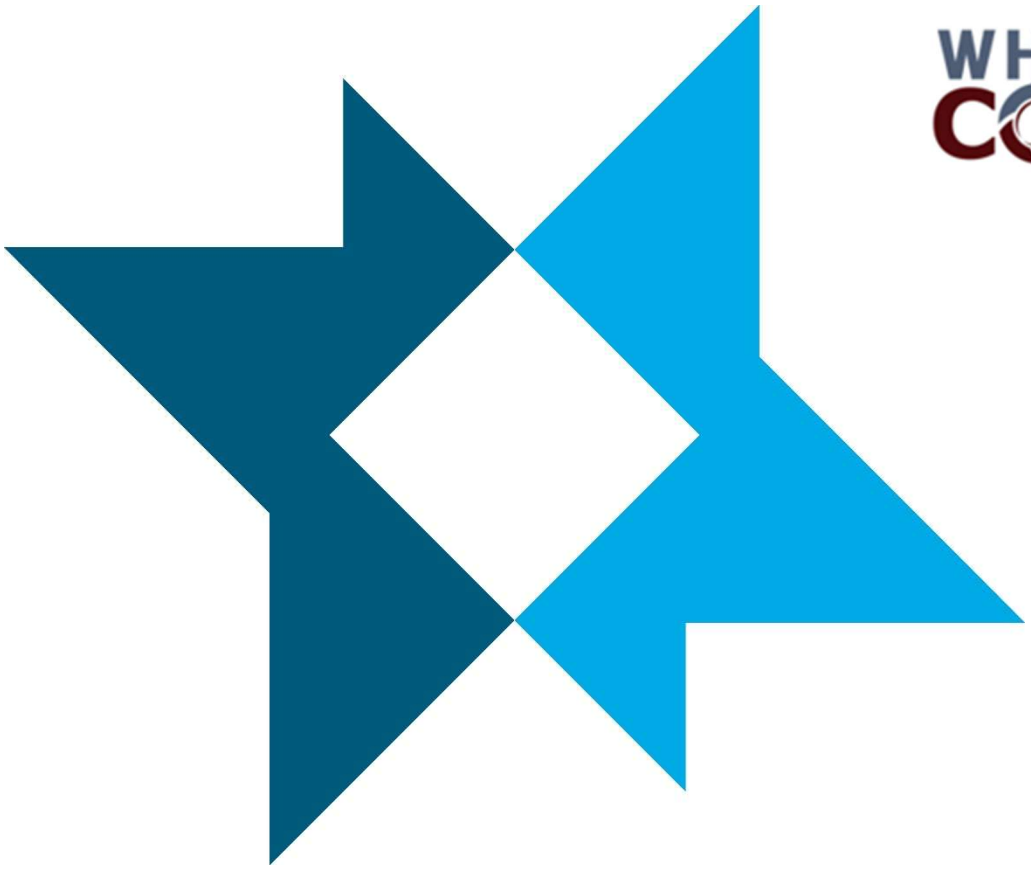
Mining Permit Application
White Pine North Project
Volume IA
Appendices C through E-2



White Pine Copper LLC
White Pine, Michigan

September 2026

Project I.D.: 23W200

Solving our clients' toughest science and engineering challenges.

Appendix C

Mine Stability Analysis

C-1 White Pine North Geotechnical Study
C-2 White Pine North Project - Bedrock Deflection Assessment
C-3 Evaluation of White Pine North Mine Subsidence on Tailings Storage Facility, Groundwater Wells, and Neighboring Surface Structures

C-1

White Pine North Geotechnical Study

REPORT

FINAL REPORT
White Pine North Geotechnical Study

Prepared for:

White Pine Copper LLC

Submitted to:

WSP USA Inc.

Submitted by:

WSP Canada Inc.

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1 e-copy to White Pine Copper

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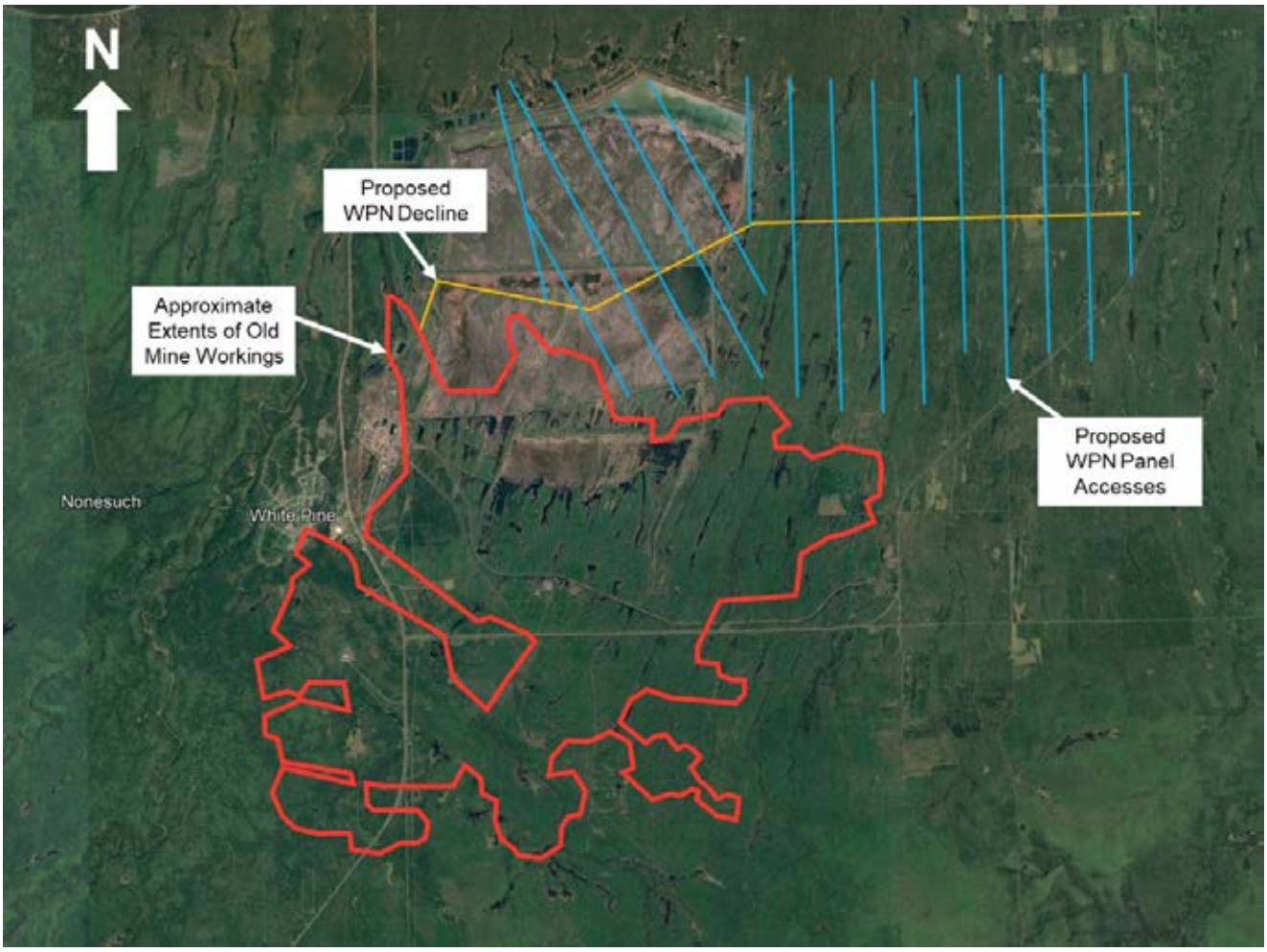
1.0 INTRODUCTION

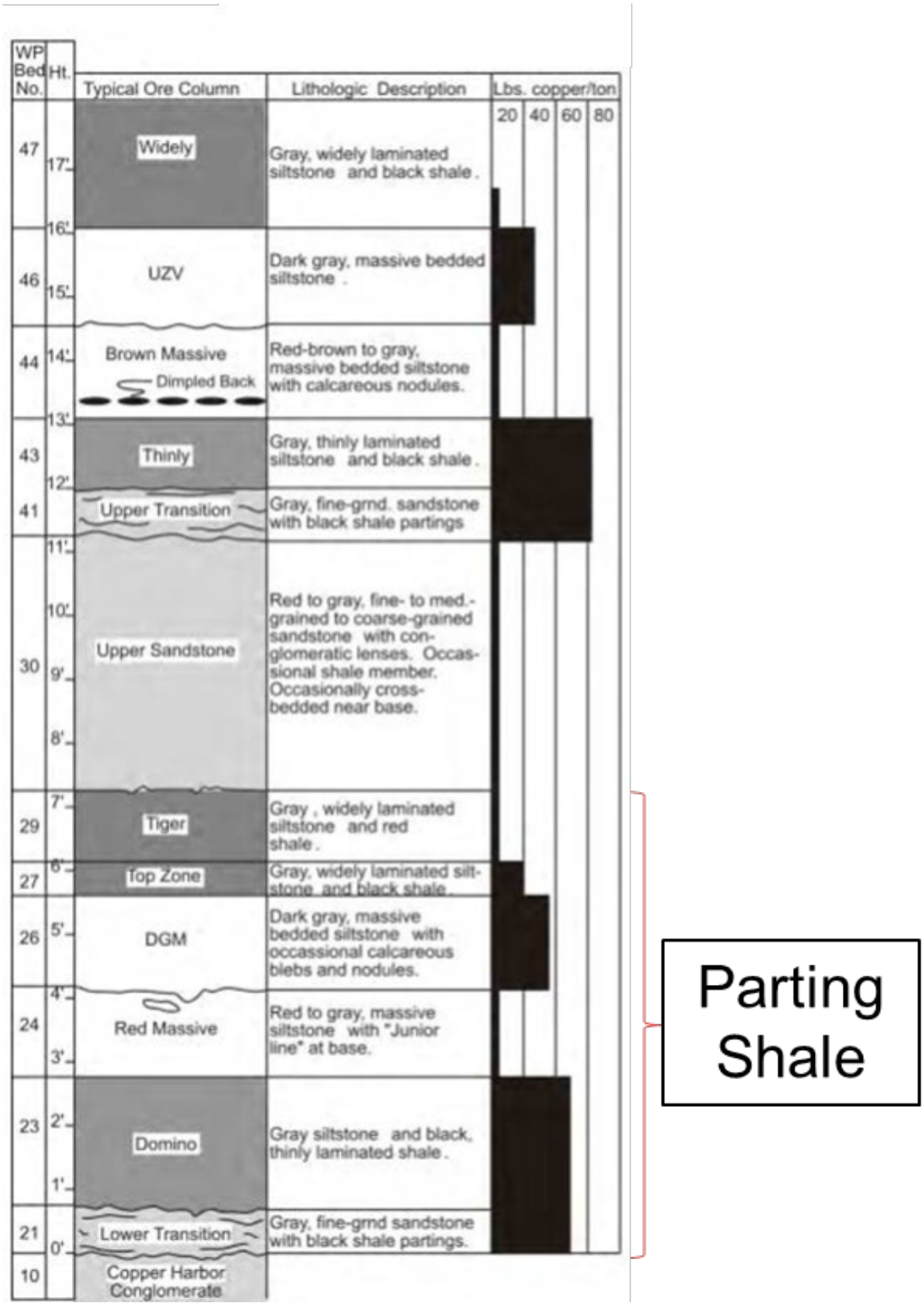
The White Pine North (WPN) copper project is located adjacent to the historic White Pine mine in the Upper Peninsula of Michigan, USA (Figure 1). The historic White Pine mine produced copper from 1953 to 1995 using mainly the room and pillar mining method to depths of approximately 750 m. White Pine Copper LLC (WPC) requested WSP to provide geotechnical guidance and recommendations for the WPN project which is targeting an orebody to the north of the historic mine at depths ranging from approximately 400 m to 1150 m.



Note: Showing the proposed WPN decline and panel accesses and the extents of the historic mine

Figure 1: Aerial Photo of the White Pine Area

The copper bearing mineralization at WPN is strata bound within the Nonesuch Formation. The ‘Parting Shale’ (comprises TIGR, TOPZ, DGMA, RMAS, DOMN, and LTRA), at the base of the Nonesuch Formation represents the target of interest for WPN (Figure 2). Above the Parting Shale is a sandstone layer ‘Upper Sandstone’ (UPSA) which does not contain mineralization. An upper zone of mineralization exists above the sandstone, but this is not currently an economic target for WPN. The mineralization in the Parting Shale is laterally persistent over a very wide area (approximately 9 km along dip and at least 5 km along strike), making the deposit amenable to tabular mining methods.



Source: Modified from G Mining Services, 2023

Figure 2: Stratigraphic Column of Copper Bearing Units and Those Near to the Copper Bearing Units

The proposed mining method is room and pillar and requires stable pillars to prevent surface impacts to meet permitting and environmental constraints. High horizontal stresses have been recognized in the historic mine and the scoping level geotechnical study recognized these high horizontal stresses as a key challenge for the mining depths of interest (SRK, 2024). A key output from the geotechnical assessments was the development of sequencing for the room and pillar mining method to address the stress challenges.

2.0 AVAILABLE DATA

Data available for the geotechnical study includes:

- Geotechnical information captured from two investigations, one in 2015 and one in 2024 (Figure 3, Table 1). Data available from these investigations include:
  - Geotechnical logging of drill core for 23 drillholes completed in 2015, which totals 7,512.57 m of logging data. Nine of these drillholes had geotechnical data collected for the entire length of the drillhole, the remaining drillholes only have geotechnical data recorded around the ore body and proposed mining area. Data provided from this campaign includes:
    - Point Load Tests (PLTs) both axial and diametral
    - Laboratory intact strength tests
    - Collection of geotechnical data (RQD, fracture count, discontinuity data, etc.)
    - Downhole acoustic televiewer (ATV) surveys, in PDF format, for nine drillholes (only eight were provided as excel databases).
  - Geotechnical logging of drill core from 27 drillholes and 29 wedges in 2024 which totals 21,902.31 m of logging data. Every drillhole, but two (WP578 and WP579) have geotechnical data for the entire length of the drillhole. WP578 and WP579 only have data near the top of the hole suggesting these holes may have been abandoned prior to completion. Data provided from this campaign includes:
    - Point Load Tests (PLTs) both axial and diametral
    - Core box photographs
    - Laboratory intact strength tests
    - Collection of geotechnical data (RQD, fracture count, discontinuity data, etc.)
    - Downhole Acoustic Televiewer (ATV) surveys, in PDF and Excel format, for 26 drillholes (2 of these drillholes do not have geotechnical logging data from either campaign; WP545 and WP547).
- Fault and lithological model developed by SRK (2025). The lithological model grouped the individual lithologies into groups (e.g., parting shale ore); two individual lithologies were separated into individual solids at WSP's request (Top Zone and Tiger).
  - The fault model does not contain a description of the ground characteristics around the fault zone.
- The mine design layout from the recently completed trade-off study (Stantec, 2024).
- Reports and documents from the historic White Pine mine:
  - Historic mine layout (Copper Range Company, 1995)
  - Various reports reflecting mine stability and some information on ground support (Parker, 1969; Agapito, 1992; Agapito & Gilbride, 2002; Itasca, 2024; Talvensaari, 1974; Agapito and Litsenberger 1993; Rock Mechanics Session, 1966; Hanninen et al., 1973; Fletcher et al, 1989; Agapito, 1994; Garfield, 1969; Parker, 1964).

- WPN scoping study (SRK, 2024)
- WPN Preliminary Economic Assessment (PEA) (G Mining, 2023)

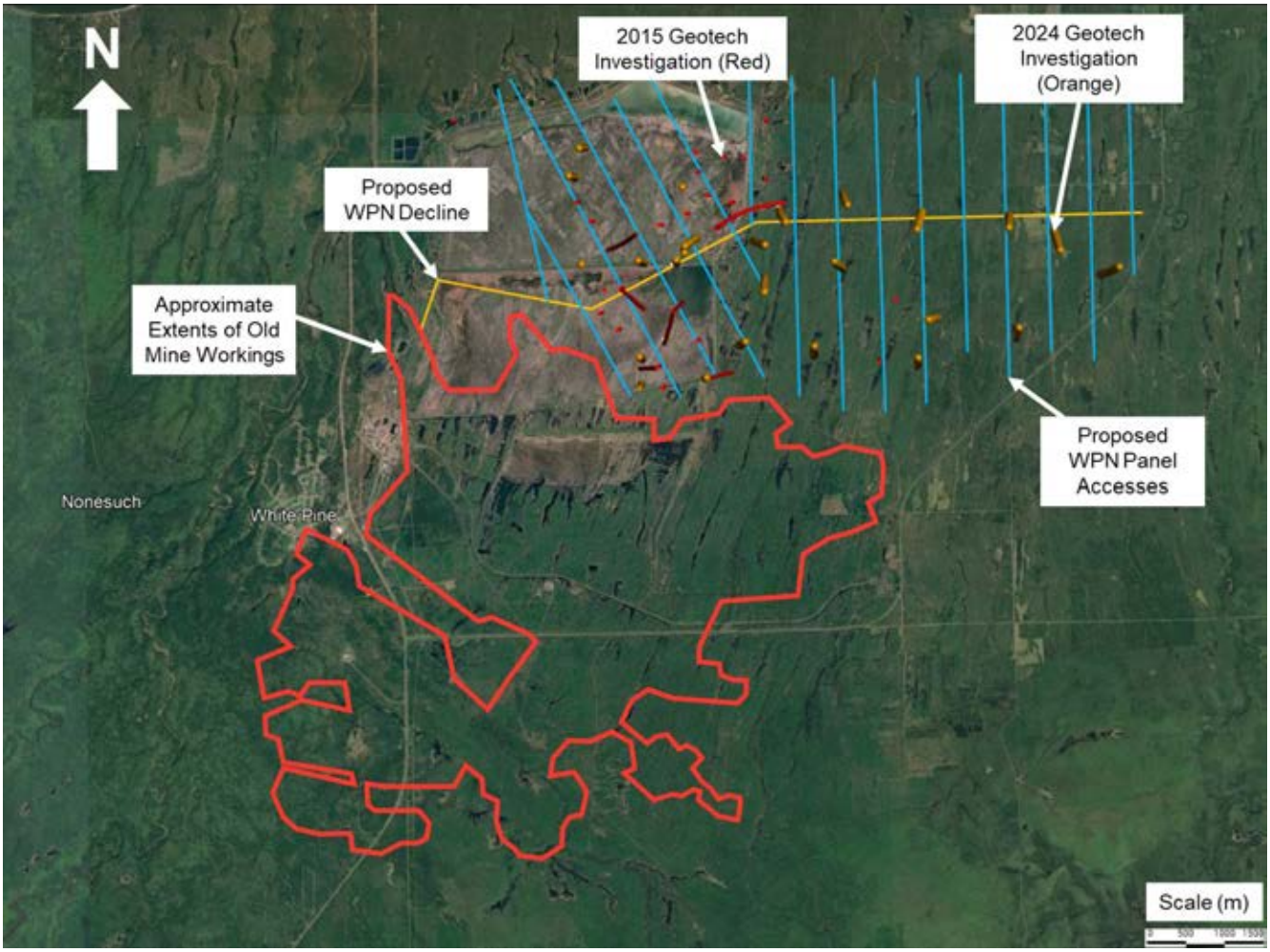


Figure 3: Plan View of the Provided WPN Layout, the Old Mine Workings, and the Geotechnical Investigation Data From 2015 and 2024

Table 1: Types of Data Collected by Drillhole ID

Drillhole ID	Wedges	Televiewer	Geotechnical Logging Targets	Length of Geotechnical Logging (including wedges)
WP527	0		Mining Horizon Only	30
WP529	0		Mining Horizon Only	27
WP530	0	Yes	Entire Drillhole	832
WP550	0		Mining Horizon Only	30
WP551	0	Yes	Entire Drillhole	602
WP552	0		Mining Horizon Only	25
WP553	0		Mining Horizon Only	24

Drillhole ID	Wedges	Televiewer	Geotechnical Logging Targets	Length of Geotechnical Logging (including wedges)
WP554	0		Mining Horizon Only	24
WP555	0	Yes	Entire Drillhole	786
WP556	0		Mining Horizon Only	30
WP557	0	Yes	Entire Drillhole	833
WP558	0		Mining Horizon Only	24
WP559	0	Yes	Entire Drillhole	569
WP560	0		Entire Drillhole	762
WP561	0		Mining Horizon Only	21
WP562	0		Mining Horizon Only	24
WP563	0	Yes	Entire Drillhole	1367
WP564	0		Mining Horizon Only	24
WP565	0		Mining Horizon Only	30
WP566	0	Yes	Entire Drillhole	600
WP567	0	Yes	Entire Drillhole	797
WP568	0		Mining Horizon Only	27
WP569	0		Mining Horizon Only	24
WP575	0	Yes	Entire Drillhole	572
WP576	0	Yes	Entire Drillhole	557
WP577	0	Yes	Entire Drillhole	557
WP578	0		Near Surface	148
WP579	0		Near Surface	10
WP580	1	Yes	Entire Drillhole	612
WP581	0	Yes	Entire Drillhole	794
WP582	0	Yes	Entire Drillhole	842
WP583	0	Yes	Entire Drillhole	703
WP584	0	Yes	Entire Drillhole	513
WP585	0	Yes	Entire Drillhole	707
WP586	0	Yes	Entire Drillhole	764
WP587	0	Yes	Entire Drillhole	871
WP588	2	Yes	Entire Drillhole	1093
WP589	0		Entire Drillhole	744
WP590	2	Yes	Entire Drillhole	979
WP591	3	Yes	Entire Drillhole	1040
WP592	2	Yes	Entire Drillhole	1145
WP593	2	Yes	Entire Drillhole	1046
WP594	3	Yes	Entire Drillhole	1077
WP595	0	Yes	Entire Drillhole	880
WP596	2	Yes	Entire Drillhole	997

Drillhole ID	Wedges	Televiewer	Geotechnical Logging Targets	Length of Geotechnical Logging (including wedges)
WP597	2	Yes	Entire Drillhole	992
WP598	2	Yes	Entire Drillhole	1013
WP599	3	Yes	Entire Drillhole	938
WP600	2	Yes	Entire Drillhole	1104
WP601	3	Yes	Entire Drillhole	1135

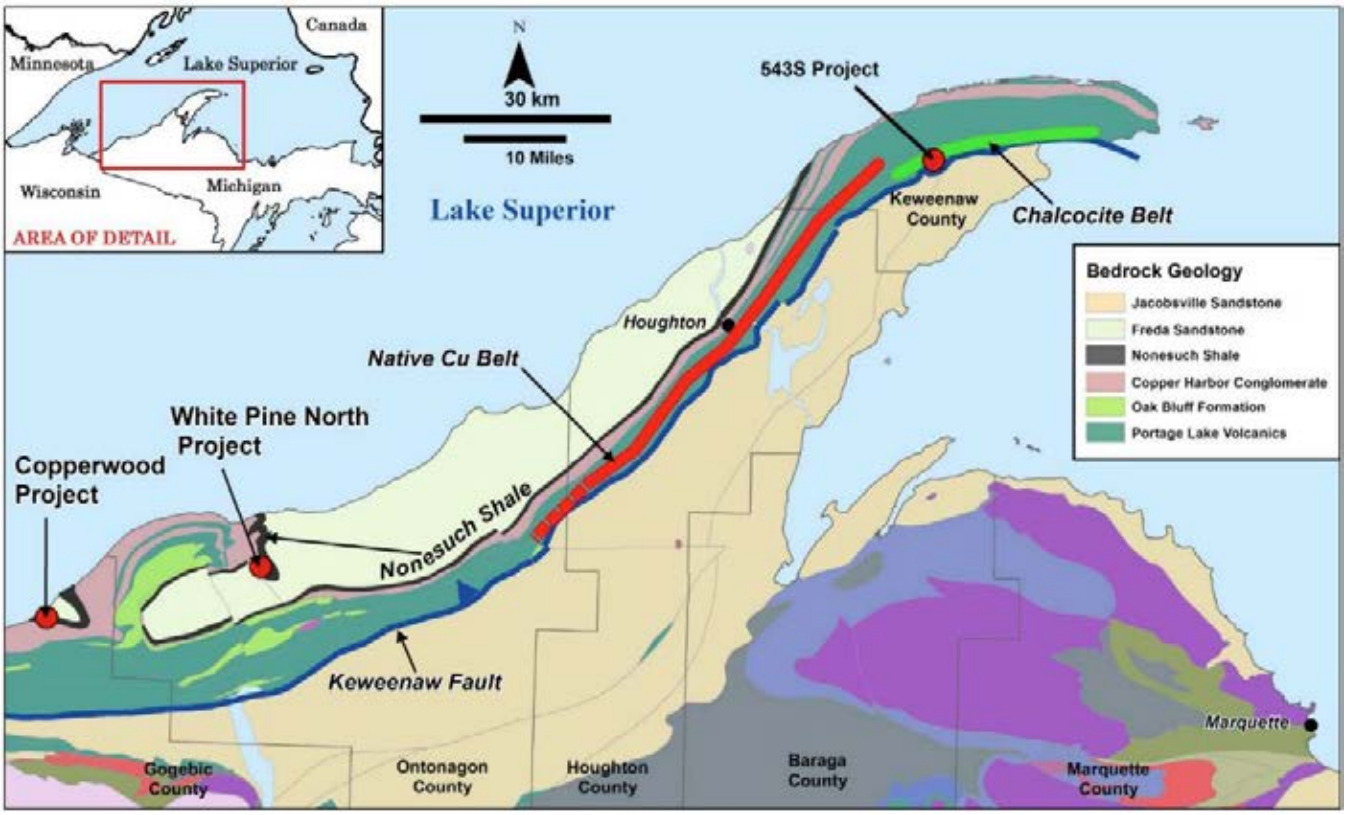
Note: WP545 and WP547 are not included in the above table as they do not have logging information and only have downhole televiewer data.

