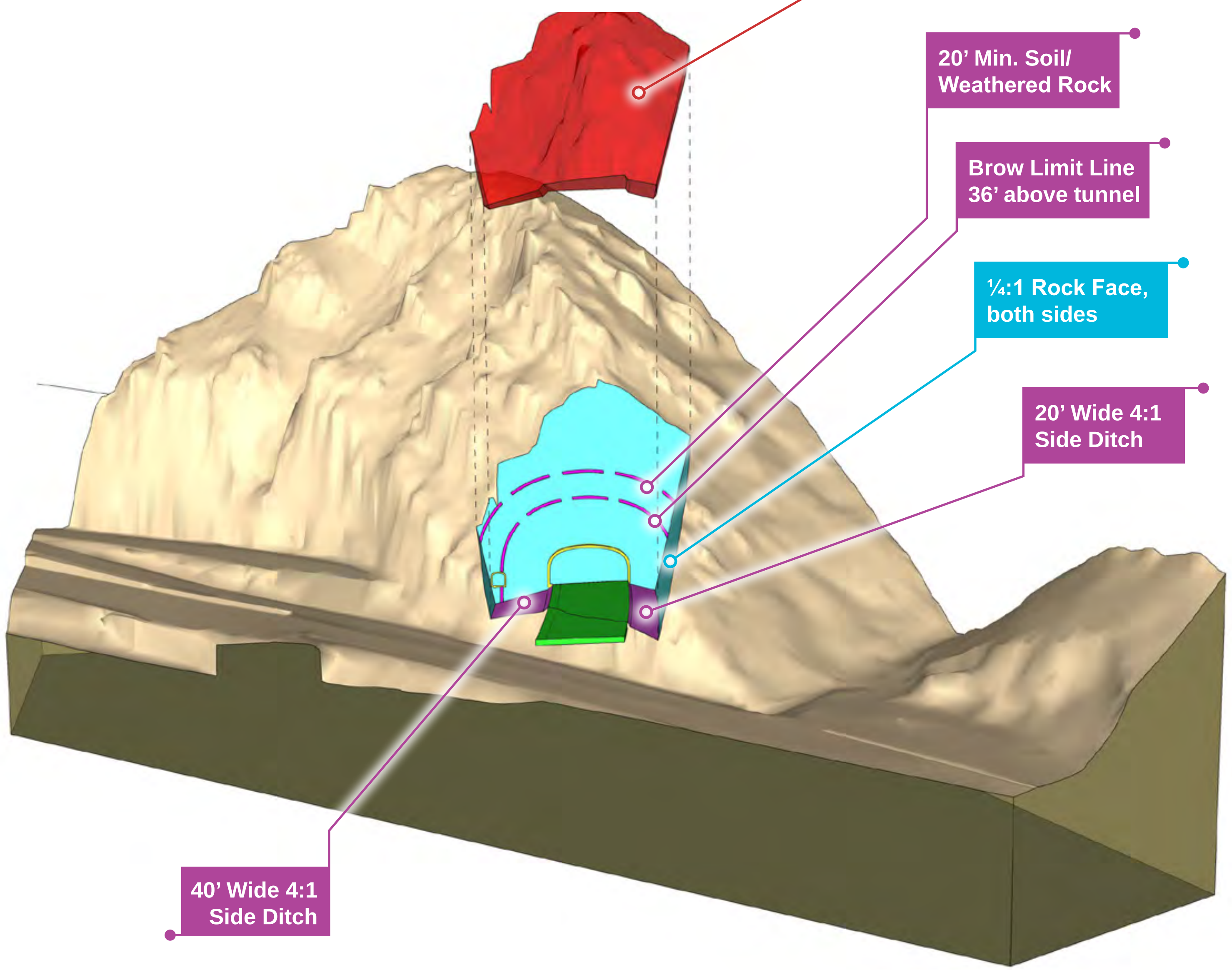
Brow Limit Line
36’ above tunnel

3:1 Rock Face

25’ Wide 4:1
Side Ditch

## East Portal

54,000 CY



Portal Excavation Volume:
54,000 CY

20’ Min. Soil/
Weathered Rock

Brow Limit Line
36’ above tunnel

¼:1 Rock Face,
both sides

20’ Wide 4:1
Side Ditch

40’ Wide 4:1
Side Ditch

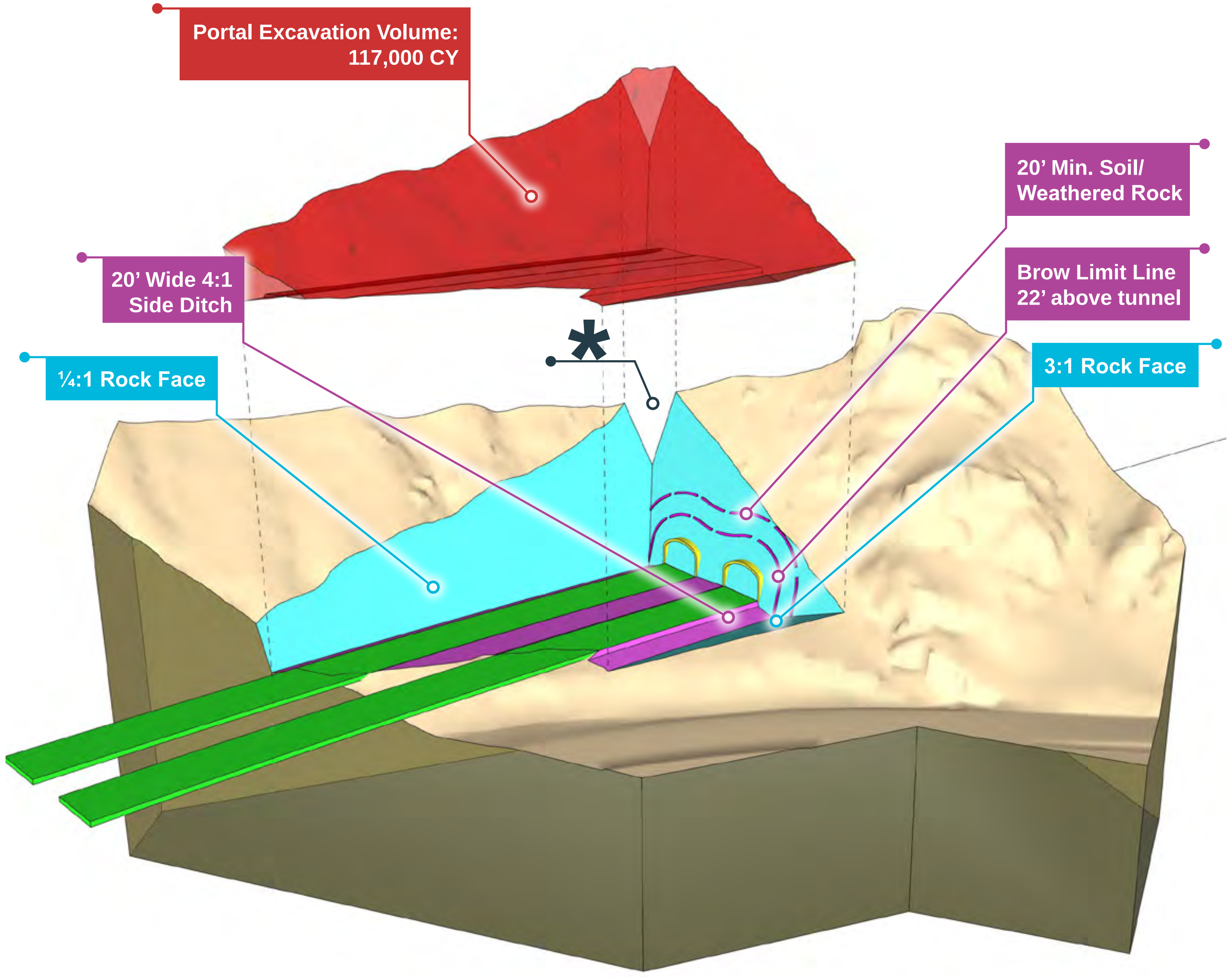
# Tunnel Layout 'D'

¼:1 Headwalls (at tunnel portals)

Total Tunnel Length: 2,260LF

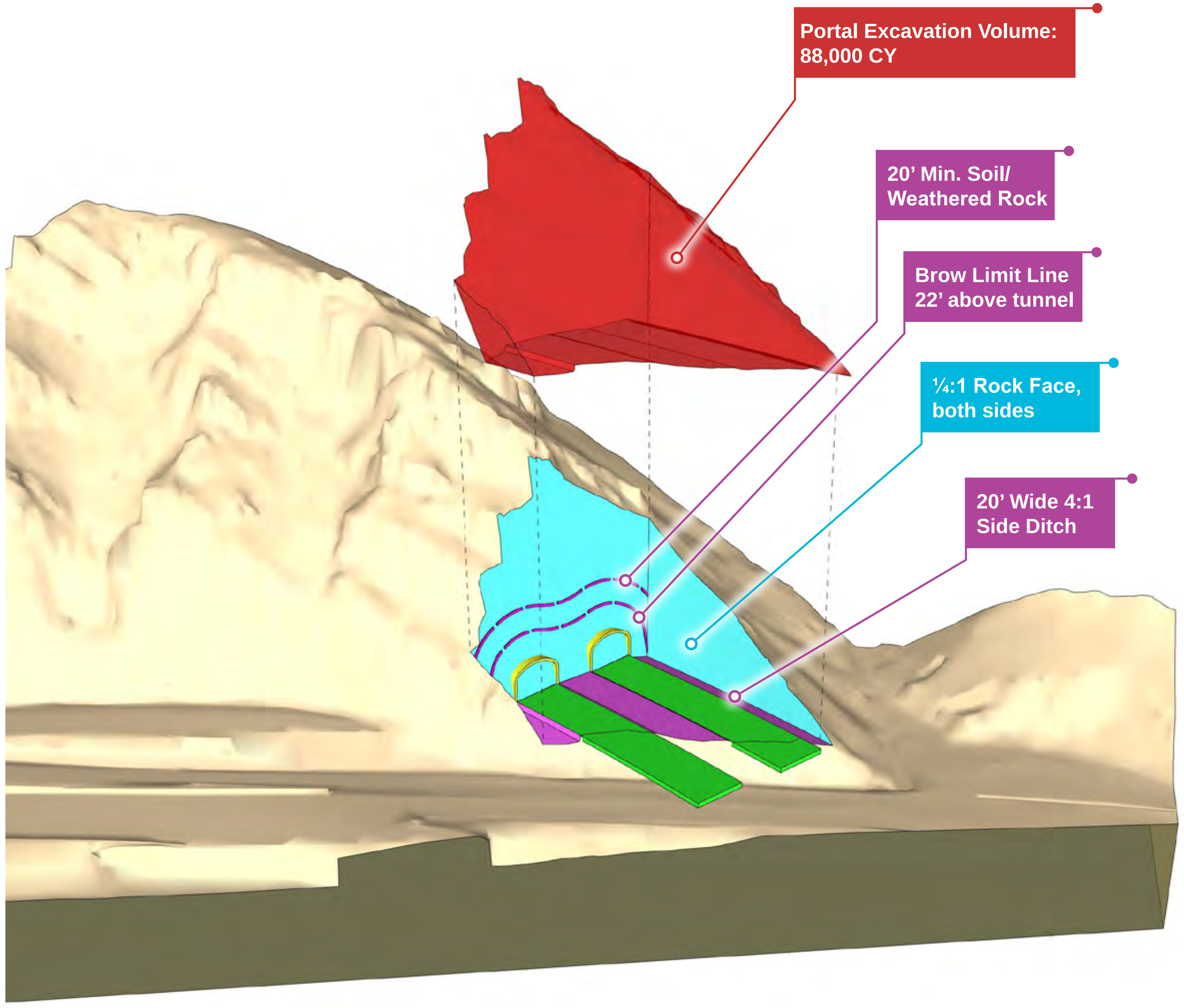
## West Portal

117,000 CY



## East Portal

88,000 CY



* Missing 'triangle' due to extents of existing terrain data.

# Appendix J. Capital Cost Estimates

## Tunnel Configuration Option A - Cost Estimate

| Tunnel Configuration Option A | | | | |
|---|---|---|---|---|
| Water Treatment Process | 6,000 | MGAL | $54.54 | $327,231.28 |
| Rock Exc CL C | 248,479 | CY | $251.33 | $62,449,427.68 |
| Main Rock Bolt 1.0" x 18' | 128,820 | LF | $37.55 | $4,836,743.88 |
| Side Wall Rock Bolt 1.0" x 9.8' | 14,765 | LF | $53.74 | $793,452.18 |
| Optical Survey Target | 1 | LS | $149,856.13 | $149,856.13 |
| Roadbed Subbase Special | 17,928 | TN | $22.61 | $405,277.82 |
| Conc Pavement 24" | 14,940 | SY | $167.68 | $2,505,157.63 |
| MC 12 x 50 Steel Sets (Contingency) | 7,530 | LF | $143.44 | $1,080,091.43 |
| Steel Sets Install (Contingency) | 38 | EA | $14,613.67 | $555,319.54 |
| Conc Footing (Assumed) | 750 | CY | $504.40 | $378,301.06 |
| Conc Lining Secondary (Less Portal) | 19,814 | CY | $792.40 | $15,700,669.63 |
| Fireboard | 348,492 | SF | $22.90 | $7,980,888.91 |
| Re-Steel Footing Epoxy | 150,000 | LB | $1.66 | $248,292.19 |
| Re-Steel Lining Blk | 3,403,177 | LB | $3.29 | $11,184,274.65 |
| Re-Steel Lining Epoxy | 3,403,177 | LB | $3.77 | $12,842,131.71 |
| Geocomp Drain Tunnel | 426,462 | SF | $1.23 | $523,544.45 |
| Waterproof Membrane | 426,462 | SF | $1.89 | $804,189.08 |
| Carrier/Invert Drain | 2,260 | LF | $505.64 | $1,142,737.82 |
| G Rail Type 7 CD Special | 4,520 | LF | $163.11 | $737,275.46 |
| Compartment Wall W/Doors Walkways | 2,260 | LF | $390.78 | $883,155.32 |
| Const Surveying Tunnel | 1 | LS | $541,579.97 | $541,579.97 |
| Vibration Monitoring | 1 | LS | $319,703.53 | $319,703.53 |
| Shotcrete Overbreak | 5,136 | CY | $780.69 | $4,009,615.55 |
| Shotcrete Steel fiber Reinforced | 5,136 | CY | $844.54 | $4,337,544.49 |
| Fixed Fire Fighting System | 1 | LS | $1,092,937.14 | $1,092,937.14 |
| Hydrants and Standpipes | 1 | LS | $458,024.23 | $458,024.23 |
| Electrical Distribution | 1 | LS | $1,148,975.30 | $1,148,975.30 |
| Tunnel Light System | 1 | LS | $1,156,152.32 | $1,156,152.32 |
| Fire Alarm and Detection | 1 | LS | $250,543.17 | $250,543.17 |
| Tunnel Closure System and Signage | 1 | LS | $685,078.97 | $685,078.97 |
| Tunnel Portable Fire Extinguisher | 1 | LS | $87,429.13 | $87,429.13 |
| Tunnel Ventilation | 1 | LS | $1,903,377.73 | $1,903,377.73 |
| Tunnel Comand Center | 1 | LS | $5,872,105.45 | $5,872,105.45 |
| Tunnel SCADA | 1 | LS | $1,587,621.57 | $1,587,621.57 |
| Access Platform/Ramp | 1 | LS | $1,362,028.57 | $1,362,028.57 |
| Tunnel Portal | 2 | EA | $13,469,125.03 | $26,938,250.06 |
| | | | **Bid Total:** | **$177,278,985.03** |

| | |
|---|---|
| Tunnel Total Length | 2260 LF |
| Unit Cost $/LF | $78,442 |

QC Checklist:

Estimate Description: _Tunnel Option A REV3___________________________________________________

Version (number and or date): _REV 3 02/04/19______

Quantities received from the design team (date): _02/04/19_and 02/06/19_____

Estimate completed by (initials and date): __02/07/19______(Dean)_____DL_________________

Estimate checked by (initials and date): ______(Ben)_____________________ BW 2/4/19

Estimate reviewed by (initials and date): ______(Jeff)____________________ JW/2/7/19

Changes made and verified (initials and date): _____(Jeff)_________________________

# Tunnel Configuration Option B - Cost Estimate

| Tunnel Configuration Option B | | | | |
|---|---|---|---|---|
| Water Treatment Process | 6,210.000 | MGAL | $54.54 | $338,688.76 |
| Rock Exc CL C | 202,770.000 | CY | $251.31 | $50,959,131.77 |
| Main Rock Bolt 1.0" x 16.4' | 95,940.000 | LF | $37.55 | $3,602,237.06 |
| Side Wall Rock Bolt 1.0" x 9.8' | 15,288.000 | LF | $53.74 | $821,563.34 |
| Optical Survey Target | 1.000 | LS | $149,856.97 | $149,856.97 |
| Roadbed Subbase Special | 17,928.000 | TN | $22.61 | $405,280.11 |
| Conc Pavement 24" | 14,940.000 | SY | $167.68 | $2,505,171.80 |
| MC 12 x 50 Steel Sets (Contingency) | 7,530.000 | LF | $143.44 | $1,080,097.54 |
| Steel Sets Install (Contingency) | 38.000 | EA | $14,613.75 | $555,322.68 |
| Conc Footing (Assumed) | 750.000 | CY | $504.40 | $378,303.20 |
| Conc Lining Secondary (Less Portal) | 17,533.000 | CY | $769.34 | $13,488,819.88 |
| Fireboard | 320,814.000 | SF | $22.90 | $7,347,070.74 |
| Re-Steel Footing Epoxy | 150,000.000 | LB | $1.66 | $248,293.59 |
| Re-Steel Lining Blk | 2,375,414.000 | LB | $3.29 | $7,806,656.13 |
| Re-Steel Lining Epoxy | 2,375,414.000 | LB | $3.77 | $8,963,845.21 |
| Geocomp Drain Tunnel | 385,398.000 | SF | $1.23 | $473,135.54 |
| Waterproof Membrane | 385,398.000 | SF | $1.89 | $726,755.62 |
| Carrier/Invert Drain | 2,340.000 | LF | $504.64 | $1,180,860.98 |
| G Rail Type 7 CD Special | 4,680.000 | LF | $163.11 | $763,378.02 |
| Const Surveying Tunnel | 1.000 | LS | $541,583.03 | $541,583.03 |
| Vibration Monitoring | 1.000 | LS | $319,705.33 | $319,705.33 |
| Shotcrete Overbreak Area | 4,725.000 | CY | $780.69 | $3,688,773.52 |
| Shotcrete Steel fiber Reinforced | 4,725.000 | CY | $844.54 | $3,990,462.09 |
| Fixed Fire Fighting System | 1.000 | LS | $1,096,583.57 | $1,096,583.57 |
| Hydrants and Standpipes | 1.000 | LS | $458,934.58 | $458,934.58 |
| Electrical Distribution | 1.000 | LS | $1,150,557.77 | $1,150,557.77 |
| Tunnel Light System | 1.000 | LS | $1,161,744.15 | $1,161,744.15 |
| Fire Alarm and Detection | 1.000 | LS | $251,754.94 | $251,754.94 |
| Tunnel Closure System and Signage | 1.000 | LS | $688,392.42 | $688,392.42 |
| Tunnel Portable Fire Extinguisher | 1.000 | LS | $87,851.98 | $87,851.98 |
| Tunnel Ventilation | 1.000 | LS | $1,852,336.70 | $1,852,336.70 |
| Tunnel Comand Center | 1.000 | LS | $5,872,138.66 | $5,872,138.66 |
| Tunnel SCADA | 1.000 | LS | $1,595,300.24 | $1,595,300.24 |
| Access Platform/Ramp | 1.000 | LS | $1,362,036.28 | $1,362,036.28 |
| Tunnel Portal | 2.000 | EA | $9,223,849.86 | $18,447,699.72 |
| Egress Tunnel | 900.000 | LF | $5,768.20 | $5,191,381.62 |
| | | | **Bid Total:** | **$149,551,705.54** |

| | |
|---|---|
| Tunnel Total Length | 2340 LF |
| Unit Cost $/LF | $63,911 |

QC Checklist:

Estimate Description: _Tunnel Option B REV3__________________________________________________
Version (number and or date): _REV3 02/04/19______
Quantities received from the design team (date): _02/04/19_and 02/06/19____
Estimate completed by (initials and date): __02/07/19______(Dean)_____DL________________
Estimate checked by (initials and date): ______(Ben)__________________ BW 2/4/19
Estimate reviewed by (initials and date): ______(Jeff)_________________ JW/2/7/19
Changes made and verified (initials and date): _____(Jeff)________________________

# Tunnel Configuration Option C - Cost Estimate

| Tunnel Configuration Option C | | | | |
|---|---|---|---|---|
| Water Treatment Process | 6,060 | MGAL | $54.54 | $330,511.33 |
| Rock Exc CL C | 198,004 | CY | $251.32 | $49,762,207.63 |
| Main Rock Bolt 1.0" x 16.4' | 93,685 | LF | $37.55 | $3,517,558.19 |
| Side Wall Rock Bolt 1.0" x 9.8' | 14,929 | LF | $53.74 | $802,270.14 |
| Optical Survey Target | 1 | LS | $149,856.60 | $149,856.60 |
| Roadbed Subbase Special | 18,487 | TN | $22.61 | $417,915.87 |
| Conc Pavement 24" | 15,406 | SY | $167.68 | $2,583,305.28 |
| MC 12 x 50 Steel Sets (Contingency) | 7,530 | LF | $143.44 | $1,080,094.86 |
| Steel Sets Install (Contingency) | 38 | EA | $14,613.72 | $555,321.30 |
| Conc Footing (Assumed) | 750 | CY | $504.40 | $378,302.26 |
| Conc Lining Secondary (Less Portal) | 17,533 | CY | $769.34 | $13,488,786.42 |
| Fireboard | 313,274 | SF | $22.90 | $7,174,376.83 |
| Re-Steel Footing Epoxy | 150,000 | LB | $1.66 | $248,292.98 |
| Re-Steel Lining Blk | 2,319,582 | LB | $3.29 | $7,623,148.70 |
| Re-Steel Lining Epoxy | 2,319,582 | LB | $3.77 | $8,753,136.25 |
| Geocomp Drain Tunnel | 376,340 | SF | $1.23 | $462,010.03 |
| Waterproof Membrane | 376,340 | SF | $1.89 | $709,675.09 |
| Carrier/Invert Drain | 2,285 | LF | $504.77 | $1,153,392.83 |
| G Rail Type 7 CD Special | 4,570 | LF | $163.11 | $745,433.52 |
| Const Surveying Tunnel | 1 | LS | $541,581.69 | $541,581.69 |
| Vibration Monitoring | 1 | LS | $319,704.54 | $319,704.54 |
| Shotcrete Overbreak Area | 4,614 | CY | $780.69 | $3,602,107.66 |
| Shotcrete Steel fiber Reinforced | 4,614 | CY | $844.54 | $3,896,708.27 |
| Fixed Fire Fighting System | 1 | LS | $1,092,940.61 | $1,092,940.61 |
| Hydrants and Standpipes | 1 | LS | $458,025.68 | $458,025.68 |
| Electrical Distribution | 1 | LS | $1,148,978.95 | $1,148,978.95 |
| Tunnel Light System | 1 | LS | $1,156,155.99 | $1,156,155.99 |
| Fire Alarm and Detection | 1 | LS | $250,543.97 | $250,543.97 |
| Tunnel Closure System and Signage | 1 | LS | $685,081.14 | $685,081.14 |
| Tunnel Portable Fire Extinguisher | 1 | LS | $87,429.40 | $87,429.40 |
| Tunnel Ventilation | 1 | LS | $1,903,383.78 | $1,903,383.78 |
| Tunnel Comand Center | 1 | LS | $5,872,124.09 | $5,872,124.09 |
| Tunnel SCADA | 1 | LS | $1,587,626.62 | $1,587,626.62 |
| Access Platform/Ramp | 1 | LS | $1,362,032.90 | $1,362,032.90 |
| Tunnel Portal | 2 | EA | $11,731,466.18 | $23,462,932.36 |
| Egress Tunnel | 2,285 | LF | $5,768.19 | $13,180,308.44 |
| Cross Passage Tunnel | 31 | LF | $6,748.78 | $210,561.94 |
| | | | **Bid Total:** | **$160,753,824.14** |

| | |
|---|---|
| Tunnel Total Length | 2285 LF |
| Unit Cost $/LF | $70,351.78 |

QC Checklist:

Estimate Description: _Tunnel Option C REV3_____________________________________________________
Version (number and or date): _REV 3 02/04/19______
Quantities received from the design team (date): _02/04/19_and02/06/19_____
Estimate completed by (initials and date): __02/07/19______(Dean)_____DL_________________
Estimate checked by (initials and date): ______(Ben)___________________ BW 2/4/19
Estimate reviewed by (initials and date): ______(Jeff)__________________ JW/2/7/19
Changes made and verified (initials and date): _____(Jeff)_________________________

# Tunnel Configuration Option D - Cost Estimate

| Tunnel Configuration Option D | | | | |
|---|---|---|---|---|
| Water Treatment Process | 8,000 | MGAL | $54.54 | $436,310.11 |
| Rock Exc CL C | 293,311 | CY | $251.31 | $73,712,018.68 |
| Main Rock Bolt 1.0" x 16.4' | 108,555 | LF | $37.55 | $4,075,850.19 |
| Side Wall Rock Bolt 1.0" x 9.8' | 47,987 | LF | $53.74 | $2,578,755.83 |
| Optical Survey Target | 1 | LS | $149,855.75 | $149,855.75 |
| Roadbed Subbase Special | 26,457 | TN | $22.61 | $598,081.77 |
| Conc Pavement 24" | 22,047 | SY | $167.68 | $3,696,858.77 |
| MC 12 x 50 Steel Sets (Contingency) | 7,530 | LF | $143.44 | $1,080,088.69 |
| Steel Sets Install (Contingency) | 38 | EA | $14,613.64 | $555,318.13 |
| Conc Footing (Assumed) | 1,500 | CY | $504.40 | $756,600.72 |
| Conc Lining Secondary (Less Portal) | 20,412 | CY | $769.33 | $15,703,617.78 |
| Fireboard | 546,016 | SF | $22.90 | $12,504,396.04 |
| Re-Steel Footing Epoxy | 300,000 | LB | $1.66 | $496,583.13 |
| Re-Steel Lining Blk | 1,285,297 | LB | $3.29 | $4,224,017.12 |
| Re-Steel Lining Epoxy | 1,285,297 | LB | $3.77 | $4,850,147.66 |
| Geocomp Drain Tunnel | 601,160 | SF | $1.23 | $738,005.27 |
| Waterproof Membrane | 601,160 | SF | $1.89 | $1,133,617.50 |
| Carrier/Invert Drain | 4,520 | LF | $360.15 | $1,627,860.94 |
| G Rail Type 7 CD Special | 9,040 | LF | $163.11 | $1,474,547.19 |
| Const Surveying Tunnel | 1 | LS | $541,578.59 | $541,578.59 |
| Vibration Monitoring | 1 | LS | $319,702.72 | $319,702.72 |
| Shotcrete Overbreak Area | 6,637 | CY | $780.69 | $5,181,415.03 |
| Shotcrete Steel fiber Reinforced | 6,637 | CY | $844.54 | $5,605,180.96 |
| Fixed Fire Fighting System | 1 | LS | $1,529,817.91 | $1,529,817.91 |
| Hydrants and Standpipes | 1 | LS | $739,883.42 | $739,883.42 |
| Electrical Distribution | 1 | LS | $1,456,147.96 | $1,456,147.96 |
| Tunnel Light System | 1 | LS | $1,560,801.66 | $1,560,801.66 |
| Fire Alarm and Detection | 1 | LS | $375,813.79 | $375,813.79 |
| Tunnel Closure System and Signage | 1 | LS | $1,370,154.46 | $1,370,154.46 |
| Tunnel Portable Fire Extinguisher | 1 | LS | $131,143.35 | $131,143.35 |
| Tunnel Ventilation | 1 | LS | $3,462,248.53 | $3,462,248.53 |
| Tunnel Comand Center | 1 | LS | $5,872,090.55 | $5,872,090.55 |
| Tunnel SCADA | 1 | LS | $1,930,156.17 | $1,930,156.17 |
| Access Platform/Ramp | 1 | LS | $1,362,025.12 | $1,362,025.12 |
| Tunnel Portal | 2 | EA | $15,532,742.13 | $31,065,484.26 |
| Cross Passage Tunnel | 36.1 | LF | $6,748.74 | $243,629.56 |
| | | | **Bid Total:** | **$193,139,805.31** |

| | | |
|---|---|---|
| Tunnel Total Length | 2260 | LF |
| Unit Cost $/LF | $85,460.09 | |

QC Checklist:

Estimate Description: _Tunnel Option D REV3_____________________________________________________
Version (number and or date): _REV 3 02/04/19______
Quantities received from the design team (date): _02/04/19_and 02/06/19_____
Estimate completed by (initials and date): __02/07/19______(Dean)_____DL_________________
Estimate checked by (initials and date): ______(Ben)___________________ BW 2/4/19
Estimate reviewed by (initials and date): ______(Jeff)__________________ JW/2/7/19
Changes made and verified (initials and date): _____(Jeff)_________________________