3.0 GEOLOGY AND OREBODY GEOMETRY

The geological and orebody knowledge available for WPN is derived from the description of the geological setting in the PEA (G Mining, 2023) and the orebody geometry and knowledge are derived from the lithological model (SRK, 2025) and the grade shells provided to WSP (Stantec, 2025).

3.1 Regional Geology

The WPN project is located in the Western Upper Peninsula of Michigan at the southern edge of the Canadian Shield on the south end of Lake Superior (G Mining, 2023) (Figure 4). The stratigraphy of the WPN project is contained within the Oronto Group comprising of (from surface downwards) the Freda Formation (mainly comprised of fluvial sandstones), the Nonesuch Formation (the host of the mineralization and comprised of interbedded siltstone, mudstone, shale, and sandstone), and the Copper Harbor Conglomerate (which interfingers with the base of the Nonesuch Formation) (G Mining, 2023). The ore is hosted at the base the Nonesuch Formation where the formation contacts the Copper Harbor Conglomerate (G Mining, 2023). The ore zone and surrounding rock masses comprise a mixture of shales, siltstones, and sandstone layers ranging from a few centimetres to a few metres thick.



Source: G Mining, 2023

Figure 4: Regional Geology of the Upper Peninsula of Michigan

3.2 Local Stratigraphy

The lithological model local to WPN was produced by SRK (2025) and provided to WSP. This model contains definition of lithological units from surface to the floor of the mineralized zone. Each lithological unit has some relevance to the selected mining method (room and pillar) and may be incorporated to some degree in stability assessments. The stratigraphy at WPN and its relevance to the mining method is summarized in Table 2. A short description for each term in the mining relevance column is provided below:

- **Crown**: This represents the rock mass that will comprise the crown pillar for the proposed mine. This rock will need to be described to a reasonable degree of confidence to assess potential for surface impacts and crown pillar stability.
- **Hanging Wall**: This represents the rock mass that will comprise the backs (roofs) of the rooms. These rock masses are incorporated when assessing the back stability of the mine openings.
- **HW / top of pillar**: The Tiger and Top Zone are weak shale rock masses which are typically located in the back of the rooms and top of the pillars of the proposed mine.
- **Ore (pillar)**: This represents the rock masses that contain the ore mineralization targeted by the proposed mine. The pillars in the proposed mine will mostly contain these rock masses.
- **Footwall**: This represents the rock masses in the floor or beneath the floor in the proposed mine.

Table 2: Summary of the White Pine North Stratigraphy and WPN Mining Relevance

Formation	Lithology	Code	Relevance to WPN Mining
Freda	Overburden	OVBD	Overburden
	Undifferentiated Freda	UNDF	Crown
Nonesuch	Undifferentiated Nonesuch	UNDN	Crown
	Marker Bed	MARK	Crown
	Massive Black Siltstone	MBSI	Crown
	Stripey	STRP	Crown
	Tiebel Sandstone	TBLS	Crown
	Massive Gray Siltstone	MGSI	Crown
	Red and Grey	RAGR	Crown
	Widely	WIDE	Hanging Wall
	Upper Zone of Values	UZVA	Hanging Wall
	Brown Massive	BMAS	Hanging Wall
	Thinly	THIN	Hanging Wall
	Upper Transition	UTRA	Hanging Wall
	Upper Sandstone	UPSA	Hanging Wall
	Tiger	TIGR	HW / top of pillar

Formation	Lithology	Code	Relevance to WPN Mining
	Top Zone	TOPZ	HW / top of pillar
	Dark Grey Massive	DGMA	Ore (pillar)
	Red Massive	RMAS	Ore (pillar)
	Domino	DOMN	Ore (pillar)
	Lower Transition	LTRA	Footwall
Copper Harbor Conglomerate	Lower Sandstone	LWSA	Footwall
	Copper Harbor Conglomerate	CCGL	Footwall

3.3 Orebody Geometry

The WPN claim boundary is approximately 9 km (E-W) by 5 km (N-S) and contains relatively low grade ore (<2% Cu) dipping to the East (between 0° and 20°) and varying in depth from approximately 250 m to 1200 m. The provided grade shell encompasses most of the claim boundary (Figure 5 and Figure 6) with some gaps where mining is not economic. The grade shell thickness is a minimum of 2 m and encompasses, at a minimum, the DOMN to DGMA lithologies (known as the Parting Shale) and can occasionally include parts of the LTRA, TOPZ, TIGR, and LWSA (where the ore zone is thinner than the minimum grade shell thickness). There is a thinner mineralized zone above the UPSA which is not targeted by WPN; this upper mineralized zone is part of the lithology grouping called Upper Siltstone (THIN to WIDE).

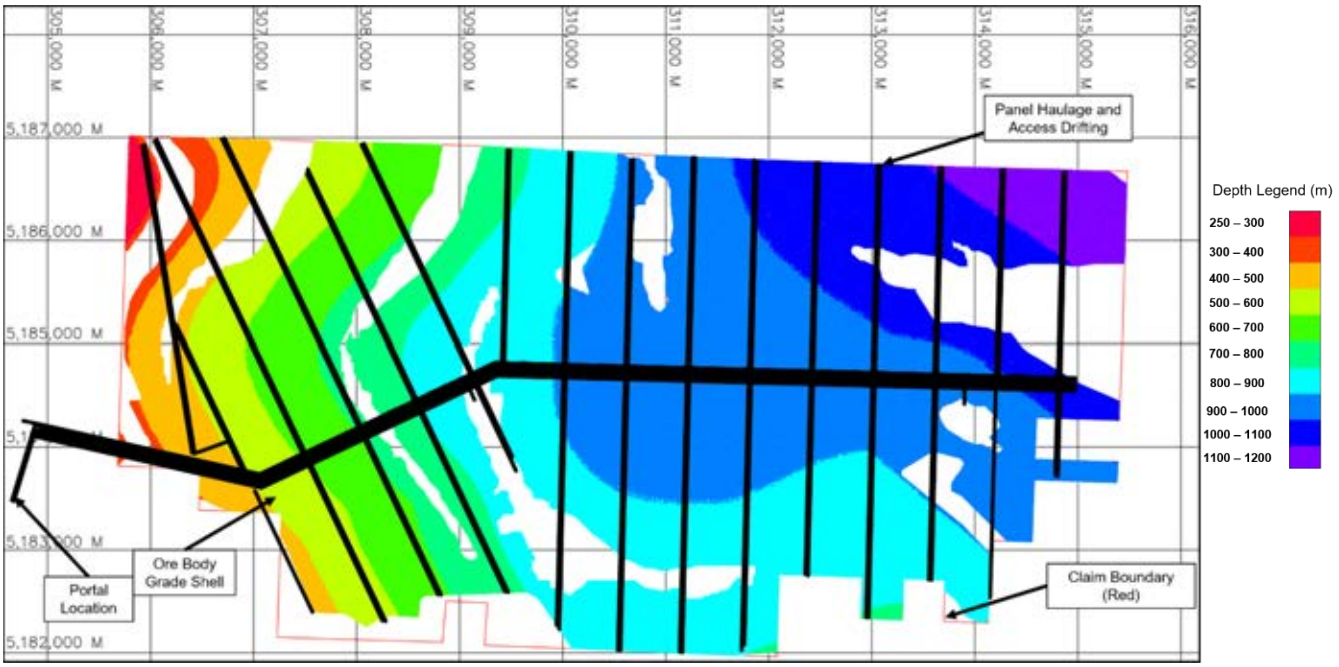


Figure 5: Grade Shell Contoured by Depth Increments in Meters Below Ground Surface (bgs)

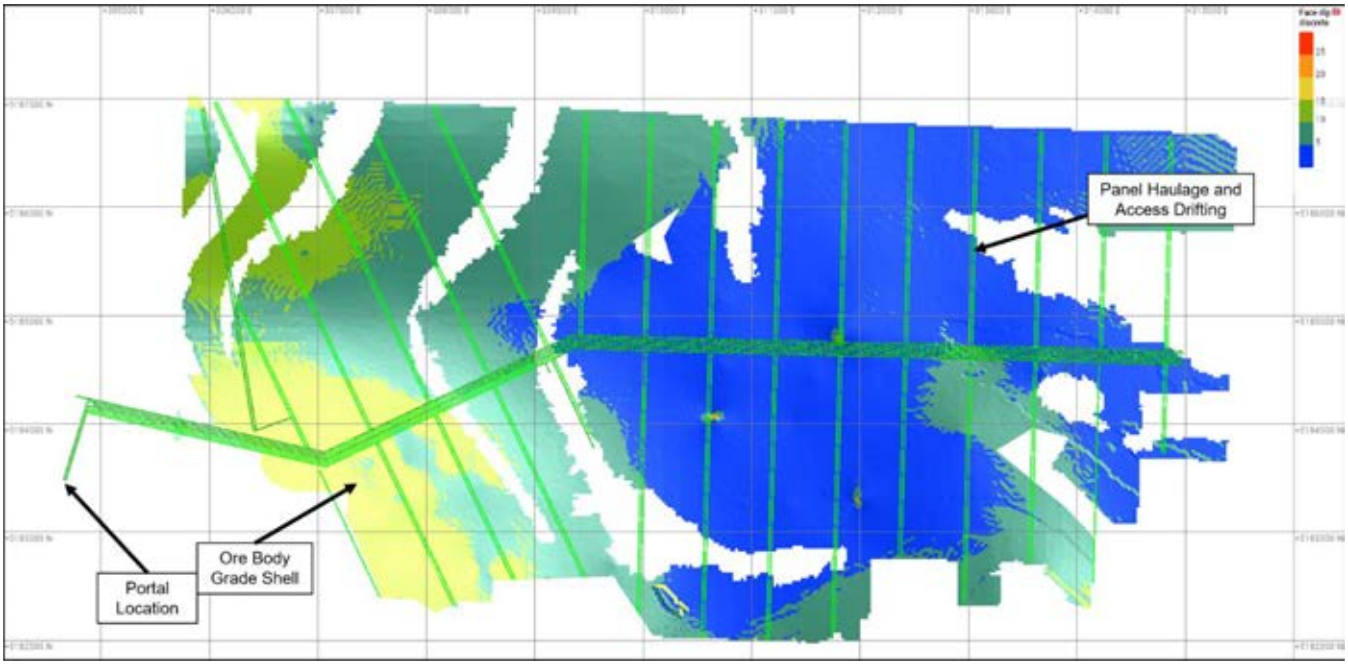
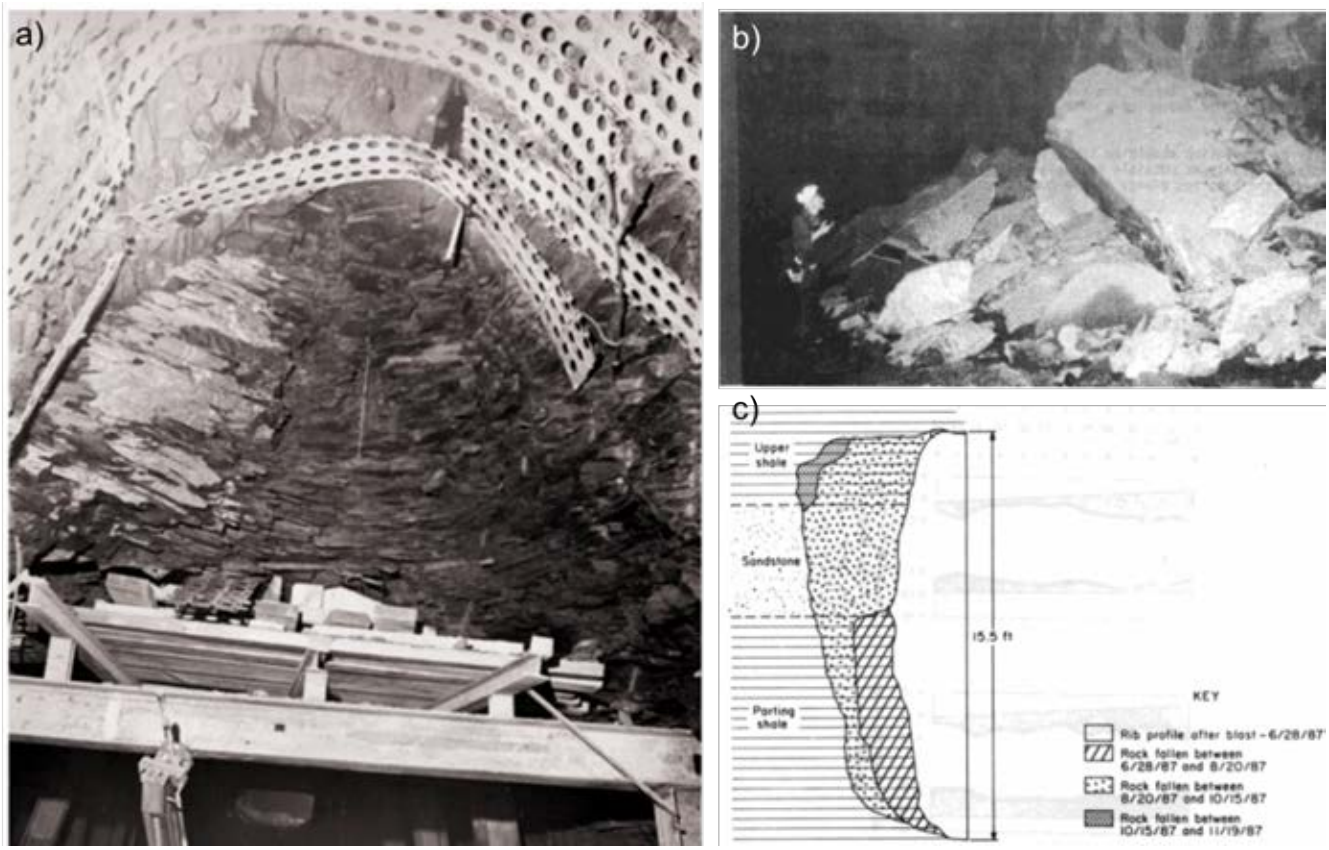


Figure 6: Grade Shell Contoured by 5-degree Dip Increments

4.0 GEOTECHNICAL CONTEXT FROM THE HISTORIC MINE

Mining in the historic mine began in 1953 and last production was in 1995. Most mining was done using the room and pillar method to depths up to 700 m. In most areas, both the Parting Shale and Upper Siltstone mineralized zones were targeted for mining resulting in mining heights of approximately 4.3 m on average (Agapito, 1992). Management of high horizontal stresses and instability of the back (roof) is a common theme in the historical reports. Figure 7a shows an example of a back fall above a conveyor decline (Agapito & Gilbride, 2002) where mechanically anchored bolts were unable to support the thinly bedded shales and siltstones. The straps and steel sets shown in the photo were installed as part of rehabilitation efforts.

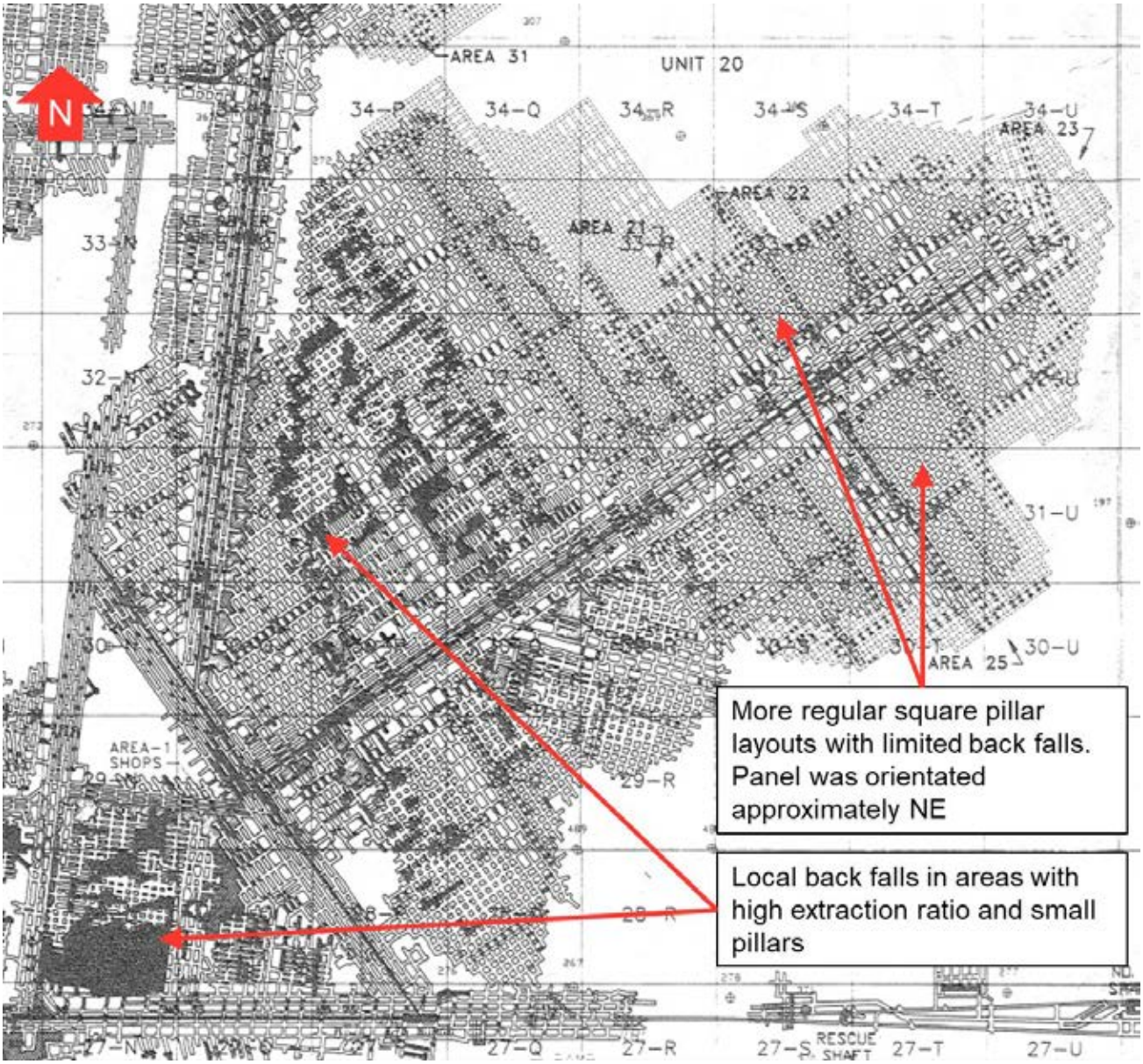


Note: a) Back failure in a conveyor decline in thinly bedded shales and siltstones (Agapito & Gilbride, 2002), b) photo of large blocks of sandstone which unravelled from the pillar walls after gradual spalling of the Parting Shale below (Fletcher et al. 1989), c) schematic showing an example sequence of pillar wall damage (Fletcher et al. 1989).

Figure 7: Historic Geotechnical Context

The back conditions were generally reported to be good in production areas where the full column was mined (Fletcher et al., 1989). In areas where faulting was observed, rock bolts spacing was tightened in the back. The main geotechnical hazard in the Unit 95 area evaluated by Fletcher et al. (1989) were large blocks of sandstone unravelling from the pillars (Figure 7b and Figure 7c). This was described as a gradual process where the shales would be observed to gradually spall and undercut the sandstone which would eventually unravel in large blocks. Blast damage was also considered to be a key contributor to the rock falls from the ribs of the pillars and testing of controlled blasting was shown to improve conditions in the ribs.

Of particular relevance to the WPN project, is the commentary from the North East (NE) Mine, the deepest portion of the historic mine (Agaptio, 1992). The NE Mine was described as being situated within a ‘high-stress environment’ but where jointing and faulting were much less pronounced in comparison to the shallower portions of the mine. When instability in the backs of the rooms was observed, it was noted that the depth of overburden correlated with deeper failures. At 700 m depth, the maximum depth of instability in the backs of the rooms was noted to be 0.6 to 1.5 m. However, it was noted that for the most part, the backs remained stable when bolted, even when some stress fracturing was observed. At the time of the NE mining, resin grouted rebar had been fully adopted with lengths up to 10 ft. (3 m) on 4 ft. (1.2 m) centers. Prior to this, mechanically anchored bolts were applied which were often observed to fail in challenging ground conditions and were found to be more susceptible to corrosion (typically over a time span of 10+ years).



Note: The dark shaded areas represent back falls.

Figure 8: Plan View Showing the NE Mine Layout in 1995

Solution mining was also trialed at the historic mine using mass blasting techniques resulting in large, caved zones extending to surface (Agapito, 1994). These caved zones were intentional and not the result of challenging geotechnical conditions. However, observations of pillars adjacent to the caved zones, which were highly stressed due to redistribution of stresses from the caved zones to the abutments, confirms that the dominant mechanism of instability in the pillars is stress fracturing leading to spalling / sloughing (Figure 9). These highly stress damaged pillars were not however observed in the areas of mining with conventional room and pillar mining.

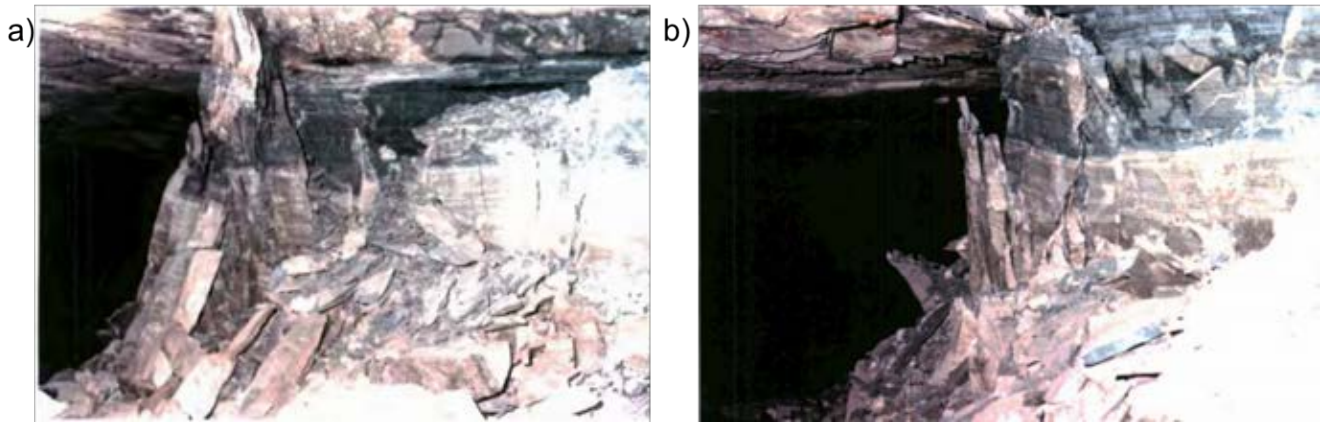
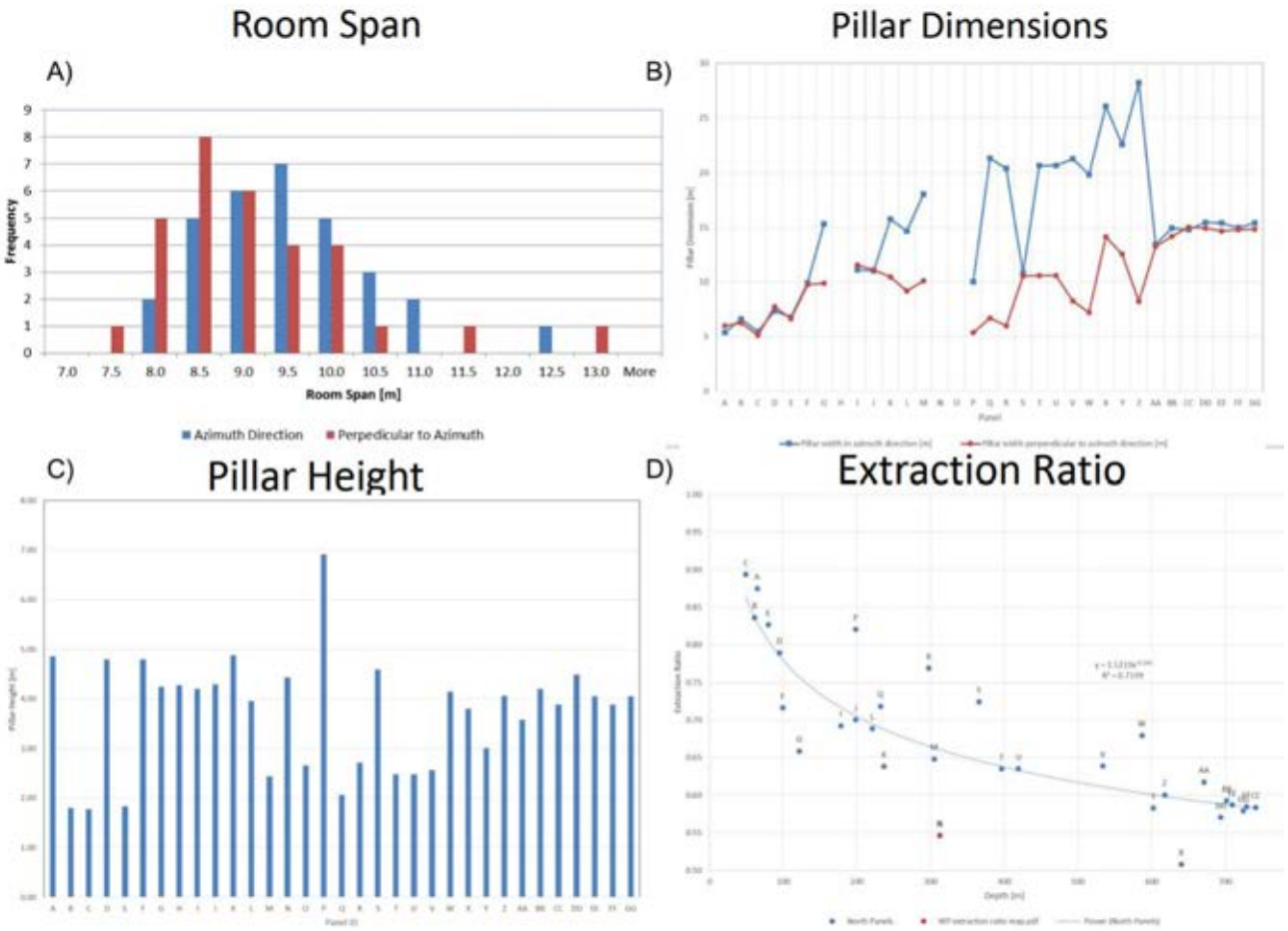


Figure 9: Examples of Stress Fracturing and Spalling of Pillar Edges Adjacent to Caved Zones for Solution Mining Demonstrating that the Pillars are Brittle and Fail by Stress Induced Fracturing

Parker (1969) noted that moisture has been observed to have a control on the long-term strength of the rock at White Pine. Mining areas adjacent to ventilation shafts, which were exposed to wet / dry cycles due to seasonal changes in humidity and temperature, often showed gradual strength loss which led to local collapses in areas with high extraction ratios and small pillars. Laboratory testing also suggested that the wet strength of the rock is approximately half the dry strength. No swelling clay minerals were reported to be found in the rocks, although the analysis methods were not described. Shotcrete was promoted to 'seal' the rock from moisture introduced from humidity and conserve the rock strength.

4.1 As-Built Mine Geometry

More recently Itasca (2024) completed a review of the achieved mining dimensions and estimated pillar loads experienced by the historic White Pine mine (Figure 10). This review found that the historic mine originally had extraction ratios above 0.8 during shallow mining but that extraction ratio dropped to 0.5 at the lowest parts of the mine. Achieved pillar sizes are varied across the mine but were consistently 15 m (square) at the lowest areas of the mine. Based on assumed inputs Itasca (2024) estimated that most pillars achieved a minimum of 1.5 factor of safety (FoS) with the deepest areas being close to 1.5 FoS. Itasca's review does not indicate which method was used to estimate the pillar FoS. Again, it is important to note that the historic mining was done to full column heights, and therefore the achievable extraction ratios are lower due to taller pillar sizes.



Note: A) Measured Historic Room Span. B) Pillar Size (Square Pillars) by Historic Mining Panel. C) Pillar Height by Historic Mining Panel. D) Historic Extraction Ratio by Depth.

Figure 10: As-built Mine Geometry

4.2 Information Corroborated by Former White Pine Employees

The historical reports were also corroborated by former White Pine employees (personal communications with Mr. Daniel Saint Don – Mine Engineer and Mr. John Litsenberger – Ground Control Engineer) who noted that 'flat back' conditions were routinely observed with primary mining. It wasn't until pillars were fully rounded by slabbing in retreat, that some observations of back instability were made in the rooms. The backs in the rooms were sometimes observed to fail locally but many were also observed to be stable with 'flat backs'. The back failures were not associated with global mine stability issues in the NE mine, and the mine had a system in place to limit personnel entry after pillars were rounded in retreat and this was not considered to be a major geotechnical concern. Alignment of mining panels with the inferred maximum horizontal stress was noted to be favourable for back stability with primary mining.

Stress fracturing and sloughage of pillar walls was often observed after pillars were rounded as these walls were unsupported. The sloughage was generally superficial and not associated with failure of the cores of the pillars. However, the pillars were made larger with increasing depth to address the stress damage to the outer portion of the pillars. The pillar wall sloughage was not considered to be an operational challenge at the time. The health and safety risk of wall sloughage was managed by personnel avoiding close proximity to the pillar walls.

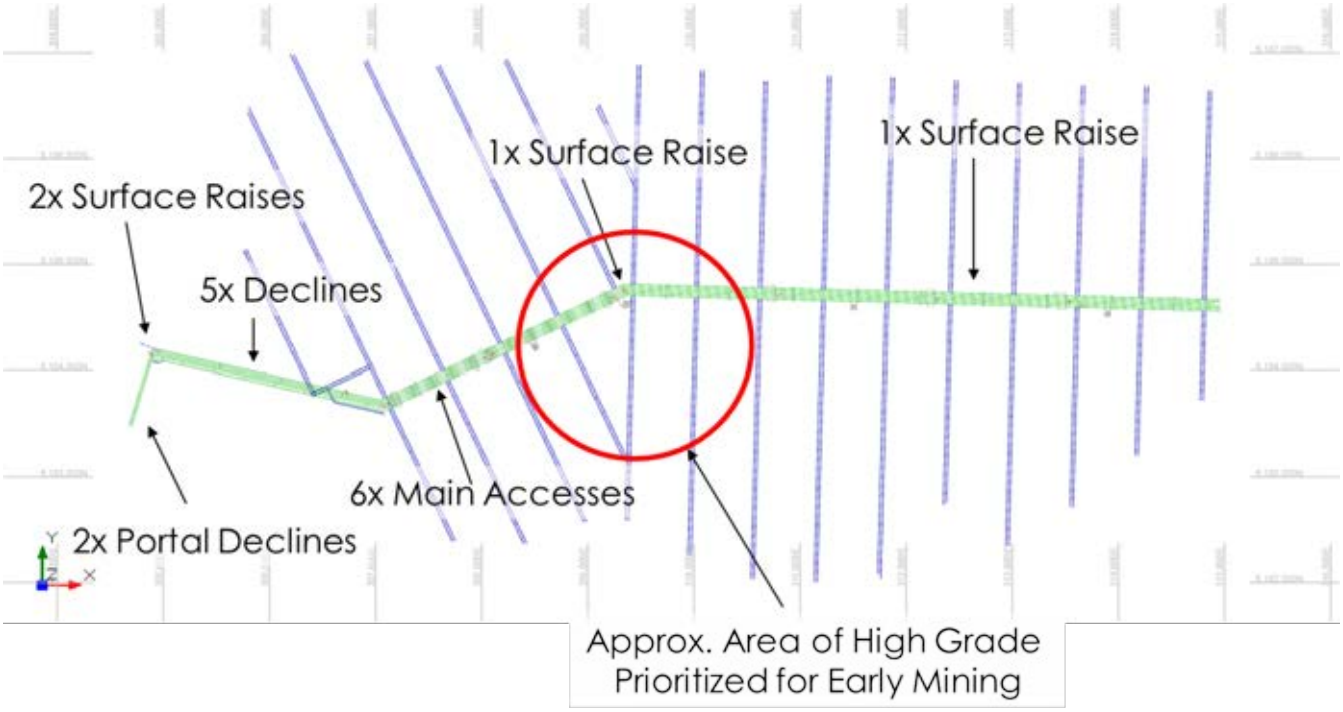
Specific to ground control, it was noted during conversations with the former White Pine employees that:

- Back failures were common in the NE mine and could extend 3-to-4 ft into the backs of rooms.
- The switch to resin rebar bolting was found to improve ground conditions and limit some of the damage previously observed in the mine.
- Stable back was typically found to be along the ‘dimple back’ which was a bedding plane with calcium nodules within the BMAS lithology.
- Shotcrete was rarely utilized at site unless there was a specific need (e.g., near the ventilation raises to mitigate moisture-based strength loss of the rock masses).
- Fault intersections in the headings were typically associated with poor ground characteristics and performance.
- Water presence in the mine was limited, and most areas were considered dry. Water had limited influence on mine stability (outside of areas near ventilation raises which may have experienced moisture-based strength loss).
- Corrosion of rockbolts was observed in localized areas but did not impact rockbolt performance until many years after installation.
- The historic areas of collapse within the room and pillar areas were due to retreat pillar robbing and removal of pillar cores.

5.0 MINE ACCESS, MINING METHOD, AND PLAN

WPN is proposed as a room and pillar mine with mine access proposed via two portals at the west end of the deposit, near the historic mine surface infrastructure. Initial geotechnical guidance and recommendations for the portals have been prepared under separate cover (WSP, 2025). Two declines from surface are split into multiple accesses as drifting approaches the ore body. Originally, the two declines split into five and then six main accesses (Figure 11); however, WPC has produced multiple updates to the access layout and drift geometry – the latest iteration only has four main declines through the ore body. The geotechnical assessments reflect the latest proposed main access layout (four accesses); however, the conceptual mine images produced by WPC, and included in this section, have not been updated to reflect this layout and instead show the version with six main accesses. The alignment of the main accesses has not changed from what is shown in this section.

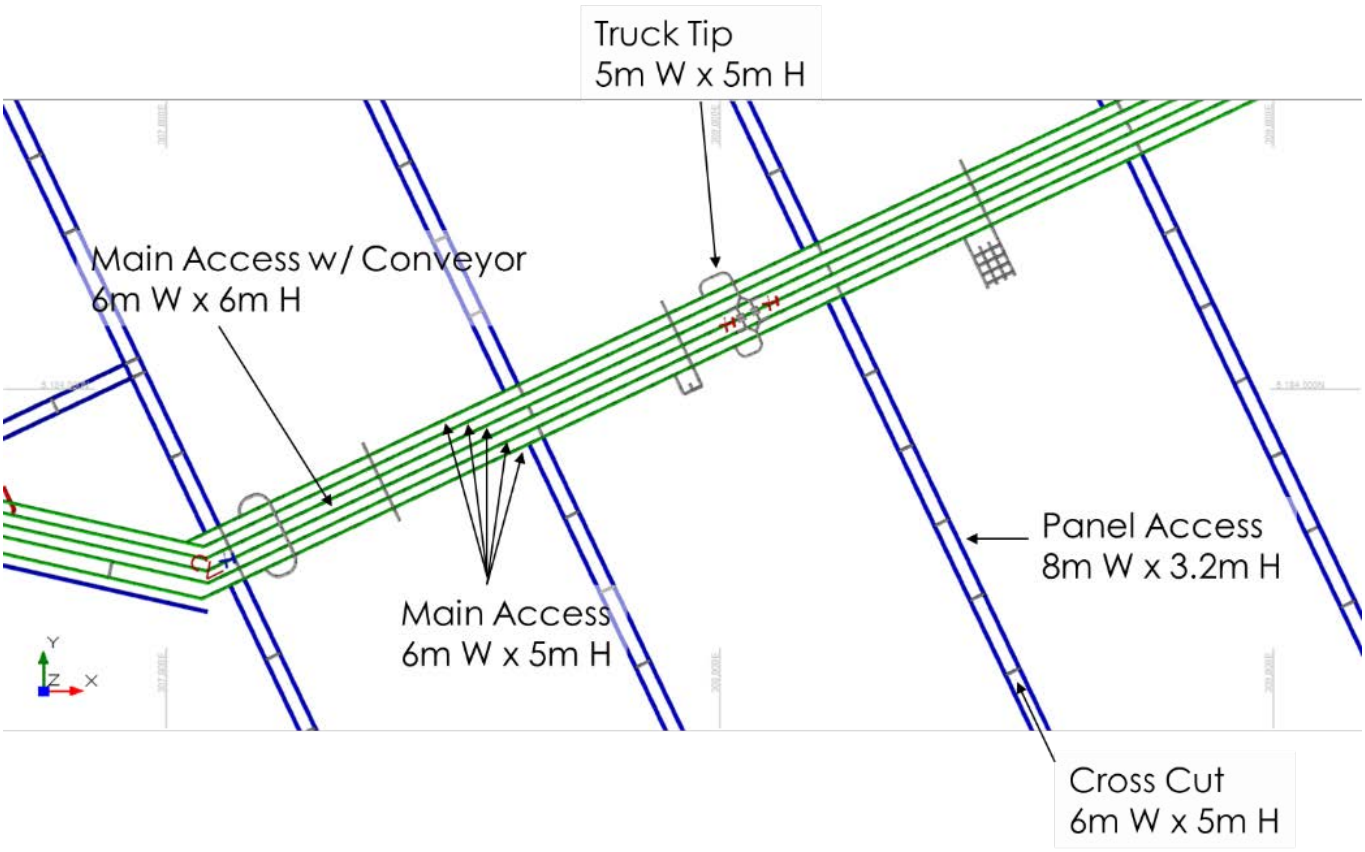
The declines are proposed to be excavated approximately in the center of the ore deposit, in an approximately East-West trend, providing central access to mining panels. The detailed dimensions and configuration for the proposed for the central declines are shown in Figure 12 (the outdated 6 main accesses version is shown). A detailed sequence of the mine access development has not been provided to WSP, but it is assumed that the central declines will be advanced approximately in parallel to reach the orebody and the first panels targeted for mining.



Source: Stantec, 2025

Note: Showing the approximate area of high grade ore that is prioritized for early mining.

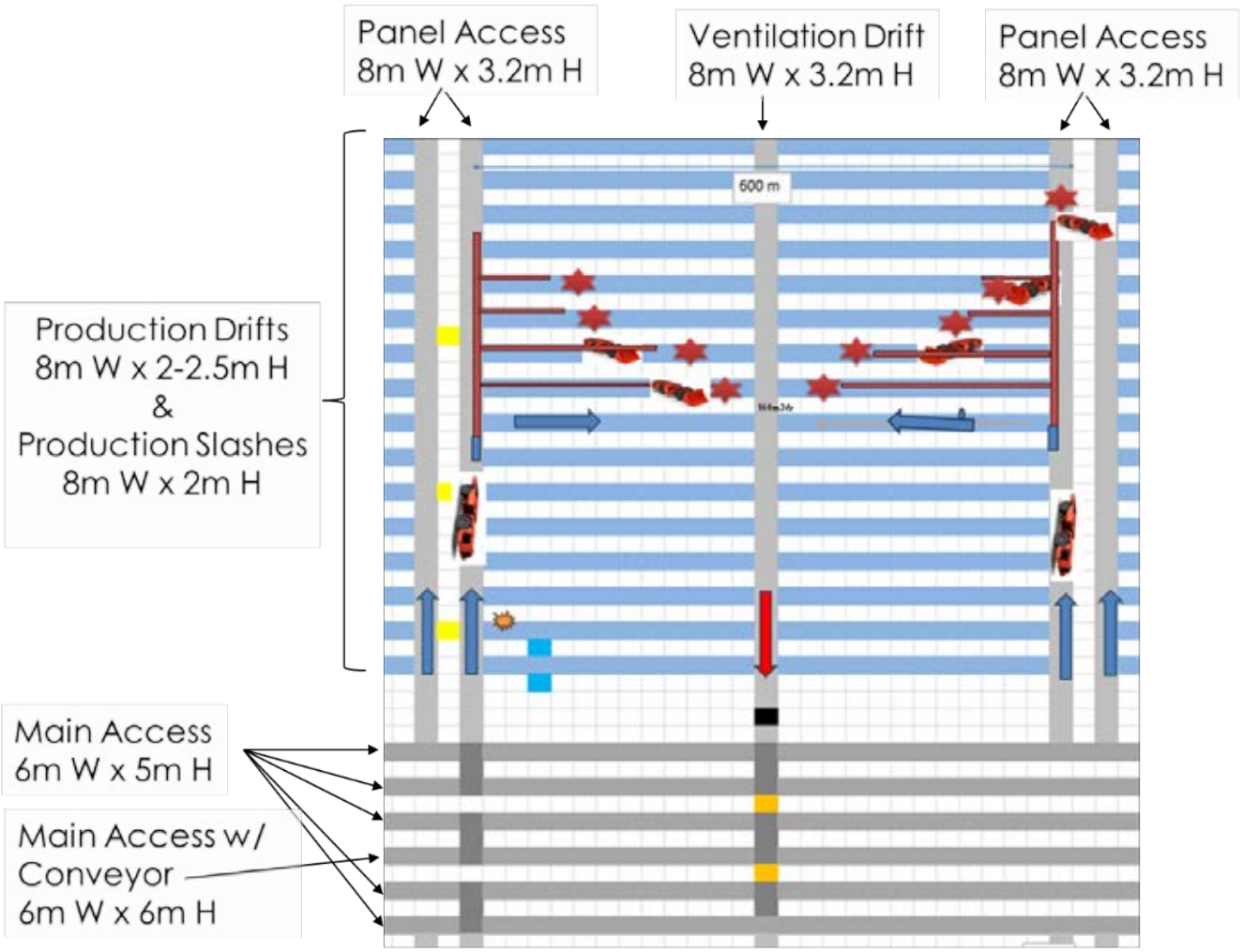
Figure 11: Plan Image of the Proposed Mine Access Configuration for WPN



Source: Stantec, 2025

Figure 12: Plan Image Showing the Details of the Declines Within the Orebody

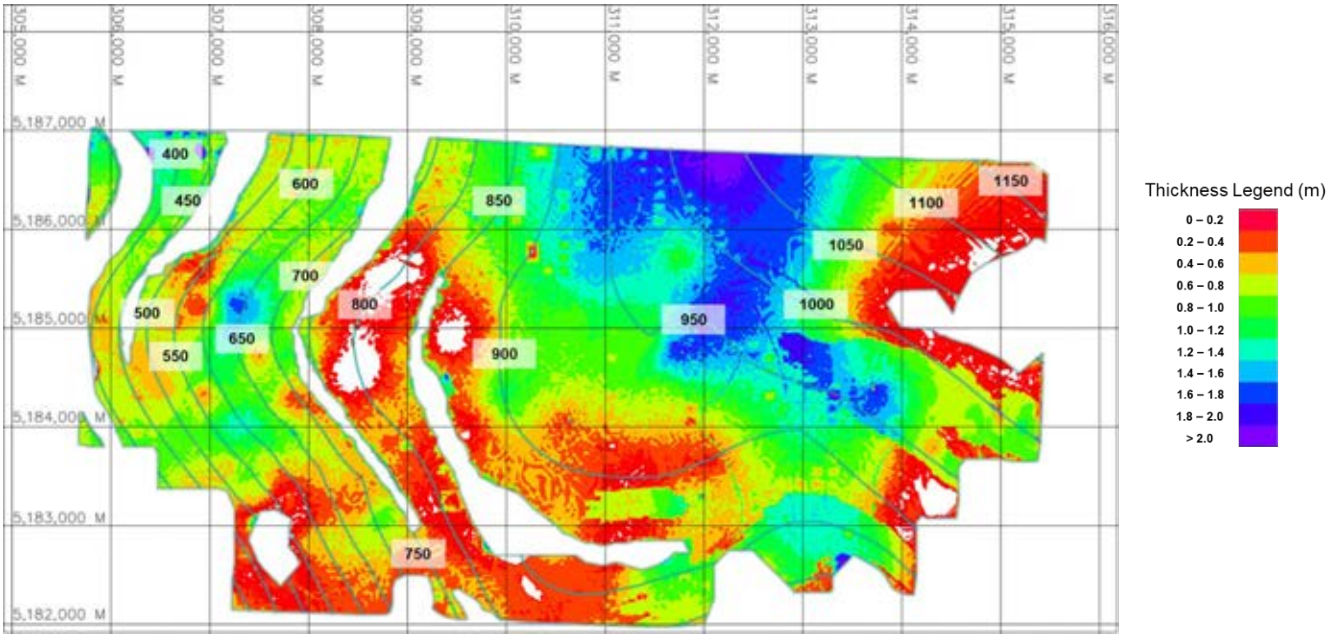
The mining method being evaluated for WPN is conventional room and pillar, with approximately 300 m wide panels extending off the central declines. The panel access arrangement and drift/drive sizes are shown in Figure 13. WSP understands that the current mine plan involves early production from shallow dipping panels, while advancing the central declines to a high grade zone at approximately 800 m depth (Figure 11). Additionally, at WSP's recommendation, the proposed mining sequence involves primary drive development and then, in retreat, the cross cut 'stubs' are excavated and mucked with remote loaders such that the backs of stubs do not need to be supported and personnel are not exposed to the highest mining induced stress changes.



Source: Stantec, 2025

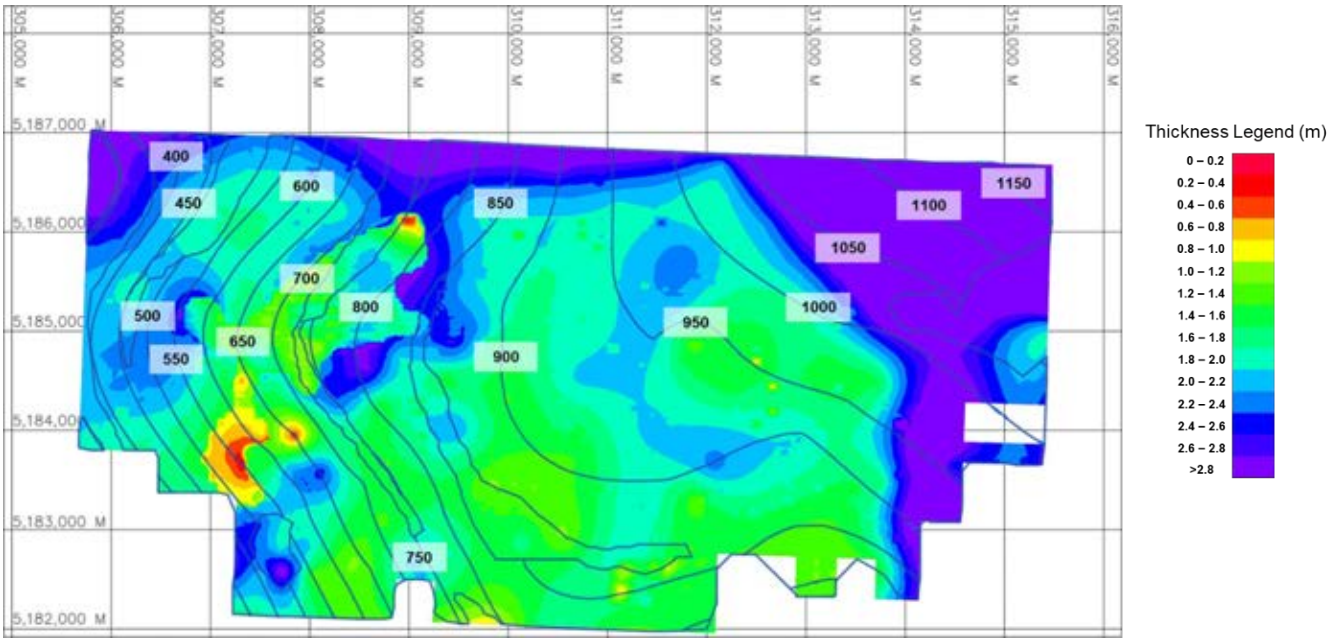
Figure 13: Plan Image Showing the Proposed Production Block Access and Primary Mining Sequence

The backs of the rooms will typically be comprised of the TOPZ and TIGR shale units overlain by the UPSA sandstone layer. The thickness of the TOPZ and TIGR varies across the site, and some areas will have the back exposed only within the UPSA. Figure 14 shows the potential thickness of TOPZ and TIGR estimated to be located in the backs of the rooms while Figure 15 shows the thickness of the UPSA across the entire claim boundary. These three units (TOPZ, TIGR, and UPSA) will be important for geotechnical assessment of the mine design as they will likely control the back stability. There will be some areas with minimal shale in the back of the rooms and other areas which will have > 1.5 m of shale in the room backs.



Note: Grade shell shown is an earlier iteration of the grade shell that was based on a 2-m room height.

Figure 14: Potential Thickness of the TOPZ and TIGR in the Back of a Room with a 2-M Height



Note: Thickness surface is clipped to the claim boundary and not to a specific grade shell.

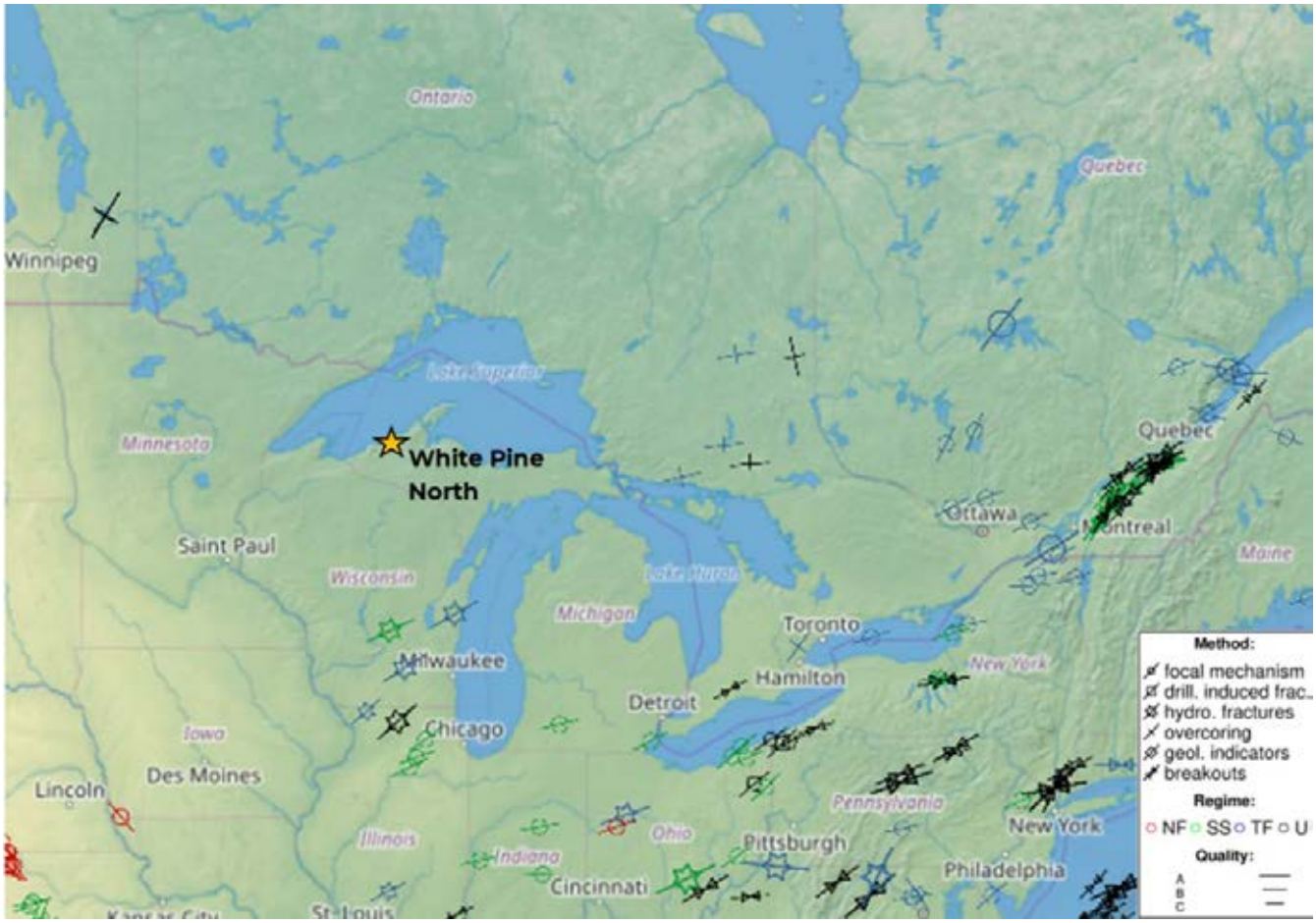
Figure 15: Thickness of the UPSA with Depth Contouring

6.0 IN SITU STRESS

In situ stress assumed for assessments at WPN are based on nearby historical in situ stress tests, regional information on in situ stress in the Canadian shield, and publicly available databases. Given the depth of the grade shell (up to 1200 m) in situ stress is likely to be a key driver for mine geometry and ground support in some areas of WPN.