# Appendix K. Construction Programmes

## Tunnel Configuration Option A - Construction Programme

| Activity | Duration (months) | Programme (month from project start) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 | 28 | 29 | 30 | 31 | 32 | 33 | 34 | 35 | 36 | 37 | 38 | 39 | 40 | 41 | 42 | 43 | 44 | 45 | 46 | |
| Mobilisation & Site Setup | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Welfare and staff facilities | 2 |🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Site preparation (removal of vegetation, rocks etc.) | 2 |🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Provision of key service connections | 2 | | |🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Portal Construction | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| East Portal | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Slope cutting | 6 | | | | |🟩|🟩|🟩|🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Portal structure construction | 3 | | | | | | | | | | |🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| West Portal | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Slope cutting | 6 | | | | |🟩|🟩|🟩|🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Portal structure construction | 3 | | | | | | | | | | |🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Tunnelling | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Excavation and primary support | 18 | | | | | | | | | | |🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | Based on 5ft (≈ 1.5m)/day advance rate with 50% downtime |
| Permanent Lining(s) and Civils Work | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Secondary Lining | 4 | | | | | | | | | | | | | | | | | | | | | | | | | | | | |🟩|🟩|🟩|🟩| | | | | | | | | | | | | | | Based on 40ft (≈ 12m) secondary lining/day with 50% downtime |
| Drainage and structural invert | 4 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |🟩|🟩|🟩|🟩| | | | | | | | | | | | | Assumed to follow the secondary lining works |
| Egress compartment construction/installation | 7 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |🟩|🟩|🟩|🟩|🟩|🟩|🟩| | | | | | | | Assumed 330ft (≈ 100m)/month to install blockwork/precast sections |
| Highway Construction | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Paving & installation of barriers | 1 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |🟩| | | | | | | |
| Mechanical, Electrical and Plumbing Works | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Fit-out | 4 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |🟩|🟩|🟩|🟩| | | |
| Testing and Commission | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Fit-out | 2 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |🟩|🟩| |

## Tunnel Configuration Option B - Construction Programme

| Activity | Duration (months) | Programme (month from project start) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | |1|2|3|4|5|6|7|8|9|10|11|12|13|14|15|16|17|18|19|20|21|22|23|24|25|26|27|28|29|30|31|32|33|34|35|36|37|38|39|40|41|42|43|44|45|46| |
| Mobilisation & Site Setup | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Welfare and staff facilities | 2 |🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Site preparation (removal of vegetation, rocks etc.) | 2 |🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Provision of key service connections (Dave to review) | 2 | | |🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Portal Construction | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| East Portal | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Slope cutting | 6 | | | | |🟩|🟩|🟩|🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Portal structure construction | 3 | | | | | | | | | | |🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| West Portal | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Slope cutting | 6 | | | | |🟩|🟩|🟩|🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Portal structure construction | 3 | | | | | | | | | | |🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Egress Portal | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Slope cutting | 3 | | | | | | | | | | |🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Portal structure construction | 3 | | | | | | | | | | | | | |🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Tunnelling | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Main highway tunnel excavation and primary support | 18 | | | | | | | | | | |🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | Based on 5ft (≈ 1.5m)/day advance rate with 50% downtime |
| Egress tunnel no.1 | 5 + 1 | | | | | | | | | | | | | | |🟩|🟩|🟩|🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | | +1 month for opening in highway tunnel |
| Egress tunnel no.2 | 2 + 1 | | | | | | | | | | | | | | | | | | |🟩|🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | | +1 month for opening in highway tunnel |
| Permanent Lining(s) and Civils Work | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Secondary Lining (Egress tunnel no.1) | 1 | | | | | | | | | | | | | | | | | | | | |🟩|🟩| | | | | | | | | | | | | | | | | | | | | | | | | Based on 40ft (≈ 12m) secondary lining/day with 50% downtime |
| Secondary Lining (Egress tunnel no.2) | 1 | | | | | | | | | | | | | | | | | | | | | | | |🟩|🟩| | | | | | | | | | | | | | | | | | | | | | |
| Secondary Lining (Highway Tunnel) | 4 | | | | | | | | | | | | | | | | | | | | | | | | | | | | |🟩|🟩|🟩|🟩| | | | | | | | | | | | | | | |
| Drainage and structural invert | 4 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |🟩|🟩|🟩|🟩| | | | | | | | | | | | | |
| Egress compartment dividing wall | 7 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |🟩|🟩|🟩|🟩|🟩|🟩|🟩| | | | | | | | |
| Highway Construction | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Paving & installation of barriers | 1 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |🟩| | | | | | | |
| Mechanical, Electrical and Plumbing Works | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Fit-out | 4 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |🟩|🟩|🟩|🟩| | | |
| Testing and Commission | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Fit-out | 2 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |🟩|🟩| |

## Tunnel Configuration Option C - Construction Programme

| Activity | Duration (months) | Programme (month from project start) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 | 28 | 29 | 30 | 31 | 32 | 33 | 34 | 35 | 36 | 37 | 38 | 39 | 40 | 41 | 42 | 43 | 44 | |
| Mobilisation & Site Setup | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Welfare and staff facilities | 2 | 🟩 | 🟩 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Site preparation (removal of vegetation, rocks etc.) | 2 | 🟩 | 🟩 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Provision of key service connections (Dave to review) | 2 | | | 🟩 | 🟩 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Portal Construction | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| East Portal | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Slope cutting | 6 | | | | | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Portal structure construction | 3 | | | | | | | | | | | 🟩 | 🟩 | 🟩 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| West Portal | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Slope cutting | 6 | | | | | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Portal structure construction | 3 | | | | | | | | | | | 🟩 | 🟩 | 🟩 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Tunnelling | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Egress tunnel no.1 | 9 | | | | | | | | | | | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | | | | | | | | | | | | | | | | | | | | | | | | | | Based on 5ft (≈ 1.5m)/day advance rate with 50% downtime. Start egress tunnel first as a 'pilot', with 3 month lag to allow feedback. |
| Main highway tunnel excavation and primary support | 18 | | | | | | | | | | | | | | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | 🟩 | | | | | | | | | | | | | | |
| Permanent Lining(s) and Civils Work | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Secondary Lining (Highway Tunnel) | 4 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | 🟩 | 🟩 | 🟩 | 🟩 | | | | | | | | | | Based on 40ft (≈ 12m) secondary lining/day with 50% downtime |
| Secondary Lining (Egress Tunnel) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | 🟩 | 🟩 | 🟩 | 🟩 | | | | | | | | | | |
| Drainage and structural invert | 4 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | 🟩 | 🟩 | 🟩 | 🟩 | | | | | | | | |
| Highway Construction | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Paving & installation of barriers | 1 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | 🟩 | | | | | | | |
| Mechanical, Electrical and Plumbing Works | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Fit-out | 4 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | 🟩 | 🟩 | 🟩 | 🟩 | | | |
| Testing and Commission | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Fit-out | 2 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | 🟩 | 🟩 | |

## Tunnel Configuration Option D - Construction Programme

| Activity | Duration (months) | Programme (month from project start) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 | 28 | 29 | 30 | 31 | 32 | 33 | 34 | 35 | 36 | 37 | 38 | 39 | 40 | 41 | 42 | 43 | |
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DRAFT

PRELIMINARY GEOTECHNICAL REPORT

# I-70 Floyd Hill Preliminary Design

CLEAR CREEK COUNTY

SHANNON & WILSON

January 2019

Shannon & Wilson No: 100864-001

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Submitted To: Atkins North America, Inc.
7604 Technology Way,
Suite 400
Denver, CO 80237
Attn: Mr. Anthony Pisano, PE

Subject: DRAFT PRELIMINARY GEOTECHNICAL REPORT, I-70 FLOYD HILL PRELIMINARY DESIGN, CLEAR CREEK COUNTY

We are pleased to submit our report for the above-referenced project. The enclosed report provides preliminary-level geotechnical design recommendations and construction considerations for the project.

We appreciate the opportunity to be of service to you on this project. If you have any questions or require further information, please contact me at 303-825-3800.

David A. Varathungarajan, PE
Associate

Reviewed by:

Gregory R. Fischer, PhD, PE
Senior Vice President

DKM:DAV:GRF/ksm

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|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| CDOT | Colorado Department of Transportation |
| CIPC | Cast-in-Place Concrete |
| CR | County Road |
| EB | Eastbound |
| FHWA | Federal Highway Administration |
| GDR | Geotechnical Data Report |
| HBSN | Hollow-Bar Soil Nails |
| I-70 | Interstate 70 |
| LFRD | Load Resistance Factor Design |
| MOT | Maintenance of Traffic |
| MP | Milepost |
| MSE | Mechanically Stabilized Earth |
| psi | Pounds per square inch |
| SAA | Shape Accel Array |
| SFMSE | Stable Feature Mechanically Stabilized Earth |
| SMSE | Shored Mechanically Stabilized Earth |
| US-6 | U.S. Highway 6 |
| USGS | U.S. Geological Survey |
| WB | Westbound |

# 1 INTRODUCTION

This preliminary geotechnical report discusses subsurface conditions and geologic hazards and provides preliminary-level geotechnical design and construction considerations for the Interstate 70 (I-70) Floyd Hill to Veterans Memorial Tunnels Project (Project) in Clear Creek County, Colorado.  Our services were conducted in general accordance with our Master Subcontract Agreement Number 17-HA1-XB-00195-ZD0001, dated June 12, 2017, and Task Order numbers 1 through 4 with Atkins North America, Inc.

Shannon & Wilson, Inc.'s (Shannon & Wilson's) geotechnical scope was limited to reviewing existing geotechnical data, completing several additional geotechnical explorations along Floyd Hill (from approximately U.S. Highway 6 [US-6] interchange to milepost [MP] 245), and providing conceptual-level geotechnical recommendations for bridges and retaining structures.  We have prepared a separate geotechnical data report (GDR) presenting subsurface explorations, laboratory testing, and geophysical testing that we completed for the project, as well as existing subsurface information by others.  Geotechnical services for the proposed tunnel, rock cuts, and rockfall mitigation are being completed by Yeh and Associates, Inc. (Yeh).

# 2 PROJECT AND SITE DESCRIPTION

As shown in Figure 1, the Project is located on I-70 between approximately MP 242 (just east of the Veterans Memorial Tunnels) and MP 248 (Floyd Hill/Beaver Brook exit).

The Project includes providing three-lane capacity for westbound I-70 from Floyd Hill to the Veterans Memorial Tunnels; a multimodal trail and frontage road between US -6 and Idaho Springs; and physical and/or operational improvements to three interchanges—the Floyd Hill/Beaver Brook exit (Exit 248) near the top of Floyd Hill; the Floyd Hill/Hyland Hills exit (Exit 247); and the junction with US-6 (Exit 244) near the base of Floyd Hill.  The project will also improve curves through the corridor.

Based on preliminary plans, the major structural elements include nine new bridges (designated Bridges A through I) and associated abutment walls, a new tunnel for westbound (WB) I-70, several retaining walls, and several rock cuts.  The approximate locations of the structures provided by Atkins and Wood (the project structural engineer) are shown schematically in Figure 2.

The proposed bridges will range in length from approximately 300 to 1,200 feet. The longest bridge (Bridge A) will be constructed where the WB alignment descends Floyd Hill. At this location, the bridge alignment will be located approximately mid-slope of the existing embankment (see Exhibit 1). As such, the bridge will utilize relatively tall columns and will require retaining structures up to about 60 feet in height at the abutments. Retaining walls at other locations along the alignment are anticipated to have heights of about 35 feet or less.

Exhibit 2-1: Photo of side-hill fill on Floyd Hill section of alignment, view west from US-6.

Beginning at the Veterans Memorial Tunnels (the western limit of the project) and heading east towards the US-6 interchange, the alignment generally follows Clear Creek through Clear Creek Canyon (with the exception of the tunnel segment of the WB alignment, which diverges from the eastbound [EB] alignment between the US-6 interchange and east of the Hidden Valley Interchange). The topography through Clear Creek Canyon is relatively steep and rugged (see Exhibit 2). At the US-6 interchange, the alignment crosses Clear Creek and climbs Floyd Hill to the eastern limit of the project. In this reach, the alignment traverses several large side-hill fills and cuts (see Exhibit 1).

Exhibit 2-2: Typical conditions of existing alignment through Clear Creek Canyon (image courtesy of Goggle Earth Pro Street View).

# 3 SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

Shannon & Wilson completed a subsurface exploration program consisting of:

- Seven geotechnical borings,
- Laboratory testing of selected samples recovered from the borings,
- Geophysical testing, and
- Installation of instrumentation to monitor groundwater conditions and potential slope movement.

A discussion of the subsurface exploration program and associated data are provided in our GDR, along with relevant previous geotechnical data by others. Figure 2 presents the locations of the borings and geophysical testing provided in the GDR.

# 4 SUBSURFACE CONDITIONS

## 4.1 Generalized Geological Units

As described in the GDR, we divided subsurface profile into four units based on geologic origin. Generalized descriptions of the units are provided below.

- Fill: Fill soils along the alignment generally consisted of medium dense to very dense granular soils with variable proportions of sand, gravel, cobbles, boulders, and non-plastic fines (typically less than about 15 percent fines). The fill materials are associated with the existing I-70 construction, but may include reworked mine tailings (Colorado Department of Transportation [CDOT] and Federal Highway Administration [FHWA], 2012). Cobbles and boulders were the fill constituents between the east portal of the twin tunnels and the Hidden Valley interchange (approximately EB Station 1000+00 to 1030+00). The side-hill fill on Floyd Hill (approximately WB Station 2107+00 to 2137+00) also contained a significant portion of cobbles and boulders (likely material derived from nearby cut slopes).
- Alluvium: Alluvial soils (deposited by moving water) generally consisted of medium dense to very dense, clean to silty sand to gravel, with variable cobble and boulder content. Alluvial soils were encountered along the entire length of the alignment. A 3-foot thick layer of hard, sandy, fat clay was observed in boring SW-2-01.
- Colluvium: Colluvium (soils deposited by downslope movement) was noted in borings SW-1-02, SW-1-04, and consisted of very dense, silty sand.
- Bedrock: Bedrock along the alignment predominantly consisted of gneiss, with occasional zones of schist, granite, and pegmatite. Unconfined compressive strengths ranged from 440 to 37,000 pounds per square inch (psi), but were generally greater than about 3,000 psi. As noted on the boring logs, bedrock weathering and fracturing were variable, but generally decreased with depth. Bedrock outcrops are visible in the existing cut on the west side of Clear Creek at the location of proposed Bridges B, C, and G (US-6 Interchange).

Exhibit 4-1: Bedrock visible from existing I-70 EB bridge over Clear Creek at base of Floyd Hill (image courtesy of Google Earth Pro Street View).

### 4.2 Subsurface Profiles

Subsurface profiles presenting boring logs and our interpretation of geologic conditions are presented in Figures 3 through 6 for the profile locations shown in Figure 2. In these profiles, we have combined the alluvium and colluvium units into a single unit referred to as granular overburden because of their similarity. Additional discussion of the profile through the Floyd Hill landslide (Figure 6) is provide in Section 5.4.2.

## 5 GEOLOGIC HAZARDS

### 5.1 Seismic Hazards

Based on recent geologic map by the U.S. Geological Survey (USGS), no faults with activity in the last 1.6 million years are mapped within the vicinity of the project site (USGS, 2018a). Therefore, in our opinion, the potential for ground surface fault rupture is low.

Liquefaction may occur in loose, saturated, cohesionless soils when subjected to earthquake ground shaking. Based on the subsurface conditions encountered at the project site, it is our opinion that the risk of liquefaction is low.

Using the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO, 2017) criteria, and based on subsurface conditions encountered in the borings, we generally recommend assuming Site Class D conditions. However, Site Class B conditions are appropriate at locations with surficial bedrock (substructures located on the west side of Clear Creek at the US-6 Interchange).

Ground motion parameters were determined for the project site using the USGS U.S. Seismic Design Map Web Application (USGS, 2018b) and procedures recommended by AASHTO (2017). Recommended seismic design ground motion parameters for Site Class B and D conditions are summarized in Exhibit 3.

Exhibit 5-1: Recommended Seismic Design Parameters

| Site Class | Parameter | Value |
|---|---|---|
| Site Class B | Peak Design Spectral Acceleration, $A_S$ | 0.067 g |
| | Short-period Design Spectral Acceleration, $S_{DS}$ | 0.141 g |
| | Long-period Design Spectral Acceleration, $S_{D1}$ | 0.036 g |
| | $T_O$ | 0.051 sec. |
| | $T_S$ | 0.255 sec. |
| Site Class D | Peak Design Spectral Acceleration, $A_S$ | 0.108 g |
| | Short-period Design Spectral Acceleration, $S_{DS}$ | 0.225 g |
| | Long-period Design Spectral Acceleration, $S_{D1}$ | 0.087 g |
| | $T_O$ | 0.077 sec. |
| | $T_S$ | 0.387 sec. |

## 5.2 Rockfall

The relatively steep rock slopes above portions of the project have the potential to generate rockfall, particularly at cut slopes in the fractured bedrock. The project includes several areas that have produced prior rockfall events, as documented in the CDOT Geohazard Event Tracker database. We understand that rock cuts and rockfall mitigation for the project are being addressed by Yeh.

## 5.3 Mining and Subsidence

Hardrock and placer mining occurred in the vicinity of the project between about 1859 and the 1950s (CDOT and FHWA, 2012). Mining activities generally occurred west of the project area. However, the Gold Bar Placer Mine operated near the Hidden Valley Interchange in the late 1800s. "Underground placer mining" reportedly occurred at the mine, resulting in subsidence affecting the roadway circa 1981 (CDOT and FHWA, 2012). As part of a subsidence investigation, CDOT (1981) completed numerous borings at the interchange and encountered voids up to approximately 40 feet in vertical dimension in the overburden. CDOT reportedly mitigated the subsidence (CDOT and FHWA, 2012), but we did not find any information documenting these activities.

Yeh (2014) conducted geophysical testing just west of the Hidden Valley Interchange (near the east abutment of the I-70 EB bridge over Clear Creek) to identify potential voids. The Yeh evaluation identified several relatively shallow (less than a few feet deep) and thin (less than 3 feet in vertical dimension) voids. No voids were noted in nearby borings completed as part of the same study. However, voids were noted in the overburden in several borings completed by others inside the project limits.

The Yeh (2014) log of boring YA-W-10 (which is located near the west abutment of proposed Bridge F, west of the Hidden Valley Interchange) noted mining debris and a void from a depth of 25.5 to 27.5 feet. "Possible voids" were noted on the log of Yeh (2014) boring YA-W-18 (located just west of the US-6 Interchange). The log of boring B-2 (CDOT, 1972), which was completed at the existing I-70 EB bridge over Clear Creek at the US-6 interchange, indicated "open space" from a depth of 10 to 17 feet. We are not aware of any mitigation activities that have been completed near these borings.

We did not observe any indications of distress consistent with mine subsidence along the existing alignment, nor are we aware of any recent reports of such distress. Based on the performance of the existing roadway and structures, and subsurface conditions in the available explorations, it is our opinion that there is a relatively low risk of mining related subsidence affecting the project. However, during final design we recommend further study to characterize the extents of potential voids and abandoned mine workings, particularly near borings YA-W-10, YA-W-18, and B-2.

## 5.4 Landslides

### 5.4.1 General

We reviewed geologic maps, aerial imagery, and previous reports to characterize known landslides that could potentially affect the project. The geologic quadrangle map encompassing the project does not show any landslides along the project alignment. However, based on our review of aerial imagery and a study by Yeh (2008), an active landslide is occurring between approximately MP 244 and 244.5 (referred to as the Floyd Hill Landslide).

### 5.4.2 Floyd Hill Landslide

The approximate location of the Floyd Hill Landslide is shown on Figure 2. The landslide appears to extend beyond the current CDOT right-of-way. Yeh (2008) indicated that landslide activity at this area has been occurring since at least 1947. The movement was reportedly exacerbated by a rock cut associated with the original I-70 construction in 1959. Since then, movement has continued, with episodes of accelerated movement during periods of increased precipitation.

Based on Yeh inclinometer data, the landslide mass appears to toe-out adjacent to the EB lanes of I-70, and during past episodes of accelerated movement, the landslide has displaced into the roadway and affected traffic. About 500 feet of the roadway is affected during these episodes. However, because the landslide appears to toe out near the roadway elevation, the roadway does not appear to be part of the landslide mass. Additionally, we are not aware of any reports of pavement distress associated with the landslide, nor did we observe any indications of such pavement distress.

A subsurface profile through the landslide is shown in Figure 6. Based on the Yeh (2008) study, the landslide mass is comprised of highly weathered and fractured bedrock that has been displaced by landslide movement (referred to as landslide debris in Figure 6). The failure appears to occur in a zone of highly fractured bedrock (referred to as the shear zone in Figure 6), based on inclinometer data reported by Yeh (2008). Beneath the displaced shear zone, the quality of the bedrock improves. A fault is mapped (Sheridan and Marsh, 1976) as crossing near the toe of the landslide (see Figure 2 for the location of the fault). Yeh (2008) attributed this fault zone to the severely fractured and sheared bedrock near the toe of the landslide.

#### 5.4.2.1 Landslide Monitoring

To further characterize the extents of landslide movement and to confirm that the existing I-70 roadway is not part of the landslide mass (which would have significant effects on proposed nearby retaining walls), we completed boring SW-1-04 with a Shape Accel Array (SAA). The boring is located on the outboard shoulder of the roadway, about 100 feet away from the interpreted landslide toe.

Data from the SAA is presented in Figure 7 as a plot of cumulative displacement versus depth. While the data do not show a distinct shear failure, which would usually be indicative of a slope failure, the data show relatively minor lateral tipping movement (less than 0.05 inch since December) in the bedrock between a depth of 30 and 83 feet. In our opinion, the apparent tipping movement is indicative of compression occurring in the SAA sensor string inside the inclinometer casing. Although the monitoring period is limited, in our opinion it is unlikely that the apparent movement is indicative of actual slope movement.

#### 5.4.2.2 Summary

We understand that the proposed improvements will be located on the outboard fill slope adjacent to the Floyd Hill landslide, and no cuts into the landslide mass are proposed. The existing data suggests the landslide toes out at or near the elevation of the EB roadway. As such, we do not anticipate that the landslide will affect the proposed WB construction. However, landslide movement is likely to continue in an episodic manner, occasionally affecting the EB roadway during periods of increased precipitation. We recommend continued site observations and monitoring of the SAA for any indications that slope movement is affecting the existing I-70 roadway.

## 5.5 Debris Flows

Debris flows occur when intense precipitation leads to concentrated surface flow of water in constrained drainage areas that transports soil, rock, and debris. Previously mapped fan deposits (Sheridan and Marsh, 1976) related to past debris flows are shown in Figure 2. It is difficult to predict the frequency of debris flow events, as they depend on the storm events. We are not aware of any debris flow incidents in recent history that have affected I-70 within the project limits.

# 6 GEOTECHNICAL CONSIDERATIONS AND RECOMMENDATIONS

## 6.1 Bridge Foundations

Suitable foundation types for the bridges included with this project include spread footings, driven piles, and drilled shafts. We anticipate that these are likely the most cost-effective options. Micropiles also may be appropriate in some applications. Considerations and preliminary design parameters are provided in the following sections for these foundation options.

## 6.1.1 Spread Footings

### 6.1.1.1 General

We anticipate that spread footings will be feasible on the west side of Clear Creek at the US-6 Interchange, because bedrock is relatively shallow in this area. Shallow bedrock (less than 10 feet of soil cover) is anticipated at Abutment 1 and Piers 2 and 3 of Bridge B and Abutment 1 of Bridge C (see Figure 2). However, borings completed near the west bank of the creek indicate that the depth to bedrock varies significantly over short distances and generally increases to about 30 feet below ground surface adjacent to the creek. As the depth to bedrock increases, excavations to construct rock-supported footings may impact adjacent traffic or require shoring. Therefore, deep foundations should be considered in these areas.

### 6.1.1.2 Design Parameters

Footings bearing on bedrock may be designed for a service bearing pressure (bearing pressure corresponding to 1 inch of settlement) of 70 kips per square foot, based on AASHTO (2017) criteria. Sliding resistance may be evaluated assuming a nominal coefficient of sliding resistance of 0.7 and a strength limit resistance factor of 0.8.

## 6.1.2 Driven Piles

### 6.1.2.1 General

H-piles driven to bedrock are a suitable foundation option for locations where rock-supported footings are not feasible. However, cobbles and boulders may cause difficult driving conditions, potentially damaging piles or stopping piles above the bedrock. Therefore, we do not recommend using driven piles for Bridge A, where nearby boring SW-1-02 indicated that the existing side hill fill has a significant cobble and boulder content (rock fill).

Although borings further west (within Clear Creek Canyon) indicated some cobbles and boulders in the overburden (generally less prevalent than along Floyd Hill), our review of as-built plans indicates that several bridges in this area (between the Veterans Memorial Tunnels and the US-6 Interchange) are supported by driven pile foundations. As such, we anticipate that driven piles could be successfully installed for structures within this section of the alignment. Nevertheless, some piles may encounter refusal due to cobbles and boulders during driving. In these cases, predrilling may facilitate pile installation, or the pile may have to be relocated. It may be preferable to use a comparatively heavier pile section (minimum HP 14x89) to decrease the likelihood of damage during driving.

Pile penetration into the relatively high strength bedrock will be limited, and could be only a few inches, depending on factors such as the rock condition, the pile type, and the driving system. Therefore, the design team should consider uplift requirements, the minimum pile length required for lateral stability, and scour in evaluating the suitability of driven piles for a given structure. If bedrock embedment is required, a drilled-in foundation type (e.g., drilled shafts, micropiles) would be more appropriate.

#### 6.1.2.2 Design Parameters

Piles driven to refusal in bedrock can be designed using a factored axial compressive resistance of 18 kips per square inch times the cross-sectional area of the pile section. We recommend a minimum spacing of three widths/diameters to reduce stress overlap between adjacent piles. We also recommend that a protective rock tip be welded to the end of each pile to reduce the possibility of pile damage during driving. Recommended parameters for lateral analysis of deep foundations using LPILE by Ensoft, Inc. (2018) are provided in Table 1, and the recommend soil/rock stratigraphy associated with these parameters is provided in Table 2 for the proposed bridges. Group action can be analyzed using p-multipliers within LPILE. P-multipliers (i.e., group reduction factors) for loading perpendicular and parallel to a line of shafts or piles are presented on Figure 8.

### 6.1.3 Drilled Shafts

#### 6.1.3.1 General

Rock-socketed drilled shafts may be used at Bridge A, where the existing rock fill will create challenges for installation of driven piles, and at other locations where foundation penetration into the bedrock is required for design (e.g., for scour mitigation). Compared to driven piles, drilled shafts generally provide higher axial and lateral resistance, meaning fewer foundation elements can be used for a given structure. Drilled shafts can also be connected directly to columns, eliminating pile caps and associated excavations. However, site and subsurface conditions will present several challenges for drilled shaft construction.

Construction of drilled shafts requires several pieces of relatively large equipment, necessitating a large work area. To construct a suitable work area and access road for installation of drilled shafts along Bridge A, temporary shoring will likely be required. Because the existing ground is relatively steep, these temporary walls may become relatively tall, significant structures.

#### 6.1.3.2 Special Considerations

Subsurface conditions at the site will present several challenges for the constructability of drilled shafts. The relatively clean granular soils at the site will require the use of temporary excavation support during the drilling process. This could be accomplished by using temporary casing, drilling fluid, or a combination thereof. Loss of drilling fluid may occur in areas with significant cobbles and boulders, such as Bridge A. To overcome this issue, the contractor could construct the shafts using an oscillator to simultaneously advance casing with the excavation. In this procedure, the shaft casing is equipped with cutting teeth or a cutting shoe and installed by either rotating or oscillating the casing. Such methods are advantageous in that they can advance through cobbles, boulders, obstructions, and rock. Groundwater will likely be encountered in shafts installed below Clear Creek. In these locations, drilling fluid would be required to prevent heave at the base of the shaft excavation.

Installation of drilled shafts will require advancing the excavation through cobbles and boulders. Cobbles and boulders can sometimes be excavated by conventional augers, but modified single-helix augers, designed with a taper and sometimes with a calyx bucket mounted on the top of the auger, a.k.a. boulder rooters, are generally more successful at extracting smaller boulders (Brown and others, 2010). However, the extraction of large boulders and rock fragments can cause considerable difficulty and significantly reduce drilling production. Boulders that are solidly embedded can likely be cored, while coring through boulders loosely embedded in soil may be ineffective. The removal of loosely embedded boulders may require breaking the boulder in the hole with percussion methods or a rock breaker tool (or other appropriate methods).

Construction of drilled shafts will also require drilling sockets in relatively high strength rock. Unconfined compressive strength testing (see the GDR) indicated strengths ranging from 440 to 37,000 psi (but generally greater than 3,000 psi). While the borings indicated a variable degree of fracturing, areas of higher strength intact bedrock will likely require specialized drilling techniques, such as coring, full-face rotary drilling, or down-hole impact hammers, to advance the excavation. Use of these techniques will decrease production rates and increase costs. In weaker, more fractured bedrock, less aggressive drilling methods, such as conventional rock augers, may be suitable.

#### 6.1.3.3 Design Parameters

We recommend designing drilled shafts for a combination of side resistance and end bearing in the bedrock using the parameters provided in Tables 1 and 2. We recommend a minimum bedrock penetration of one shaft diameter and no less than 5 feet. Recommended parameters for lateral analysis of deep foundations using LPILE by Ensoft, Inc. (2018) are also provided in the tables. Group action can be analyzed using p-multipliers within LPILE. P-multipliers (i.e., group reduction factors) for loading perpendicular and parallel to a line of shafts or piles are presented on Figure 8.

### 6.1.4 Micropile Foundations

#### 6.1.4.1 General

Micropiles consist of small diameter (typically less than 12 inches, but larger sizes are feasible) drilled and grouted foundation elements. Compared to drilled shafts, micropiles can be more easily installed through boulders and in strong bedrock. Micropiles can also be installed with much smaller equipment. Micropiles may be constructed with permanent steel casing to provide lateral resistance in the upper portion of the element. A central reinforcing bar may also be included to transfer uplift loads to the bond zone.

While we generally anticipate that spread footings, driven piles, and drilled shafts will be the most cost-effective foundation options for bridges on this project, micropiles may be considered for some applications. Micropiles may be considered in locations with difficult access (equipment to install micropiles is much smaller than equipment to install driven piles and drilled shafts) and at locations with lighter loads. One possible application would be modifying or widening an existing bridge.

#### 6.1.4.2 Design Parameters

We recommend designing micropiles for a nominal bond strength of 100 psi in the bedrock. Recommended parameters for lateral analysis of deep foundations using LPILE by Ensoft, Inc. (2018) are provided in Table 1, and the recommend soil/rock stratigraphy associated with these parameters is provided in Table 2 for the proposed bridges. Group action can be analyzed using p-multipliers within LPILE. P-multipliers (i.e., group reduction factors) for loading perpendicular and parallel to a line of shafts or piles are presented on Figure 8.

## 6.2 Retaining Structures

The project will require cut and fill walls to accommodate the proposed alignment. The walls will generally have heights of about 30 feet or less. However, some walls will be taller, including the fill walls associated with Bridge A, which will have a maximum height of about 60 feet. Fill walls at the west portal of the tunnel will have a maximum height of about 45 feet.

The selection of appropriate walls systems for the project will depend on several issues, including cost, maintenance of traffic (MOT), constructability, and design requirements. The following sections provide a discussion of wall types that may be considered for different applications and locations on the project and presents advantages and disadvantages associated with each wall type.

Based on the stage of the design at the time this report was prepared, we understand that limited wall design parameters are required because the wall selection process is in ongoing. Therefore, we have only provided preliminary design parameters for Mechanically Stabilized Earth (MSE) wall options.

## 6.2.1 Mechanically Stabilized Earth Walls

### 6.2.1.1 General

MSE walls are generally used in fill situations and essentially consist of a facing element (typically precast concrete panels for CDOT projects), a reinforced zone comprised of granular backfill and soil reinforcement (usually metallic strips or geosynthetics), and a backfill zone (1 horizontal to 1 vertical [1H:1V] area) behind the reinforced zone. MSE walls are routinely constructed for CDOT projects. MSE walls have been constructed with heights greater than 100 feet. However, in most CDOT applications, wall height does not exceed about 30 to 40 feet. As such, there will be greater uncertainty in estimating costs for tall MSE walls required for the project.

Subsurface conditions along the alignment are generally favorable for MSE walls, and we anticipate that MSE walls will be a cost-effective option for most fill situations on the project. However, MSE walls are not inherently scour resistant. Instead, scour resistance is obtained by imbedding the base of the wall below the scour depth, necessitating increased excavation depth (and potentially shoring). Therefore, other systems may be preferable at locations susceptible to scour.

The primary disadvantages of MSE walls are the issues related to the excavation (when required due to site grades) to construct the reinforced zone. Sloped excavations along the alignment can generally be sloped at 1H:1V (assuming excavations above the groundwater table). This may result in significant excavation into existing embankments, particularly for tall walls (e.g., walls adjacent to Bridge A). In these situations, temporary shoring may be required to construct the excavation and wall without affecting adjacent structures or MOT. To reduce the width of the reinforced zone, excavation shoring could be incorporated into the permanent retaining wall, resulting in a hybrid system (see Section 6.2.1.3).

### 6.2.1.2 Design Parameters

Except for abutment walls at Bridge A (see Section 6.2.5), for preliminary design we recommend assuming the reinforcement length specified in the AASHTO LRFD Bridge Design Specifications (AASHTO, 2017) of 0.7 times the wall height and at least 8 feet.

During final design, the it may be necessary to increase the reinforcement length for some walls, particularly for walls supported by sloping ground.

At locations where the wall is supported by bedrock or very dense granular soil, and no toe slope is present, a non-uniform (trapezoidal) reinforcement configuration may be utilized to reduce excavation volumes (Berg and others, 2009). For such walls, the cross -sectional area of the reinforced block (wall height times reinforcement length) should be equal to the area of a reinforced zone assuming a reinforcement length of 0.7 times the wall height. The base reinforcement should be at least 0.4 times the wall height (but no less than 8 feet). The difference in reinforcement length between adjacent steps should be less than 0.15 times the wall height.

#### 6.2.1.3 Shored Mechanically Stabilized Earth Walls

Shoring (e.g., soldier pile walls) behind the reinforced zone of an MSE wall may be designed as a permanent structure to improve global stability and reduce earth pressures acting on MSE walls, thereby decreasing the required width of the reinforced zone. These structures are referred to as shored MSE (SMSE) walls (Berg and others, 2009; Morrison and others, 2006). Berg and others (2009) recommend a minimum reinforcement length of 0.3 times the wall height and no less than 5 feet. To eliminate the potential for a tension crack at the back of the soil reinforcement, the upper layers of reinforcement should be lengthened to 0.6 times the wall height.

SMSE walls may be appropriate for locations where relatively large cuts would be required to construct a conventional MSE wall, such as walls adjacent to Bridge A. However, SMSE structures have been designed and implemented for limited applications - namely low-volume roads in mountainous terrain and for low seismic hazards (Berg and others, 2009). Before selecting SMSE structures, the design team should consider that SMSE structures are not routinely used on CDOT projects and conduct more in-depth analyses.

#### 6.2.1.4 Stable Feature Mechanically Stabilized Earth Walls

Like SMSE walls, MSE walls that are constructed in front of a stable feature (referred to as stable feature MSE [SFMSE] walls), such as a rock cut or an existing wall, can designed using a decreased reinforcement length, because there is essentially no earth pressure applied to the back of the reinforced zone. For such structures, Berg and others (2009) recommend a minimum reinforcement length of 0.3 times the wall height (but no less than 5 feet) and increasing the length of reinforcement in the upper three feet of the wall an additional 6.5 feet to reduce the likelihood of a pavement crack developing at the back of the reinforced zone.

SFMSE walls may be considered for locations where fill walls are required adjacent to rock cuts (as may be the case at the west portal of the proposed tunnel).

### 6.2.2 Cast-in-Place Concrete Cantilever Walls

Cast-in-place concrete (CIPC) cantilever walls are often used for fill situations on CDOT projects but may also be used for cut situations. Based on our experience, CIPC cantilever walls are generally more cost effective than MSE walls for heights less than about 10 feet.

Subsurface conditions along the alignment are favorable for CIPC walls, and we anticipate that CIPC walls will be cost effective for relatively short fill walls. CIPC walls have similar disadvantages to MSE walls (i.e., lack of scour resistance, impacts from excavation required to place granular backfill in the retained zone), which may limit their use along Clear Creek and in locations where excavation may interfere with MOT requirements.

### 6.2.3 Soldier Pile Walls

Soldier pile walls consist of drilled or driven vertical elements (driven piles or drilled shafts), usually spaced about 5 to 10 feet on centers, with facing elements spanning the vertical elements. Facing options include precast concrete lagging, shotcrete, cast-in-place concrete, and precast concrete aesthetic panels. Soldier piles walls are generally used in cut situations to facilitate top-down construction. However, soldier pile walls may also be used in fill situations. As such, soldier pile walls may be advantageous in sections with both cut and fill. In fill applications, backfill can be placed directly on the existing grade, meaning excavation is not required to construct a retained zone as is required for MSE walls.

Solider pile walls can usually be cantilevered to heights on the order of 15 to 20 feet (depending subsurface conditions, the ground slope in front of the wall, the backslope behind the wall, deflection criteria, etc.). Taller walls can be constructed by installing ground anchors (strand or bar anchors, depending on design loads and the required anchor lengths) through the vertical elements or through horizontal waler beams connecting the vertical elements. Ground anchors bonded in the granular overburden or bedrock along the alignment will achieve relatively high capacity. Scour resistance can be obtained by embedding the facing elements below the scour depth. Alternatively, the spacing of vertical elements can be decreased such that the vertical elements form a continuous structure (a tangent or secant pile wall).

Due to the presence of cobble- and boulder-sized soil particles along the alignment, installation of driven vertical elements may be challenging. Specifically, large soil particles may limit penetration of driven piles and may create challenges in maintaining vertical alignment. As such, drilled vertical elements (e.g., drilled shafts, micropiles) may be preferable for retaining structures.

We anticipate that soldier pile walls will be a suitable alternative for locations where the existing roadway must be widened but MOT and other considerations limit the potential excavation area in the retained zone of the wall. Soldier pile walls may also be used to provide shoring in combination with SMSE walls, thereby reducing excavation requirements for tall walls. This system may be suitable for the tall walls located adjacent to Bridge A. Additionally, ground anchors installed through the wall could be used improve provide an adequate factor of safety for global stability, as required.

### 6.2.4 Soil Nail Walls

Soil nail walls are a top-down cut wall that consist of steel nail bars installed and grouted in a drill hole. At each excavation lift, shotcrete is applied to the excavation face before installing the next row of soil nails and continuing the process. After reaching the bottom of the excavation, a permanent facing is applied to the full height of the wall. Permanent facing may consist of structural shotcrete or CIPC. Aesthetic finishes or features, such as sculpted shotcrete, precast concrete panels, or masonry facing may be installed in front of the structural facing. Soil nail walls are routinely used on CDOT projects and were included in the Clear Creek County Road (CR) 314 project (which was completed in 2014 and overlaps the west end of the current project area).

Soil nail walls require site soils with sufficient "stand-up time" (the ability of the soil to stand unsupported) to permit excavation and installation of facing between subsequent excavation lifts. The granular soils along the alignment vary from clean to silty, and as such, we anticipate that areas of running soils are likely to be encountered during excavation, making soil nail construction challenging. Stand-up time will also be limited in talus deposits located above CR 314, where cut walls are anticipated. Running soils and over-break at the excavation face (i.e. removal of additional soil from behind the excavation neat line) may result shotcrete quantity over-runs.

Talus and coarse-grained soils will present drilling and grouting challenges for soil nail installation. Cased drilling techniques, which increase cost and slow production, will likely be required to mitigate collapse of drill holes. Additionally, grout loss is likely to occur, particularly in talus deposits above CR 314. Grout socks and multiple stages of grouting may be required to mitigate grout loss. While soil nail walls are feasible for the project (and have been implemented on past projects in the area), soil nail construction will be more expensive and production rates will be lower than for typical projects. At locations where cobbles and boulders are not prevalent, hollow-bar soil nails (HBSNs) could be considered as an alternative to cased drilling. However, HBSNs are not routinely used by CDOT for permanent structures.

### 6.2.5 Bridge A Abutment Walls

Relatively tall walls are proposed along Floyd Hill and at the abutments of Bridge A. While we anticipate that a reinforcement length of 0.7 times the wall height will provide a minimum factor of safety (FS) of 1.3 for global stability in this area, an FS of 1.5 is required at bridge abutments based on CDOT (2017) and AASHTO (2017) criteria. There are several possible approaches to achieve a minimum FS of 1.5 at the bridge abutments.

- Increase the reinforcement length or wall embedment. Depending on the wall embedment, preliminary analyses suggest a minimum required reinforcement length of 0.8 to 1.0 times the wall height.
- Incorporate ground anchors used for shoring into the permanent design of an SMSE wall. An anchored solider pile wall could be used to provide shoring behind the reinforced zone. The ground anchors could also be designed to provide an improvement to the FS for global stability. Use of this type of system would also allow for a decreased reinforcement length. While soil nails could be considered instead of ground anchors, based on preliminary analyses we anticipate that ground anchors will be more cost effective because of the relatively high capacity that can be obtained for a single ground anchor bonded into bedrock.
- Backfill the abutment walls with lightweight fill (e.g. geofoam).
- Support the abutment retaining walls on deep foundations.

# 7 LIMITATIONS

The recommendations provided in this report are preliminary in nature and based on limited subsurface information and limited structure information provided by the design team. These recommendations should not be relied on for final design.

Within the limitations of scope, schedule and budget, our observations were completed in accordance with generally accepted professional geotechnical and geological principles and practice in the locale at the time the work was done. We make no other warranty, express or implied.

The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands, or hazardous or toxic substances in the soil, surface water, groundwater, or air, on or below or around this site, or for the evaluation or disposal of contaminated soils or groundwater should any be encountered. If a service is not specifically indicated in this report, do not assume that it was performed.

We have prepared an Appendix, Important Information About Your Geotechnical Report, to assist you and others in understanding the use and limitations of our report.

# 8 REFERENCES

American Association of State Highway Transportation Officials (AASHTO), 2017, LRFD Bridge Design Specifications, 8th edition.

Berg, R.R., Christopher, B.R., Samtani, N.C., 2009, Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume II, U. S. Federal Highway Administration, FHWA-NHI-10-025, GEC 011.

Brown, D.A.; Turner, J.P.; and Castelli, R.J., 2010, Drilled shafts: construction procedures and LRFD design methods: Washington, D. C., U. S. Federal Highway Administration, FHWA NHI-10-016, NHI course no. 132014, GEC 010.

Colorado Department of Transportation, 2017, Geotechnical Design Manual.

Colorado Department of Transportation and Federal Highway Administration, 2012, Twin Tunnels Environmental Assessment and Section 4(f) Evaluation, Clear Creek County, June 28.

Colorado Division of Highways (now CDOT), 1981, Engineering Geology, Hidden Valley Subsidence, Project No. IR 70-3(154), 5 sheets.

Colorado Division of Highways (now CDOT), 1972, Log of Boring B-1, Project I-70-3(29), Floyd Hill - Beaverbrook.

Ensoft, Inc., 2018, LPILE 2018, A Program for Analyzing Stress and Deformation of Individual Piles or Drilled Shafts Under Lateral Load, Austin, Texas.

Morrison, K.F., Harrison, F.E., Collin, J.G., Dodds, A., Arndt, B., 2006, Shored Mechanically Stabilized Earth (SMSE) Wall Systems, U. S. Federal Highway Administration, FHWA-CFL/TD-06-001.

Sheridan, Douglas M. and Marsh, Sherman P., 1976, Geologic Map of the Squaw Pass Quadrangle, Clear Creek, Jefferson, and Gilpin Counties, Colorado, U.S. Geological Survey, Squaw Pass Quadrangle, Colorado Map GQ-1337, scale 1:24,000.

U.S. Geological Survey (USGS), 2018a, Quaternary Fault and Fold Database of the United States, Interactive Fault Map Application, available from: https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561a9b0aadf88412fcf

USGS, 2018b, U.S. Seismic Design Maps Web Application, available from: http://earthquake.usgs.gov/designmaps/us/application.php, accessed November 29, 2018.

Yeh and Associates, Inc., 2014, Final Combined Geotechnical Data Report, Eastbound Twin Tunnel and Westbound Twin Tunnel Addendum, Idaho Springs, Colorado, February 27, project number 212-090.

Yeh and Associates, Inc., 2008, Final Geotechnical Investigation Report, I-70 Floyd Hill Landslide, Clear Creek County, Colorado, August 25, project number 27-207.

# Important Information

## About Your Geotechnical/Environmental Report

IMPORTANT INFORMATION

## CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals.  A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated.  No one other than you should apply this report for its intended purpose without first conferring with the consultant.  No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

## THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors.  Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client.  To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations.  Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

## SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity.  Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time.  Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report.  The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

## MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken.  The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions.  The actual interface between materials may be far more gradual or abrupt than your report indicates.  Actual conditions in areas not sampled may differ from those predicted in your report.  While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts.  Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

## A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

## THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

## BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

## READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

**The preceding paragraphs are based on information provided by the ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland**.

# Table 1 - Geotechnical Parameters For Deep Foundation Design

| | DRILLED SHAFT AXIAL RESISTANCE PARAMETERS [1, 2, 3, 4, 5] | | | | LPILE PARAMETERS FOR LATERAL ANALYSIS [6, 7, 8, 9] | | | |
|---|---|---|---|---|---|---|---|---|
| Representative Soil/Rock Description | Nominal Unit Side Resistance $f_s$ (ksf) | Side Resistance - Resistance Factor | Nominal Unit Base Resistance $q_b$ (ksf) | Base Resistance - Resistance Factor | LPile Soil Type | Total Unit Weight [10] $\gamma$ (pcf) | Drained Friction Angle $\phi'$ (deg) | Unconfined Compressive Strength $q_u$ (psi) |
| Granular Overburden: Silty SAND with Gravel | - | - | - | - | Sand (Reese) | 125 | 34 | - |
| Rock Fill: GRAVEL with Silt and Sand | - | - | - | - | Sand (Reese) | 135 | 42 | - |
| Bedrock: GNEISS/SCHIST | 17 | 0.55 | 175 | 0.5 | Stong Rock (Vuggy Limestone) | 145 | - | 1000 |

NOTES:

1 Factored shaft end bearing resistance should be calculated by multiplying the nominal unit end bearing (qb) by the end area of the shaft and the given resistance factor. Factored shaft side resistance should be calculated by multiplying the nominal unit side resistance (fs) by the side surface area of the shaft within each layer and by the given resistance factor. Total factored axial compressive capacity for the shaft is determined by summing the factored end bearing resistance and the factored side resistance. If a non-redundant single shaft is used, the above resistance factors should be reduced by 20 percent (AASHTO, 2017).

2 A resistance factor of 1.0 is appropriate for the Service and Extreme Event Limit States.

3 Factored uplift resistance should be calculated by multiplying the total nominal side resistance over the embeded shaft length by the resistance factor provided above minus 0.1. A resistance factor of 0.8 is appropriate for uplift at the Extreme Event Limit State.

4 The nominal shaft resistance parameters above are based on a single shaft and do not consider group action of closely spaced shafts (closer than 2 diameters, center to center).

5 Side resistance should be ignored in the top 2 feet of bedrock.

6 The LPile parameters shown are for a single shaft. For group effects, see Figure 8 for recommended p-multipliers.

7 The LPile parameters shown are for a horizontal ground surface. Sloping ground surface modifications should be included, as necessary, per Ensoft, Inc.'s recommendatinos for the LPile program.

8 A resistance factor of 1.0 should be used for lateral analyses.

9 The lateral loading analysis should consider the potential for loss of material from flooding and scour.

10 For design below the water table, the effective unit weight ($\gamma' = \gamma - \gamma_w$ , $\gamma_w = 62.4$ pcf) must be used.

deg = degrees; ksf = kips per square foot; pcf = pounds per cubic foot; psi = pounds per square inch

# Table 2 - Soil/Rock Elevations For Deep Foundation Design

| Bridge | Location, (Boring ID) | Depth | | Approximate Elevation [1] | | Representative Groundwater Elevation [2] | Representative Soil/Rock Description |
|---|---|---|---|---|---|---|---|
| | | Top (ft) | Bottom (ft) | Top (ft) | Bottom (ft) | (ft) | |
| Bridge A | Abutment 1 (SW-1-04) | 0 | 20 | 7,325 | 7,305 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 20 | 75 | 7,305 | 7,250 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 75 | 82 | 7,250 | 7,243 | | Bedrock: GNEISS/SCHIST |
| | Pier 2 (SW-1-02 and SW-1-04) | 0 | 20 | 7,320 | 7,300 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 20 | 70 | 7,300 | 7,250 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 70 | 100 | 7,250 | 7,220 | | Bedrock: GNEISS/SCHIST |
| | Pier 3 (SW-1-02 and SW-1-04) | 0 | 20 | 7,310 | 7,290 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 20 | 60 | 7,290 | 7,250 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 60 | 100 | 7,250 | 7,210 | | Bedrock: GNEISS/SCHIST |
| | Pier 4 (SW-1-02) | 0 | 20 | 7,310 | 7,290 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 20 | 60 | 7,290 | 7,250 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 60 | 100 | 7,250 | 7,210 | | Bedrock: GNEISS/SCHIST |
| | Pier 5 (SW-1-01 and SW-1-02) | 0 | 20 | 7,320 | 7,300 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 20 | 70 | 7,300 | 7,250 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 70 | 100 | 7,250 | 7,220 | | Bedrock: GNEISS/SCHIST |
| | Pier 6 (SW-1-01 and SW-1-02) | 0 | 20 | 7,340 | 7,320 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 20 | 25 | 7,320 | 7,315 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 25 | 100 | 7,315 | 7,240 | | Bedrock: GNEISS/SCHIST |
| | Abutment 7 (SW-1-01) | 0 | 10 | 7,370 | 7,360 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 10 | 20 | 7,360 | 7,350 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 20 | 61 | 7,350 | 7,309 | | Bedrock: GNEISS/SCHIST |

NOTES:
1 Ground surface elevations are approximate.
2 Groundwater was not encountered during drilling, except where indicated.
3 Geophysical testing considered in conjunction with borings to estimate bedrock depth.

## Table 2 - Soil/Rock Elevations For Deep Foundation Design

| Bridge | Location, (Boring ID) | Depth | | Approximate Elevation [1] | | Representative Groundwater Elevation [2] | Representative Soil/Rock Description |
|---|---|---|---|---|---|---|---|
| | | Top (ft) | Bottom (ft) | Top (ft) | Bottom (ft) | (ft) | |
| Bridge B | Abutment 1 and Piers 2 and 3 (SW-2-03) | 0 | 5 | 7,242 | 7,237 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 5 | 43 | 7,237 | 7,199 | | Bedrock: GNEISS/SCHIST |
| | Pier 4 and Abutment 5 (B-12) | 0 | 15 | 7,250 | 7,235 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 15 | 37 | 7,235 | 7,213 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 37 | 45 | 7,213 | 7,205 | | Bedrock: GNEISS/SCHIST |
| Bridge C | Abutment 1 (YA-W-22) | 0 | 10 | 7,260 | 7,250 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 10 | 34 | 7,250 | 7,226 | | Bedrock: GNEISS/SCHIST |
| | Pier 2 (B-6) | 0 | 25 | 7,245 | 7,220 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 25 | 45 | 7,220 | 7,200 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 45 | 60 | 7,200 | 7,185 | | Bedrock: GNEISS/SCHIST |
| | Pier 3 (B-4) | 0 | 21 | 7,240 | 7,219 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 21 | 36 | 7,219 | 7,204 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 36 | 55 | 7,204 | 7,185 | | Bedrock: GNEISS/SCHIST |
| | Pier 4 (B-9) | 0 | 10 | 7,222 | 7,212 | 7,206 | Granular Overburden: Silty SAND with Gravel |
| | | 10 | 25 | 7,212 | 7,197 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 25 | 40 | 7,197 | 7,182 | | Bedrock: GNEISS/SCHIST |
| | Abutment 5 (B-12) | 0 | 16 | 7,251 | 7,235 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 16 | 38 | 7,235 | 7,213 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 38 | 43 | 7,213 | 7,208 | | Bedrock: GNEISS/SCHIST |

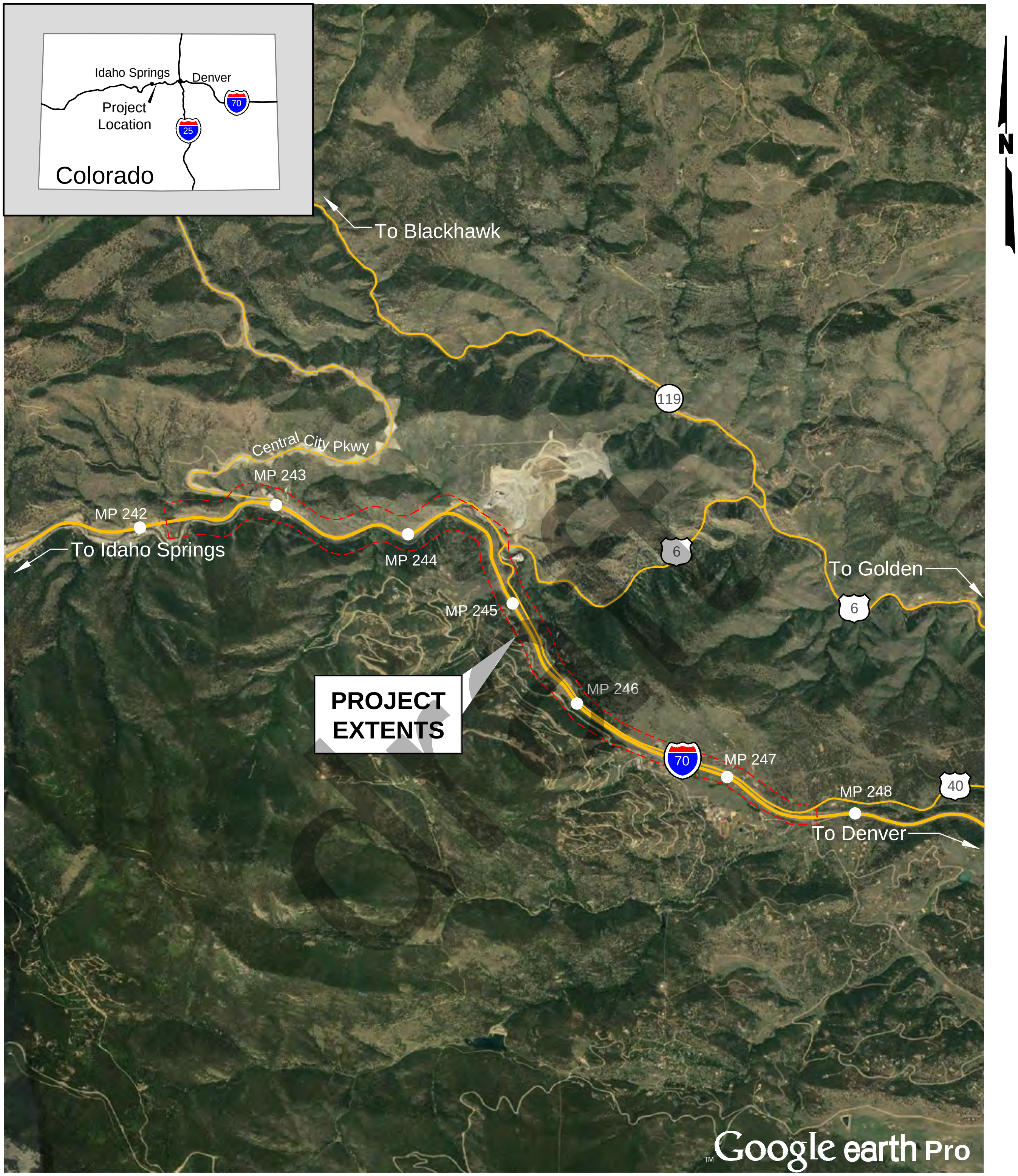
NOTES:

1 Ground surface elevations are approximate.
2 Groundwater was not encountered during drilling, except where indicated.
3 Geophysical testing considered in conjunction with borings to estimate bedrock depth.

# Table 2 - Soil/Rock Elevations For Deep Foundation Design

| | | Depth | | Approximate Elevation [1] | | Representative Groundwater Elevation [2] | |
|---|---|---|---|---|---|---|---|
| Bridge | Location, (Boring ID) | Top (ft) | Bottom (ft) | Top (ft) | Bottom (ft) | (ft) | Representative Soil/Rock Description |
| Bridge D | All Substructure Locations (YA-07) | 0 | 6 | 7,337 | 7,331 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 6 | 33 | 7,331 | 7,304 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 33 | 50 | 7,304 | 7,287 | | Bedrock: GNEISS/SCHIST |
| Bridge E | All Substructure Locations (YA-11, YA-W-10) | 0 | 28 | 7,345 | 7,317 | 7,337 | Granular Overburden: Silty SAND with Gravel |
| | | 28 | 42 | 7,317 | 7,303 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 42 | 50 | 7,303 | 7,295 | | Bedrock: GNEISS/SCHIST |
| Bridge F | All Substructure Locations (TH2) | 0 | 23 | 7,306 | 7,283 | -NA- | Granular Overburden: Silty SAND with Gravel |
| | | 23 | 34 | 7,283 | 7,272 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 34 | 37 | 7,272 | 7,269 | | Bedrock: GNEISS/SCHIST |
| Bridge G | All Substructure Locations (B-1) | 0 | 13 | 7,224 | 7,211 | 7,211 | Granular Overburden: Silty SAND with Gravel |
| | | 13 | 24 | 7,211 | 7,200 | | Rock Fill: GRAVEL with Silt and Sand |
| | | 24 | 37 | 7,200 | 7,187 | | Bedrock: GNEISS/SCHIST |
| Bridge H | All Substructure Locations (YA-W-17) | 0 | 49 | 7,258 | 7,209 | 7,227 | Granular Overburden: Silty SAND with Gravel |
| Bridge I | All Substructure Locations (B-6-1) | 0 | 24 | 7,287 | 7,263 | 7,275 | Granular Overburden: Silty SAND with Gravel |
| | | 24 | 32 | 7,263 | 7,255 | | Bedrock: GNEISS/SCHIST |

NOTES:
1 Ground surface elevations are approximate.
2 Groundwater was not encountered during drilling, except where indicated.
3 Geophysical testing considered in conjunction with borings to estimate bedrock depth.



0 1 2

Approximate Scale in Miles

NOTE

Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth ™ Mapping Service.