6.1 Regional Context from Publicly Available Databases

The World Stress Map (WSM) (Heidbach et al., 2025) is a database that compiles public information related to in situ stress magnitude and/or orientation estimates. The database includes (but is not limited to) in situ stress measurements (e.g., overcoring and hydraulic fractures), estimation of in situ stress orientation from seismic events, and estimation of in situ stress orientation from borehole information (e.g., breakouts). The WSM is a reasonable first indicator for regional stress conditions which may apply at WPN (Figure 16). The nearest indicators on the WSM are from overcoring and hydraulic fractures which indicate NE-SW to E-W trend to the maximum horizontal stress and a combination of strike slip and thrust fault regimes.

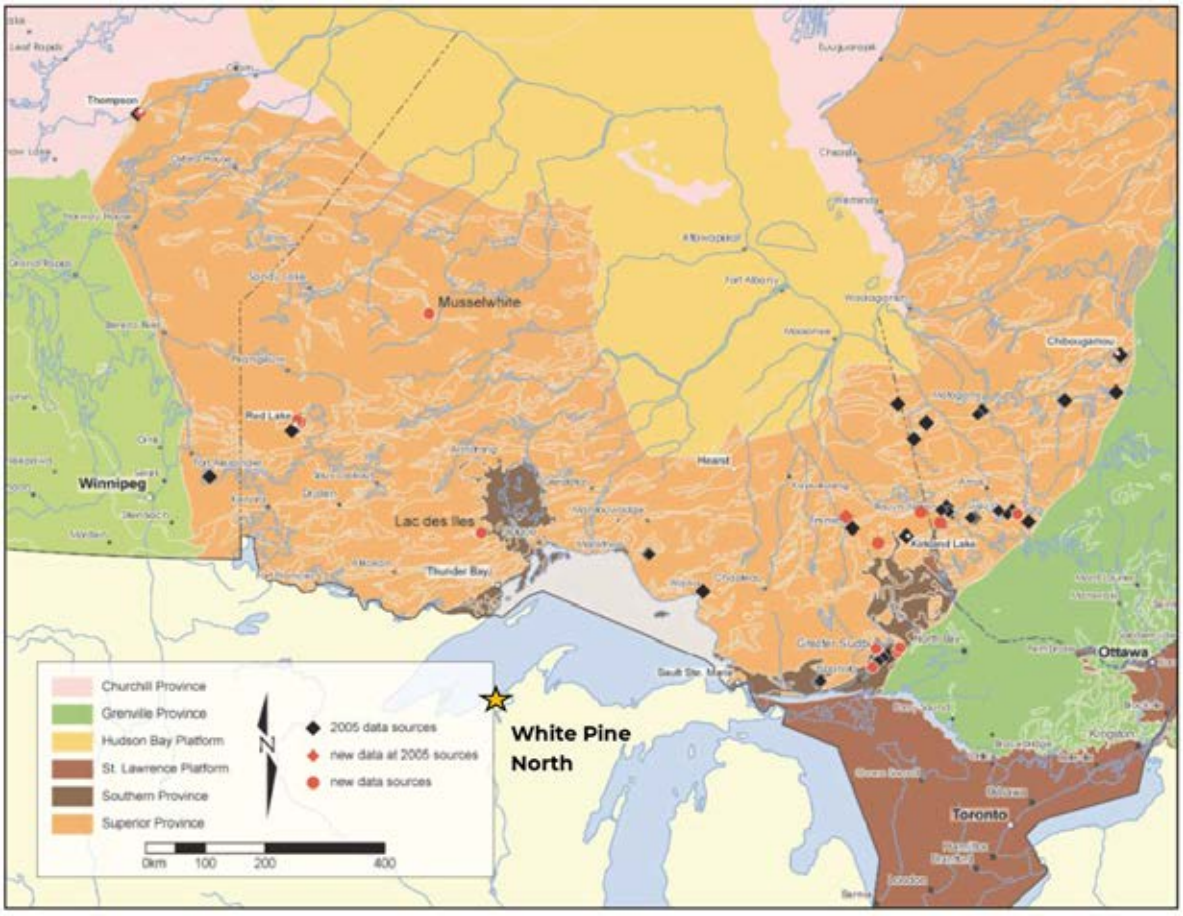


Note: Showing regional stress indicators.

Figure 16: World Stress Map Output

The regional in situ stress context estimated for the Canadian Shield is also applicable to the WPN project. Stress measurements, mainly from Canadian hard rock mines, have been summarized by Yong and Maloney (2015) and can provide a reasonable understanding of the range of in situ stress magnitudes and orientations for the Canadian Shield. The locations of stress measurement data compiled by Yong and Maloney (2015) relative to the White Pine North project are shown in Figure 17. Figure 18 shows the ratio of maximum and minimum horizontal stresses relative to vertical stress (KH and Kh, respectively). Key observations from the Canadian Shield dataset are:

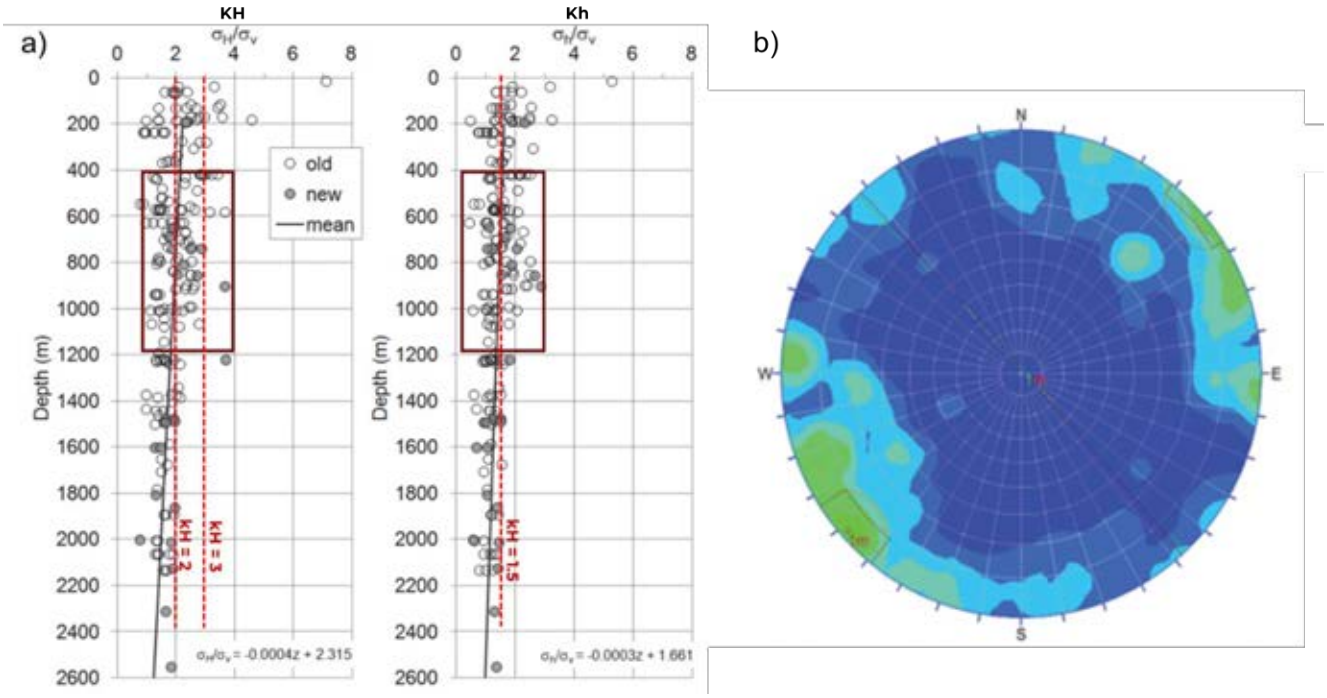
- There is considerable scatter in the magnitudes but the trends show that both KH and Kh tend to decrease with increasing depth.
- Within the mining depths of interest at WPN, the mean KH is approximately 2 with an upper bound of 3 (ignoring outliers).
- Within the mining depths of interest at WPN, the mean Kh is approximately 1.5 with an upper bound of 3 (ignoring outliers).



Source: Yong and Maloney (2015)

Note: with the WPN project location overlain.

Figure 17: Locations of Stress Measurement Data in the Canadian Shield



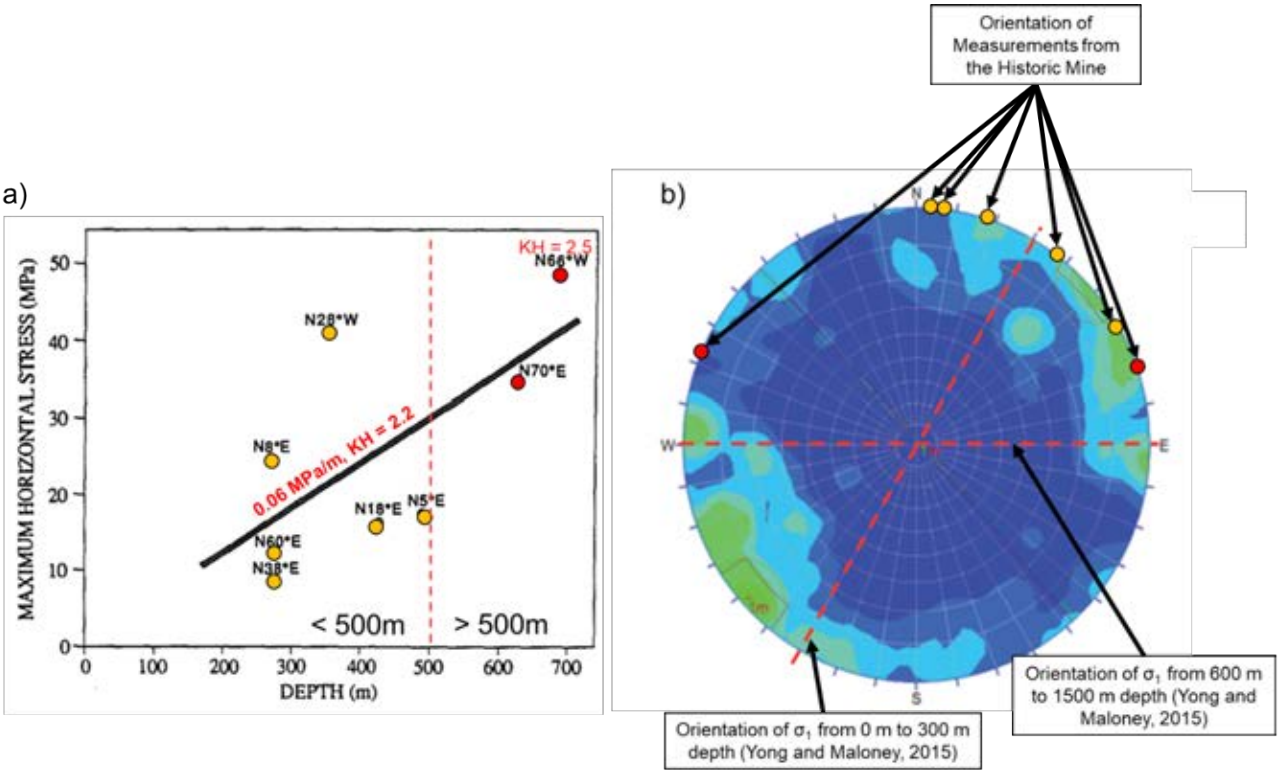
Note: a) Stress measurement data from the Canadian Shield (Yong and Maloney 2015) with the depths of interest for the WPN project shown in the black boxes; and b), stereonet showing contours of trend and plunge of the major principal stress measured in the Canadian Shield, which are dominantly NE-SW and sub-horizontal.

Figure 18: Ratio of Maximum and Minimum Horizontal Stresses Relative to Vertical Stress

6.2 Historical Stress Testing

Stress measurements have not been conducted within the proposed WPN mine limits; however, testing using the overcoring method was conducted in the historic White Pine mine using a USBM gauge from underground boreholes (Agapito, 1992). The average stress magnitudes measured at various testing locations in the historic mine are shown in Figure 19a. Agapito (1992) proposed that the maximum horizontal stress could be defined by the best fit curve to the average stress measurements which shows an increase of 0.06 MPa /m of depth which is equivalent to a KH of 2.2 assuming an overburden stress of 0.027 MPa /m. This is in good agreement with the data from the Canadian Shield albeit slightly higher than the mean magnitude proposed by Yong and Maloney (2015) at the depths of interest. Given the close proximity of the overcoring tests to the mine openings (e.g., within 4 ft in the backs of rooms), it is likely that these measurements were influenced by mining induced stresses and in situ KH may be lower.

Like the testing in the Canadian Shield (Yong and Maloney, 2015), the available information from the historic mine suggests that the major principal stress is dominantly sub-horizontal and the trend ranges from N-S to E-W, with the deepest measurements in the NE Mine showing a more E-W orientation (Figure 19b). The intermediate principal stress is also sub-horizontal, and the minor principal stress is vertical, corresponding to the weight of overburden.



Source: Adapted from Yong and Maloney 2015

Note: a) Average magnitude from overcoring measurements at different sites in the historic mine (modified from Agapito and Litsenberger 1993), and b) orientations of the overcoring measurements overlain on the stereonet of measured in situ stress orientations in the Canadian Shield

Figure 19: Historical Stress Testing

6.3 WPN Local Data

In situ stress orientation can be inferred from the orientation of borehole breakouts while high in situ stress to rock strength ratios can be inferred from core discing observed in drill core. In addition to the borehole information, fault zones may also influence local in situ stress conditions or compartmentalize the in situ stress regime; although, WPN's fault model (SRK, 2025) did not contain information regarding potential local in situ stress influences. Data available from within the WPN claim includes instances of core discing and borehole breakout measurements from downhole geophysical televiewer surveys.

Only six instances of core discing were observed in the WPN core, and they were dominantly influenced by bedding (i.e., weaker planes perpendicular to the core axis). These instances of discing are likely less relevant when defining high stress to strength scenarios as the strength of the defects can be more variable than the intact rock.

Six drillholes had records of borehole breakouts and were used to infer approximate orientation of maximum horizontal in situ stresses (Figure 20). The breakouts typically occurred around and below 700 m depth and mostly suggested E-W maximum horizontal in situ stress orientations (Table 3). The breakouts are also found to be near to the interpreted fault traces, potentially indicating some fault related influence.

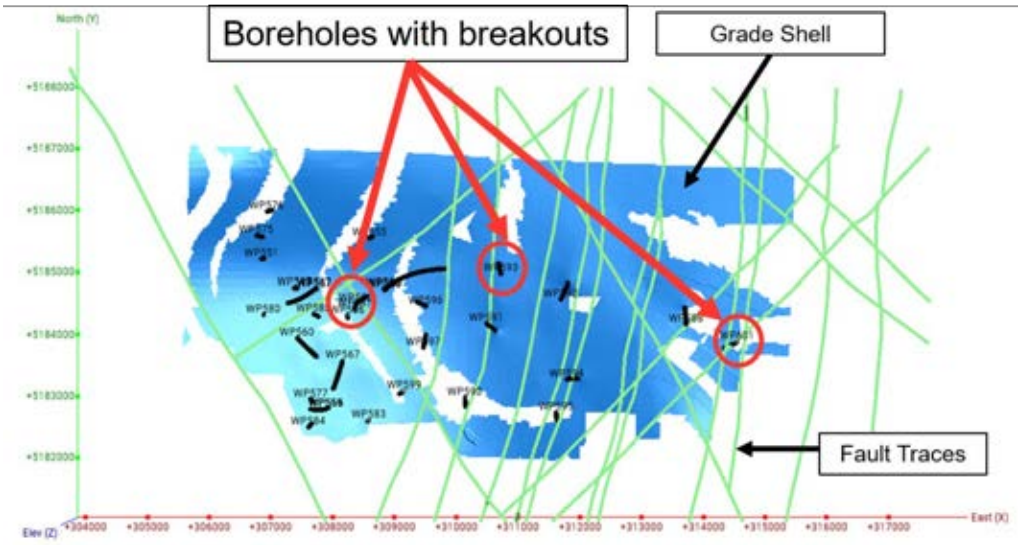


Figure 20: Plan View of the Grade Shell, Fault Model Traces, and Locations of Drillholes with Borehole Breakouts

Table 3: Borehole Breakouts at WPN

BHID	Downhole Depth (Top) Nearest m	Downhole Depth (Bottom) Nearest m	Breakout Orientation (° Azimuth)	Lithology	Comments
WP581	781	789	000 / 180	RMAS / LWSA / CCGL	Five individual breakouts with three additional one-sided breakouts.
WP582	837	838	000 / 180	CCGL	One breakout
WP586	755	758	100 / 280	LWSA	Two breakouts with four additional one-sided breakouts.
WP593	932	934	045 / 225	TBLS	One breakout
WP601	681	682	000 / 180	UNDF	One breakout
WP601	986	987	000 / 180	CCGL	One breakout

6.4 Assumed Stress Condition

A range of in situ stress magnitude and orientation is assumed for the geotechnical assessments (Table 4) due to the residual uncertainty for the stress condition at depth for WPN. The uncertainty in the principal horizontal stress orientation (possibly ranging from E-W to N-S) is due to the range in regional estimates, the historic testing, and the potential interaction between faulting and the borehole breakouts recorded at WPN. The in situ stress magnitude is better constrained as publicly available data, nearby mine sites, and the historic testing suggest that the principal horizontal stress magnitude is likely between 2 and 2.5 x σ_v.

Table 4: Stress Conditions Assumed for the Geotechnical Assessments

Component	Trend	Plunge	Magnitude with Depth
Minor, σ_3 (vertical)	0	90	0.027 MPa/m
Intermediate, σ_2 (horizontal)	90 to 180	0	1.5 x σ_v
Major, σ_1 (horizontal)	0 to 90	0	2 to 2.5 x σ_v

7.0 STRUCTURAL CONDITIONS

Structural interpretation at WPN can be compartmentalized by rock mass fabric information (e.g., the characteristics and orientation of open joints, bedding, veins, etc.) and by major features (e.g., regional and mine scale faults). Rock mass fabric data (e.g., joints, bedding, etc.) was collected from oriented geotechnical core logging and from Acoustic Televiewer (ATV) and/or Optical Televiewer (OTV) surveys that were completed during the 2015 and 2024 investigation campaigns. Major feature data was collected from the drill core and televiewer surveys in 2015 and 2024, but was also supported by airborne geophysical surveys of the WPN subsurface; WSP was not involved in the development of major features nor the estimation of their characteristics and most of the data described in this section for regional faults is informed by the PEA (G Mining, 2023) and the mine scale faults are informed by the fault model (SRK, 2025).

7.1 Regional Faults

The White Pine deposit is located between the Keweenaw Fault and the eastern extension of the Porcupine Mountain Fault (G Mining, 2023) (Figure 21). These regional faults, together with the nearby synclines and folding structures, contributed to the dip, orientation, and composition of the lithological units of interest at WPN.

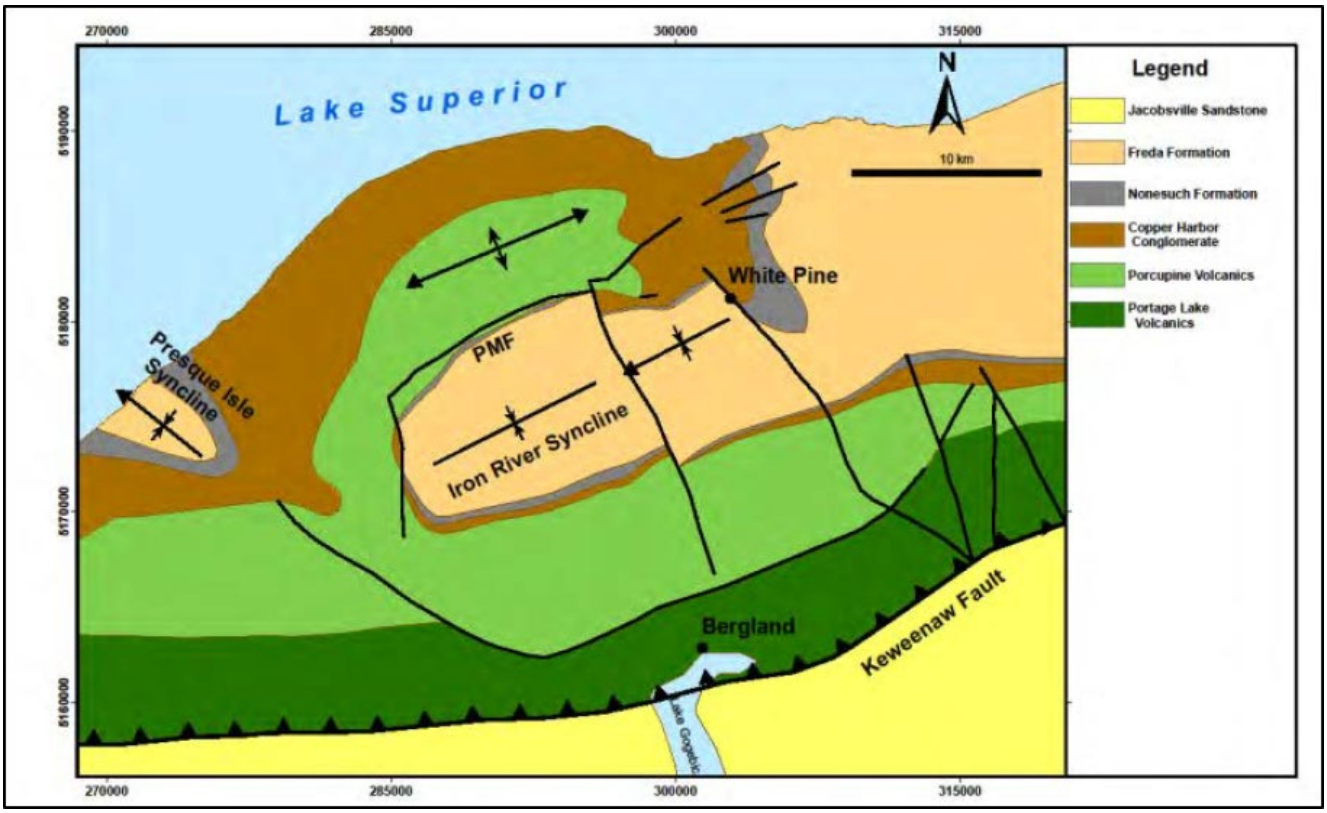


Figure 21: Geologic Map of the Region Containing the WPN Project

7.2 Mine Scale Faults

Estimation of faulting within the WPN project area is based on available geotechnical investigation data, geophysical aerial surveys, and mapping of the historic White Pine mine. WPC requested that a mine scale fault model be produced by SRK (2025), which was then provided to WSP. The mine scale fault model includes 19 wireframes, which strike N-S, NE-SW, and NW-SE (Figure 22). The inferred fault locations are based on a geomagnetic survey of the area which was examined to develop inferred fault trace and levels of confidence (Figure 22). The dip of the structures was not modelled and was instead inferred by drillhole and downhole televiewer data. The fault model deliverable (SRK, 2025) suggested that the mine scale fault model was reviewed against historic underground mapping from the White Pine mine; this increased confidence in the fault traces for those WPN mine scale faults that bordered the historic mine workings (Figure 23). The mine scale fault model does not include information on possible rock mass characteristics or local in situ stress influences related to the fault wireframes and the associated deliverable (SRK, 2025) indicates that this remains an interpretation gap.

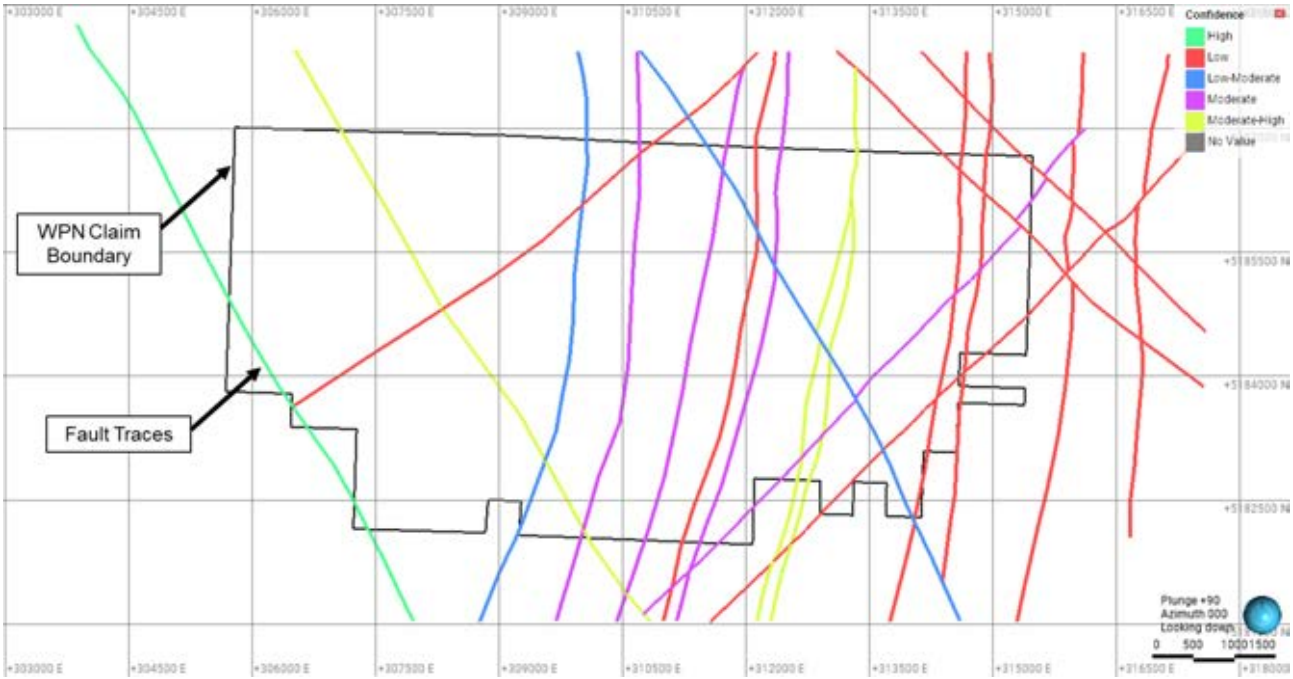


Figure 22: Plan View of the WPN Claim Boundary Showing the Mine Scale Fault Traces Colour-Coded Based on their Assigned Confidence

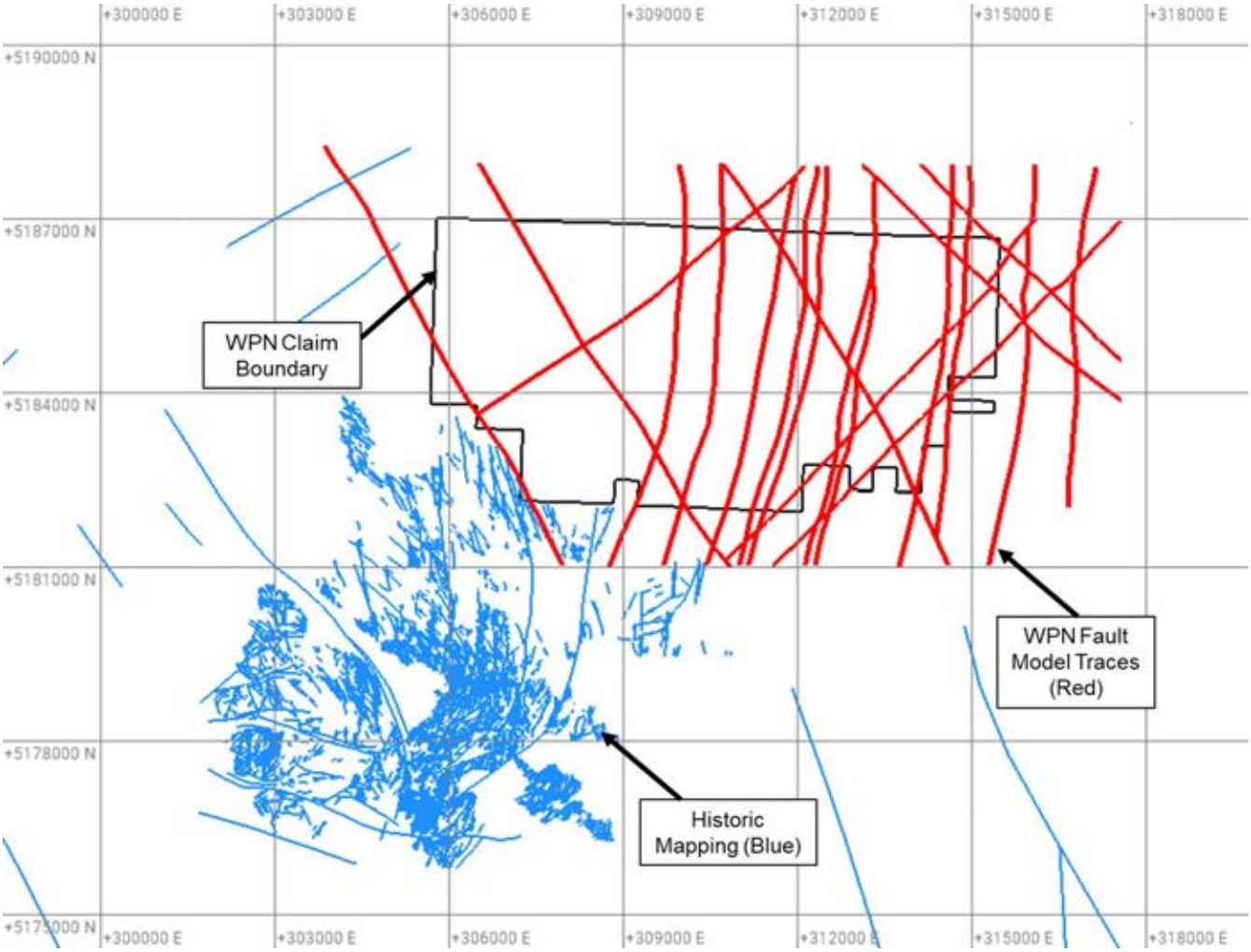


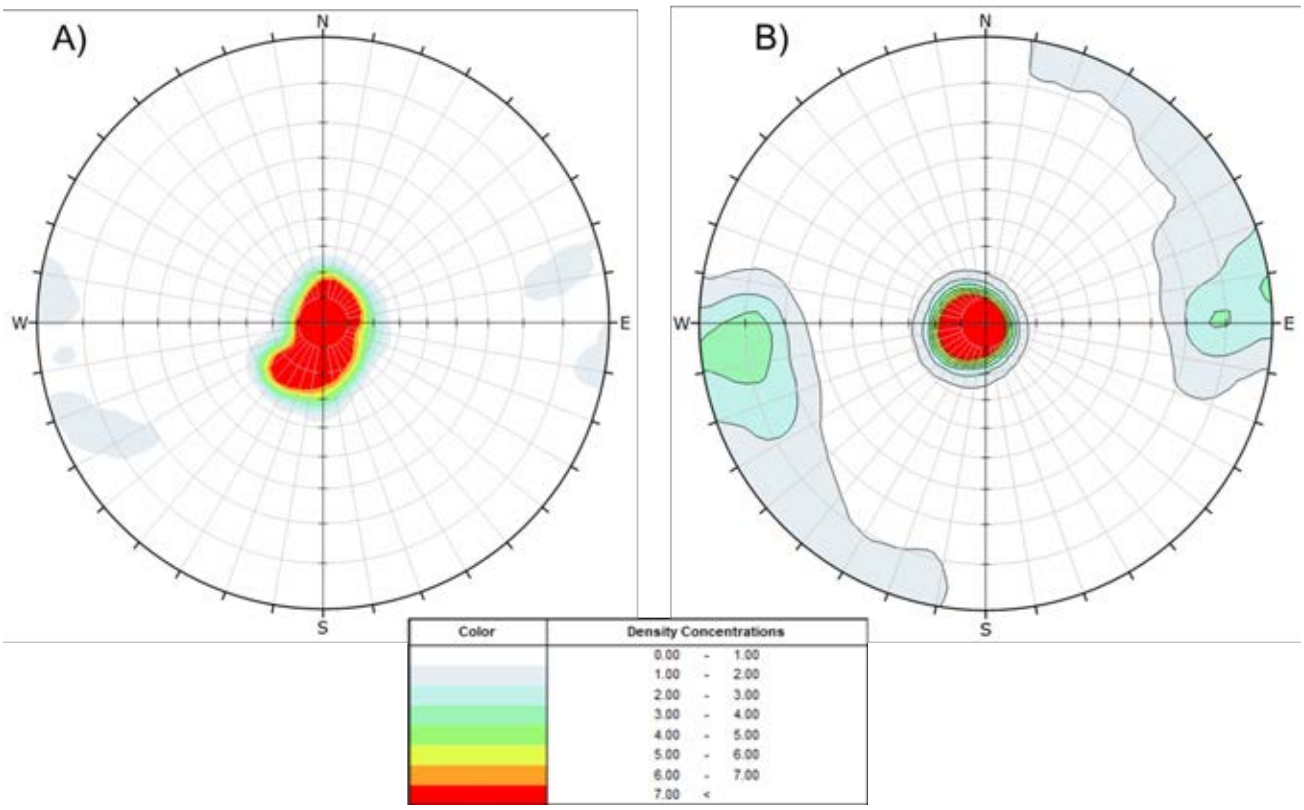
Figure 23: Plan View Showing Structure Mapping from Historic Workings, the WPN Claim Boundary, and Inferred Mine Scale Fault Model

WSP reviewed available drill core photographs from the 2024 investigation campaign for those intervals which intersected the inferred fault wireframes or were logged as intersecting a fault. This review attempted to identify if there was observable core damage near to the fault zones which may suggest a relationship between poorer rock mass characteristics and fault zone proximity. The fault wireframes did not show a correlation with damage zones in the 2024 geotechnical core, this may be because of the limited intersections with the wireframes (i.e., only 3 of the 2024 drillholes intersected an inferred fault wireframe). Reviewing the geotechnical logs for fault structures showed that most structures logged as faults contained minor lengths (< 30 cm) of damaged ground or fault gouge (Figure 24). Based on the available data, the rock mass damage related to fault wireframes appears manageable for mine stability but the damage interpretation is of relatively low confidence given that most faults are interpreted to be vertical to sub-vertical, the drillholes are also sub-vertical, and the spacing of the drillholes is quite large (> 500 m on average); this should be specifically investigated in future studies.

Figure 24: Example of Logged Fault Zones within the Geotechnical Core from the 2024 Investigation Campaign

7.3 Rock Mass Fabric

The rock mass at WPN is comprised mostly of bedding fabric, some veins, and some open structure. The data available for review regarding the spacing, characteristics, and orientation of the structural fabric includes oriented core data and downhole televiewer interpretations. There are 15 drillholes which contain structural orientation data (these drillholes are from the 2024 investigation campaign) and 34 drillholes with downhole televiewer survey data (these drillholes are split between the 2015 and 2024 investigation campaigns). The oriented core information includes orientation for features observed in the drill core and covers cemented joints, filled joints (e.g., bedding and foliation), and open joints. The downhole televiewer provides orientation for open fractures (joints, faults, broken zones, etc.) and closed features (e.g., bedding planes, veins, etc.). Stereonets showing orientation data from oriented core and downhole televiewer data sets are in Figure 25.



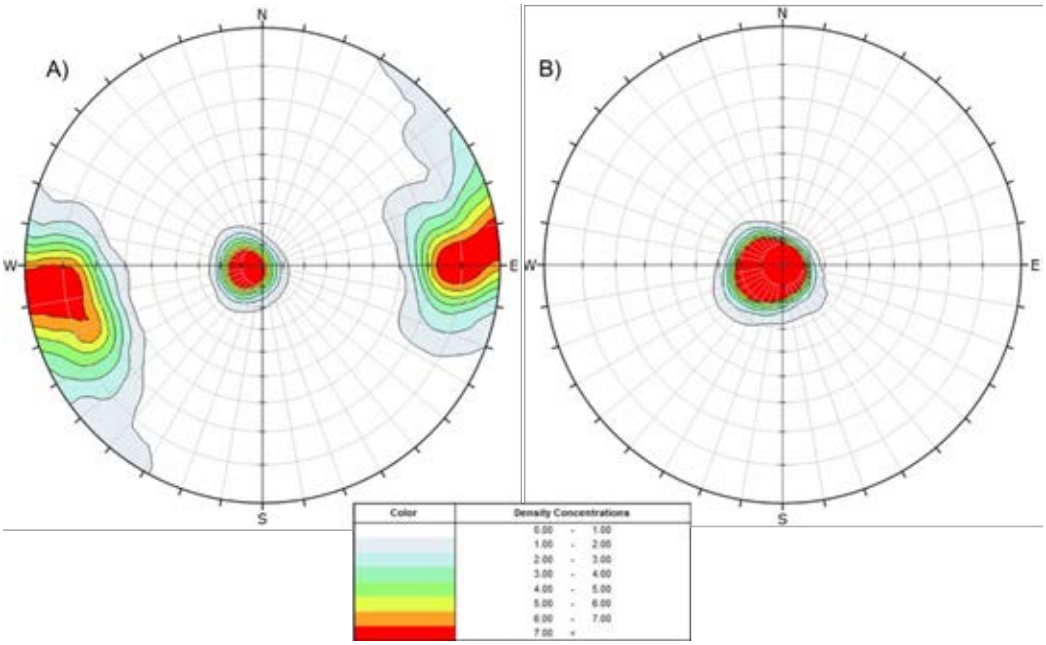
Note: A) Stereonet of oriented core data regardless of feature type. B) Stereonet of downhole televiewer data regardless of feature type.

Figure 25: Stereonets Showing Orientation Data from Oriented Core and Downhole Televiewer Data Sets

For structure orientation, downhole televiewer is typically of higher confidence as it records structural orientation and depth of fractures observed in the rock in situ, whereas the oriented core data can be influenced by mechanical drill induced damage which will typically result in a greater number of structures than would be observed in the downhole televiewer. The interpretation of structural fabric orientation for the WPN study relied largely on the downhole televiewer data (more data and likely better data confidence) although the conclusions were compared with the oriented core data. At WPN, two main structural sets were identified; a sub-horizontal set mainly comprising bedding planes and some open structure (which may also be bedding related) and a sub-vertical set comprised mostly of closed veins and open structure (Table 5, Figure 26, and Figure 27). The oriented core database shows a similar orientation for bedding planes; however, the logged open structure and cemented joints show less sub-vertical structure and less consistency (i.e., no consistent sub-vertical set orientation).

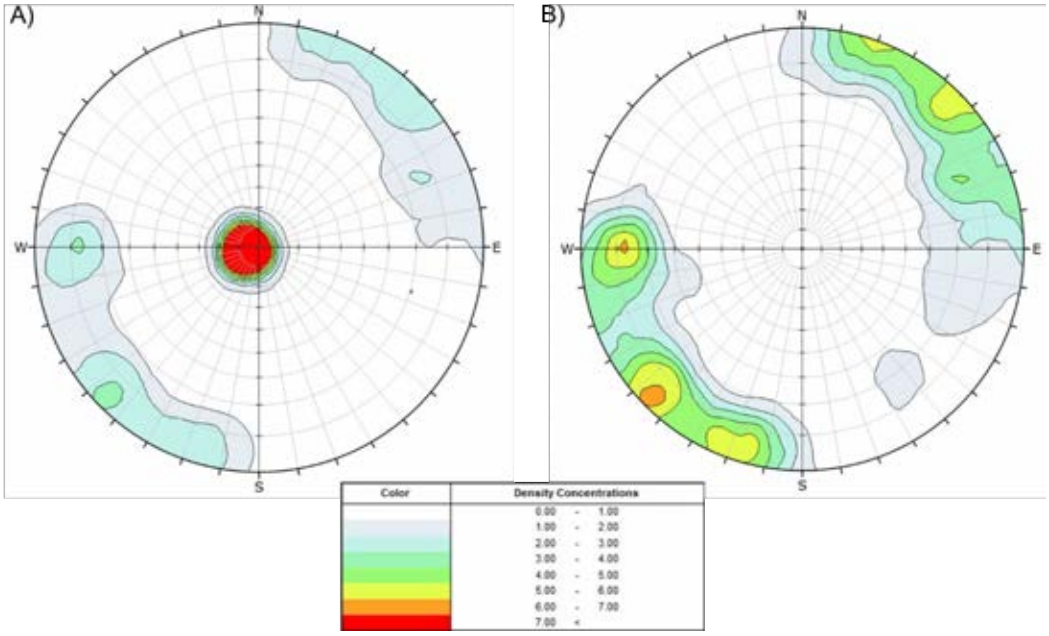
Table 5: Number of Structures Recorded in the Downhole Televiewer Dataset

Structure Type	Count
Open Fracture (different apertures)	2,980
Bedding Planes (closed feature)	1,369
Veins (closed feature)	1,666
Broken Zone	14
Filled Fractures (closed feature)	35
Fault Contact	112
Lithological Contact (closed feature)	1,243



Note: A) Stereonet of vein structures recorded in the televiewer dataset. B) Stereonet of bedding structures recorded in the televiewer dataset.

Figure 26: Veing Bedding Structures in the Televiewer Dataset



Note: A) Orientations of open structure from the downhole televiewer dataset. B) Orientations of open structure that has a dip different than bedding from the downhole televiewer dataset.

Figure 27: Orientations of Open Structures from the Downhole Teleview Dataset

For the stability assessments for WPN, the fabric structure of most interest is open structure which could impact the strength of the backs and pillars in the mine. The area of interest for mining (from the MARK lithology to the LWSA) show minimal open structure that are sub-vertical (eight structures total from the televiewer dataset) and sub-horizontal (seven structures total from the televiewer dataset). Most of the open structure (2,677 structures from the televiewer dataset) are within the Undifferentiated Nonesuch and Freda domains which are above the mining horizon and includes structures recorded from surface to the bottom of the drillholes. The count of open sub-vertical structures is influenced by the plunge of the drillholes as it would be less likely that they would intersect sub-vertical features.

The character of open structures is informed by the geotechnical drillholes which recorded roughness, infill characteristics, and weathering. Averages for the area of interest suggest that roughness is typically smooth to slightly rough with most structures exhibiting no evidence of weathering or infill.

The spacing of the open structures cannot be meaningfully calculated within the area of interest due to the limited number of open structures recorded by the downhole televiewer; however, the spacing of the sub-vertical open structures in the Undifferentiated Nonesuch and Freda domains is approximately 6 m (on average) along the drillhole axis. This equates to approximately 1 sub-vertical open structure every 3 m laterally (when accounting for drillhole plunge) which suggests that it may be possible that some rooms may have 1 or 2 open structures in the back if it is assumed that the open structure spacing from the Nonesuch and Freda continues into the area of interest. Stability assessments completed for this study will examine implications of sub-vertical to vertical open features in the back of excavations.

Other structures of interest include closed bedding planes; the spacing of these at WPN is understood to be millimetre and centimetre scale (from core observations) and the bedding plane strength is shown to impact the intact rock strength (discussed in Section 8.1). Sub-vertical vein structures may also impact back and pillar

stability; however, the downhole televiewer dataset has only recorded 21 such structures in the lithologies of interest (MARK to LWSA). It may be possible that individual veins may be present in the back or pillars of certain rooms; but it is unlikely (based on the available dataset) to be a widespread concern and would have less impact than the potential for sub-vertical open structures in the backs of the room.

7.3.1 Direct Shear Testing

Fourteen (14) direct shear test results were available for review (Advanced Terra Testing, 2025). The direct shear tests were completed on a mixture of four (4) saw cut structures and ten (10) naturally open bedding planes. No sub-vertical or inclined open structures were tested. The shear strength interpreted by the laboratory varies in both friction angle (from ~20° to ~59°) and apparent cohesion (from ~170 kPa to ~533 kPa at peak strength).

The value of the saw cut tests are minimal given they are examining shear strength of an artificial structure. There may be some value in the direct shear strengths estimated for the naturally open bedding planes if the stability assessments examine open bedding planes sliding past each other. The shear strengths interpreted by the laboratory were not reinterpreted by WSP.

7.4 Lithological Contacts

The lithological contacts in the zone of interest at WPN are tight and not associated with an increase in rock mass structure or alteration based on observations made from the rock core and downhole televiewer surveys. An example of the rock mass characteristics near the lithological contacts is provided in Figure 28. The contacts generally appear to be tight and are occasionally associated with a fracture in the core (but this fracture is not typically observed in the downhole televiewer survey).

Note: Approximate locations of contacts are shown with a yellow line.

Figure 28: Example Core from Drillhole WP591 within the Mining Area of Interest

8.0 ROCK MASS CHARACTERIZATION

Rock mass characteristics can be defined by rock mass classification, rock strength and elastic properties, and interpretation of rock mass behaviour. This section summarizes the estimation of rock mass characteristics using the geotechnical core logging database and laboratory strength database.

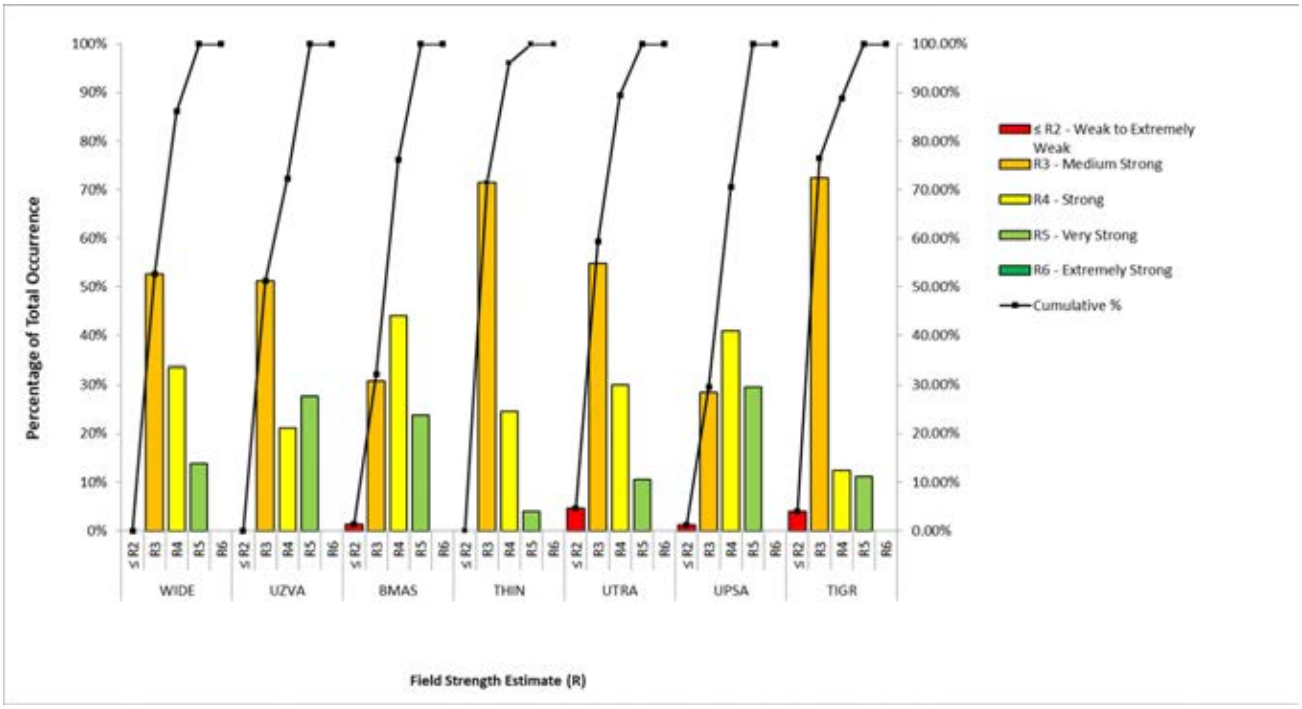
8.1 Rock Strength and Elastic Properties

WSP conducted a review of the available strength testing data which consisted of ISRM field strength estimates, field Point Load Testing (PLT), Uniaxial Compressive Strength Testing (UCS), Triaxial Compressive Strength Testing (TCS), Multi-Stage Triaxial Compressive Strength (MTCS) testing, and Biaxial Strength (BTS) testing. The data available for the geotechnical study included:

- ISRM field strength estimates (Brown, 1981) from the 2015 and 2024 geotechnical investigation campaigns. This data includes 9,203 data points the majority of which suggest R3 (5,284 data points), R4 (3,000 data points), and R5 (775 data points).
- PLT data from the 2015 and 2024 geotechnical investigation campaigns. These included both diametral and axial tests with records of how the sample failed (e.g., along bedding, through homogenous rock, etc.). 4,628 tests were performed in 2024 and 1,759 were performed in 2015.
    - Additional PLT tests (162) were collected by WSP in 2025 on metallurgical samples at WPN's Kamloops metallurgical laboratory.
- 333 laboratory intact strength tests (UCS, TCS, BTS, and MTCS) collected over multiple testing campaigns:
    - UCS, MTCS, and TCS tests completed from core collected in the historic White Pine mine in 1992 (Agapito, 1992).
    - UCS, TCS, and two MTCS tests completed from core collected during the 2014 geotechnical investigation (University of Minnesota, 2015 & Geoengineering North LLC, 2014). The tests from the Geoengineering North LLC report were completed at the Michigan Technological University in Houghton Michigan.
    - TCS, MTCS, and 3 UCS tests completed from core collected during the 2014 geotechnical investigation (Advanced Terra Testing, 2024 & 2025).
    - TCS and UCS tests completed from select core collected from White Pine's metallurgical laboratory and tested by WSP (2025).