I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

**VICINITY MAP**

January 2019 100864-001

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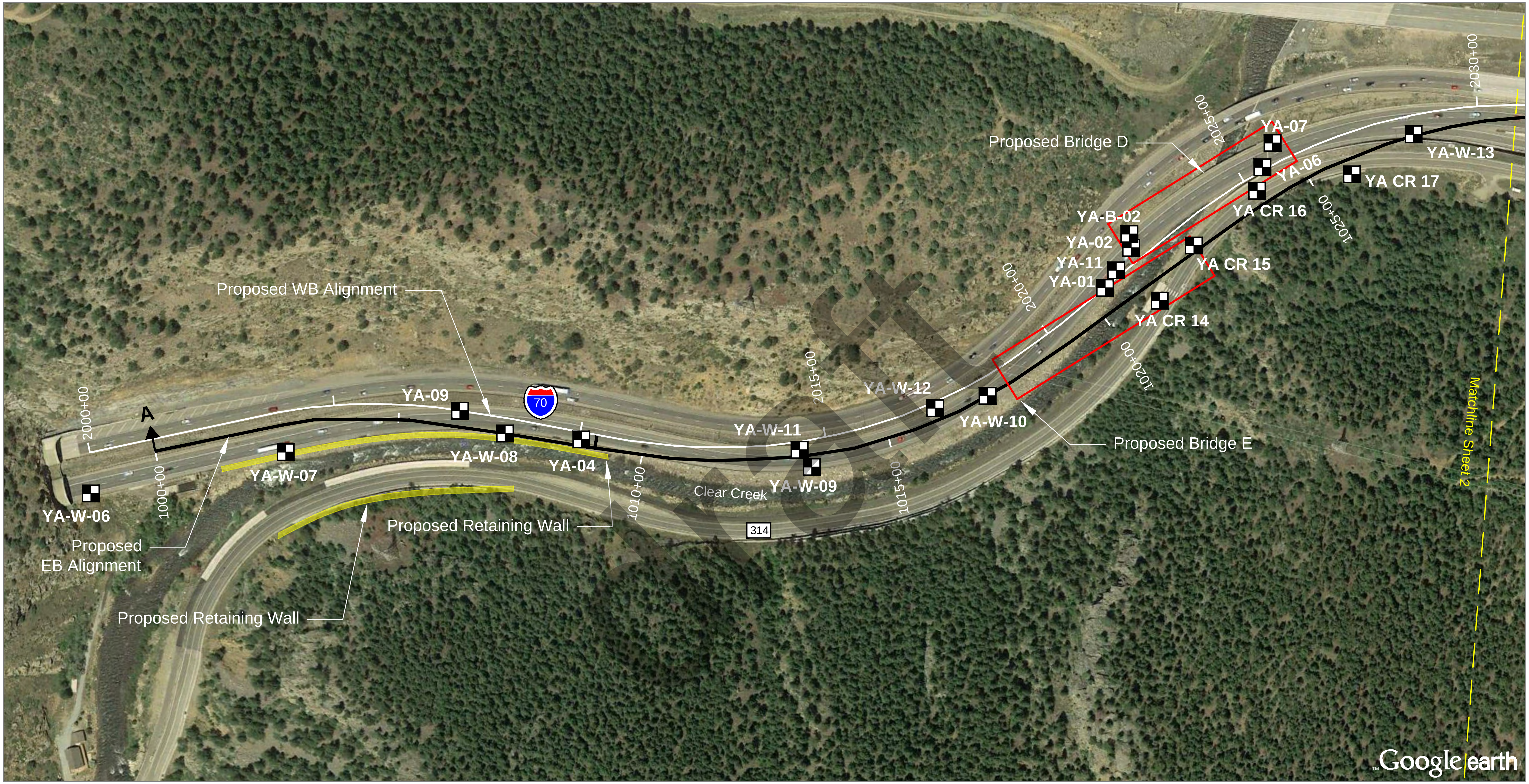
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

**FIG. 1**

Filename: I:\EF\DEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F1.dwg Date: 01-09-2019 Login: dav



LEGEND

| | | |
|---|---|---|
| SW-1-01 | ◕ | Current Boring Designation and Approximate Location |
| SW-1-04 | ⊙ | Current SAA Designation and Approximate Location |
| YA-B-1 | ▚ | Yeh & Associates Boring Designation and Approximate Location (2008, 2014) |
| B-6-1 | ⊕ | CDOT Boring Designation and Approximate Location (see report for references) |

A A' Profile Designation and Approximate Location

0 200 400
Approximate Scale in Feet

NOTES

1. Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth™ Mapping Service.
2. Locations of borings by others should be considered approximate and were estimated based on available documents.
3. Alignment stationing adopted from Atkins files Opt01-I70-WB.dgn and Opt01-I70-EB.dgn.
4. Proposed structure locations are shown schematically based on information provided by Wood.

N

I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

## SITE AND EXPLORATION PLAN

January 2019
100864-001

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**FIG. 2**
Sheet 1 of 5

Filename: I:\EF\DEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F2.dwg Date: 01-15-2019 Login: dav

## LEGEND

| | | |
|---|---|---|
| SW-1-01 | ◐ | Current Boring Designation and Approximate Location |
| SW-1-04 | ⊙ | Current SAA Designation and Approximate Location |
| YA-B-1 | ▚ | Yeh & Associates Boring Designation and Approximate Location (2008, 2014) |
| B-6-1 | ⊕ | CDOT Boring Designation and Approximate Location (see report for references) |

A A' Profile Designation and Approximate Location

0 200 400

Approximate Scale in Feet

## NOTES

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N

I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

### SITE AND EXPLORATION PLAN

January 2019 100864-001

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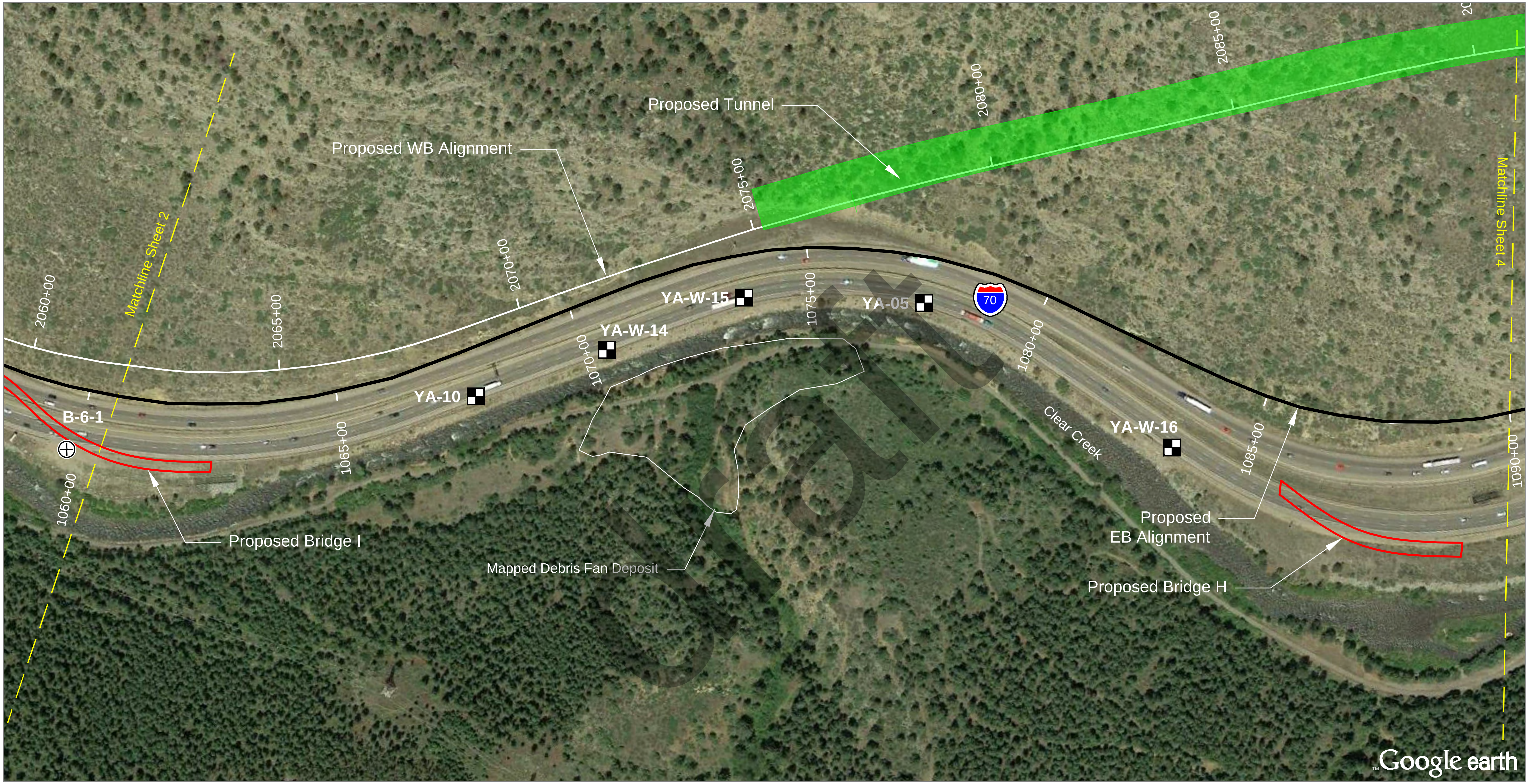
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 2
Sheet 2 of 5

Filename: I:\EF\DEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F2.dwg Date: 01-15-2019 Login: dav



LEGEND

| | |
|---|---|
| **SW-1-01** | Current Boring Designation and Approximate Location |
| **SW-1-04** | Current SAA Designation and Approximate Location |
| **YA-B-1** | Yeh & Associates Boring Designation and Approximate Location (2008, 2014) |
| **B-6-1** | CDOT Boring Designation and Approximate Location (see report for references) |
| A A' | Profile Designation and Approximate Location |

0 200 400
Approximate Scale in Feet

NOTES

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N

I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

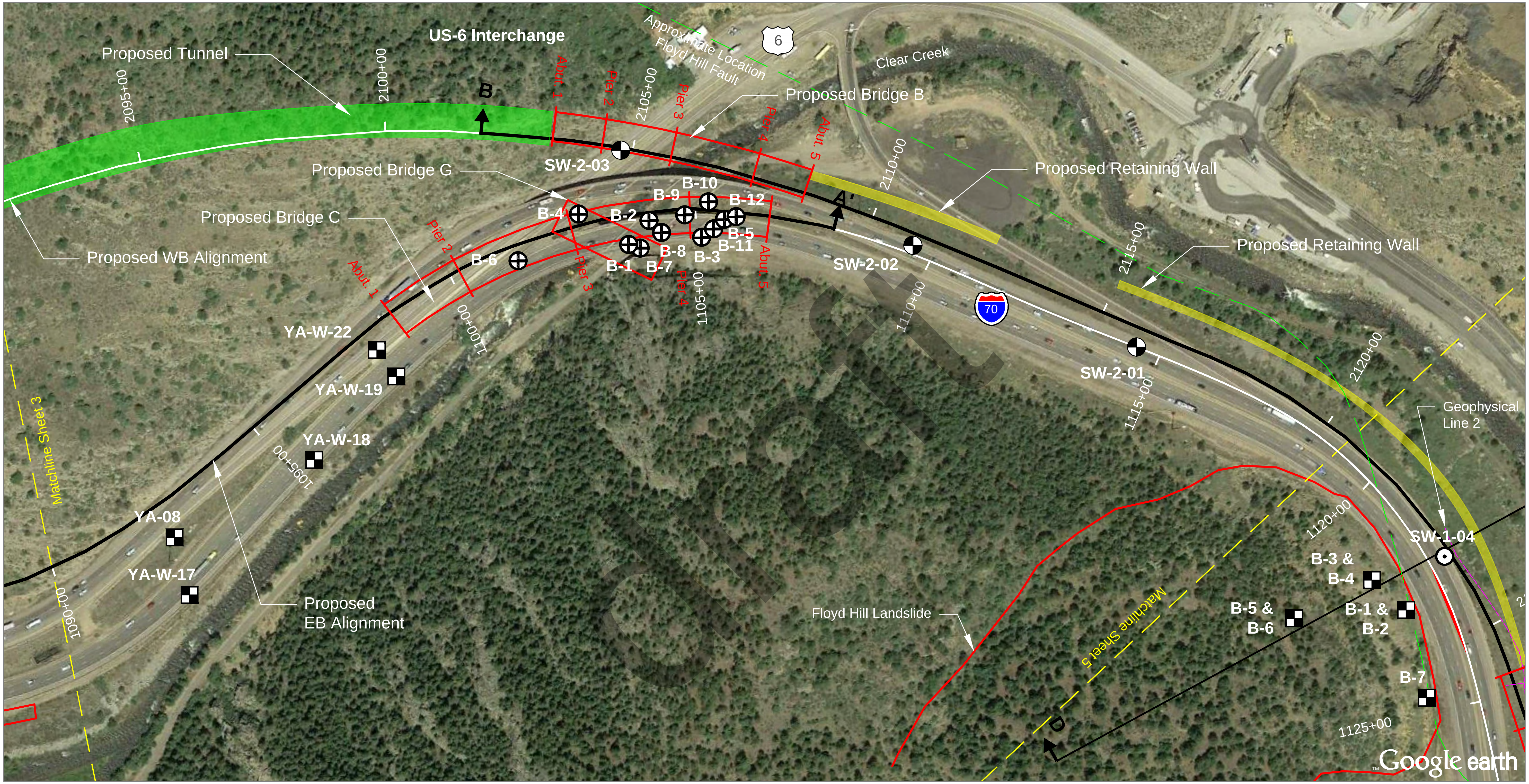
## SITE AND EXPLORATION PLAN

January 2019
100864-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

**FIG. 2**
Sheet 3 of 5

Filename: I:\EF\DEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F2.dwg Date: 01-15-2019 Login: dav



## LEGEND

| | |
|---|---|
| **SW-1-01** ◕ | Current Boring Designation and Approximate Location |
| **SW-1-04** ⊙ | Current SAA Designation and Approximate Location |
| **YA-B-1** ▞ | Yeh & Associates Boring Designation and Approximate Location (2008, 2014) |
| **B-6-1** ⊕ | CDOT Boring Designation and Approximate Location (see report for references) |
| **A** ⌊ **A'** | Profile Designation and Approximate Location |

0 200 400
Approximate Scale in Feet

## NOTES

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I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

## SITE AND EXPLORATION PLAN

January 2019
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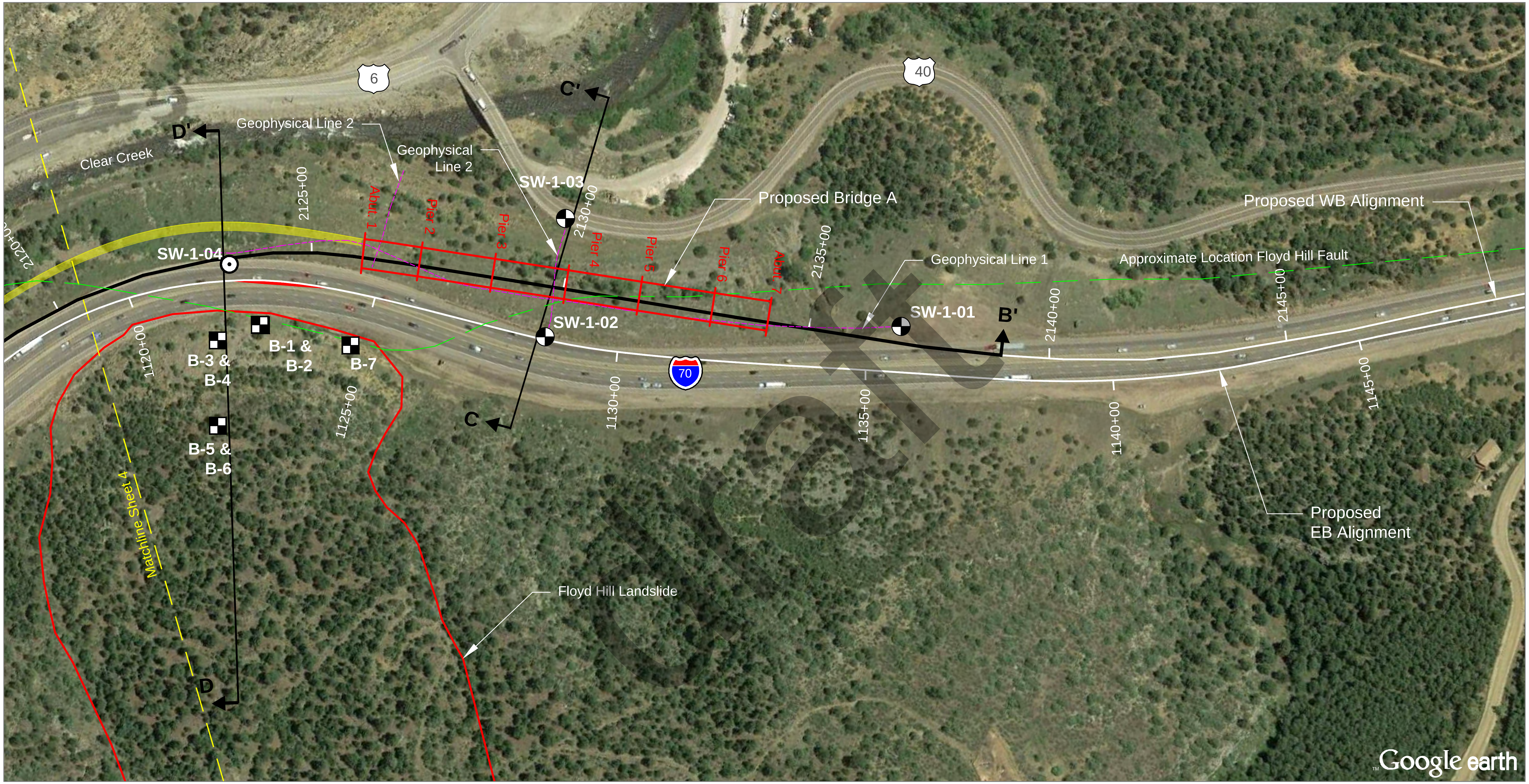
FIG. 2
Sheet 4 of 5

Filename: I:\EF\DEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F2.dwg Date: 01-15-2019 Login: dav



LEGEND

| | | |
|---|---|---|
| SW-1-01 | ◐ | Current Boring Designation and Approximate Location |
| SW-1-04 | ⊙ | Current SAA Designation and Approximate Location |
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A A' Profile Designation and Approximate Location

N

0 200 400
Approximate Scale in Feet

NOTES

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Preliminary Design
Clear Creek County, Colorado

## SITE AND EXPLORATION PLAN

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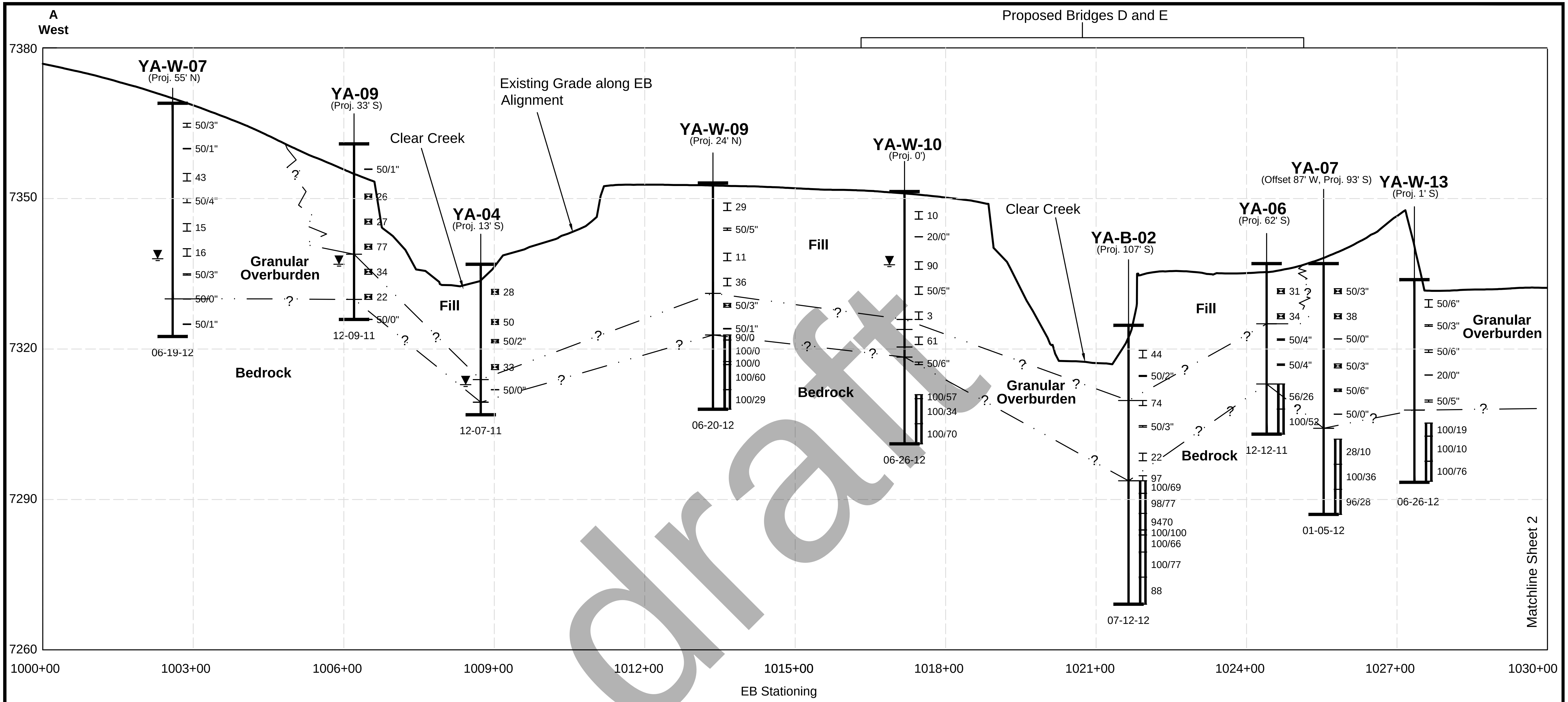
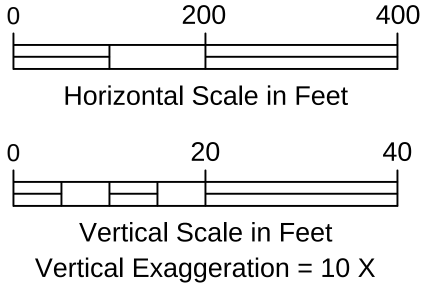
FIG. 2
Sheet 5 of 5

Filename: I:\EF\DEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F2.dwg Date: 01-15-2019 Login: dav



NOTES

1. Ground surface and alignment stationing was developed from Atkins file Opt01-I70-EB.dgn.
2. See Figure 2 for location of subsurface profile in plan view.
3. This subsurface profile is generalized from materials observed in borings. Variations may exist between profile and actual conditions.
4. See Preliminary Geotechnical Report for generalized description of geologic units shown on profile.



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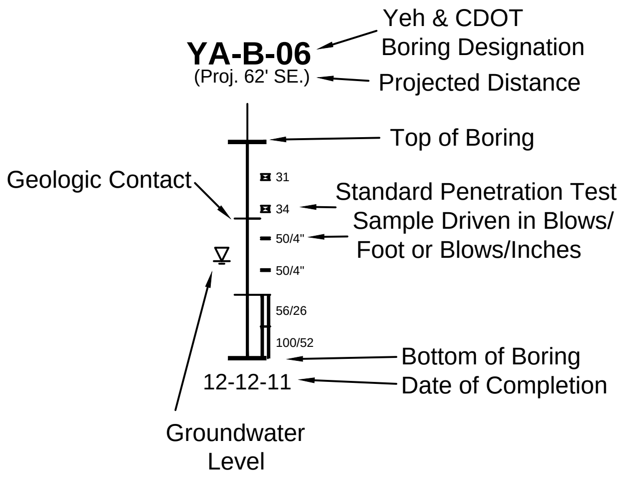

I-70 Floyd Hill
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Clear Creek County, Colorado

## GENERALIZED SUBSURFACE PROFILE A - A'

January 2019 100864-001

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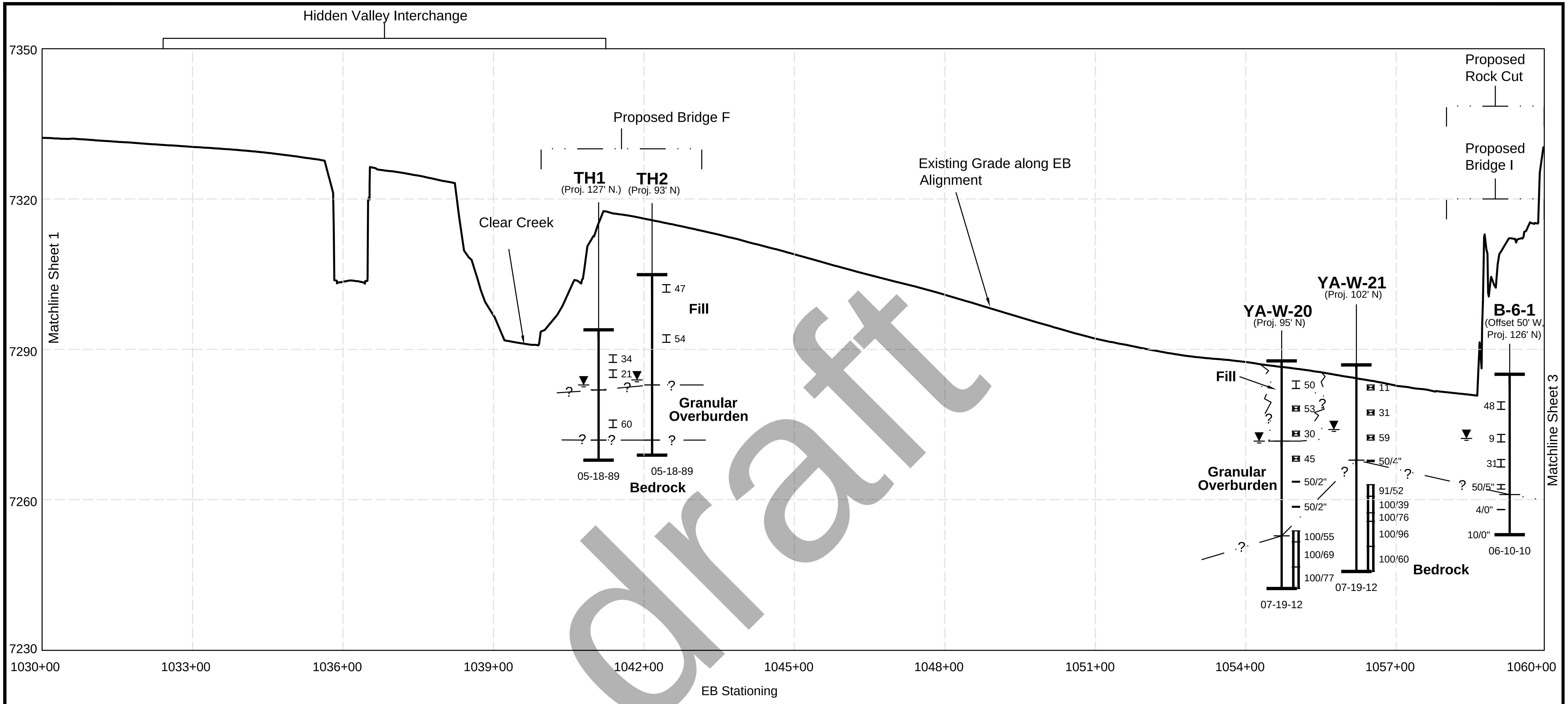
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 3
Sheet 1 of 4

Filename: I:\EF\DEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F3.dwg Date: 11-09-16 Login: ----



Filename: I:\EF\DEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F3.dwg  Date: 11-09-16  Login: ----

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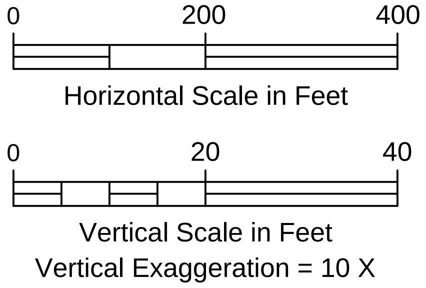

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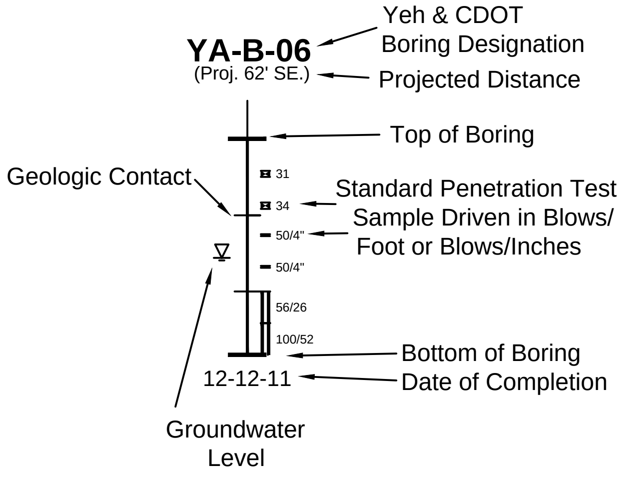


I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

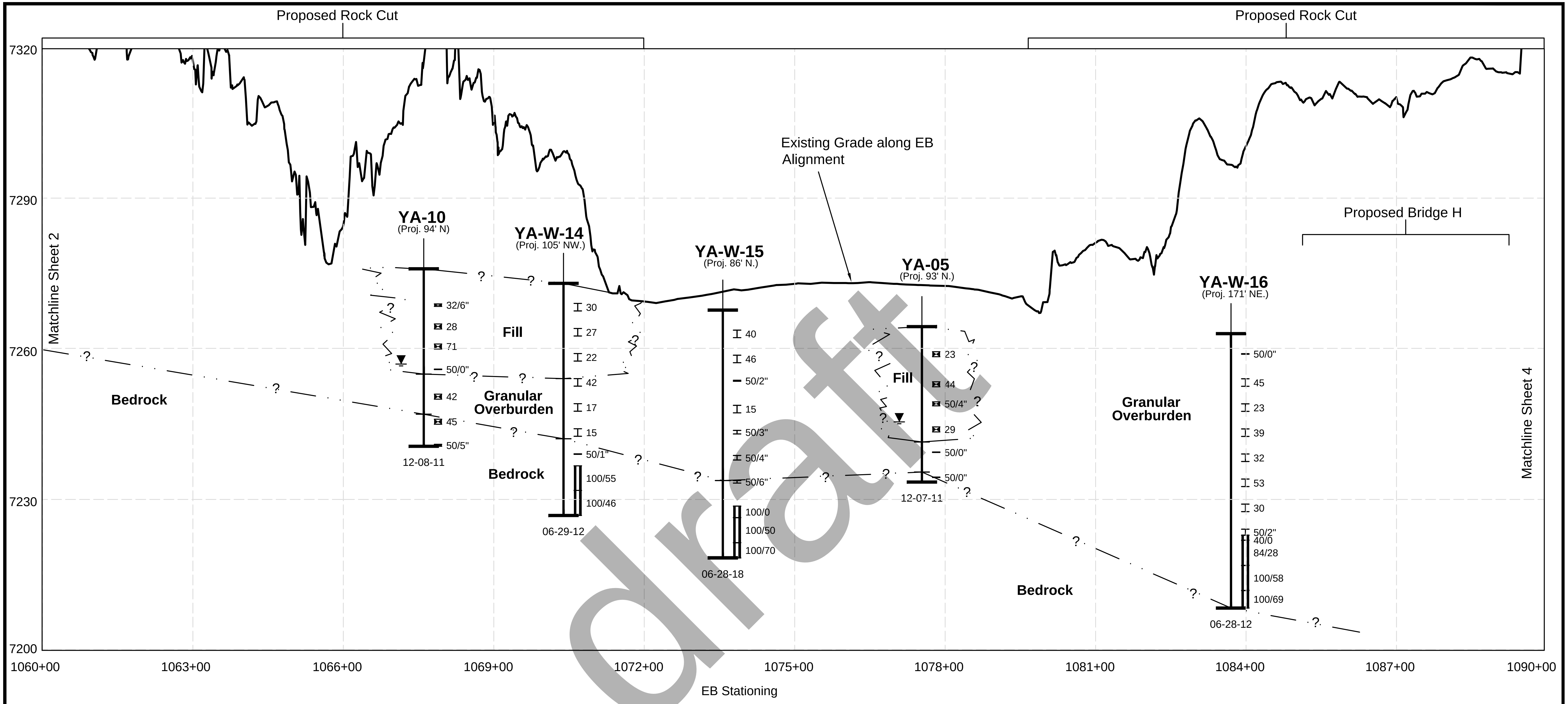
**GENERALIZED SUBSURFACE PROFILE A - A'**