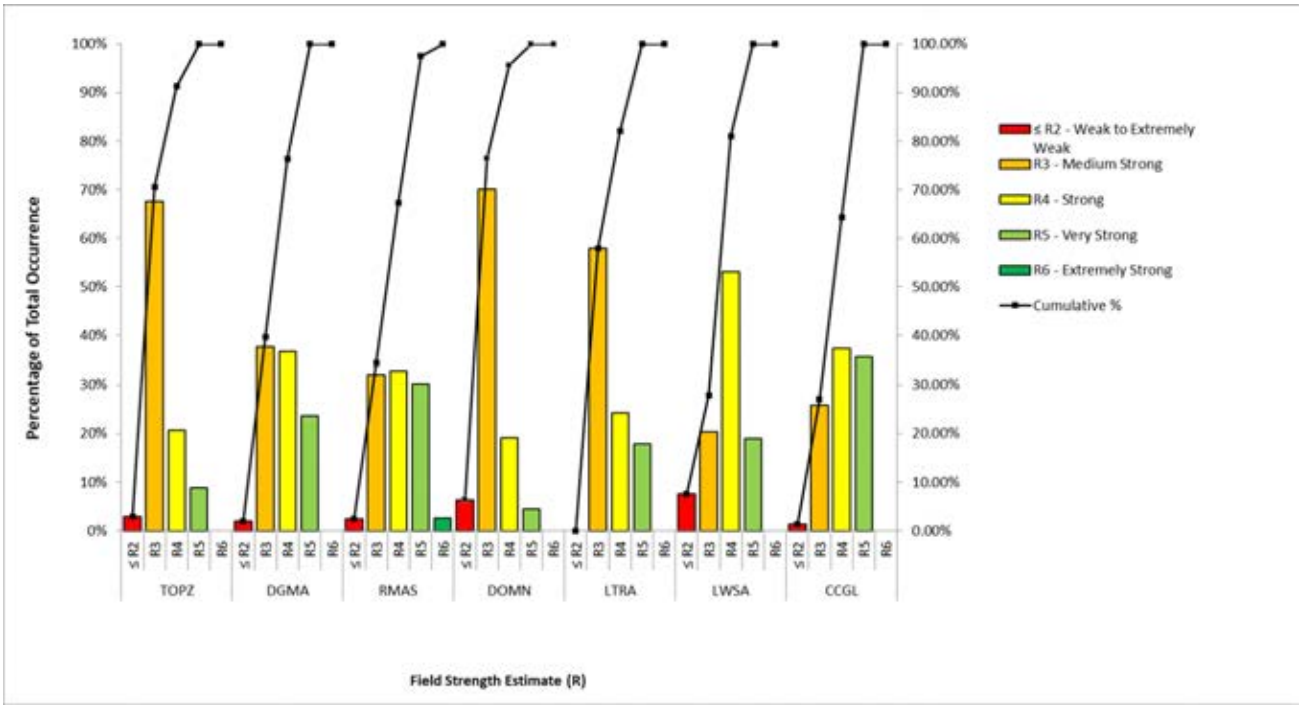
8.1.1 Field Strength Estimates and PLT

Field strength estimates are only available for the 2024 drilling campaign and typically suggest R3 to R5 rock strengths were common within the mining horizon (WIDE to CCGL) (Figure 29 and Figure 30). The sandstone units (UPSA and CCGL) typically were observed to have R4 rock strengths while the weakest units were found to be the THIN, DOMN, TIGR, and TOPZ units, which predominantly were observed to be R3. Minimal R2 strength were estimated from the core. The way in which the core failed during the field strength estimates was not recorded; although, it is possible many failed along bedding planes.



Note: WIDE to TIGR

Figure 29: ISRM Field Strength Estimates from the 2024 Campaign



Note: WIDE to TIGR

Figure 30: ISRM Field Strength Estimates from the 2024 Campaign

The PLT data collected during the 2015 and 2024 investigation campaigns included both axial and diametral tests and recorded the method in which the core failed (e.g., through homogeneous rock, along a bedding plane, along a preexisting defect, a combination of a bedding plane / defect and homogeneous rock, or if the failure is considered invalid according to ASTM). The PLT data was filtered for invalid test results and plotted for diametral and axial samples taken from the 2015 and 2024 investigation campaigns.

The records of the type of failure for the PLTs from the 2015 investigation campaign are not consistent in their labelling but suggest that most axial samples exhibited failure through homogeneous rock while the diametral samples mostly exhibited failure along bedding planes; this is reasonable given how the bedding planes would be oriented to the core axis. Part of the labelling of failure types from the 2024 investigation campaign appears unusual as approximately 250 of the diametral samples were recorded as failing through homogeneous rock and not bedding planes when shales and siltstones were tested; this is unusual due to the orientation of the core axis to the bedding planes. Is50 (MPa) values were found to vary significantly for most lithological units based on sample type (axial and diametral) and investigation campaign (Figure 31).

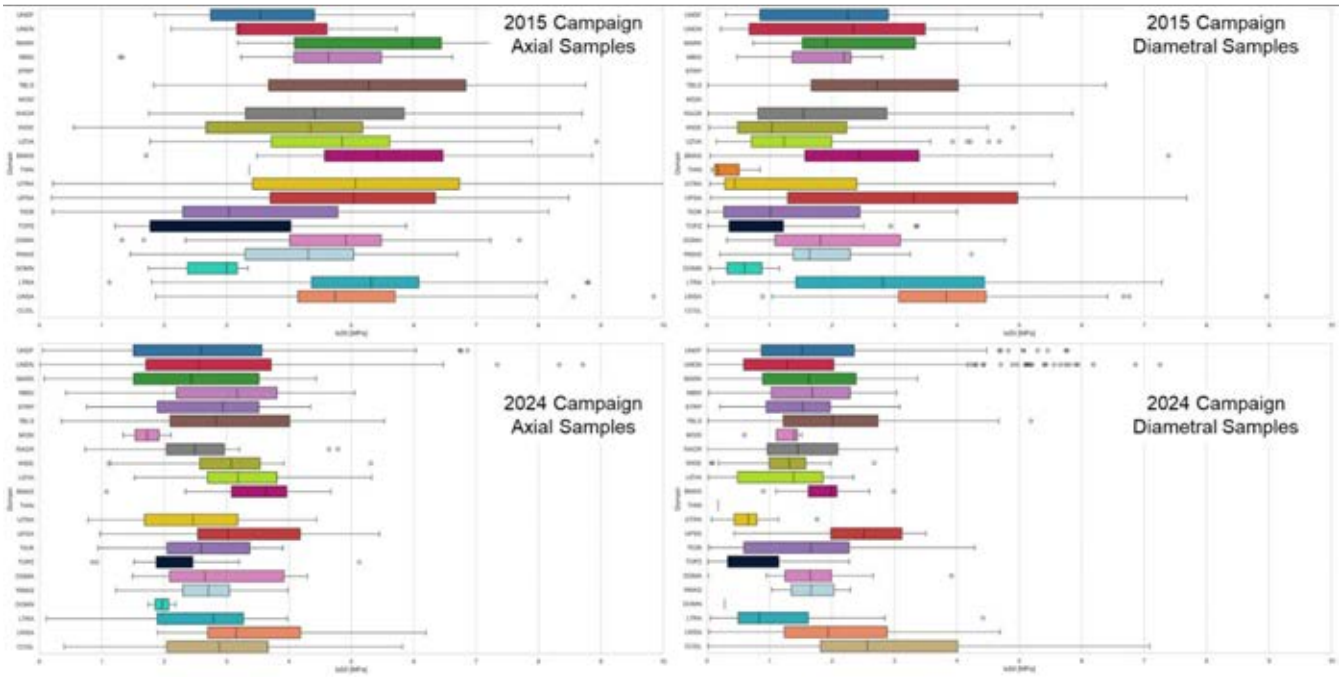


Figure 31: Is50 Plots Separated by Sample Type and Investigation Campaign

The 2024 investigation campaign PLT results are consistently lower than the 2015 investigation campaign PLT results; typically, the average axial strengths recorded for each lithology in 2024 are approximately half the strength of what was recorded in 2015. The diametral strength difference between the 2015 and 2024 campaign is less than the axial strength difference, but certain lithologies (e.g., LWSA, LTRA, UPSA, etc.) has lower average diametral strengths in the 2024 campaign.

WSP conducted additional PLT tests on a limited number of core samples from metallurgical samples collected during the 2024 investigation campaign and stored by WPC in their Kamloops laboratory. PLTs were performed in intervals adjacent to intervals that contained PLT results. The WSP testing suggests that the 2024 PLT results are likely anomalously low (Figure 32). The WSP axial testing results aligned reasonably well with the results obtained during the 2015 investigation, the WSP diametral testing suggested that some lithologies have higher strengths

than suggested from the 2024 investigation campaign. It is possible that the WSP samples were impacted by a strength bias as only those samples which remained intact after transportation from site to Kamloops were able to be tested, potentially biasing the results towards stronger sections of each lithology; this potential bias cannot be resolved without additional testing completed continuously during an investigation campaign. For the purposes of this geotechnical study, the 2015 PLT dataset was considered the highest confidence dataset as it is likely that the 2024 campaign yielded anomalous results which could be the result of a non-calibrated or malfunctioning PLT apparatus.

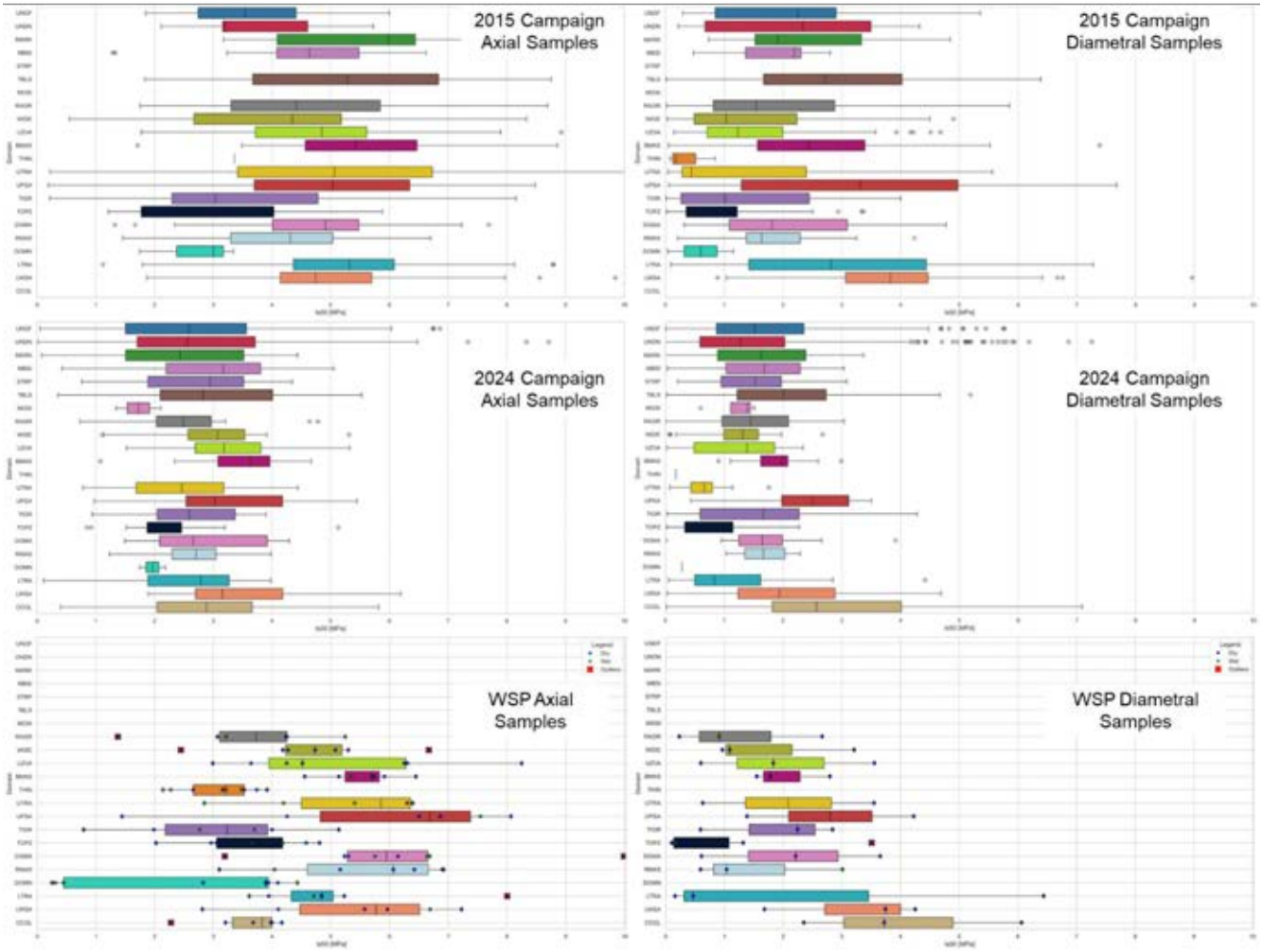


Figure 32: Is50 Plots Separated by Sample Type and Investigation Campaign Including WSP Samples

8.1.2 Laboratory Testing

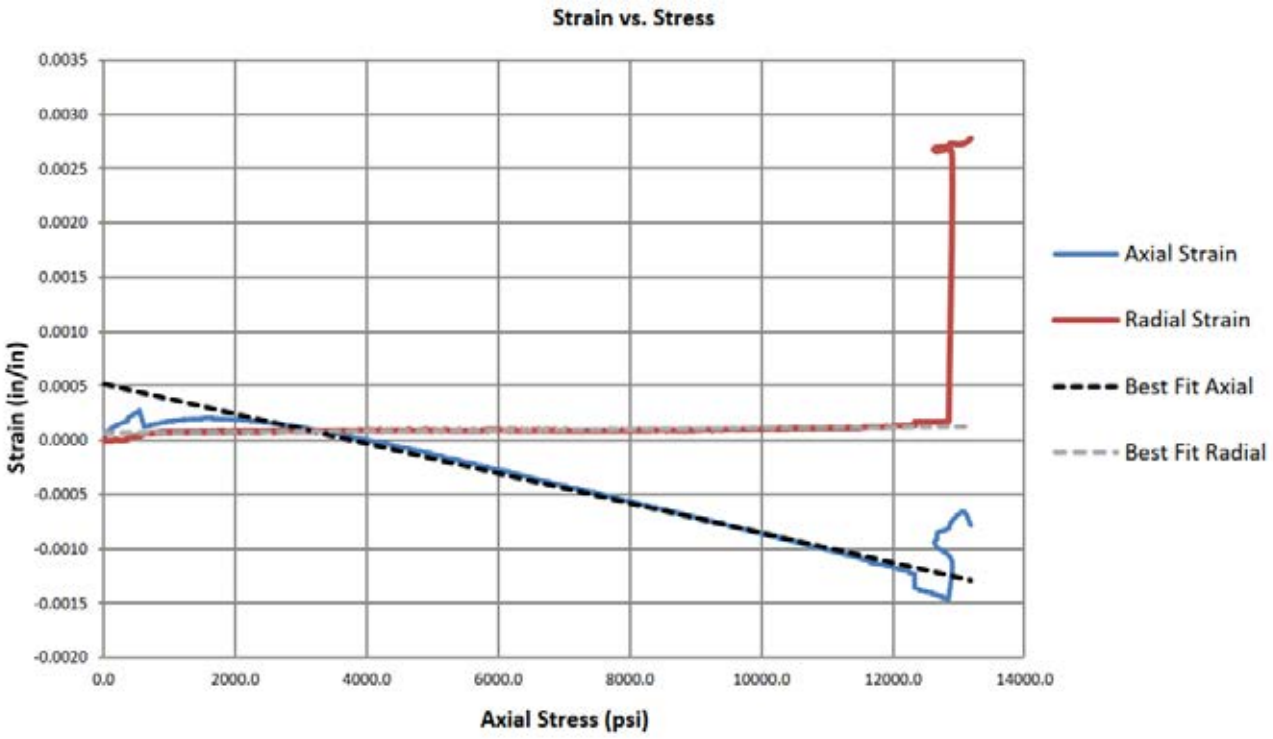
The available laboratory or study reports, which contain descriptions of the laboratory testing methods completed to date as well as plots of strain and load measured during testing, were reviewed for the lithologies of interest (RAGR to LWSA). The review produced the following commentary:

- The 1992 samples selected for laboratory testing were collected in NX core sizes from the roof and ribs of active mining areas of the old White Pine mine (Agapito, 1992). The report indicates that there were challenges in collecting appropriate core samples due to rock mass fracturing (Agapito, 1992); this may

indicate that the areas of sampling were already experiencing stress or blast related damage which could have influenced the resulting strength interpretation. These samples should be considered lower confidence due to being sampled within the area of mining influence.

- The 2014 and 2015 laboratory samples were tested by the University of Minnesota and the Michigan Technological University; parts of the testing methods were non-ASTM standard, some of the tests performed are not appropriate for the type of rock behaviour present at WPN, and some of the data typically produced during laboratory testing was not included in the reports.
  - ASTM standards (D7012) require that the ends of the rock core specimens be parallel within a tolerance of 0.025 mm. None of the samples tested had the ends of the core sample ground to be within the ASTM tolerance for compression testing. Some samples were naturally within tolerance but were not ground to ensure a flat testing surface for the compression apparatus. The impact of this would likely reduce the intact rock strength interpreted by the laboratory tests.
  - Some of the reports (typically from Michigan Technological University) did not include before and after testing photographs of the sample.
  - None of the reports provided commentary on the way in which the sample failed. Understanding what failed within the sample is important for how the results are used in the development of rock strength parameters.
  - Two of the triaxial compressive strength tests were done in multiple loading stages; defined as a ‘multi-stage triaxial compressive strength test’ (MTCS). An MTCS test repeatedly loads the sample under different confining stresses until the sample is eventually failed. The use of multi-stage triaxial testing for strong brittle rock is problematic and review of the WPN data suggests that the damage initiation thresholds and even peak strengths were often exceeded during the first loading stage, and therefore subsequent loading stages were measuring residual rock strength rather than peak strength. The practical issue with this testing is that it can lead to underestimation of the confined rock strength which is critical for the design of stable pillar dimensions
    - The remaining triaxial compressive strength tests were performed as a single stage – a single loading stage to sample failure. These types of tests are preferred when estimating intact rock strengths.
- The laboratory testing completed on samples collected during the 2024 investigation campaign were almost entirely a mixture of single stage and multi stage triaxial compressive strength tests. 37 single stage TCS, 42 MTCS, and 3 UCS were completed.
  - The MTCS results have the same concerns as those tested in 2014 and 2015 and generally should not be used for intact strength interpretation.
  - The UCS results appear reasonable but were only completed in the RAGR, MBSI, and MARK lithologies which are well above the critical mining units.
  - The TCS results appear lower than would be expected (in many cases the interpreted TCS strengths at confinements of 20 MPa to 40 MPa are lower than UCS tests from the 2014 and 2015 reports). The laboratory report for the single stage TCS results contained some apparently irregularities in the stress strain plots (Figure 33); these could suggest inconsistencies with how the samples were initially loaded

during testing which may have impacted the overall result, potentially explaining why some results appear unusually low.

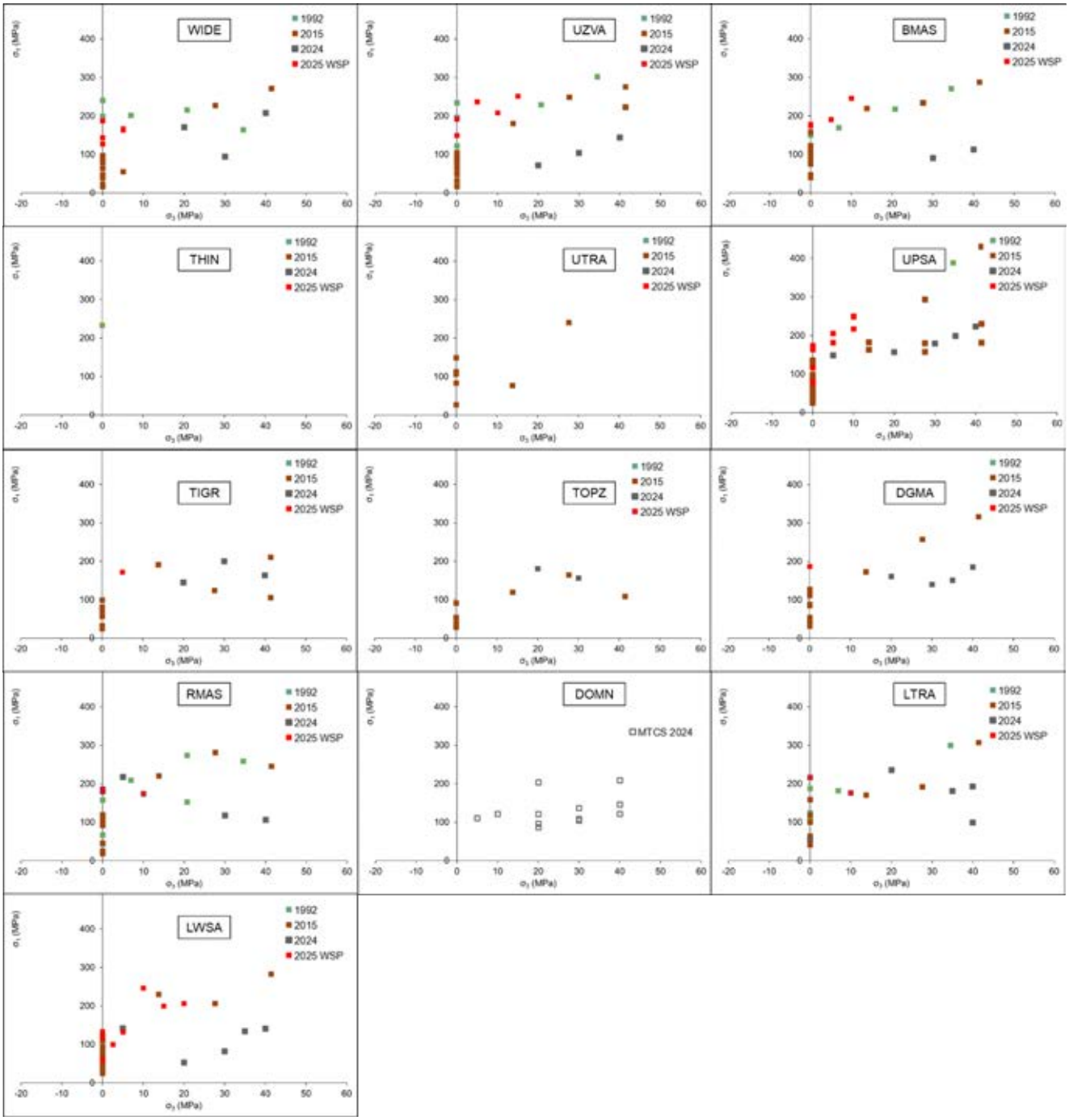


Note: It is unclear why the sample name contains 'UCS' when the sample is reported as a TCS test. The initial loading stage shows some irregularities where axial stain sharply changes in response to increasing stress.

Figure 33: Stress and Strain Measured During TCS Testing of Sample WP003611M_UCS

The laboratory testing review suggests that many of the results produced for WPN from previous campaigns may not have measured peak intact strength of the lithologies and existing rock strength interpretations may be underestimates. To examine this possibility, WSP collected and performed additional UCS and single stage TCS laboratory intact strength tests on WPN metallurgical core samples collected during the 2024 investigation campaign and stored in a Kamloops laboratory. Similar to the PLT tests performed on core from the Kamloops laboratory, the samples selected for intact strength testing may be biased to stronger core that was able to survive transportation and storage. The laboratory testing reports produced by WSP are provided in Appendix A.

The measured strength for those samples tested by WSP are typically equal or greater than previous test results (Figure 34). The principal stress plots produced by WSP filtered the available laboratory testing by method of failure (where possible) to only include samples which failed wholly or in part through homogeneous rock; MTCS results are not included (except for the DOMN unit as no other testing was available).



Note: Tests Filtered by Sample Source (1992, 2015, or 2024 investigation campaigns and 2025 WSP testing)

Figure 34: Principal Stress Plots for Each Lithology Near the Mining Horizon

The elastic parameters for each lithology were assumed based on available laboratory testing results which included an estimate of Young's Modulus and Poisson's Ratio. 19 tests performed by WSP (Appendix A), 14 tests performed by Agapito (1992), and 76 tests (includes 30 MTCS tests) performed by Advanced Terra Testing (2025) included estimates of elastic properties. Some of the available estimates of elastic properties were

considered lower confidence due to potential testing issues (listed earlier in this section); thus, elastic properties estimated for use in the geotechnical assessments were based on the following:

- Average Young's Modulus and Poisson's Ratios estimated by WSP's laboratory were assumed to be the highest confidence and were used if available for the lithology (WIDE, UZVA, BMAS, UPSA, DGMA, RMAS, LTRA, and LWSA).
  - For DOMN, TOPZ, THIN, and TIGR; no WSP laboratory estimates were available.
- THIN used the estimate from the Agapito (1992) report.
- DOMN, TOPZ, and TIGR were assumed as the only estimates available from the investigation campaigns were from MTCS tests which likely have misleading results.

Representative elastic and strength parameters by stratigraphy are shown in Table 6.

Table 6: Representative Intact Rock Material Parameters for Geotechnical Assessments

Stratigraphic Unit	Density (kg/m^3)	Youngs Modulus (GPa)	Poisson's Ratio	σ_{ci} (MPa)	m_i	Tensile Strength (MPa)
WIDE	2700	45	0.32	140	11	3.8
UZVA	2700	43	0.30	160	11	4.4
BMAS	2700	54	0.30	150	15	3.0
THIN	2700	53	0.19	80	12	2.0
UPSA/UTRA	2700	50	0.32	150	18	2.5
TOPZ/TIGR	2700	41	0.25	90	12	2.3
DGMA/RMAS/LTRA	2700	48	0.31	150	13	3.5
DOMN	2700	48	0.15	95	8	3.6
LWSA	2700	38	0.37	120	12	3.0
Mean / Representative of mining column	**2700**	**50**	**0.28**	**125**	**12**	**3.1**

8.2 Rock Mass Classification

Rock mass classification at WPN was calculated as part of the 2024 investigation campaign using Bieniawski's Rock Mass Rating (RMR_{89}) (1989) (Figure 35). The 2015 investigation campaign did not capture sufficient data to estimate an RMR_{89} for every logged interval as many intervals do not have an intact strength estimate. To provide a comparison between the 2015 and 2024 investigation campaigns, the Rock Quality Designation (RQD) was reviewed for each lithology (Figure 36). The RQD (from the 2015 and 2024 campaigns) and RMR_{89} (from the 2024 campaign) suggest the rock mass is 'Good' to 'Very Good' through the different mining domains (HW, Ore body / Room Pillars / Footwall). The TOPZ and TIGR (weaker shale domains likely to be present in the back of most rooms) are shown to have the most structure, with the other domains typically averaging an RMR_{89} greater than 80 or RQD greater than 90.

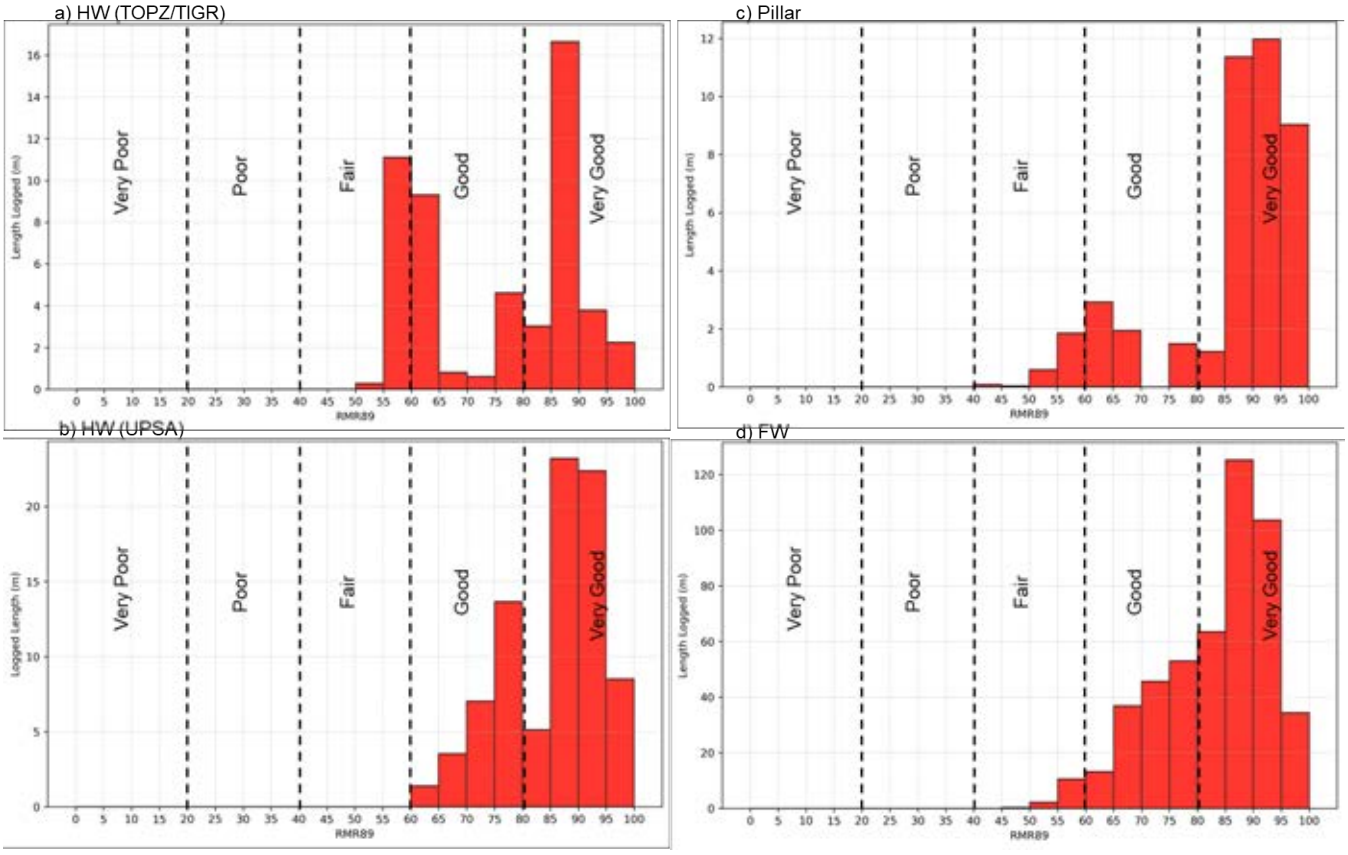
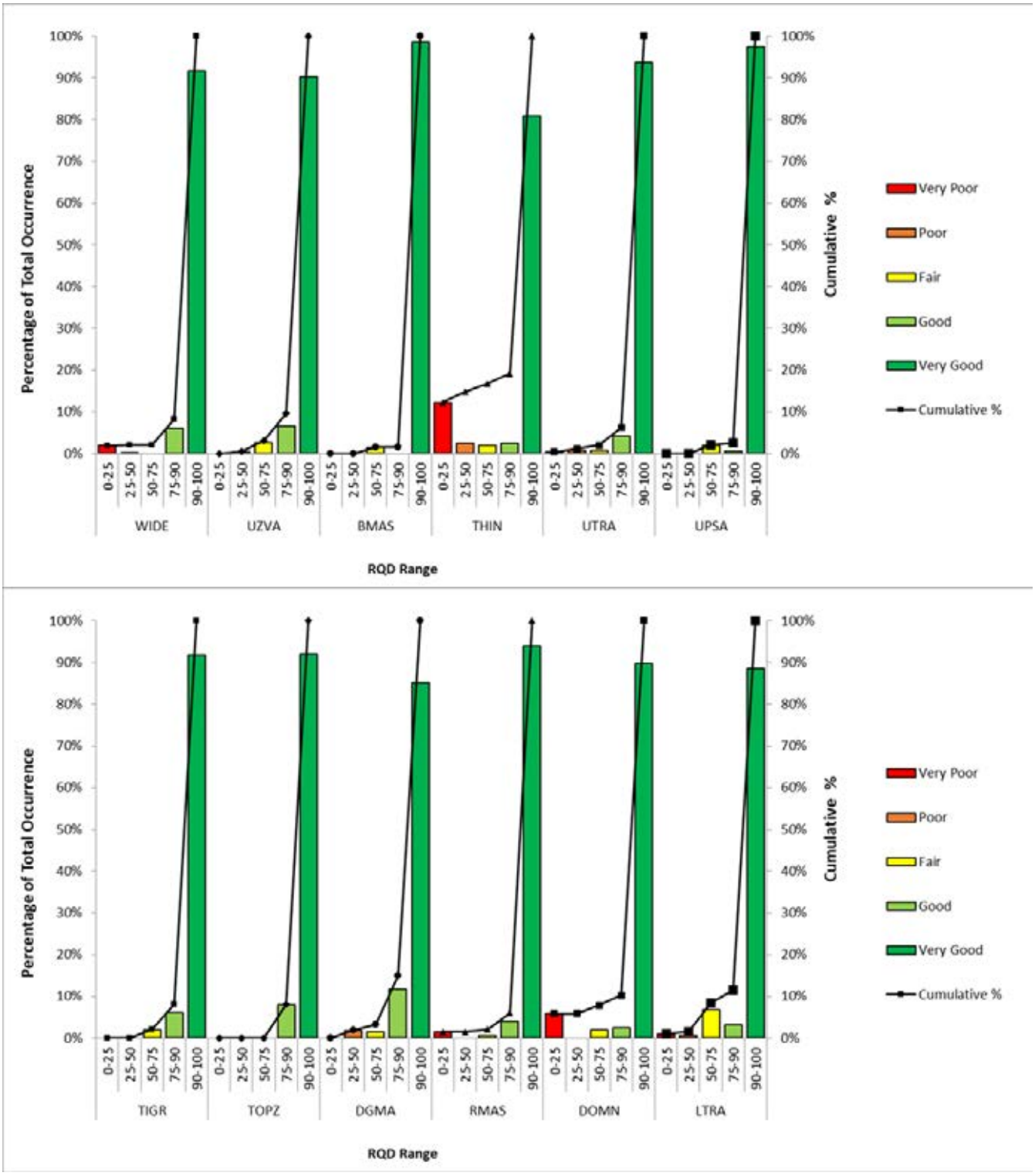


Figure 35: Histograms of RMR89 by Key Lithological Groupings for WPN



Note: Includes both 2015 and 2024 investigation datasets

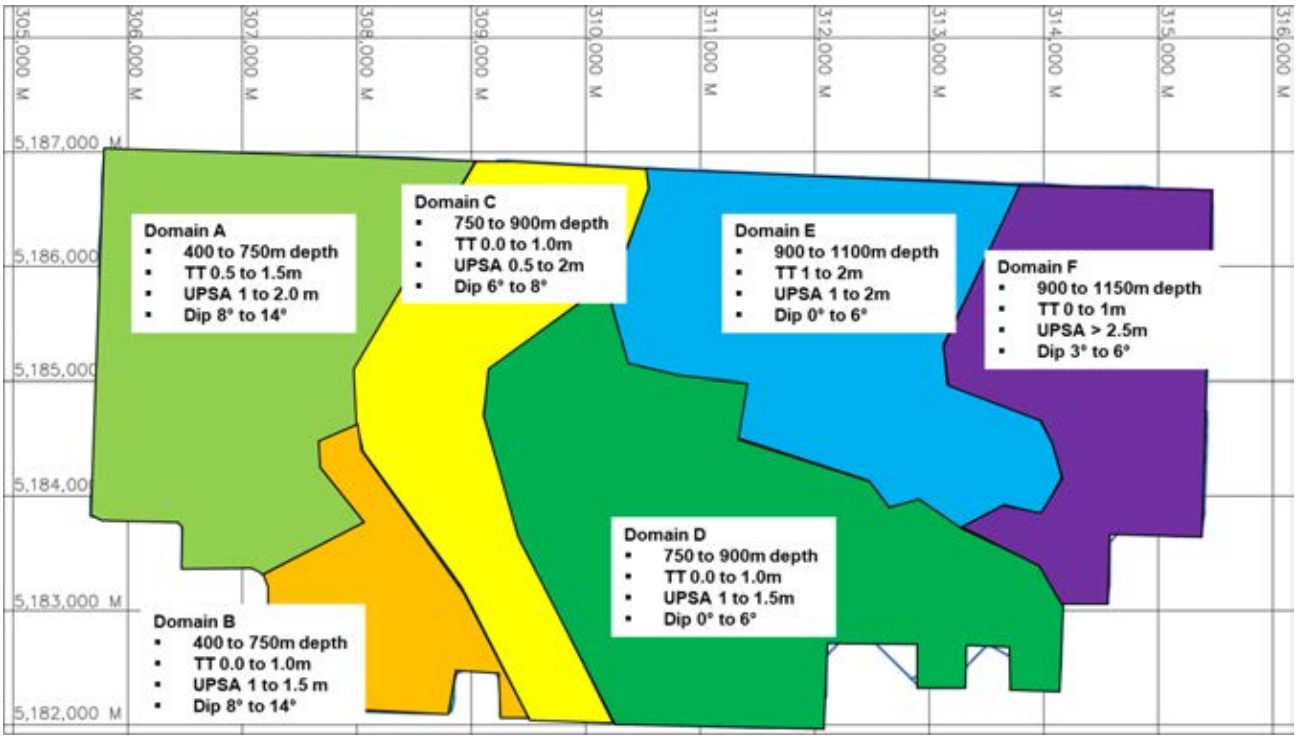
Figure 36: RQD Ranges for Key Lithologies

8.3 Geotechnical Domains and Expected Behaviour

The geotechnical data collected from the site, from mainly sub-vertical and deep boreholes, suggests that the rock mass conditions within the mining horizon are consistently of good quality rock masses with sparse open jointing. When considering the structural data and observations made from the historic White Pine mine reports, the rock mass behaviour is expected to be dominantly brittle with damage manifesting as stress induced fracturing. Thus the rock strength itself will be the main control on the rock mass strength. The available strength testing data suggests that the rock strength by stratigraphy is relatively consistent across the ore deposit and therefore domaining based on strength was not considered. However, the apparent consistency of the intact rock strength across the site may be influenced by the issues associated with both the PLT and laboratory testing; this interpretation should be revisited as additional intact strength data become available for the project.

Review of the logged stratigraphy suggests that the ore horizon consists of mainly bedded shales and siltstones. In the immediate back of the proposed rooms, there exists a layer of tightly bedded shales with relatively weak laminations (TOPZ and TIGR). These shales are expected to be challenging to manage during mining and have therefore driven many of the recommendations included in this report. Above the siltstones, however, is a band of high-quality and strong sandstone (UPSA) which ranges in thickness from 1 m to greater than 3 m and is expected to be a key lithology for maintaining stability of the backs of the rooms.

Geotechnical domains (Figure 37) have been defined for the project based on mining depth, thickness of the TOPZ/TIGR, UPSA, and dip. These parameters are each expected to have a control on the geotechnical performance of the mine.



Note: Image is clipped to the claim boundary.

Figure 37: Geotechnical Domains Selected Based on Thickness of the TOPZ and TIGR (TT), Thickness of the UPSA, Dip of the Ore, and Depth of Each Domain

9.0 NUMERICAL STRESS ASSESSMENTS

Numerical stress models in FLAC3D V7 were developed to assess stress redistribution, depth of rock mass damage, and pillar stability for the mine development and production areas. Several model schemes were developed at different scales (Table 7) which were utilized to confirm room and pillar dimensions, inform ground support selection, and identify stress related challenges with increasing depth.

Table 7: Model Schemes and Purpose

Model Scheme	Target Area	Purpose
2D Decline Model (FLAC3D)	Central declines and Protection Pillar	High resolution continuum model to evaluate stress redistribution and damage associated with driving the central declines. Used for ground support design and confirmation of pillar sizes. The impact of the adjacent panel production areas is simulated to confirm appropriate sizing of the protection pillar separating the central drives from the panels.
2D Barrier Pillar Model (FLAC3D)	Barrier pillar between WPN and the historic WP mine	High resolution continuum model to evaluate extents of stress redistribution into the barrier pillar between the WPN mine and historic WP mine. Used for evaluating barrier pillar sizing.
3D Four Pillar Model (FLAC3D)	Historic Northeast Mine	High resolution continuum model to benchmark stress and material parameter inputs against observed behaviour in the historic mine.
3D Four Pillar Model (FLAC3D)	Production Panels	High resolution continuum model to evaluate the local mining sequence (i.e., retreat stubbing), stress redistribution, and stress damage to the rock mass
3D Intersection Model (FLAC3D)	Panel access and ore drive intersections (3-way)	High resolution continuum model to evaluate the depth of stress damage in the production intersections to inform ground support selection.
3D Intersection Model (FLAC3D)	Ventilation drive and ore drive intersections (4-way)	High resolution continuum model to evaluate the depth of stress damage in the ventilation drive intersections to inform ground support selection.
3D Panel Model (FLAC3D)	Production panels and central declines	Panel scale continuum model to inform mine scale stress redistribution and rock mass damage potential, as well as to inform surface deformation potential

Details of each of model set up including geometry, boundary conditions, zone size, excavation sequencing, stress initialization, and material groups are summarized in Appendix B.

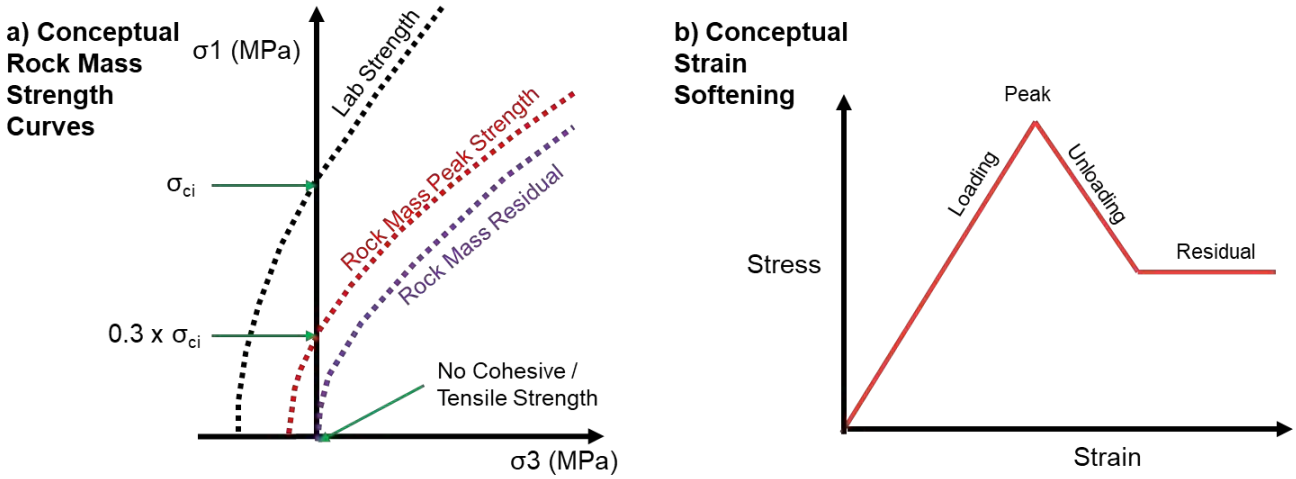
9.1 Constitutive Model and Interpretation

Based on the rock mass characterization (Section 8.0) and context from the historic mine (Section 4.0), the rock masses are massive to moderately jointed and the dominant mechanism of instability in the WPN mine is expected to be stress induced fracturing through the intact rock and separation of beds which can lead to bulking and buckling driven deformation. These mechanisms differ from the behaviour of more jointed rock masses which is driven by shearing and sliding of rock blocks along existing discontinuities. The Hoek-Brown GSI based failure

criterion commonly used for simulation of rock mass behaviour was developed for jointed rock masses and cannot be directly applied for rock masses that are susceptible to stress fracturing as this will lead to overestimation of rock mass strength at low confinement and underestimation of rock mass strength at high confinements.

A modified Hoek-Brown failure criterion (Bewick et al. 2015) was therefore utilized to develop representative brittle rock mass strength envelops for each stratigraphy of interest. This approach addresses the stress driven damage mechanism and captures the rock mass tensile strength, spalling strength (i.e., stress fracturing at low confinement which leads to fragments detaching), and the confined strength (i.e., control on pillar core stability). This approach is applicable to hard, brittle rock masses with GSI greater than approximately 65. For the tightly bedded stratigraphy (e.g., the TOPZ and TIGR) the role of the beds leads to anisotropic strength which is not directly captured in the brittle rock mass strengths. However, initial modelling suggested that these units are readily damaged in the simulation without introducing anisotropic strengths, and therefore no refinements to the approach were warranted for this stage of study.

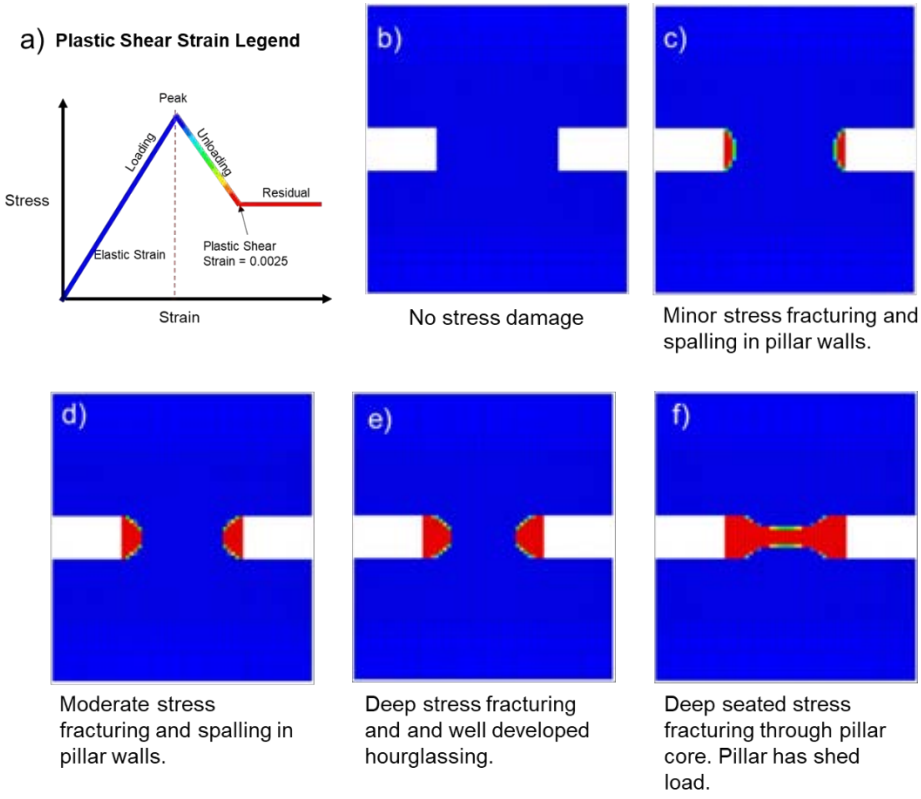
It is well documented (e.g., Martin et al. 1999) that stress fracturing and spalling around excavation tends to initiate when the compressive stresses at the boundary of the excavation exceeds 0.3 to 0.5 times the UCS. For the purposes of geotechnical modelling, WSP has assumed the low bound strength of 0.3 times the UCS, which is often considered as a proxy for the long-term strength of brittle rock (Figure 38a). Notably, the rock mass damage simulated with a rock mass strength of 0.3 times the UCS will not entirely experience a loss of cohesive strength, particularly if supported by grouted rockbolts and/or shotcrete which can provide local confinement. Thus, the damage envelopes forecast in the models should be considered as the maximum depths of damage around the excavations which may require support over the LOM. These damage depths will not be experienced everywhere in the mine but need to be planned for in the ground support selection.



Note: a) The rock mass strength curves, and b), the strain softening approach utilized in the numerical models

Figure 38: Conceptual Schematics

Strain softening is also applied in the numerical models to capture the loss of strength and stress redistribution associated with brittle rock masses (Figure 38b). For the purposes of geotechnical modelling, the unloading rate has been assumed to match the loading rate which is aligned with WSP's experience in room and pillar mines in sedimentary rock. Tracking of the rock mass damage is done by contouring of plastic shear strain which provides an indication of the level of damage the rock mass has experienced (e.g., just past peak but carrying load vs. reaching the residual strength with load mostly shed). Figure 39 shows a demonstration of how contours of plastic shear strain have been used to interpret the behaviour of brittle pillars for WPN.



Note: a) showing the legend and b) through f) showing the progression of damage for a pillar under increased vertical loading. For b) to e) the cores of the pillar are still intact and therefore still able to carry additional load. f) represents pillar failure.

Figure 39: Stage of Pillar Damage Interpreted Based on Contours of Plastic Shear Strain

9.2 Material Parameters

As described in the previous section, the rock at White Pine was simulated using a modified HB approach for brittle rock and the parameters are shown in Table 8. Due to scarce open structure observed in the core, the intact modulus was assumed as the rock mass modulus.

The HB constitutive model was applied with the compressive strength (σ_{ci}) downgraded to 30% to give the rock mass compressive strength (σ_{sp}). The m_b parameter was assumed to be equivalent to m_i which provides a good representation of the low confinement rock mass strength but underestimates the confined strength. This approach was used because the proposed pillars are very squat and therefore the main challenge as the mine extends deeper is the stress fracturing of the 'skin' of the excavations. This also provides a conservative approach for evaluating mine scale stability as the pillars are likely stronger than simulated.

The post-peak (i.e., residual) parameters were selected to provide strain softening at low confinement and near perfectly plastic behaviour at high confinement. The HB 's' parameter is dropped to near zero (0.001) to represent loss of cohesion while the m_b value is unchanged. The tensile strength is softened to zero. A constant dilation angle of 10º was applied in the simulations. The critical plastic shear strain was selected to give provide a softening slope that is similar to the rock mass loading slope.

Table 8: Material Inputs for Geotechnical Assessment

Stratigraphic Unit	Density (kg/m^3)	Youngs Modulus (GPa)	Poisson's Ratio	σ_{ci} (MPa)	σ_{sp} (MPa)[1]	m_b	s	a	Residual s	Tensile Strength (MPa)	Residual Tensile Strength (MPa)	Dilation Angle (°)	Critical Plastic Shear Strain
WIDE	2700	45	0.32	140	42	11	1	0.5	0.001	3.8	0	10	0.0025
UZVA	2700	43	0.30	160	48	11	1	0.5	0.001	4.4	0	10	0.0025
BMAS	2700	54	0.30	150	45	15	1	0.5	0.001	3.0	0	10	0.0025
THIN	2700	53	0.19	80	24	12	1	0.5	0.001	2.0	0	10	0.0025
UPSA/UTRA	2700	50	0.32	150	45	18	1	0.5	0.001	2.5	0	10	0.0025
TOPZ/TIGR	2700	41	0.25	90	27	12	1	0.5	0.001	2.3	0	10	0.0025
DGMA/RMAS/LTRA	2700	48	0.31	150	45	13	1	0.5	0.001	3.5	0	10	0.0025
DOMN	2700	48	0.15	95	28.5	8	1	0.5	0.001	3.6	0	10	0.0025
LWSA	2700	38	0.37	120	36	12	1	0.5	0.001	3.0	0	10	0.0025
Average[2]	**2700**	**50**	**0.28**	**125**	**37.5**	**12**	**1**	**0.5**	**0.001**	**3.10**	**0**	**10**	**0.0025**

Note:
[1] σ_{sp} = spalling strength; assumed to be 0.3 x σ_{ci}
[2] average parameters used only in the mine panel scale models

9.3 Stress Cases Evaluated

Because the orientation of the in situ stresses are not well constrained for the WPN project, conservative stress cases were developed to verify that the ground support and pillar stability were evaluated for the worst possible orientations of the excavations relative to the maximum horizontal stress. For example, the mine declines are proposed to be driven on an approximately east-west trend. As previously discussed (Section 6.0), the historical stress testing data from White Pine suggests that the orientation of the maximum horizontal stress ranges from N-S to E-W. Since driving the declines perpendicular to the maximum horizontal stress will result in the greatest stress concentration in the backs / floors, this 'worst case' was assumed for the simulations. Additionally, each simulation was evaluated with a KH of 2.0 and 2.5 to bracket the likely magnitude range for WPN. The results of the simulations with KH of 2.5 were used to develop the geotechnical guidance.

9.4 Model Results and Discussion

The following sub-sections outline the results of the key simulations use to develop guidance for the WPN project. Additional details on the simulations are shown in Appendix B.