January 2019    100864-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

**FIG. 3**
Sheet 2 of 4

Filename: I:\EFIDEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F3.dwg Date: 11-09-16 Login: ----



NOTES

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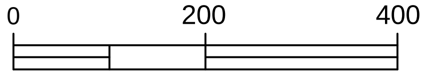
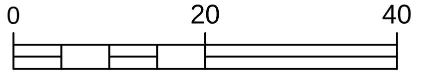
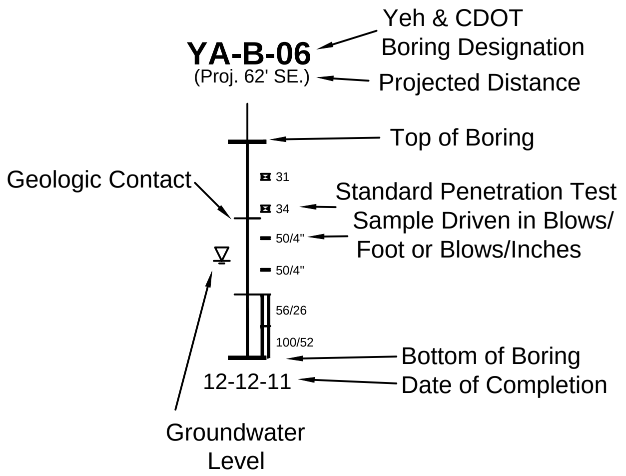


Horizontal Scale in Feet



Vertical Scale in Feet
Vertical Exaggeration = 10 X

LEGEND



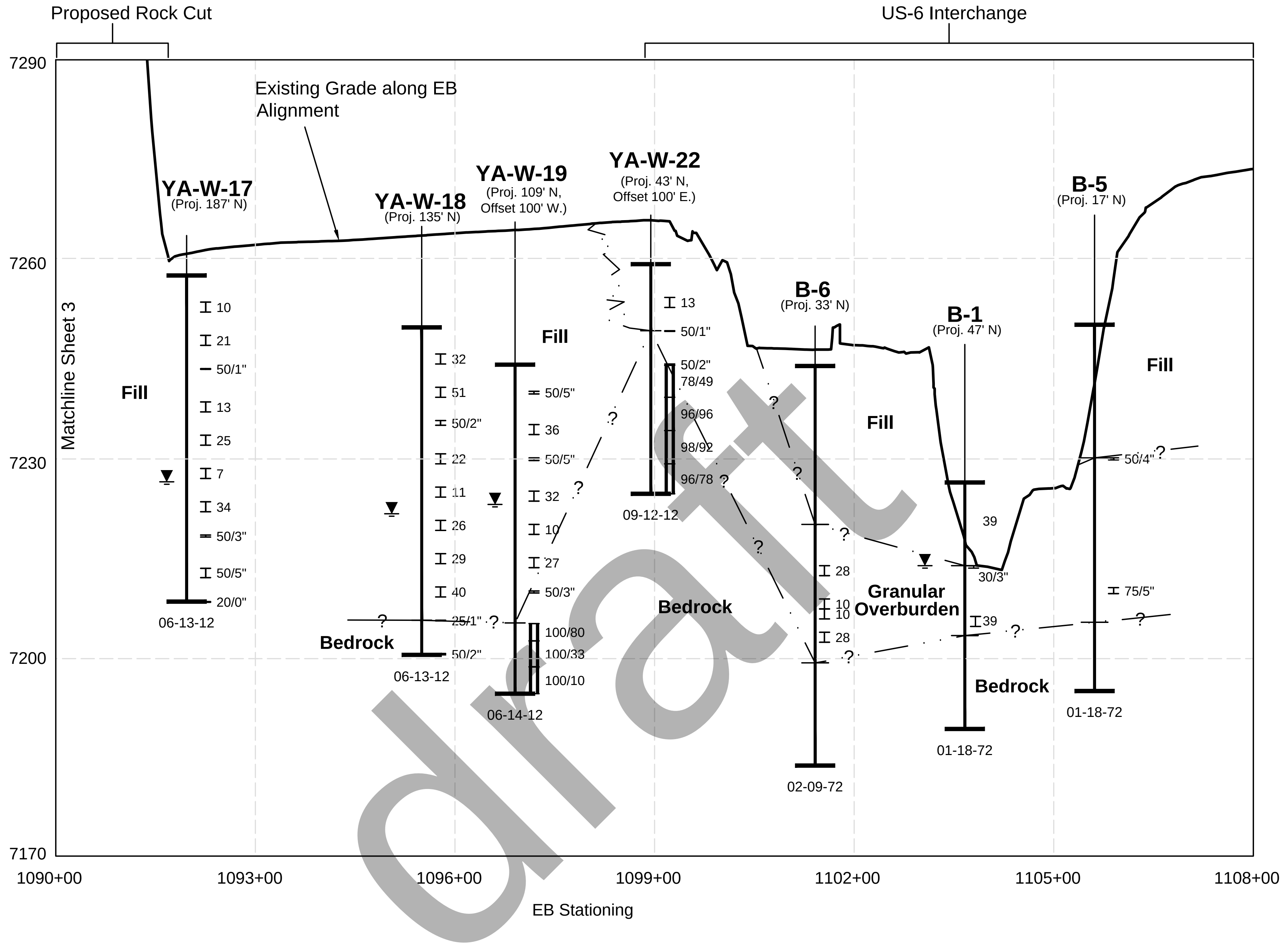
I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

**GENERALIZED SUBSURFACE PROFILE A - A'**

January 2019 100864-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

**FIG. 3**
Sheet 3 of 4



NOTES

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2. See Figure 2 for location of subsurface profile in plan view.
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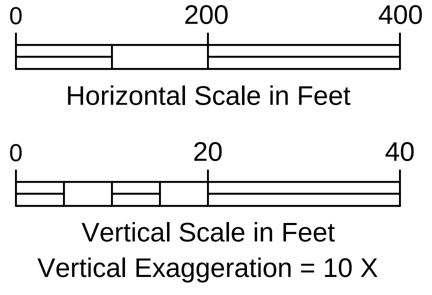
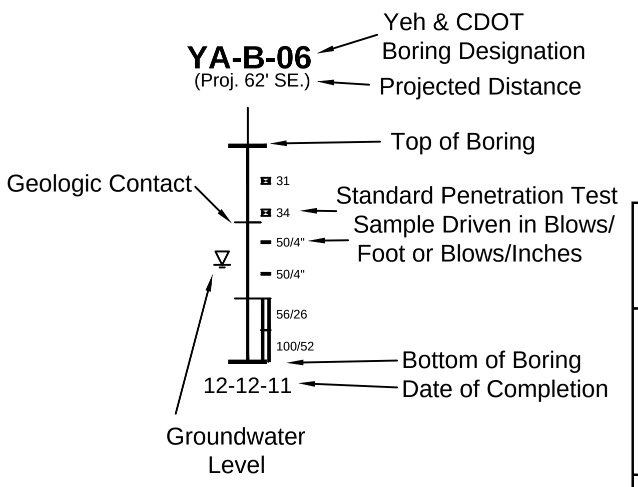


LEGEND



I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

## GENERALIZED SUBSURFACE PROFILE A - A'

January 2019 100864-001

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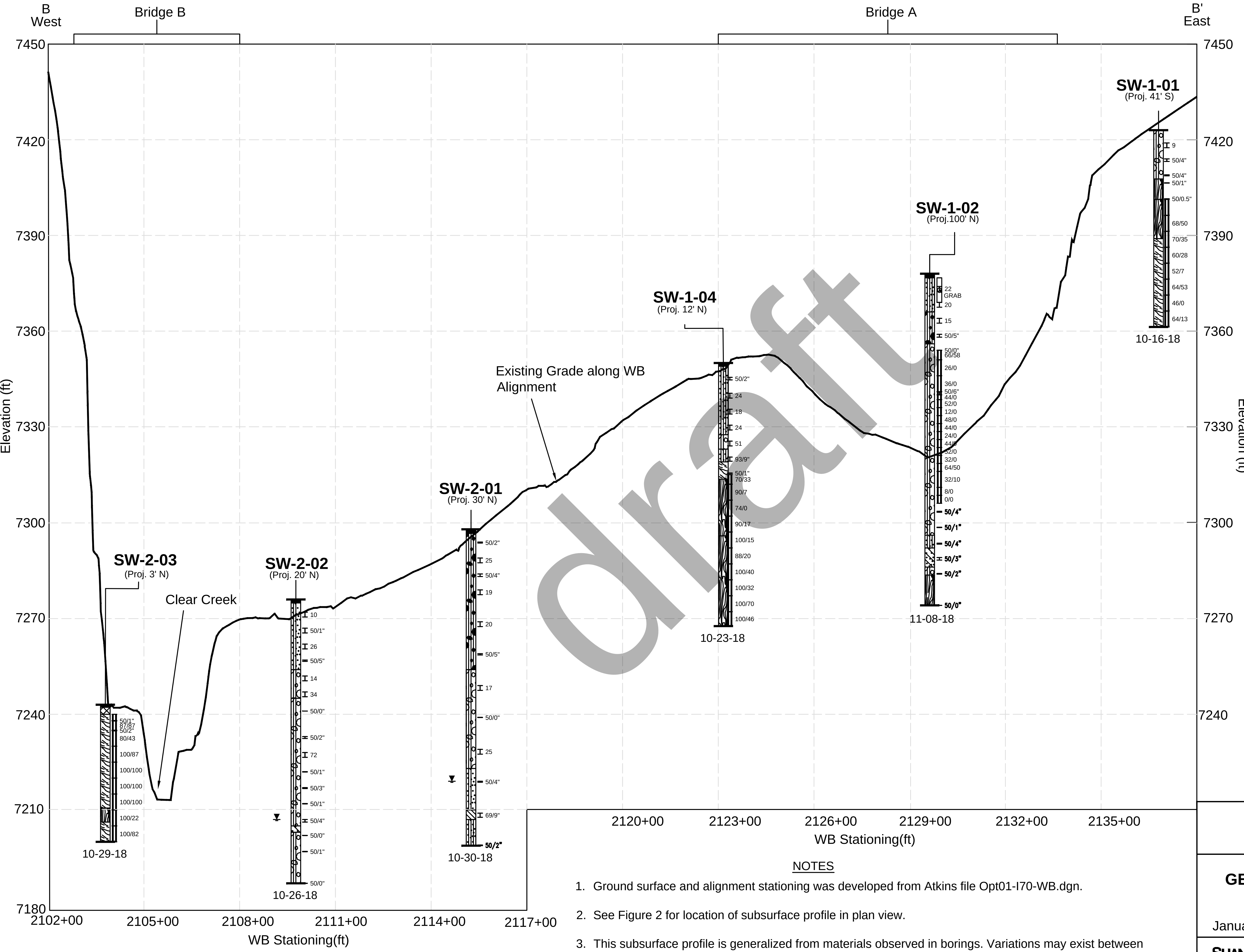
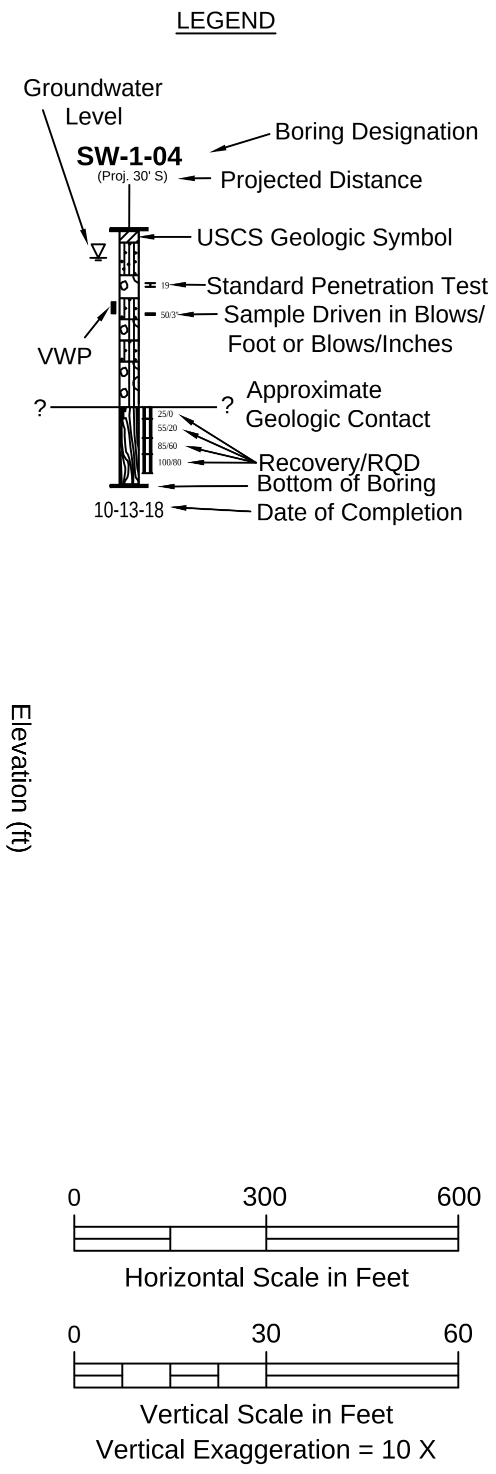
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

**FIG. 3**
Sheet 4 of 4





## NOTES

1. Ground surface and alignment stationing was developed from Atkins file Opt01-I70-WB.dgn.
2. See Figure 2 for location of subsurface profile in plan view.
3. This subsurface profile is generalized from materials observed in borings. Variations may exist between profile and actual conditions.

I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

## GENERALIZED SUBSURFACE PROFILE B - B'

January 2019
100864-001

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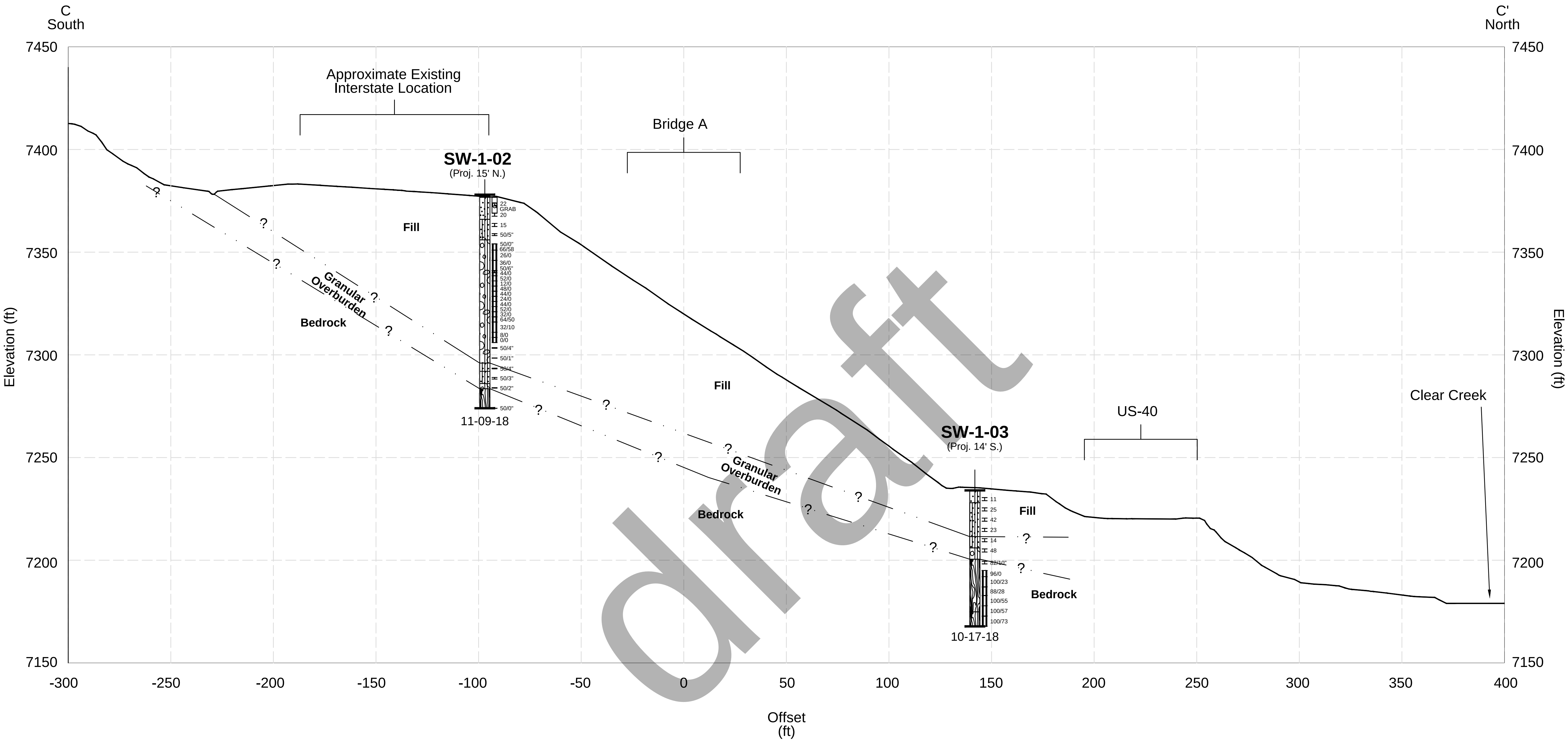
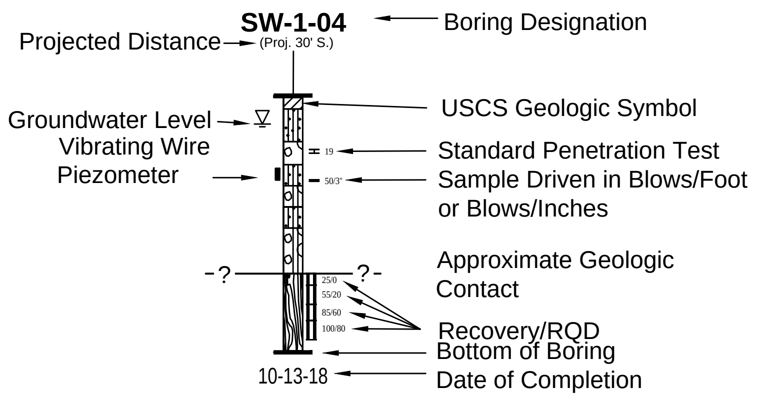
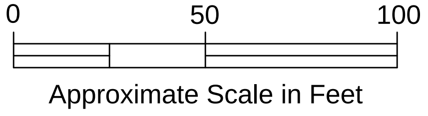
**FIG. 4**

Filename: I:\EF\DEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F4.dwg Date: 11-09-16 Login: ----



LEGEND





NOTES

1. Surface profile developed from Atkins file 21912DES_Model-Option 01.dgn.
2. See Figure 2 for location of subsurface profile in plan view.
3. This subsurface profile is generalized from materials observed in borings. Variations may exist between profile and actual conditions.
4. See Preliminary Geotechnical Report for generalized description of geologic units shown on profile.

I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

## GENERALIZED SUBSURFACE PROFILE C - C'

January 2019 100864-001

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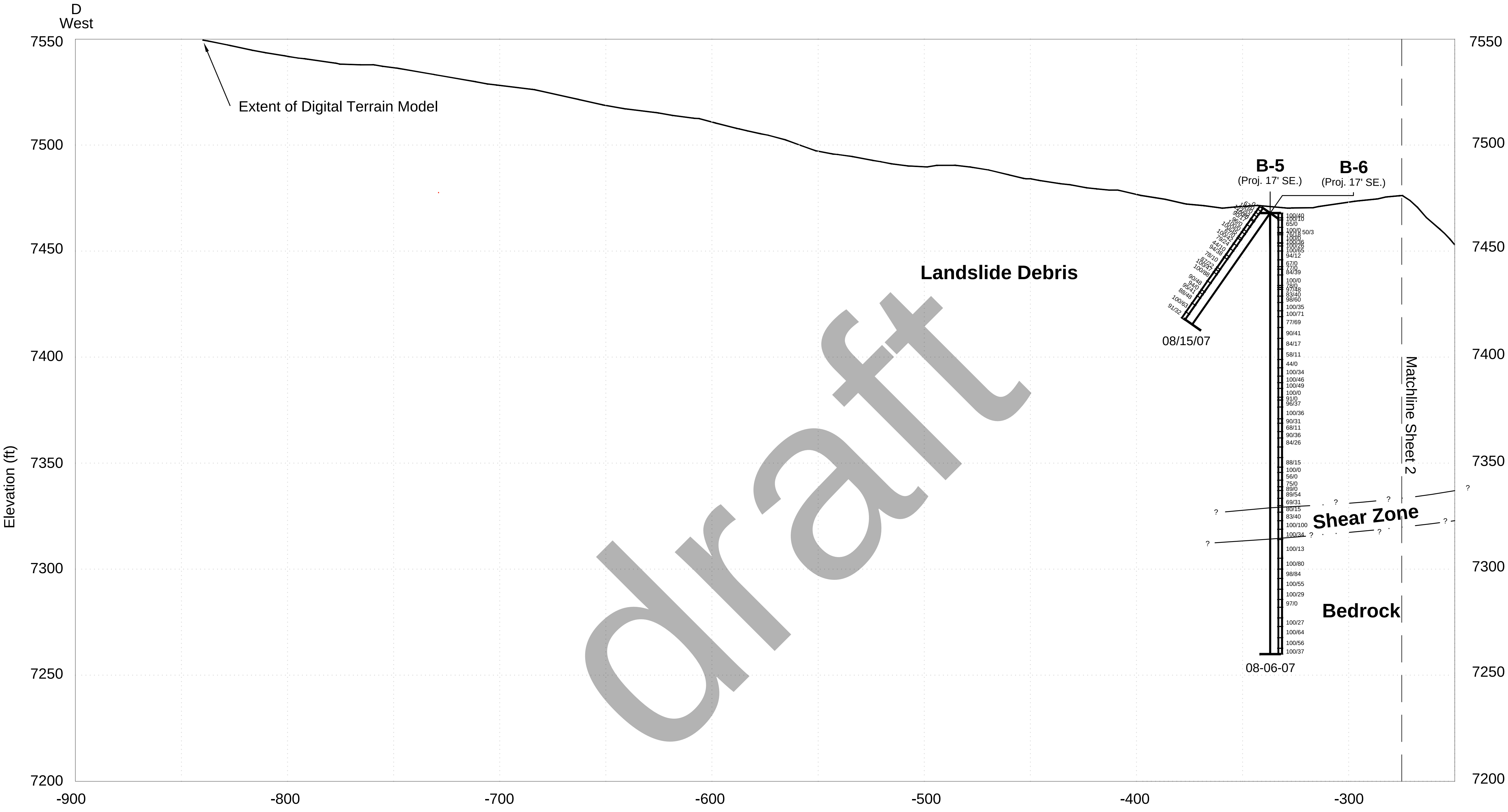
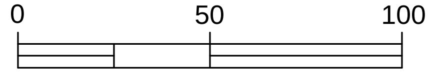
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

**FIG. 5**

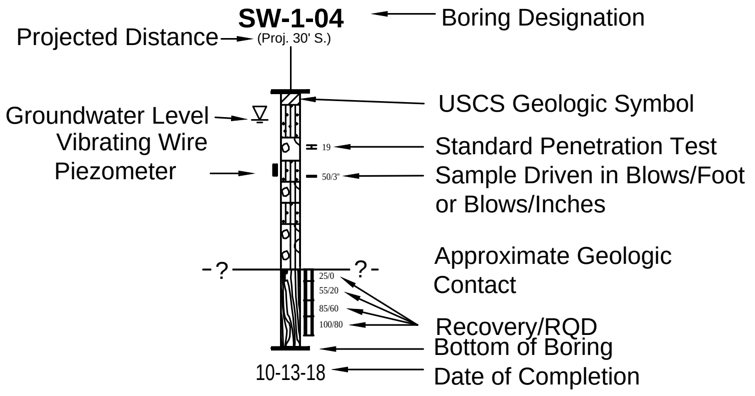
Filename: I:\EF\DEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F5.dwg Date: 01-15-2019 Login: dav

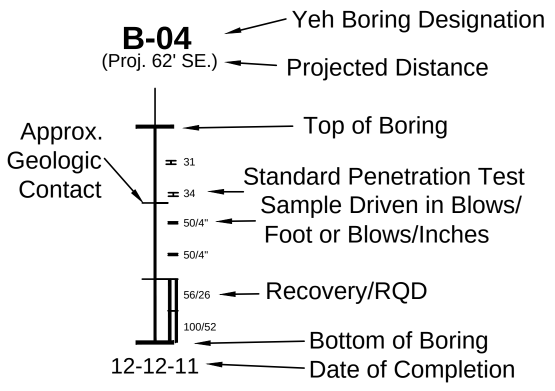




Approximate Scale in Feet

LEGEND





NOTES

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3. This subsurface profile is generalized from materials observed in borings. Variations may exist between profile and actual conditions.
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I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

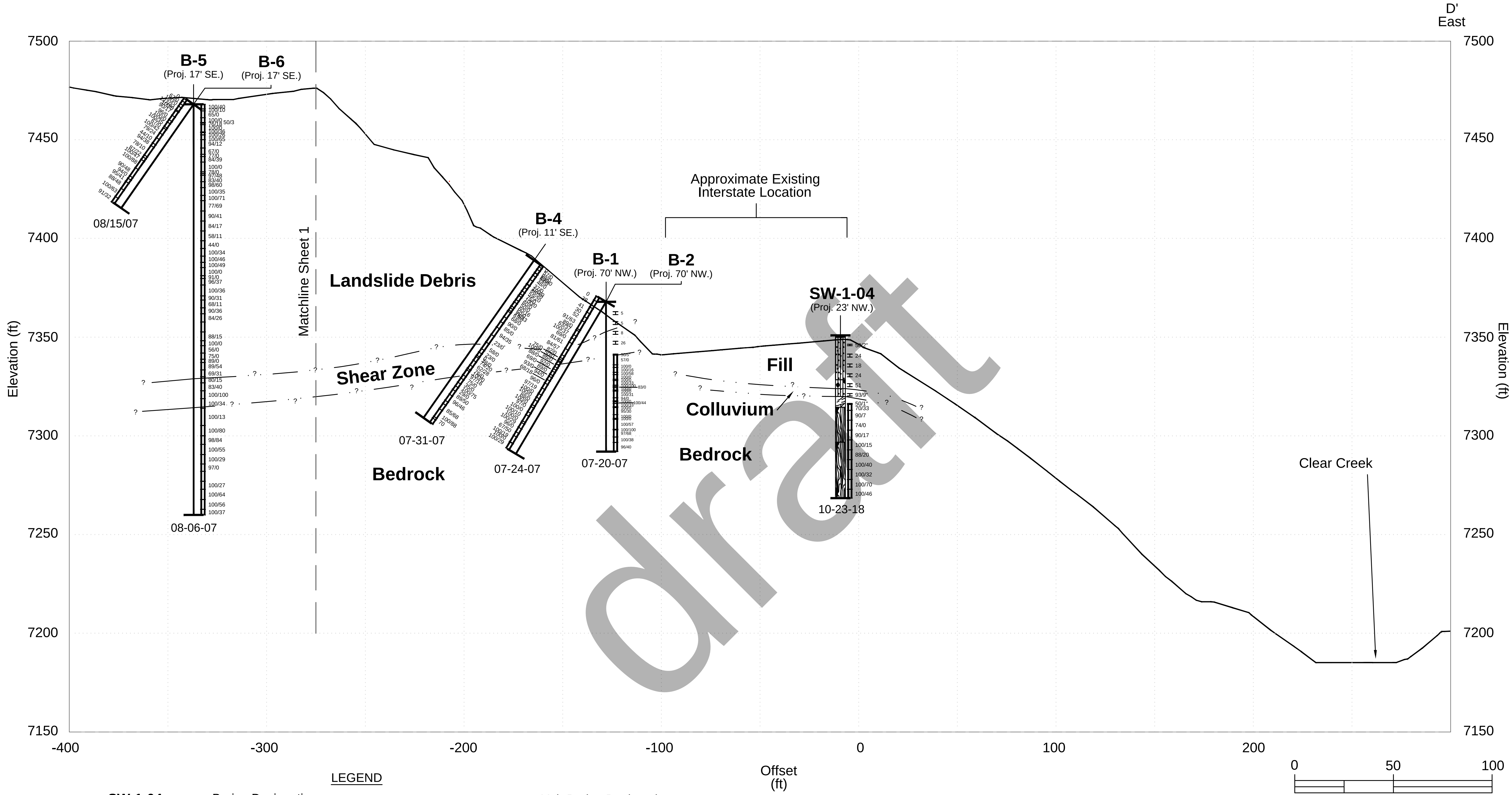
## GENERALIZED SUBSURFACE PROFILE D - D'

January 2019
100864-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

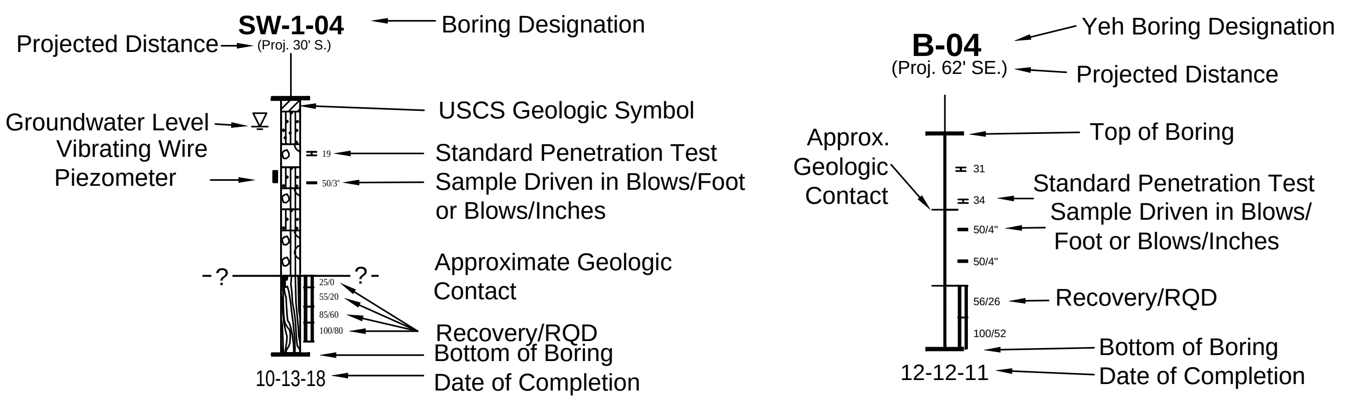
**FIG. 6**
Sheet 1 of 2

Filename: I:\EF\DEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F6.dwg Date: 01-15-2019 Login: dav



0 50 100
Approximate Scale in Feet

LEGEND



## NOTES

1. Surface profile developed from Atkins file 21912DES_Model-Option 01.dgn.
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I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

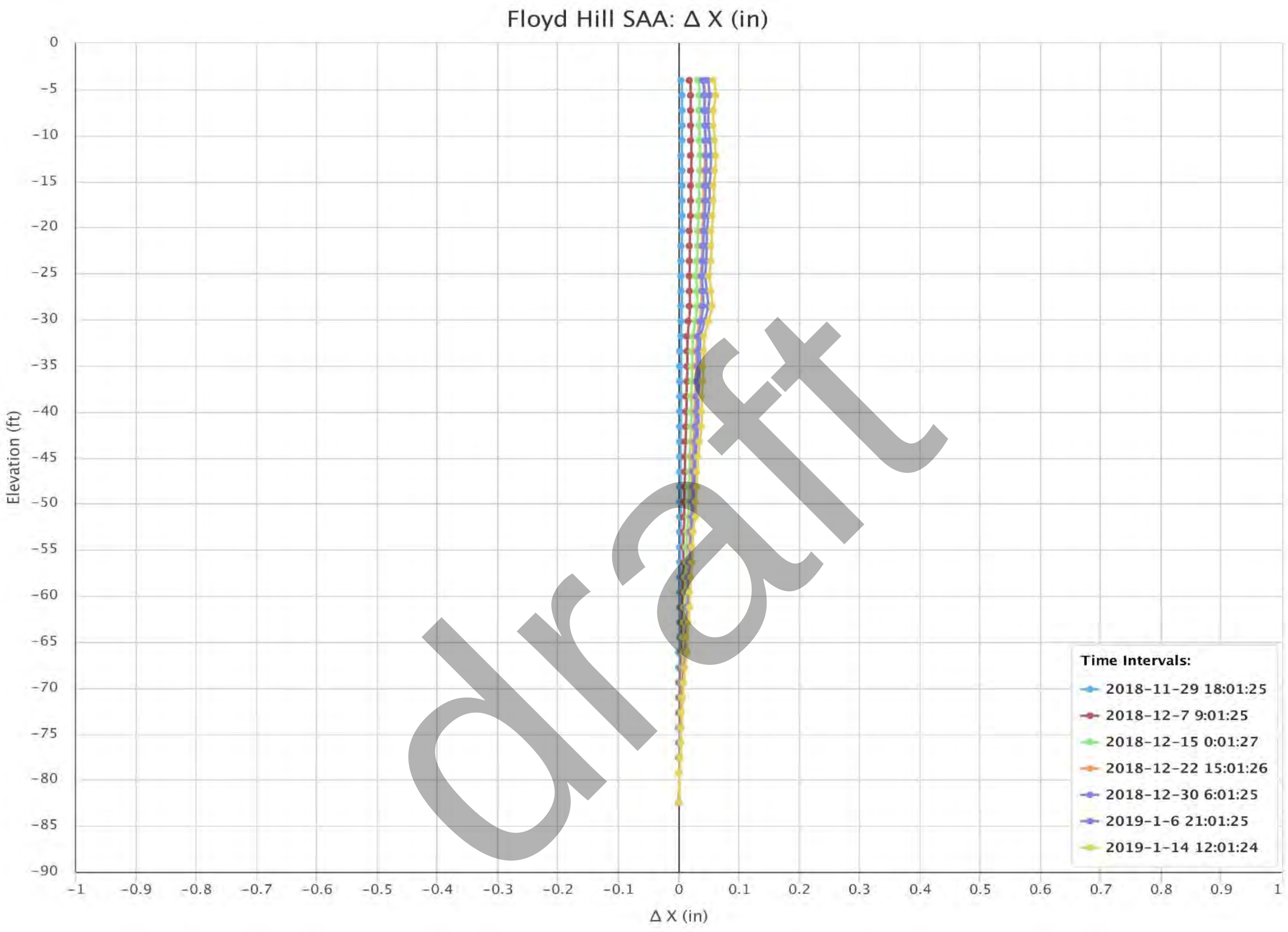
## GENERALIZED SUBSURFACE PROFILE D - D'

January 2019 100864-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

**FIG. 6**
Sheet 2 of 2

Filename: I:\EF\DEN\100000s\100864 Floyd Hill Prel\Drafting\100864-001_F6.dwg Date: 01-15-2019 Login: dav



**NOTES**

1. X-direction is oriented approximately perpendicular to the contours of the slope. Y-direction is oriented perpendicular to x-direction.

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Preliminary Design
Clear Creek County, Colorado

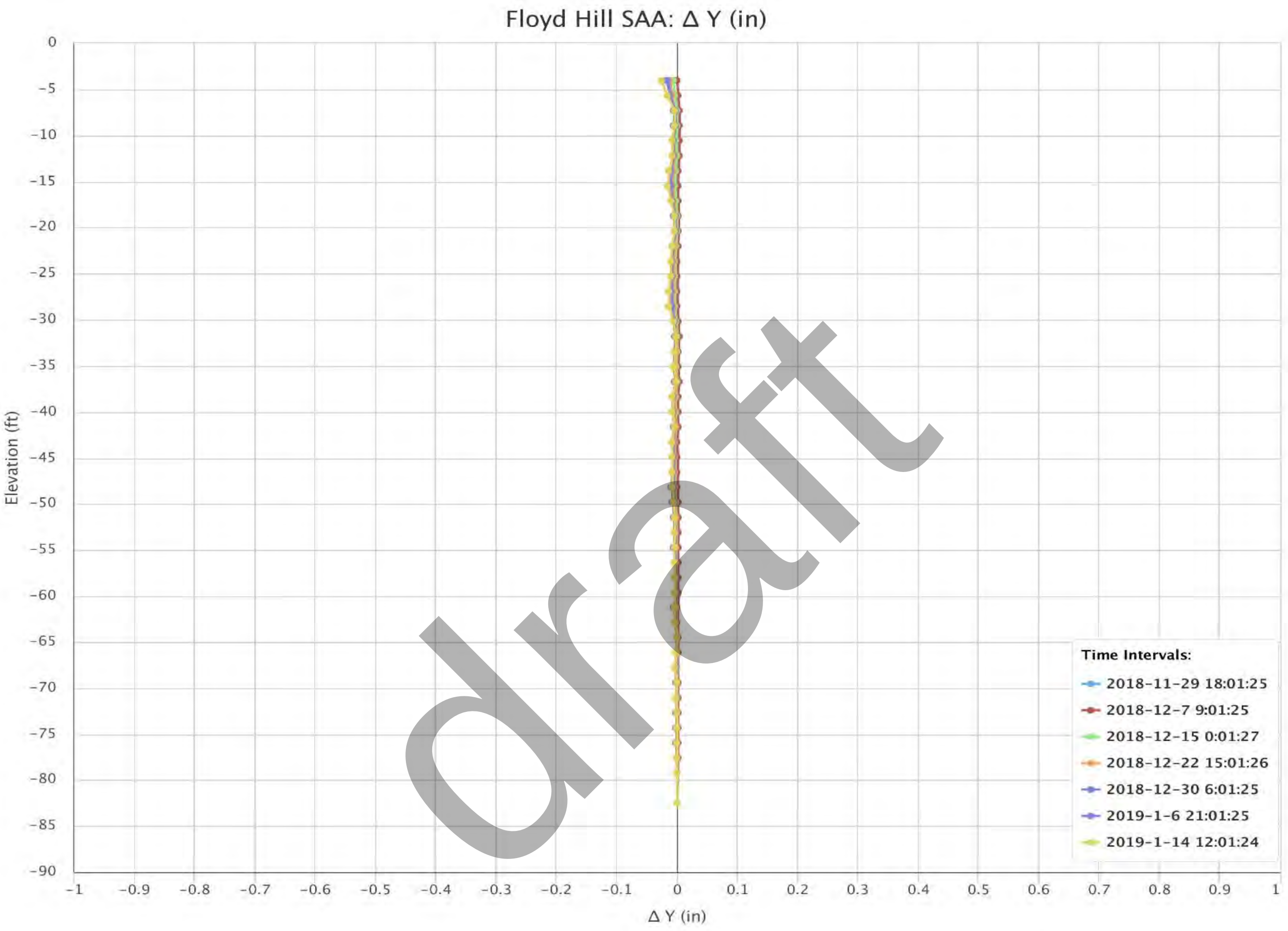
**BORING SW-1-04 SAA CUMULATIVE DISPLACEMENT**

January 2019
100864-001

**SHANNON & WILSON, INC.**
Geotechnical and Environmental Consultants

**FIG. 7**
Sheet 1 of 2

**FIG. 7**
Sheet 1 of 2



**NOTES**

1. X-direction is oriented approximately perpendicular to the contours of the slope. Y-direction is oriented perpendicular to x-direction.

I-70 Floyd Hill
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**FIG. 7**
Sheet 2 of 2

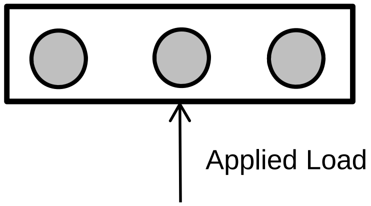
| Group Type | Shaft Spacing (Dia.) | Recommended P-Multipliers | | | |
|---|---|---|---|---|---|
| | | Loading Type-1 Row 1 | Loading Type-2 | | |
| | | | Row 1 | Row 2 | Row 3 and higher |
| Groups with 1 Row | 2.0D | 0.54 | 1.00 | 0.33 | 0.25 |
| | 3.0D | 0.80 | | 0.50 | 0.38 |
| | 5.0D | 1.00 | | 0.85 | 0.70 |

| Group Type | Shaft Spacing (Dia.) | Recommended P-Multipliers | | |
|---|---|---|---|---|
| | | Loading Type-3 | | |
| | | Row 1 | Row 2 | Row 3 and higher |
| Groups with 2 or more Rows | 2.0D | 0.53 | 0.27 | 0.20 |
| | 3.0D | 0.80 | 0.40 | 0.30 |
| | 5.0D | 1.00 | 0.85 | 0.70 |

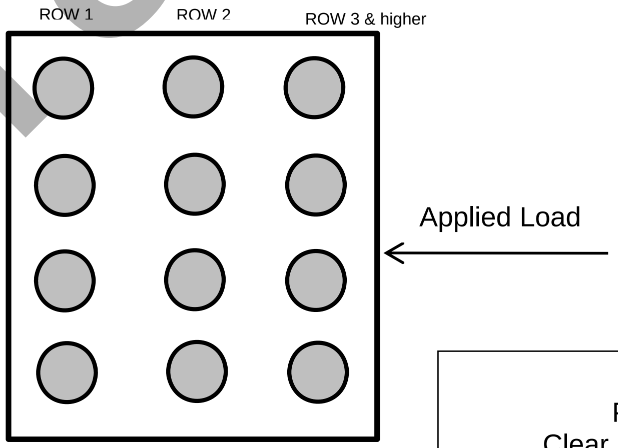
NOTES:

1. Linear interpolation can be used to calculate the P-Multipliers for the shaft spacings which are not listed above.
2. P-Multipliers are based on AASHTO LRFD Bridge Design Specifications (2017).
3. The P-Multipliers should be applied to the load portion of the P-Y curve in the LPILE software.

LOADING TYPE - 1



LOADING TYPE - 3



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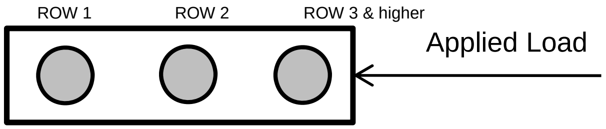

I-70 Floyd Hill
Preliminary Design
Clear Creek County, Colorado

## RECOMMENDED P-MULTIPLIERS FOR GROUP EFFECTS

January 2019
100864-001

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**FIG. 8**

**FIG. 8**

# Draft

# Technical Memorandum

# WB & EB I-70 Rock Cut Evaluation

# Floyd Hill, Clear Creek County, Colorado

Yeh Project No.: 218-300

January 29, 2019
Prepared for:

Atkins
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Yeh and Associates, Inc.
Consulting Engineers & Scientists

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# 1. PURPOSE AND SCOPE

This technical memorandum presents our preliminary geotechnical engineering investigation to support design decisions for the proposed westbound (WB) and eastbound (EB) Interstate 70 (I-70) rock cuts related to the proposed Floyd Hill Tunnel highway realignment. The study was performed in accordance with project constraints and our proposal to Atkins dated July 2, 2018.

## 1.1 Purpose of Work

The I-70 Floyd Hill Tunnel Project is proposed to improve highway safety and capacity from the Beaver Brook Interchange to the Veterans Memorial Tunnels. Improvements include a westbound bridge structure near the bottom of Floyd Hill, an approximately 2,200-ft tunnel traveling westbound from the base of Floyd Hill, and multiple rock cuts for both eastbound and westbound highway re-alignments.

## 1.2 Scope of Work

Project limits for this scope are bounded by the East Portal of the proposed Floyd Hill Tunnel, approximately CDOT Milepost 244.3 to Milepost 243.2. It is not included in this scope, but Yeh's previous experience and knowledge of the I-70 mountain corridor will provide supplemental information. The scope of work included the following:

- Field reconnaissance including reviewing published maps and reports, performing an aerial survey investigation and surficial geologic mapping.
- Rock slope mapping along the existing I-70 rock cut and along the entire slope using UAV (still in progress)
- Technical Memorandum that presents subsurface characterization based on data collected during the investigation. The information presented in the memorandum(a) will ultimately be incorporated into Preliminary Geotechnical Data Report for the proposed rock cuts along I-70.

# 2. BACKGROUND INFORMATION

The purpose of this I-70 Floyd Hill project is to perform a preliminary feasibility assessment of the tunnel portal and highway curve reduction areas and support highway design decisions. The rock cut project limits extend from the base of Floyd Hill, a long steep incline with a curving bridge at the bottom, to immediately east of the Hidden Valley/Central City Parkway interchange at I-70 exit 243. Continuing west from the base of Floyd Hill, which routinely experiences traffic backups ~~c~~onditions, the highway follows the contours of the slope above Clear Creek along the base of steep rock cuts. As shown in Figure 1 below, the approximate rock cut areas are intended to reduce highway curves and allow for additional lane capacity for both westbound and eastbound travelers.

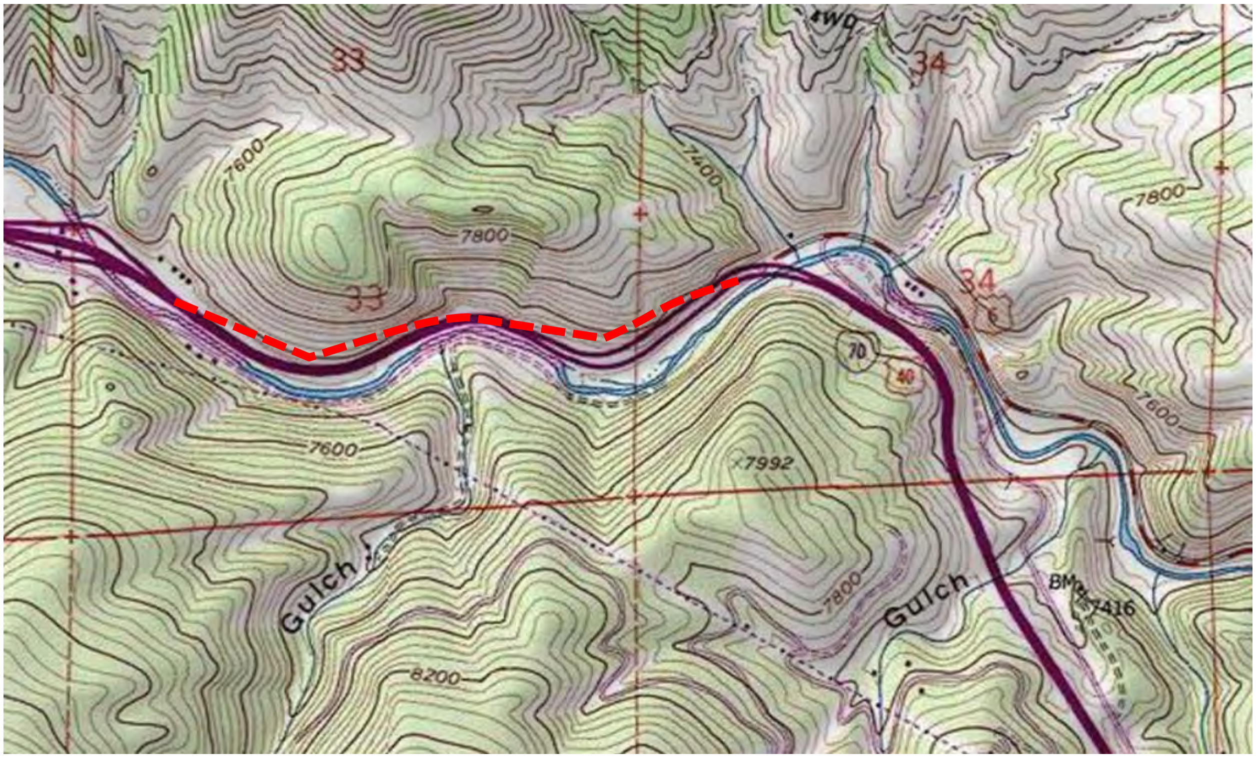


**Figure 1. I-70 Floyd Hill Curve Straightening Project Location (proposed alignment in red)**

## 2.1 Geologic Site Conditions

A wide range of geologic conditions are represented and exposed along the I-70 corridor due to the extensive period of time represented in the multiple rock formations. The geologic time reflected along the corridor ranges from recent river and debris flow deposits to Precambrian rocks between 1 and 2 billion years old. The Precambrian age metamorphic and igneous rocks are intruded by Precambrian, Tertiary and Cretaceous age stocks and numerous porphyritic dikes. The regional rock type of most relevance to the rock cuts is biotite gneiss identified in Figure 2. The most common porphyries range in composition from Precambrian pegmatite and lamprophyres to Cretaceous quartz monzonite and granodiorite.

Most of the present configuration of the area is characterized by moderately rugged topographic relief. The mountains to the south and north are deeply incised by Clear Creek Canyon and its tributaries. The maximum local relief is about 3,000 feet. The elevation in the project area ranges from slightly over 7,000 feet along Clear Creek to more than 10,000 feet at Santa Fe Mountain to the southwest. Slopes are typically steep, averaging approximately 35 degrees on the proposed east portal rock face. Topographic forms are generally influenced by minor faulting, fractures, and zones of weakness in rock. In addition, rain, snowmelt, freeze-thaw, wind and Clear Creek have created deposits of alluvium (stream deposits), talus (rockfall deposits) and alluvial fans.

Bedrock in the project area is primarily Precambrian metamorphic gneiss and migmatite. Biotite gneiss is the predominant mapped bedrock along the proposed tunnel alignment, however the geologic map includes feldspar gneiss, hornblende gneiss, calc-silicate gneiss, and amphibolite. Locally the bedrock is also known as migmatite, a composite rock consisting of igneous and metamorphic portions. In Colorado migmatite is generally a blend of quartz pegmatite or granite intruded into a metamorphic host rock and intensely deformed. Biotite gneiss in the immediate area of the proposed tunnel is generally light gray to medium gray in color and fine to medium grained. Precambrian pegmatite dikes, lamprophyre dikes, and Cretaceous quartz monzonite porphyry or granodiorite porphyry dikes are also mapped in the project area.

Some of the rock exposed in the existing cut along I-70 shows up to 1-ft thick parallel layering while some of the rock is more schistose with foliated biotite seams or banded bedrock. In many places, the rock is highly deformed by folding and faulting. Intense localized folding is commonly found in the project area and is typical of migmatite formations. Measurements of these folds is not feasible or indicative of regional tectonic movement. Pegmatite and folding is common in the migmatite/gneiss which permitted vertical and horizontal blasting techniques for the previous highway excavation as shown in Figure 3. The density and orientation of fracturing is highly variable in the studied tunnel areas, however the general rock structure follows the same orientation as the schistose banding. Banding generally trends north-northeast, and lineations generally plunge 45-55 degrees to the north-northwest based on the (Squaw Pass Quadrangle geologic map quad).

The west portal area is covered by Pleistocene alluvium. The alluvium ranges from gray to brown in color and consists of sand and silt with some gravel and larger material.

The rock cuts are in the areas of two mapped limbs of the Floyd Hill Fault. The Floyd Hill Fault generally trends north-northwest. Few fractures exposed along the existing rock cut are continuous. Those that are continuous show alteration or oxidation staining as shown in Figure 4. Some brecciated zones are also present in rock samples and excavated rock cut faces.

The highway realignment project site is located east of historical metal mining activity known as the Idaho Springs Mining District. Bedrock in the Floyd Hill tunnel area few sulfide veins and displays only very weak sulfide mineralization or alteration along select fractures. At least three prospect pits have been dug in the vicinity of the proposed tunnel as shown on the Squaw Pass Quadrangle map. Other possible sites of exploration activity or borrow pits have been observed at the west portal area in the alluvium.

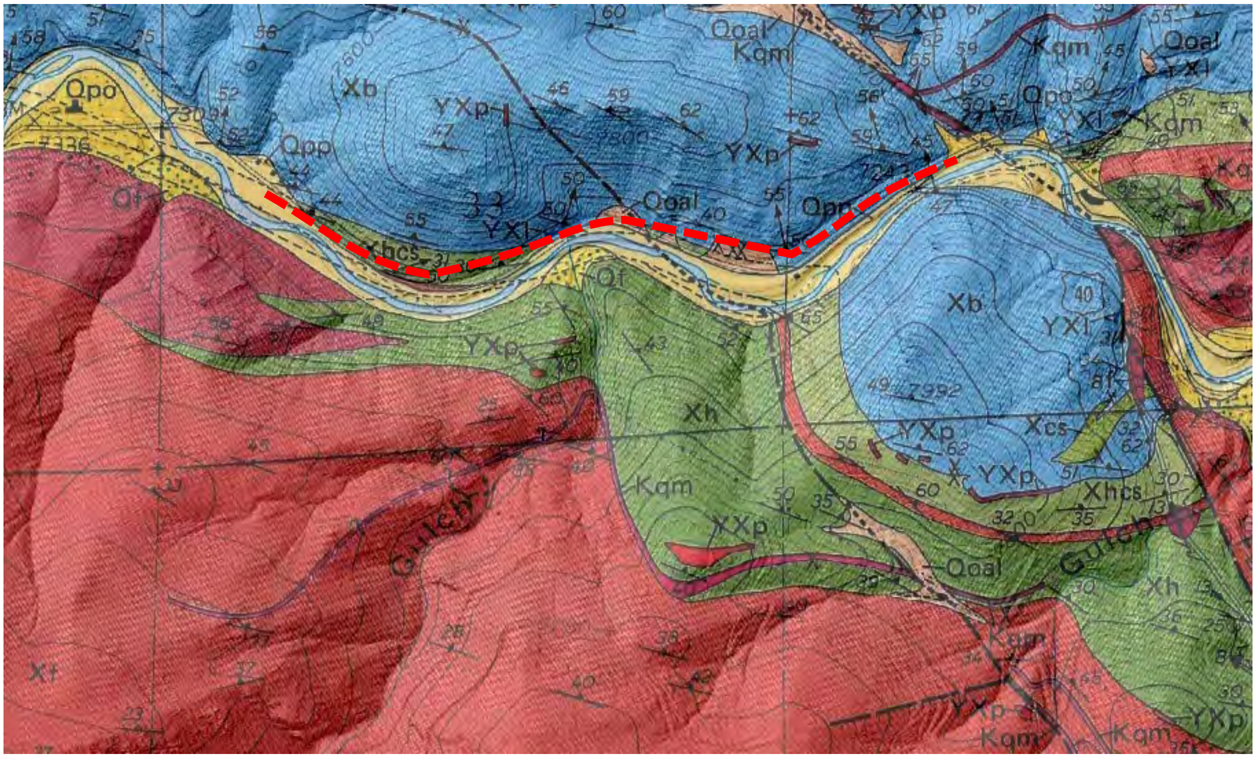


**Figure 2. Geologic Map of the Squaw Pass Quadrangle, Clear Creek, Jefferson and Gilpin Counties, Colorado by Sheridan and Marsh (1976). Published by USGS.**

**Figure 3. Existing rock cut, vertical (yellow) and fan-drilled (red) half-casts.**

**Figure 4. Example of minor oxidation and alteration along a fracture.**

## 3. SURFACE GEOLOGIC EXPLORATION

The investigation program included site reconnaissance, structural mapping and geologic data interpretation. A subsurface investigation was performed for the proposed tunnel associated with this rock cut and can be found in our previous Draft Preliminary Tech Memo Tunnel submitted to Atkins on December 21, 2018.

## 3.1 Geologic Mapping

Field geologic mapping was performed along the existing highway rock cuts to supplement the photogrammetry scanning and modeling performed with the iSite Studio 7 geotechnical module. Field geologic mapping of the I-70 rock cut occurred over a period of three days, Nov. 14, 15, and Dec. 5, 2018. Measurements of rock structures were completed using the FieldMove Clino application on mobile devices with enabled GPS. Photos and data were downloaded to a desktop computer and visualized using ArcMap 10.5.

Based on our mapping and measurements of dip/dip direction the foliation/banding planes and discontinuities (joints, fractures, shear zones) controls overall stability of the rock structure. As previously presented in our memorandum for the Floyd Hill Tunnel, the average of all the imaged boreholes had a dip of 46 degrees with a dip direction of 007 degrees. Average discontinuity measurements for the portal boreholes in Table 1 below demonstrate the variability of the rock structure.

**Table 1. Borehole rock structure imaging results.**

| Borehole | Rock Structure Type | Dip/Dip Direction | Approximate Station |
|---|---|---|---|
| YA-WP-1 | Foliation/Banding | 52/016 | 2079+50 |
| YA-WP-2 | Foliation/Banding | 45/011 | 2079+50 Rt 100’ |
| YA-WP-3 | Foliation/Banding | 50/016 | 2080+35 |
| YA-EP-1 | Foliation/Banding | 47/003 | 2101+70 |
| YA-EP-2 | Foliation/Banding | 36/348 | 2100+70 |

Data was collected along the highway rock cuts from the east portal area to the Hidden Valley exit area. The structural mapping mostly encountered discontinuity (bedding) structure with a few repeating joints and random shear zones, similar to the results from borehole imaging.

Seismic surveys and borings indicate that overburden colluvium thickness ranges between a thin veneer of a few feet to up to approximately 40 feet. The colluvium consists of silty sand to poorly graded sand with gravel and some cobbles. Boulders are evident on the surface throughout the project site and could be buried in colluvium.

## 3.2 Geologic Structure in I-70 Rock Cuts

Generally the dip and dip direction of the rock structure and banded foliation is oriented approximately north-northeast. This orientation combines with the existing south-facing I-70 road rock cuts and slope to create stepped, overhanging features. Multiple overhangs and detached blocks are evident on along the brow of the eastern segment of the rock cut and tunnel portal area. Detached blocks rotate in place downslope and serve as buttresses for colluvium wedges and fans. An example of this bedrock structure controlling topography is shown in Figure 5 below.

**Figure 5. Bedrock structure at East Portal boring YA-EP-1 (looking West).**

Depending on the angle of approach for rock excavations this creates variable conditions for controlled excavation or blasting to create stable benches in the rock mass, since the bedrock structure is dipping northeastward into the slope. Intact rock strength testing from the Floyd Hill Tunnel project indicate general compressive strengths sufficient to stand up under their own weight and provide boundary rock mass support, provided controlled excavation methods are utilized concurrently with an engineered rock reinforcement protocol. Rock core samples indicate between 10-feet of weathered bedrock for the West Portal area and 20-feet of weathered bedrock for the East Portal area. Weathered bedrock has undergone alteration or is mechanically broken to the extent that it should not be included in calculating support segments for overlying or supporting rock. Anticipated failure mechanisms that would influence the rock cuts are the structural rock fabric and the shear strength of weak foliations or banding. Other failure mechanisms such as cross-cutting joints or discontinuities are present, however those conditions are localized and not representative of the entire project length.

Preliminary rock mass discontinuity data measured manually along the exposed rock cuts on I-70 are summarized as stereonets on Figure 6 and Figure 7. The measurement locations are presented on the same figures. Measurement data including tables of the dip, dip direction and structure type and kinematic analysis stereonets can be found in Appendix B.

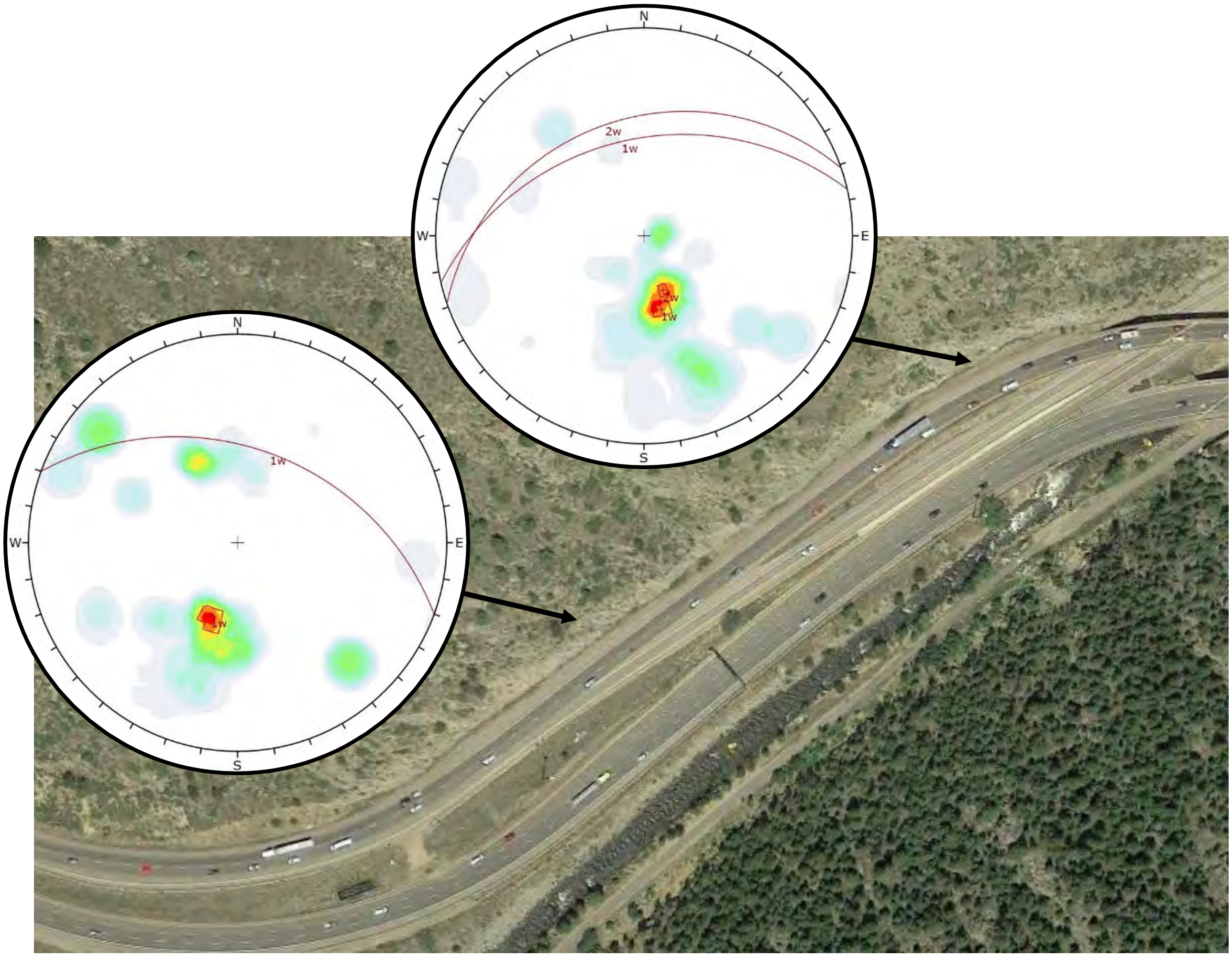


**Figure 6. November 14, 2018 rock cut mapping stereonets.**

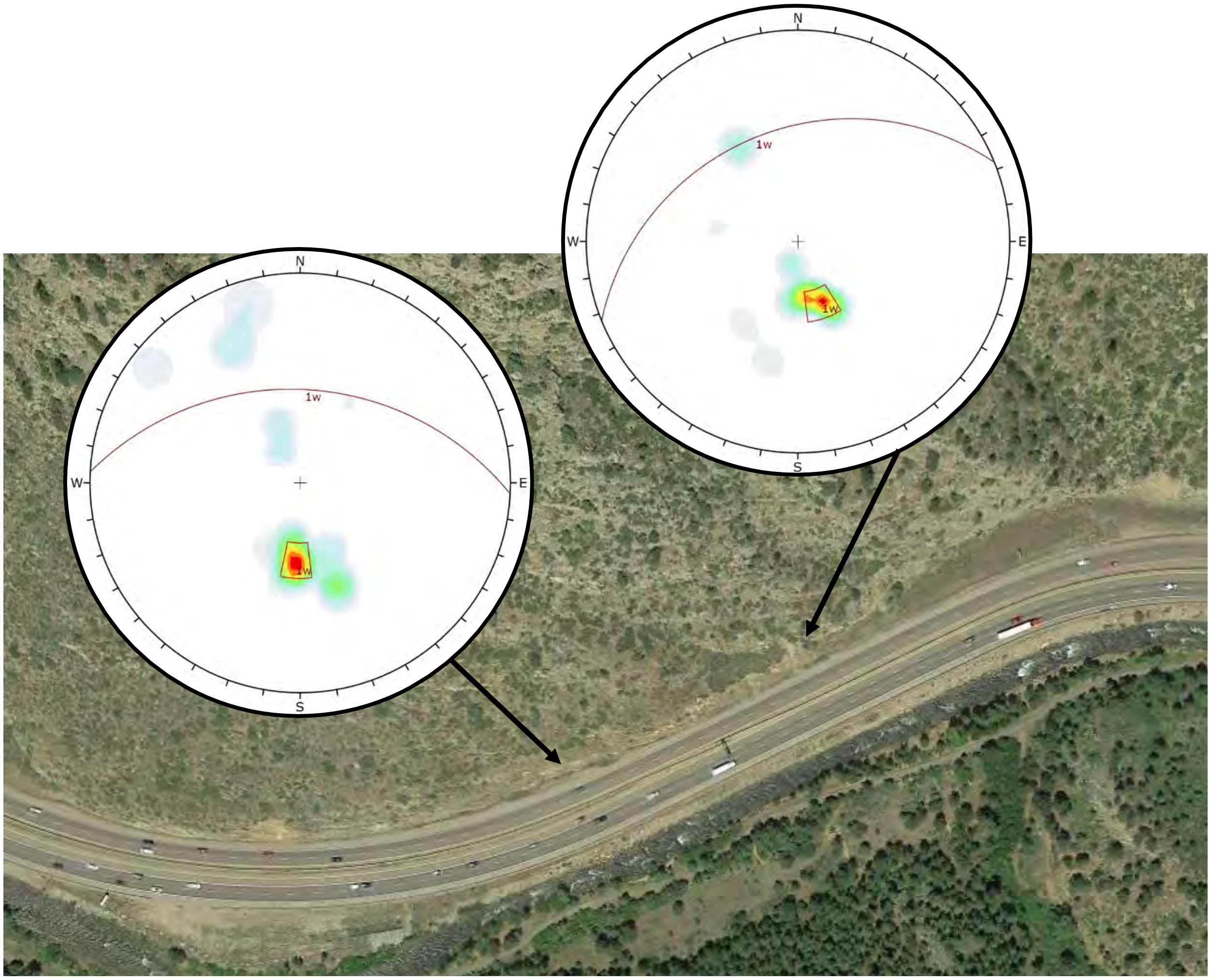


**Figure 7. November 15, 2018 rock cut mapping stereonet.**

# 4. ROCK CUTS

Based on our preliminary field investigation, and our previous experience in the corridor, the banded rock structure and bedrock composition appear to be similar. Migmatite bedrock, generally consisting of folded metamorphic gneiss and quartz pegmatite intrusions, is found at the tunnel location and along the existing I-70 road cuts. Regional dipping bedrock conditions observed in the banded structure indicate an average apparent dip direction north-northeast, at a dip angle ranging between 35° to 55° below horizontal.

Intact rock strength of the rock is dependent on the degree of weathering, which ranged from intensely weathered to moderately weathered in the first 10 feet to 20 feet of our drilled borings, not including overlying soil or colluvium. The Floyd Hill Fault bounds the east portal area, which may explain the slab-like boulders on the east aspect slopes. Natural stepped features and overhangs on the slope above the rock cuts are indicative of the more erodible bands within the rock mass, combined with the northward dipping structure of the rock. Other weak planes or discontinuities are found throughout the site, including shear zones in the face of the cuts. Up to 2 repeating joint sets were visually identified in the recovered rock core and televiewer images (see Draft Floyd Hill Tunnel Memorandum). These joint sets are visually apparent but, based on analyzed data are, not as statistically persistent as the data points representing structural foliation or banding.

## 4.1 Rock Cut Evaluation

Based on our current understanding the proposed rock cuts are 0.25H:1.0V (76° above horizontal) with a minimum 20-ft wide excavation bench. Current proposed rock cut heights may exceed 140 feet on occasion. This does not include the proposed tunnel portal cuts, which are currently 170 feet in the proposed design. With these design assumptions it should be possible to develop preliminary recommendations for the proposed rock cuts including excavation methods, rock reinforcement support and long-term design life.

### 4.1.1 Rock Cuts – I-70 MP 244.3 to MP 243.2 (Approximate)

There are two proposed rock cut segments to straighten I-70 between the base of Floyd Hill and the Hidden Valley exit. Traveling west the first proposed cut area roughly extends from the base of Floyd Hill west to the proposed West Portal area which is currently a clear zone with a culvert inlet and highway signage for Exit 243. Based on these preliminary limits and the provided cross sections, the proposed rock cuts for the new highway alignment consist of excavating a two-tier, rock face with one level supporting EB I-70 and the other level supporting WB I-70 as shown in Figure 6.

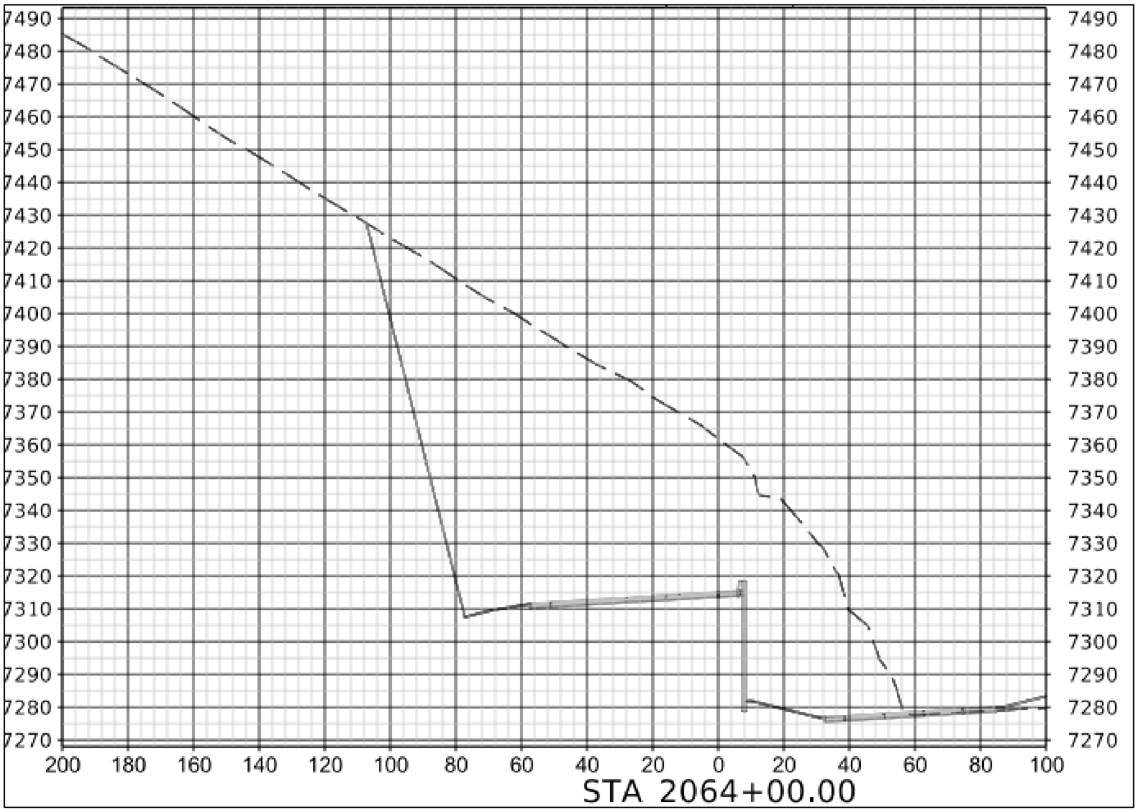


**Figure 8. Two-Tier Rock Cut WB and EB I-70**

Continuing west from the West Portal area, the second proposed rock cut would reduce the curve going toward Hidden Valley Exit 243. The proposed rock cuts vary in height depending on location.

## 4.2 Rock Cut Excavation

Controlling excavation methods for the rock cuts is critical to long-term stability and reduced maintenance for the life of the cut slope. Controlled methods of excavation already proven in the I-70 corridor are drilling and blasting or heavy mechanical excavation techniques such as rippers or hydraulic picks. As evident in Figure 3 the rock can be drilled and blasted to create space for roadway. However, horizontal presplit drilling (the fan in Figure 3) is not frequently utilized in highway construction because it is less controllable than vertical drilling and blasting techniques. Recent rock excavations for Colorado DOT projects were performed primarily with vertical drill and blast methods.

Controlled blasting for the proposed rock cut should utilize consistent hole spacing with sufficient burden to prevent significant overbreak. Presplitting or trim blast methods may be used to manage overbreak of the final rock cut. Uncontrolled blasting patterns or overpressure of the face could result in excessive damage to the rock cut, requiring rockfall mitigation or additional excavation to bring the cut face into a serviceable condition.

Mechanical excavation techniques including but not limited to hydraulic breakers, rams or picks have proven success in the I-70 corridor for hard materials too small to blast. However large-scale excavation with mechanical means is likely uneconomical, due to anticipated low production rates and heavy wear to the equipment.

Other means of excavation not mentioned here may be considered, provided a contractor can present current successful construction examples.

## 4.3 Rock Cut Slope Treatments

Final desired rock cut stability may be achieved by a combination of cut rock slope mitigation treatments concurrent with a controlled excavation method. Mitigation treatments for final cut slopes includes draped double-twist mesh, large high spiral twist mesh, cable-net mesh panels, rock bolts, rock dowels and rock face scaling (manual or mechanical). One or a combination of these mitigation treatments could be performed on the final cut slope, if required, to stabilize the face for long-term service life and to reduce potential future hazards. Rock slope treatment types, effectiveness and applicable locations are presented below in Table 2

**Table 2. Rock slope mitigation treatment options.**

| Rock Slope Treatment Option | Suitability – Effectiveness | Applicable Locations | Maintenance |
|---|---|---|---|
| Double Twist Mesh | Contains small rocks and debris in shoulder | Slopes ≤200 ft<br>Stable cuts producing small rocks, debris | Regular inspection, repair wire mesh when necessary |
| Cable Net Panel | Contains moderate-size rocks in place, prevents bouncing | Slopes ≥200 ft<br>Cuts producing rocks, debris regularly | Regular inspection<br>Replacement of rockfall impacted panels |
| High Strength Spiral Twist Mesh | Contains moderate-size rocks in place, prevents bouncing | Slopes ≥200 ft<br>Cuts producing rocks, debris regularly | Regular Inspection<br>This does not use panels. Installation is easier. Continuous diamond mesh is less obtrusive |
| Rock Bolt/Dowel | Passive or active anchoring of rocks in place | Large rock masses requiring stabilization | Limited with proper installation and corrosion protection |
| Scaling | Removes unstable rocks | Rock cut excavations<br>Incipient rockfall hazards outside right-of-way | Perform every 3 to 5 years not counting emergency rockfall response |

#### 4.3.1 Rock Cut Catchment Ditches

For any rock cut mitigation treatment option to work properly, a sufficiently sized ditch is required in order to create a buffer between traffic and rockfall sources and/or as storage for collected debris. Appropriately sized ditches must be considered not only for debris catchment but for highway clear zones and sight lines around curves. Ditch effectiveness is a function debris source height, width, storage capacity for collected debris and surface hardness as it relates to conservation of energy (for bouncing debris). Many factors must be considered when evaluating ditch effectiveness which can be modeled for further evaluation. Existing ditch widths along the current I-70 highway alignment are generally too small to effectively mitigate rockfall or serve as debris storage. New highway alignment should incorporate sufficient ditch space/rockfall mitigation treatment to contain debris from new rock cuts.

### 4.4 Subsurface Conditions and Seismicity

The site is classified as Site Class B in accordance with Table 3.10.3.1-1 of the AASHTO LRFD Bridge Design Specifications. The Peak Ground Acceleration (PGA), and the short- and long-period spectral acceleration coefficients ($S_S$ and $S_1$ respectively) for the tunnel site were obtained using the USGS/AASHTO 2007 Seismic Parameters for an event with a 7% Probability of Exceedance (PE) in 75 years and a Site Class B (reference site). An event with the above probability of exceedance has a return period of about 1,033 years. Since the site classification (B) is the same as the reference site (B), no value adjustments were necessary. The seismic parameters for this site are shown on Table 5 and Table 6.

**Table 3. Seismic Design Parameters**

| PGA (0.0 sec) | $S_S$ (0.2 sec) | $S_1$ (1.0 sec) |
|---|---|---|
| 0.067 | 0.141 | 0.036 |

**Table 4. Seismic Design Parameters for Site Class B**

| $A_S$ (0.0 sec) | $S_{DS}$ (0.2 sec) | $S_{D1}$ (1.0 sec) | Seismic Zone |
|---|---|---|---|
| 0.067g | 0.141 g | 0.036 g | 1 |

As noted previously, the nearby quarry can cause ground vibrations during excavation blasts. This is a potential external source of vibration.

## 5. GROUNDWATER CONDITIONS

The drilling program for the Floyd Hill Tunnel investigation used water, therefore the water observed during and after hole completion is attributed to drilling. No long-term groundwater monitoring was planned or has been performed since completion of the drilling program. Drill water circulation was lost in each boring, even after setting oversized collars into bedrock. This indicates sufficient open fractures or discontinuities that can transmit water. A perennial water seep along the I-70 rock cut, between the portal sites, has consistently produced enough water to dampen the rock cut throughout the year. During the spring, noticeable water seeps out of the rock cut as shown in Figure 7 below. This water is likely using fractures in the rock as a drainage pathway and may not indicate a water bearing formation.

**Figure 9. Seepage noted in I-70 rock cut at approximate mile marker 244.2 (Google Earth).**

Variations in groundwater conditions may occur seasonally. The magnitude of the variation will be largely dependent upon fluctuations in the amount of spring snowmelt, duration and intensity of precipitation, site grading changes, and the surface and subsurface drainage characteristics of the surrounding area. Seasonal perched areas of groundwater or fracture flow may also exist but were not confirmed in any of our test holes during the investigation. Generally, based on observation, the rock mass in the area appears relatively dry. This needs to be confirmed.

## 6. LIMITATIONS

This report transmits geotechnical data only, for use by Atkins Global Corporation and CDOT, for the proposed tunnel approximately between Floyd Hill and Hidden Valley, Colorado. The data submitted are based on the exploratory borings, laboratory testing, field mapping and reconnaissance included in our investigation. As previously identified, the rock and its properties in the tunnel area can vary considerably over short distances.

This Investigation was largely conducted in difficult terrain and active traffic using a single lane closure during the day in the late fall to winter months necessitating restricted space techniques and special operational procedures in order to perform work. Difficult access and weather constraints limited the degree of access and correspondingly the types and extents of activities that were possible to conduct particularly in the portal areas and along the I-70 road cut.

This Investigation has been conducted in accordance with generally accepted geotechnical engineering practices in this area. The nature and extent of subsurface variations across the site may not become evident until excavation is performed. During construction conditions may be different from those described herein. No warranty, expressed or implied, is made.

Yeh and Associates, Inc.

Prepared by:

Todd Hansen, P.E.
Project Geological Engineer

Reviewed by:
Howard Hume, Ph.D., P.G.
Senior Project Manager

# 7. REFERENCES

Barton, N.R., Lien, R., Lunde, J. 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics.* 6(4), 189-239.

Bieniawski, Z. T., 1976. Rock mass classification in rock engineering. In *Exploration for rock engineering, proc. of the symp.*, (ed. Z.T. Bieniawski), 1, 97-106. Cape Town: Balkema.

Bieniawski, Z. T., 1989. Engineering rock mass classifications. New York: Wiley.

Palmström, A., 2009. Technical note. Combining the RMR, Q and RMi classification systems. Tunneling and Underground Space Technology Vol. 24, pp. 491-492.

Sheridan, D.M. and Marsh, S.P., 1976, Geology Map of the Squaw Pass Quadrangle, Clear Creek, Jefferson, and Gilpin Counties, Colorado: U.S. Geological Survey Map GQ-1337.

Technical Memorandum. Draft Floyd Hill Tunnel Portal Investigation; I-70 Floyd Hill, Clear Creek County, Colorado. December 21, 2018. Yeh and Associates Project 218-300.

Widmann, B.L. and Rogers, W.P., 2002, Geologic Hazards of the Georgetown, Idaho Springs, and Squaw Pass Quadrangles, Colorado Geological Survey Open File Report 03-02.

Widmann, B.L., Kirkham, R.M., Morgan, M.L., and Rogers, W.P., with contributions by Crone, A.J., Personius, S.F., and Kelson, K.I., and GIS/Web design by Morgan, K.S., Pattyn, G.R., and Phillips, R.C., 2002, Colorado Late Cenozoic fault and fold database and internet map server: Colorado Geological Survey Bulletin 64a, https://pubs.er.usgs.gov/publication/70031679

# APPENDICES

# Appendix A

## PHOTOS

**Photo 1 – Ice and springs from water seeps; localized and not representative of the rock cuts.**

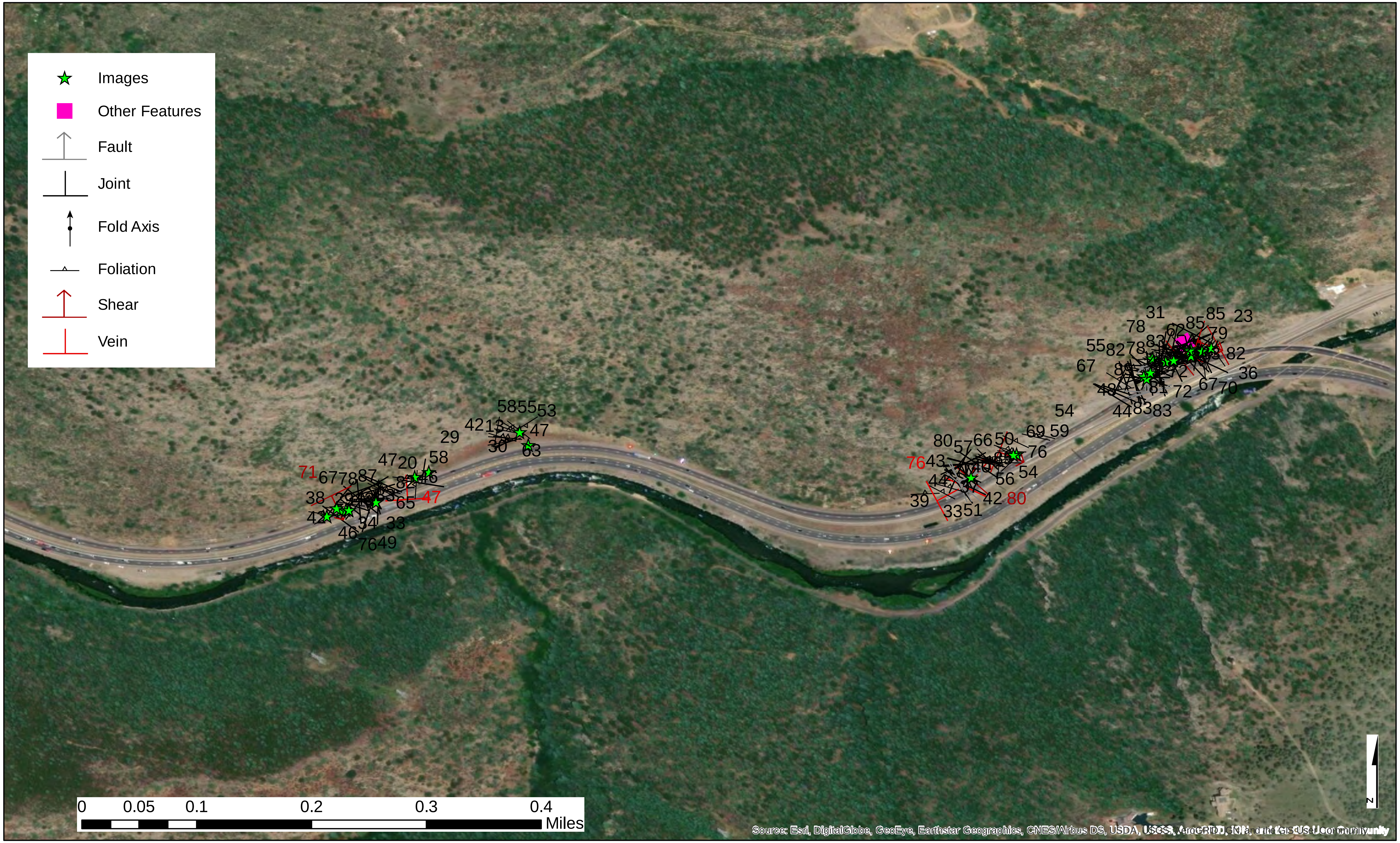
**Photo 2 – Colluvium and vegetation overlying bedrock, represents typical conditions across site.**

**Photo 3 – Existing (looking) Westbound rock cut with colluvium deposits. Note cobbles and boulders.**

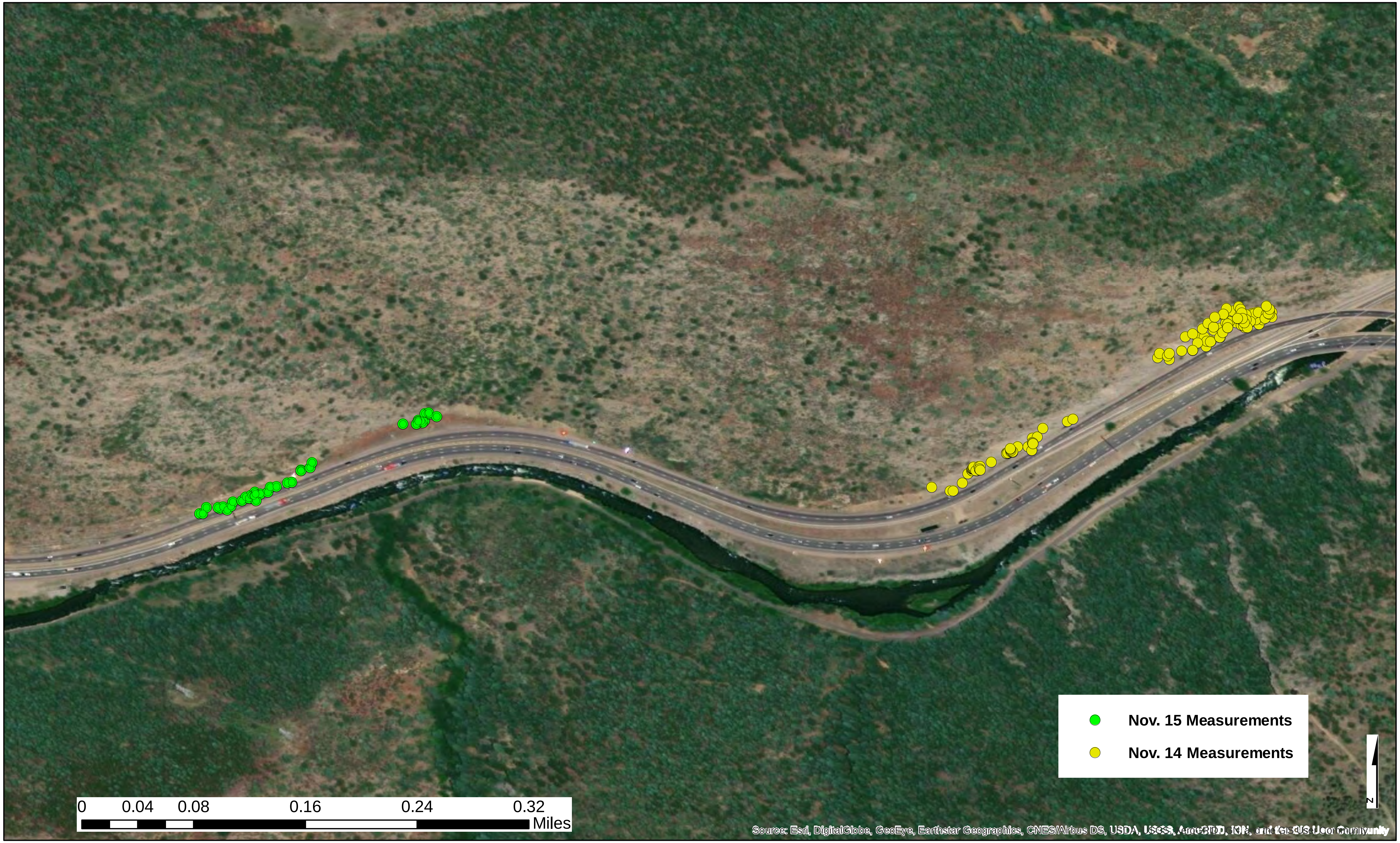
**Photo 4 – Typical variable rock conditions, differential alteration and weathering. Drill steel stuck in rock, possible indication of difficult drilling conditions.**

# Appendix B

## ROCK MASS DISCONTINUITY MEASUREMENTS



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

Rock Cut
Field Mapping Data,14th

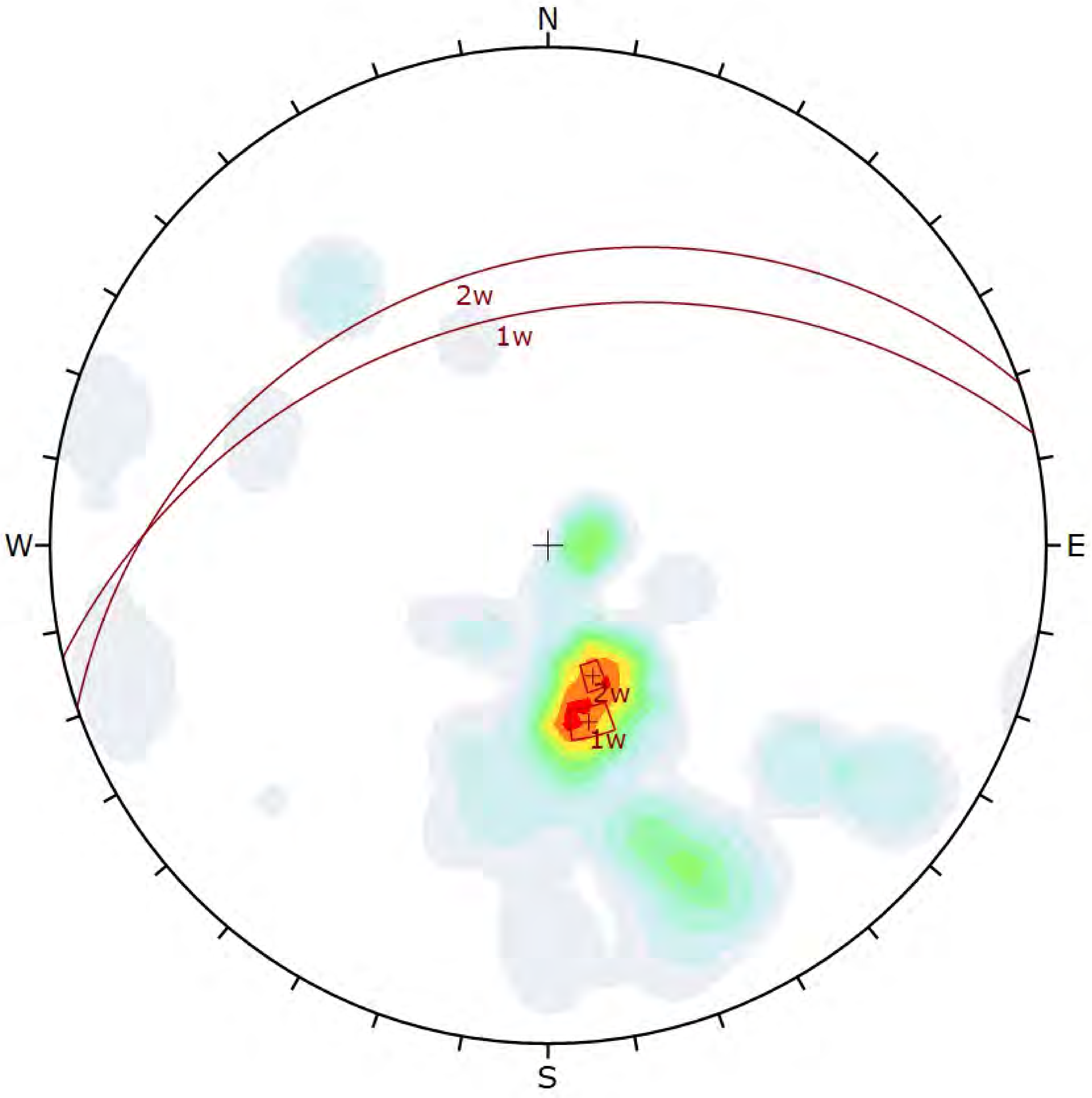
| OBJECTID | DIP | DipDirection | strike | StructureType |
|---|---|---|---|---|
| 26 | 39 | 358 | 268 | Foliation |
| 27 | 76 | 61 | 331 | Vein |
| 28 | 37 | 27 | 297 | Foliation |
| 29 | 56 | 16 | 286 | Foliation |
| 30 | 43 | 6 | 276 | Foliation |
| 31 | 57 | 25 | 295 | Vein |
| 32 | 80 | 40 | 310 | Shear |
| 33 | 51 | 351 | 261 | Foliation |
| 34 | 83 | 112 | 22 | Shear |
| 35 | 74 | 19 | 289 | Foliation |
| 36 | 57 | 2 | 272 | Foliation |
| 37 | 66 | 25 | 295 | Foliation |
| 38 | 59 | 13 | 283 | Foliation |
| 39 | 54 | 14 | 284 | Foliation |
| 40 | 67 | 31 | 301 | Joint |
| 41 | 55 | 24 | 294 | Joint |
| 42 | 48 | 11 | 281 | Foliation |
| 43 | 70 | 16 | 286 | Joint |
| 44 | 53 | 0 | 270 | Joint |
| 45 | 35 | 18 | 288 | Foliation |
| 46 | 56 | 358 | 268 | Foliation |
| 47 | 76 | 82 | 352 | Joint |
| 48 | 58 | 10 | 280 | Foliation |
| 49 | 54 | 349 | 259 | Foliation |
| 50 | 78 | 44 | 314 | Joint |
| 51 | 44 | 13 | 283 | Foliation |
| 52 | 40 | 11 | 281 | Foliation |
| 53 | 39 | 14 | 284 | Foliation |
| 54 | 81 | 63 | 333 | Joint |
| 55 | 51 | 12 | 282 | Foliation |
| 56 | 35 | 25 | 295 | Foliation |
| 57 | 31 | 346 | 256 | Joint |
| 58 | 80 | 156 | 66 | Joint |
| 59 | 34 | 347 | 257 | Foliation |
| 60 | 25 | 7 | 277 | Foliation |
| 61 | 28 | 51 | 321 | Shear |
| 62 | 58 | 347 | 257 | Foliation |
| 63 | 74 | 330 | 240 | Foliation(?) |
| 64 | 78 | 359 | 269 | Foliation |
| 65 | 11 | 265 | 175 | Foliation |
| 66 | 47 | 159 | 69 | Joint |
| 67 | 77 | 335 | 245 | Foliation |
| 68 | 10 | 253 | 163 | Foliation |
| 69 | 31 | 288 | 198 | Foliation |
| 71 | 67 | 140 | 50 | Joint |
| 72 | 58 | 280 | 190 | Foliation |
| 73 | 70 | 47 | 317 | Joint |
| 74 | 9 | 342 | 252 | Fold Axis |
| 75 | 36 | 63 | 333 | Joint |
| 76 | 89 | 241 | 151 | Shear |
| 77 | 87 | 109 | 19 | Shear |
| 78 | 60 | 4 | 274 | Foliation |
| 79 | 82 | 44 | 314 | Joint |
| 80 | 23 | 25 | 295 | Joint |
| 81 | 45 | 356 | 266 | Foliation |
| 82 | 36 | 326 | 236 | Foliation |

Rock Cut
Field Mapping Data 14th

| OBJECTID | DIP | DipDirection | strike | StructureType |
|---|---|---|---|---|
| 83 | 25 | 39 | 309 | Foliation |
| 85 | 77 | 1 | 271 | Foliation |
| 86 | 63 | 340 | 250 | Foliation |
| 87 | 82 | 234 | 144 | Foliation |
| 88 | 70 | 343 | 253 | Foliation |
| 89 | 80 | 305 | 215 | Foliation |
| 90 | 67 | 311 | 221 | Foliation |
| 109 | 39 | 26 | 296 | Foliation |
| 110 | 39 | 25 | 295 | Foliation |
| 111 | 33 | 195 | 105 | Joint |
| 112 | 44 | 194 | 104 | Joint |
| 113 | 51 | 176 | 86 | Joint |
| 114 | 43 | 155 | 65 | Joint |
| 115 | 42 | 173 | 83 | Joint |
| 116 | 80 | 129 | 39 | Joint |
| 117 | 57 | 114 | 24 | Joint |
| 118 | 56 | 53 | 323 | Joint |
| 119 | 66 | 214 | 124 | Joint |
| 120 | 85 | 26 | 296 | Joint |
| 121 | 50 | 152 | 62 | Joint |
| 122 | 71 | 68 | 338 | Fault |
| 123 | 73 | 59 | 329 | Fault |
| 124 | 50 | 40 | 310 | Fault |
| 125 | 53 | 44 | 314 | Foliation |
| 126 | 72 | 9 | 279 | Foliation |
| 127 | 61 | 45 | 315 | Foliation |
| 128 | 43 | 17 | 287 | Foliation |
| 129 | 51 | 15 | 285 | Foliation |
| 130 | 46 | 20 | 290 | Foliation |
| 131 | 46 | 27 | 297 | Foliation |
| 132 | 71 | 16 | 286 | Foliation |
| 133 | 81 | 276 | 186 | Foliation |
| 134 | 40 | 15 | 285 | Foliation |
| 135 | 58 | 2 | 272 | Foliation |
| 136 | 54 | 358 | 268 | Foliation |
| 137 | 76 | 317 | 227 | Foliation |
| 138 | 69 | 30 | 300 | Foliation |
| 139 | 44 | 353 | 263 | Foliation |
| 140 | 48 | 31 | 301 | Foliation |
| 141 | 45 | 18 | 288 | Joint |
| 142 | 83 | 73 | 343 | Joint |
| 143 | 89 | 181 | 91 | Joint |
| 144 | 83 | 63 | 333 | Joint |
| 145 | 82 | 72 | 342 | Joint |
| 146 | 72 | 253 | 163 | Joint |
| 147 | 78 | 247 | 157 | Joint |
| 148 | 72 | 206 | 116 | Joint |
| 149 | 83 | 78 | 348 | Joint |
| 150 | 32 | 342 | 252 | Fault |
| 151 | 34 | 3 | 273 | Fault |
| 152 | 23 | 348 | 258 | Fault |
| 153 | 61 | 14 | 284 | Foliation |
| 154 | 62 | 16 | 286 | Foliation |
| 155 | 57 | 357 | 267 | Foliation |
| 156 | 77 | 340 | 250 | Foliation |
| 157 | 73 | 24 | 294 | Foliation |

## Rock Cut
## Field Mapping Data,14th,

| OBJECTID | DIP | DipDirection | strike | StructureType |
|---|---|---|---|---|
| 158 | 70 | 2 | 272 | Foliation |
| 159 | 63 | 343 | 253 | Foliation |
| 160 | 65 | 334 | 244 | Foliation |
| 161 | 83 | 89 | 359 | Joint |
| 162 | 63 | 113 | 23 | Joint |
| 163 | 62 | 100 | 10 | Joint |
| 164 | 45 | 20 | 290 | Foliation |
| 165 | 57 | 20 | 290 | Foliation |
| 166 | 47 | 348 | 258 | Foliation |
| 167 | 46 | 332 | 242 | Foliation |
| 168 | 45 | 357 | 267 | Foliation |
| 169 | 39 | 343 | 253 | Foliation |
| 170 | 85 | 102 | 12 | Joint |
| 171 | 39 | 349 | 259 | Foliation |
| 172 | 51 | 14 | 284 | Foliation |
| 173 | 79 | 207 | 117 | Joint |
| 174 | 41 | 350 | 260 | Foliation |
| 175 | 54 | 8 | 278 | Foliation |
| 176 | 85 | 346 | 256 | Joint |
| 177 | 39 | 2 | 272 | Foliation |
| 178 | 70 | 50 | 320 | Foliation |
| 179 | 30 | 340 | 250 | Foliation |
| 180 | 34 | 354 | 264 | Foliation |



Stereonet Projection: East Data Set, November 14, 2018

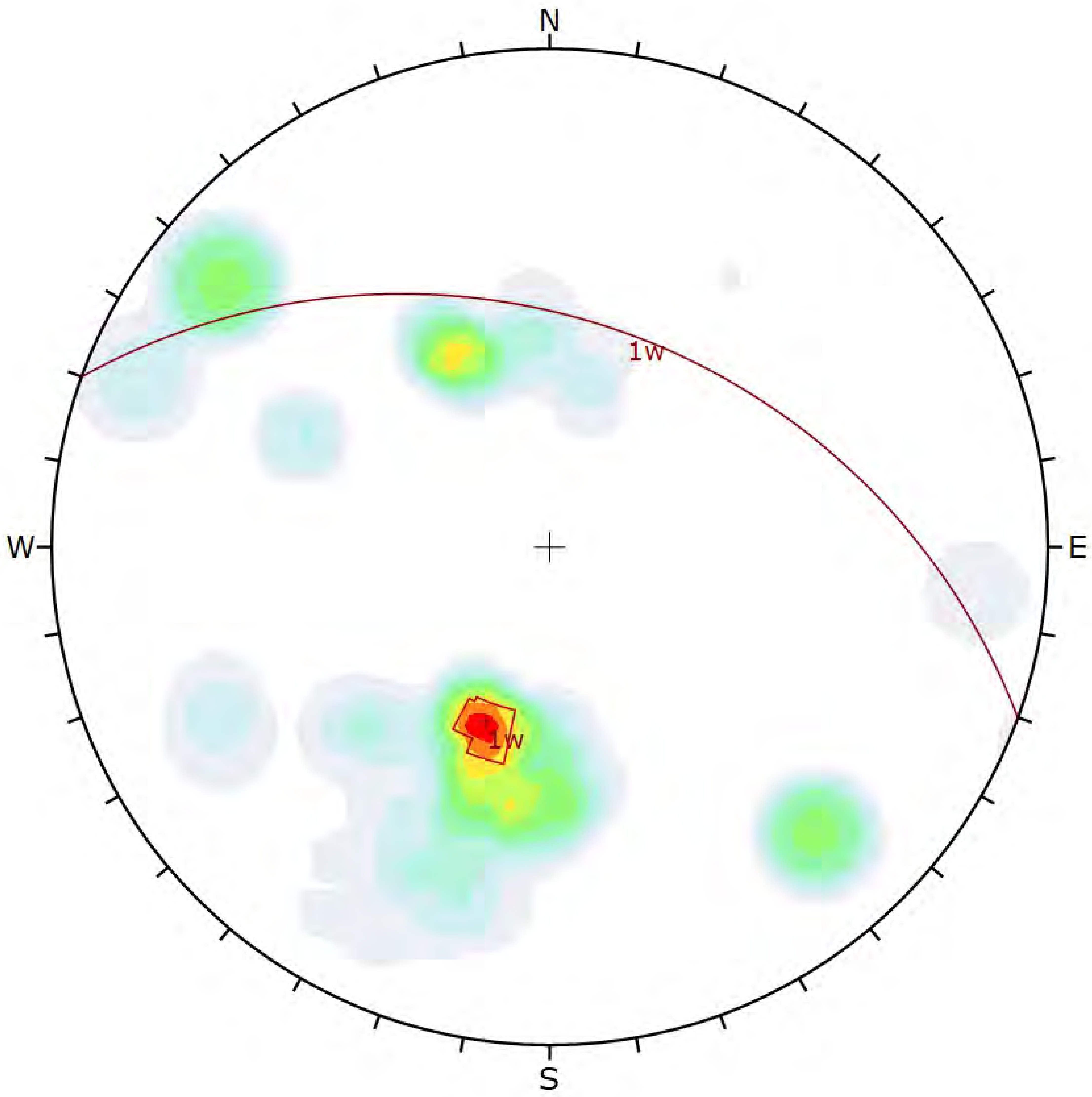
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 1.20 |
| ⬜ | 1.20 - 2.40 |
| 🟦 | 2.40 - 3.60 |
| 🟦 | 3.60 - 4.80 |
| 🟩 | 4.80 - 6.00 |
| 🟩 | 6.00 - 7.20 |
| 🟩 | 7.20 - 8.40 |
| 🟨 | 8.40 - 9.60 |
| 🟧 | 9.60 - 10.80 |
| 🟥 | 10.80 - 12.00 |

| | |
|---|---|
| **Maximum Density** | 11.47% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | Color | Trend | Plunge | Label |
|---|---|---|---|---|
| **Mean Set Planes** | | | | |
| 1w | 🟥 | 167 | 50 | |
| 2w | 🟥 | 161 | 59 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 71 (71 Entries) |
| **Terzaghi Weighting** | Minimum Bias Angle 10° |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Stereonet Projection: West Data Set, November 14, 2018



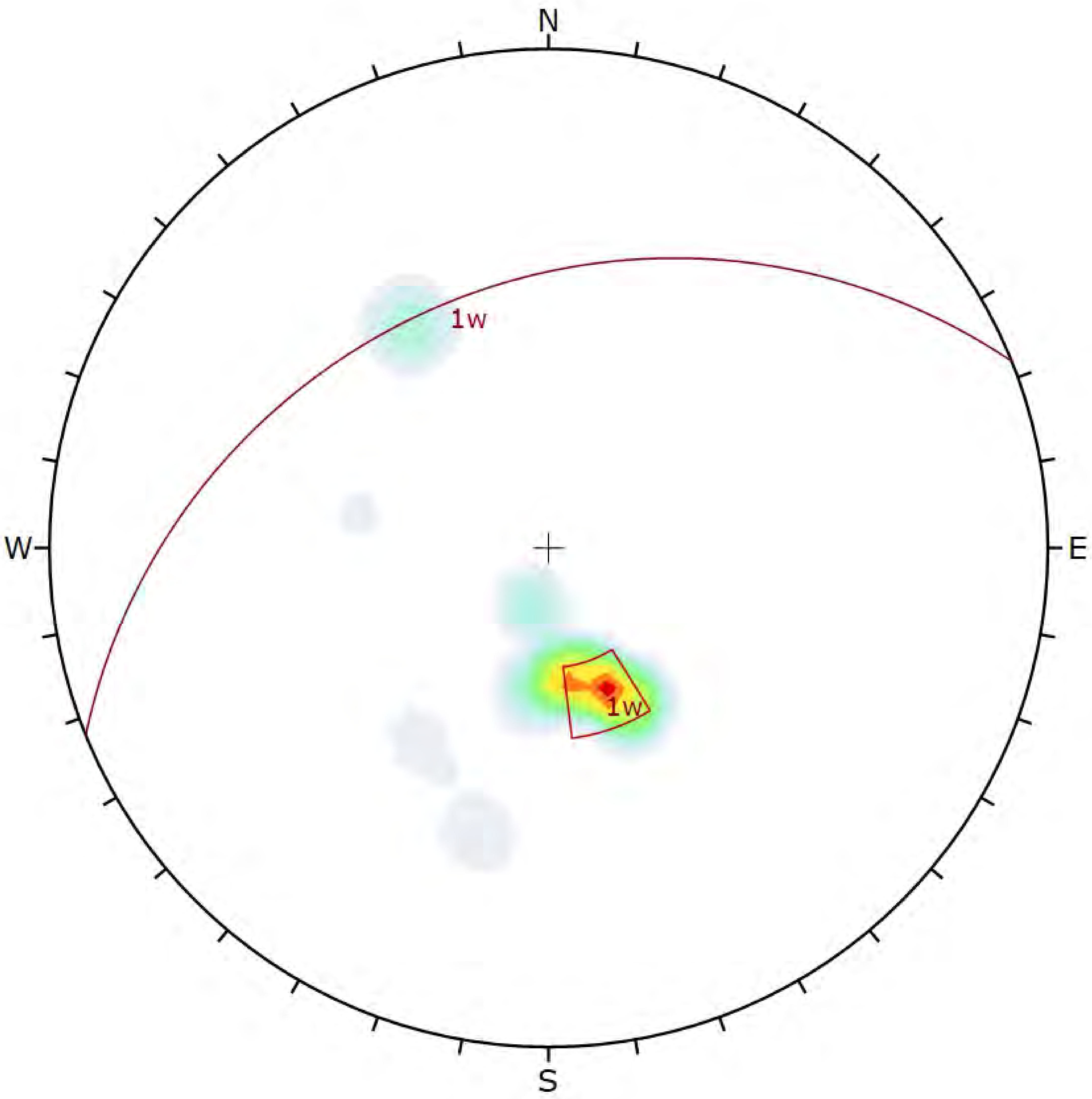
| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 1.30 |
| ⬜ | 1.30 - 2.60 |
| 🟦 | 2.60 - 3.90 |
| 🟦 | 3.90 - 5.20 |
| 🟩 | 5.20 - 6.50 |
| 🟩 | 6.50 - 7.80 |
| 🟩 | 7.80 - 9.10 |
| 🟨 | 9.10 - 10.40 |
| 🟧 | 10.40 - 11.70 |
| 🟥 | 11.70 - 13.00 |
| **Maximum Density** | 12.92% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

|  | Color | Trend | Plunge | Label |
|---|---|---|---|---|
| **Mean Set Planes** |  |  |  |  |
| 1w | 🟥 | 200 | 49 |  |

|  |  |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 44 (44 Entries) |
| **Terzaghi Weighting** | Minimum Bias Angle 10° |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |

Rock Cut
Field Mapping Data,15th

| OBJECTID | DIP | DipDirection | strike | StructureType |
|---|---|---|---|---|
| 1 | 42 | 8.59608746 | 278.6 | Foliation |
| 2 | 76 | 36.0708771 | 306.1 | Joint |
| 3 | 46 | 5.31901121 | 275.3 | Foliation |
| 4 | 83 | 128 | 38 | Joint |
| 5 | 49 | 219 | 129 | Joint |
| 6 | 67 | 288 | 198 | Joint |
| 7 | 29 | 9 | 279 | Foliation |
| 8 | 78 | 272 | 182 | Joint |
| 9 | 33 | 161 | 71 | Joint |
| 10 | 38 | 2 | 272 | Foliation |
| 11 | 42 | 2 | 272 | Foliation |
| 12 | 87 | 85 | 355 | Joint |
| 13 | 65 | 209 | 119 | Joint |
| 14 | 47 | 203 | 113 | Joint |
| 15 | 47 | 6 | 276 | Foliation |
| 16 | 39 | 25 | 295 | Foliation |
| 17 | 39 | 26 | 296 | Foliation |
| 18 | 55 | 11 | 281 | Foliation |
| 19 | 52 | 339 | 249 | Foliation |
| 20 | 47 | 356 | 266 | Vein |
| 21 | 39 | 274 | 184 | Foliation |
| 22 | 46 | 6 | 276 | Foliation |
| 23 | 58 | 341 | 251 | Foliation |
| 24 | 29 | 9 | 279 | Joint |
| 25 | 42 | 100 | 10 | Foliation |
| 91 | 38 | 336 | 246 | Foliation |
| 92 | 71 | 154 | 64 | Shear |
| 93 | 82 | 164 | 74 | Joint |
| 94 | 20 | 150 | 60 | Joint |
| 95 | 34 | 3 | 273 | Foliation |
| 96 | 43 | 358 | 268 | Foliation |
| 97 | 40 | 334 | 244 | Foliation |
| 98 | 34 | 7 | 277 | Foliation |
| 99 | 30 | 342 | 252 | Foliation |
| 100 | 37 | 331 | 241 | Foliation |
| 101 | 29 | 350 | 260 | Foliation |
| 102 | 30 | 4 | 274 | Foliation |
| 103 | 13 | 16 | 286 | Foliation |
| 104 | 63 | 10 | 280 | Foliation |
| 105 | 58 | 19 | 289 | Foliation |
| 106 | 47 | 31 | 301 | Foliation |
| 107 | 55 | 148 | 58 | Foliation |
| 108 | 53 | 38 | 308 | Foliation |



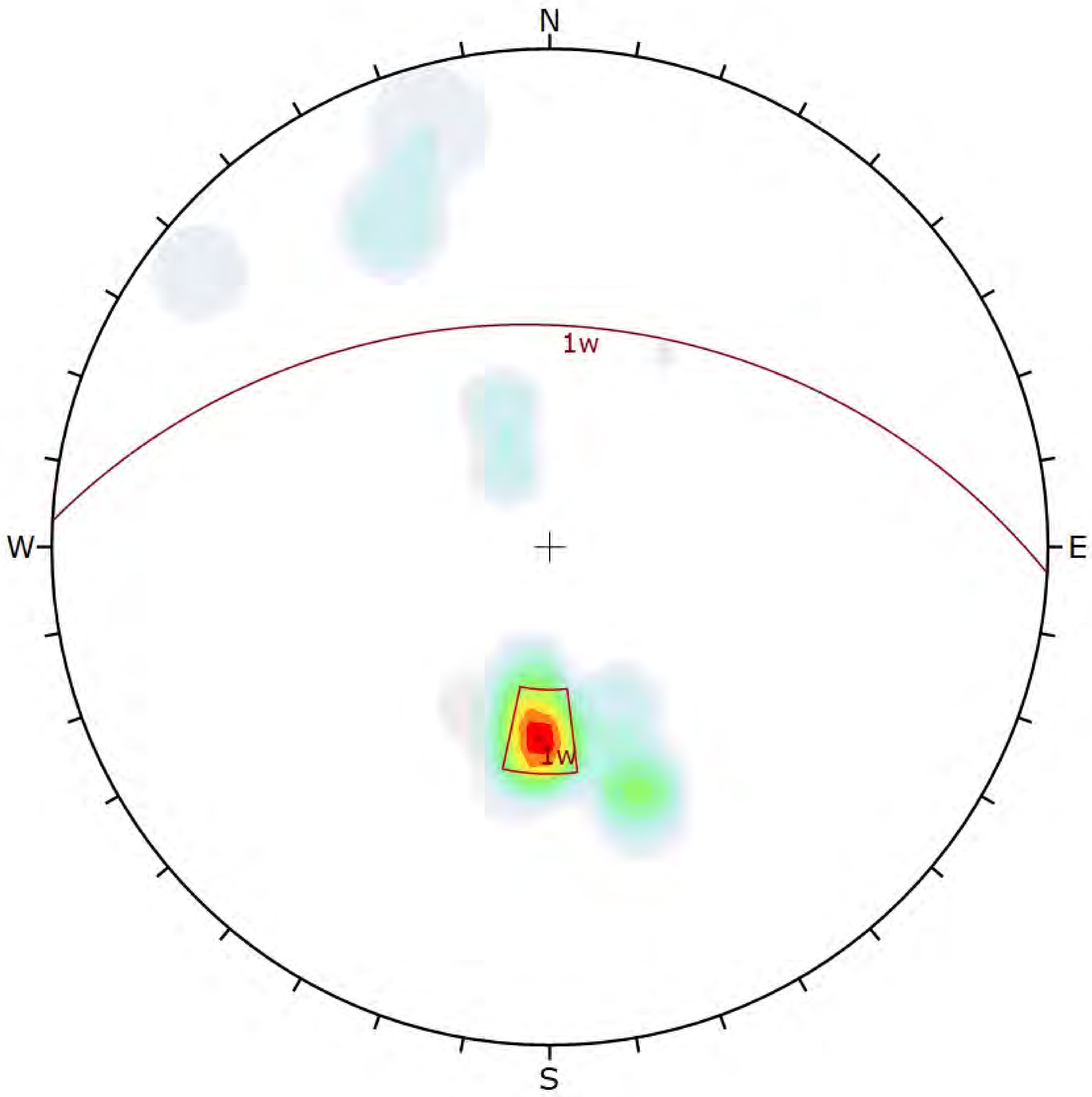
Stereonet Projection: East Data Set, November 15, 2018

| Color | Density Concentrations |
|---|---|
| ⬜ | 0.00 - 3.50 |
| ⬜ | 3.50 - 7.00 |
| 🟦 | 7.00 - 10.50 |
| 🟦 | 10.50 - 14.00 |
| 🟩 | 14.00 - 17.50 |
| 🟩 | 17.50 - 21.00 |
| 🟩 | 21.00 - 24.50 |
| 🟨 | 24.50 - 28.00 |
| 🟧 | 28.00 - 31.50 |
| 🟥 | 31.50 - 35.00 |

| | |
|---|---|
| **Maximum Density** | 34.37% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | Color | Trend | Plunge | Label |
|---|---|---|---|---|
| **Mean Set Planes** | | | | |
| 1w | 🟥 | 158 | 56 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 13 (13 Entries) |
| **Terzaghi Weighting** | Minimum Bias Angle 10° |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |



Stereonet Projection: West Data Set, November 15, 2018

| Color | Density Concentrations |
|---|---|
| | 0.00 - 2.30 |
| ⬜ | 2.30 - 4.60 |
| 🟦 | 4.60 - 6.90 |
| 🟦 | 6.90 - 9.20 |
| 🟩 | 9.20 - 11.50 |
| 🟩 | 11.50 - 13.80 |
| 🟩 | 13.80 - 16.10 |
| 🟨 | 16.10 - 18.40 |
| 🟧 | 18.40 - 20.70 |
| 🟥 | 20.70 - 23.00 |

| | |
|---|---|
| **Maximum Density** | 22.22% |
| **Contour Data** | Pole Vectors |
| **Contour Distribution** | Fisher |
| **Counting Circle Size** | 1.0% |

| | Color | Trend | Plunge | Label |
|---|---|---|---|---|
| | | **Mean Set Planes** | | |
| 1w | 🟥 | 183 | 48 | |

| | |
|---|---|
| **Plot Mode** | Pole Vectors |
| **Vector Count** | 30 (30 Entries) |
| **Terzaghi Weighting** | Minimum Bias Angle 10° |
| **Hemisphere** | Lower |
| **Projection** | Equal Angle |