9.4.1 Historical Mine Benchmark

Benchmark modelling of the historic WP Northeast mine at a depth of 700m suggests that the assumed material parameters and stress condition range (KH of 2 to 2.5) reasonably captures the observed pillar and room performance described by former WP employees and observed by WSP staff in the 1990s. The stress case with KH of 2.5 at 700m depth likely overestimates the average depth of stress damage in the room backs (model shows ~2m). Based on the historic observations, the maximum depth of damage observed was 0.6 to 1.5 m with many rooms having flat, undamaged backs (consistent with the KH = 2.0 simulations). The simulations also show damage in the floor which may have gone unnoticed in the historic mine as this was not discussed by former mine staff as causing any challenges with floor heave etc. The benchmark models provide a degree of confidence that the range of parameters and stress inputs for this geotechnical study are reasonable and adequately conservative for mine design. The results also suggest that the modelling approach using a continuum, while not capturing the influence of the tight bedding of some lithologies directly, is providing a reasonable match to the observed behaviour.

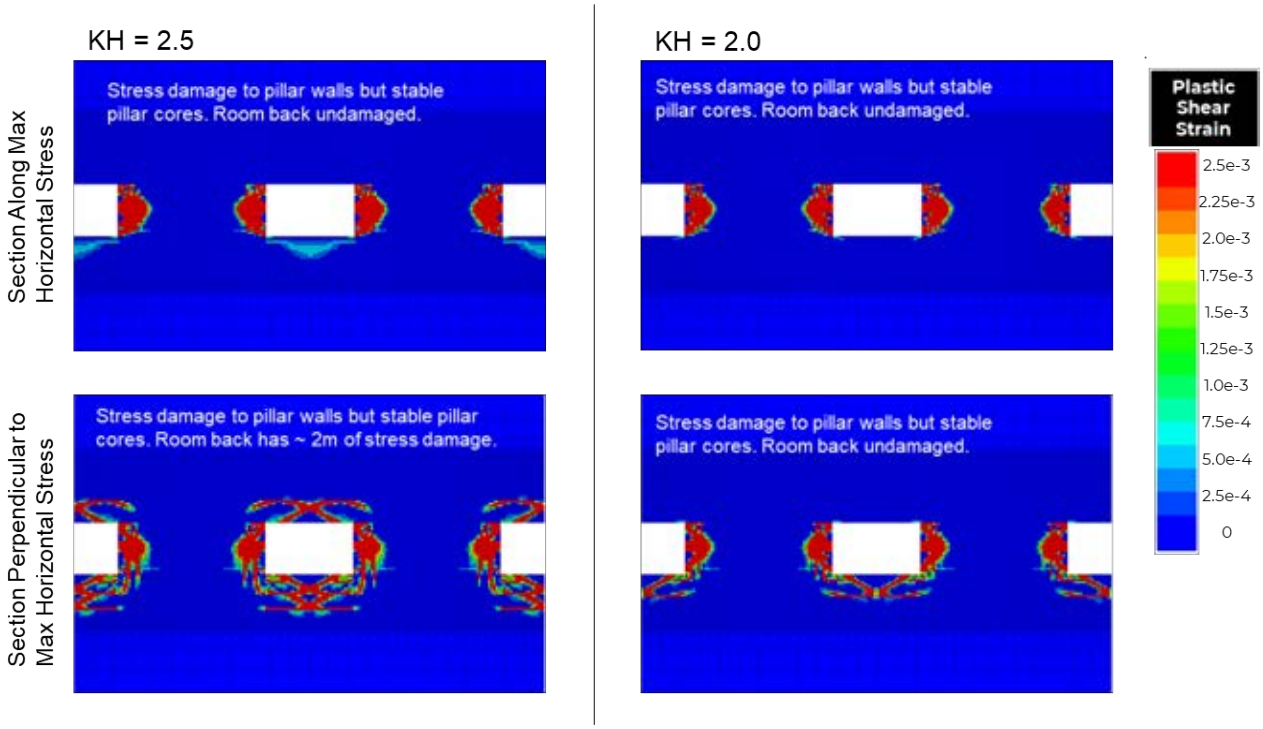
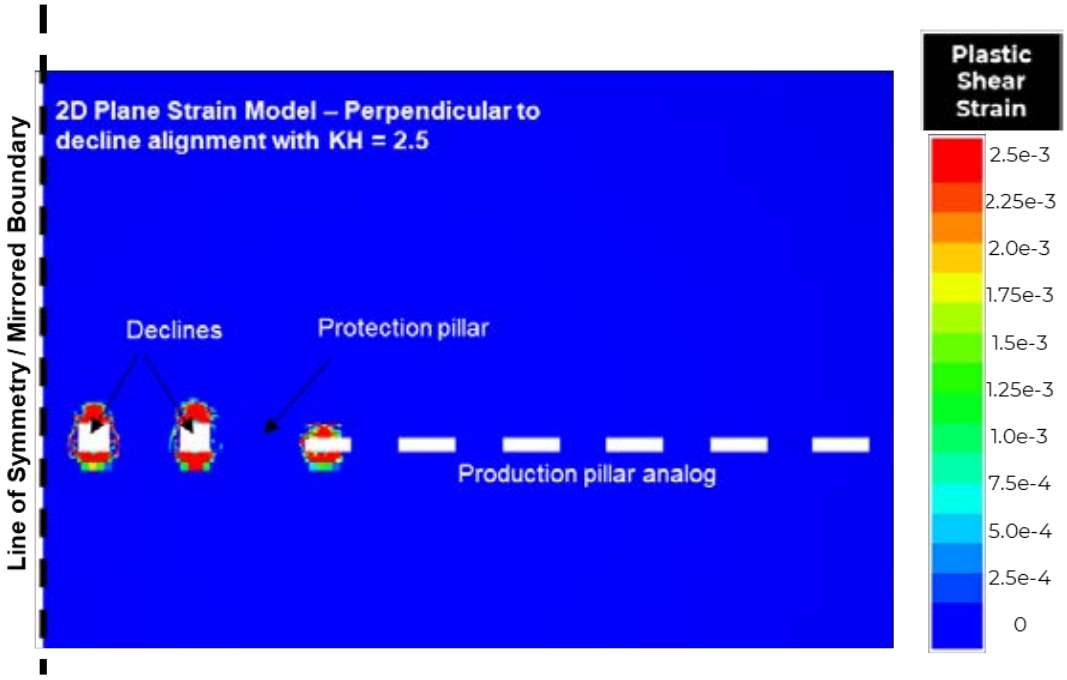


Figure 40: Benchmark Modelling of the Historic WP Northeast Mine at a Depth of 700 m

9.4.2 Decline Stability

2D simulation of the 4 central declines and protection pillar was used to identify minimum pillar dimensions between the declines and the protection pillar separating the production panels from the declines. These are key pillars which are required to remain stable for the LOM. Mining induced stress damage around the central drives, and particularly the conveyor decline, would also be very challenging to manage likely resulting in significant disruptions to production. Therefore, the pillars were sized to limit changes in stress damage profiles as the adjacent panel is mined (a 2D analog was applied to approximate the extraction ratio of the proposed panel mining).

Figure 41 shows the output of the simulation for Domain F at a depth of 1,100 m. The results of the simulations suggest that the depth of stress damage could reach up to 3 m in the backs and floors of the declines at a depth of 1100m. These simulations assume the least favourable orientation of the maximum horizontal stress (i.e., perpendicular to the decline trend) and therefore this should be considered as the maximum plausible depth of damage. The decline ribs have up to 0.5 of damage in Domain F and only minimal change (i.e., less than 0.2 m of stress damage) occurs when the adjacent panel mining is complete.

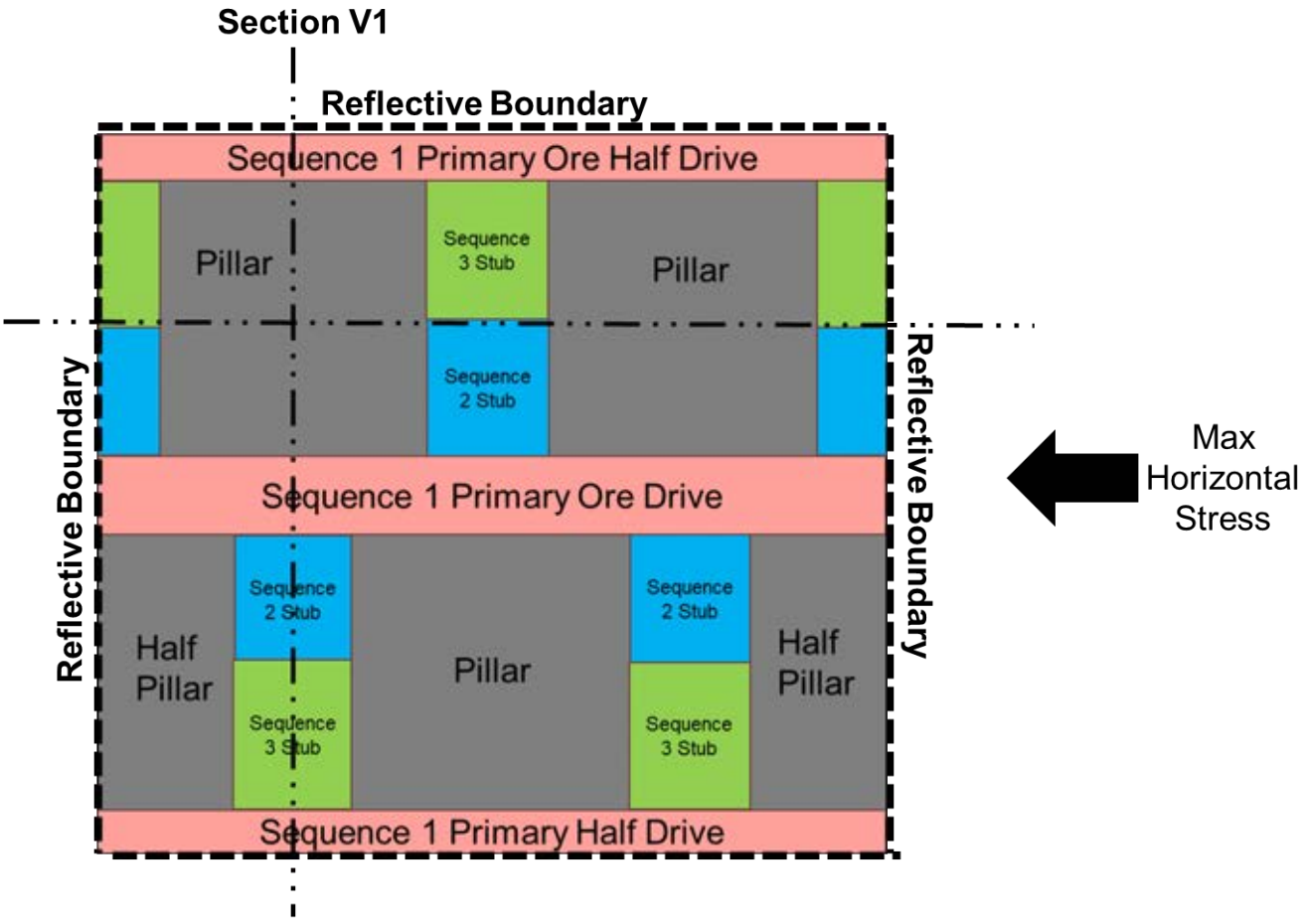


Note: Showing plastic shear strain reaching approximately 3 m into the backs. The protection pillar and pillars separating the declines are shown to be stable.

Figure 41: 2D Plane Strain Model of the Declines at their Deepest Depth (1,100 m) Production Pillars and Rooms

Four pillar models with reflective boundaries were developed to evaluate pillar and room stability for the production areas. As the proposed pillars are very squat, they are outside the range of the empirical pillar stability databases (i.e., the width to height ratios are above 3) and therefore the numerical assessments formed the basis for checks on pillar stability. The assumption with these simulations is that the simulated pillars are at the center of an infinitely wide mining panel and therefore the full tributary load is experienced in the pillars. Since the proposed panels at WPN do have very large extents, this is a reasonable approach to evaluate pillar stability, but it is emphasized that pillars towards the edges of the panels will likely not experience these full tributary loads. Arching across the mining area is also a possibility, given the high horizontal stresses. Thus, if mine performance is being evaluated against the simulation, it is imperative that pillars near the centers of large panels be considered.

The proposed mining sequence involves alignment of the ore drives with the orientation of the maximum horizontal stress to reduce stress concentration in the backs/ floors of these headings. Then, in retreat, the cross cut ‘stubs’ are excavated and mucked with remote loaders such that the backs do not need to be supported. The sequence of excavation in the four pillar models is shown in Figure 42.

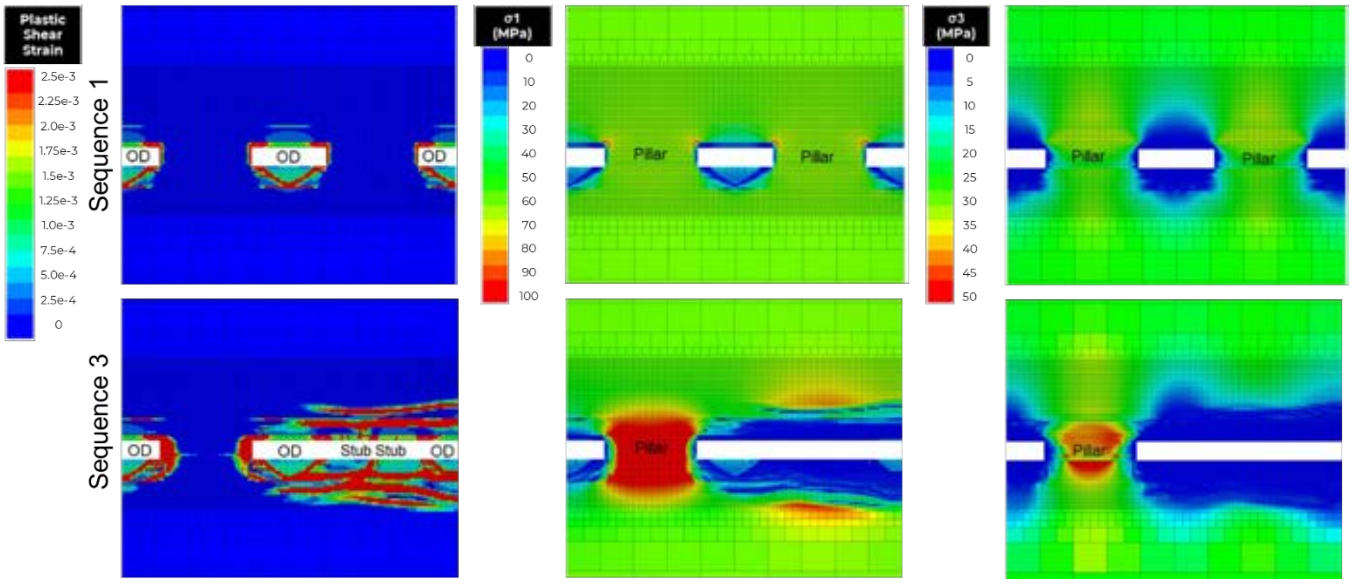


Note: showing the excavation sequence, reflective boundaries, and orientation of the maximum horizontal stress relative to the primary ore drives

Figure 42: Horizontal Section Through the Center of the Four-Pillar Model

An example of the model outputs from the 4-pillar models is shown in Figure 43 for Domain D (KH = 2.5) along the V1 section shown in Figure 41. While the depth of rock mass damage varies by domain, these model results are representative of the key findings. During initial primary ore drive mining (top row plots in Figure 43 showing Sequence 1), the depth of stress damage in the ore drive backs is limited to approximately 0.6m and stress damage to the pillar walls is less than 0.5m deep. Once retreat mining is complete (bottom row plots in Figure 43 showing Sequence 3), the pillar walls become damaged more deeply (~2m), but the cores of the pillars are undamaged, confined, and able to carry additional load. The depth of damage in the ore drive backs increases to 2 to 4m depending on proximity to the intersections.

As previously discussed, the 4-pillar models likely overestimate the stress redistribution that will occur near the front of the mining advance due to the influence of the abutment. Therefore, the stress damage shown in the models is more representative of the damage that may be expected once the panel is completely mined and not the conditions that will be encountered at the mining front.



Note: Showing damage to the pillar walls but undamaged pillar cores.

Figure 43: Example Output from a 4-Pillar Model of Domain D at 900 m Depth with KH = 2.5

The key takeaways from the 4-pillar models and implications for mining are the following:

- The backs of the primary ore drives have up to 1.5 m of stress damage (for the deepest domain F) before stubbing. Note that some of the stress damaged rock in the backs has not reached its residual state and is therefore still carrying load. However, for the purposes of ground support design, the rockbolts will need to be installed beyond the depth of stress fractured rock as the wide room spans will not benefit from arching.
- The pillar walls show initially limited damage during primary ore drive mining. Some minor slabbing / spalling of the pillar walls is possible during mining of the primaries that will need to be managed.
- During stubbing, the stress damage in both the pillar walls and drive backs (in the ‘entry’ portions of the ore drives, not the ‘non-entry’ stub intersections) grows suggesting that the ground support will need to accommodate this increased depth of damage. Because of the retreat mining sequence, the exposure to these higher stress conditions will be limited to relatively small sections of the ore drives at the mining front.
- The room backs are heavily stress damaged in retreat but this is a local stability concern that can be managed with ground support (in the primaries) and the mining sequence to eliminate personnel exposure. The local instability is not shown to destabilize the pillars. The simulations show that the stress damage is limited to a maximum of 6m above the room backs.
- The fully rounded pillars are shown to be stable, confined, and capable of taking additional load. The pillars are highly stressed in the deep domains, and therefore it is imperative that fault structures that could be exploited by the high stress condition in the confined pillars be identified and specific assessments conducted to evaluate potential for fault slip.

9.4.3 Panel Stability and Surface Deformation

Two panel scale models were run for Domain D (900m) and Domain F (1100m) to confirm that the pillars are stable after mining and to investigate load redistribution that occurs near the mining front. Figure 44 shows the setup of the panel scale models at an intermediate mining stage (i.e., before the panel has been fully mined).

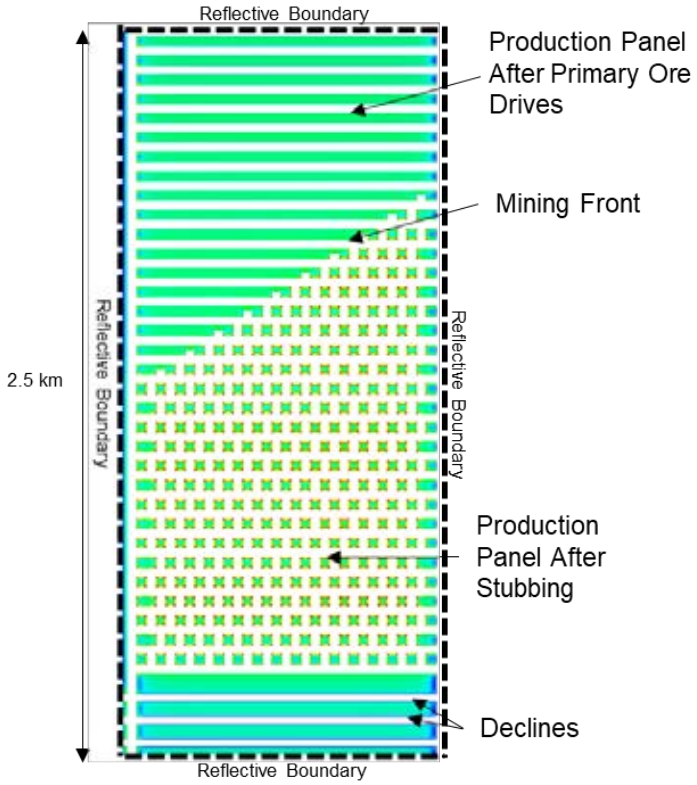
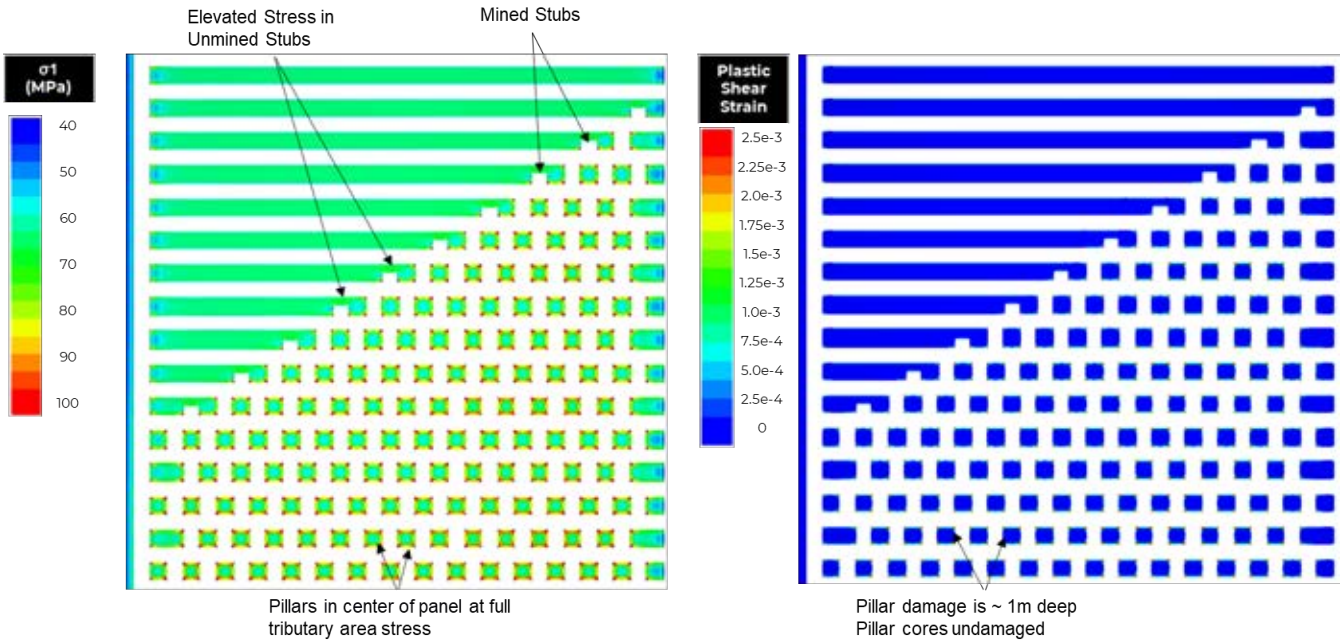


Figure 44: Example Setup of the Panel Scale Models Showing the Reflective Boundaries and the Mining Sequence

Figure 45 shows the output of the panel scale model for Domain D (KH = 2.5). Key findings from these simulations are as follows (applicable to every Domain):

- The retreat mining sequence will be effective in reducing exposure of mine personnel to the highest stress changes in the pillars which occurs away from the stubbing ‘mining front’
- There is elevated stress in some of the stub locations prior to being mined, but the simulations suggest only minimal stress damage associated with these areas. However, this will be an important consideration for drill and blasting activities, particularly in the deep domains.
- At the end of the mining sequence (i.e., the panel is fully mined), the pillars are stable, confined, and capable of taking more load. There may be opportunities for selective mining of protection pillars in retreat at the end of the mine life.
- The panel scale models (and the more conservative 4-pillar models) confirmed that the pillars remain stable and only localized stress damage to room backs is expected.



Note: Showing the maximum stress and plastic shear strain contours

Figure 45: Example Output from the Panel Scale Model for Domain D

10.0 GEOTECHNICAL GUIDANCE AND RECOMMENDATIONS

WSP has developed geotechnical guidance and recommendations for the WPN project which are summarized in the sub-sections that follow.

10.1 Decline and Protection Pillar

WPC indicated that 4 parallel declines (6.5 m height, and 6 m width) are required for mine access, conveyance, and ventilation. Design recommendations on the sizing of the pillars between the declines as well as the protection pillars which separate the production areas from the declines are as follows:

- **The pillars between the declines:** 15 m width corresponding to pillar width to height ratio of 2.3:1 or a pillar width to drive width ratio of 2.5.
- **The protection pillars:** 20m width. Stubbing should not be conducted in this pillar until the end of mine life in retreat.

10.2 Room and Pillar Sizing and Sequence

The proposed mining method for the new mine is room and pillar mining. Initial analysis of the pillar and back stability in the rooms suggests that the back stability will be a challenge for the mining drives that are driven perpendicular to the major principal stress as this orientation concentrates high differential stress in the back. This damage could be supported with long support, however due to the low mining heights of 2 to 2.5m, there is substantial benefit in reducing requirements for long support for the mine. Thus, the mine plan incorporates a mining sequence that reduces support requirements in the backs of the rooms.

The sequence requires alignment of the mining panels approximately (e.g., +/- 15 degrees) with the orientation of the major and intermediate principal stresses. The ore drives are mined parallel to the major principal stress, limiting stress damage in the backs. This is followed by stubbing of the pillars in retreat, such that the room and pillar intersections are non-entry and can be mucked using remote loaders. The back of the rooms will be unsupported above the stubs, which may cause some locally higher dilution, due to the shale units (TOPZ and TIGR) in the back, which should be accounted for in the mine plan. The pillars are also offset, rather than square, to limit the intersection spans created by the ore drives and stubs (i.e., 3-way intersections rather than 4-way).

To manage the potential for stress damage in the backs of the rooms, and to limit the amount of ground support required in the mine, WSP recommends the following mining sequence and layout considerations:

- Primary mining should be carried out in a direction that is approximately aligned with the maximum horizontal stress. Confirmation of the local stress orientations can be undertaken with a stress measurement campaign and through detailed record keeping and back analysis of observations of stress fracturing and back instabilities.
- Stubbing of pillars should be undertaken in retreat using remote mucking equipment. The rooms opened after stubbing should be considered open stopes and therefore non-entry.
- To reduce the intersection spans, pillar rows should be offset to limit the number of 4-way intersections.
- Mining heights in the deepest domains are increased to 2.5 m to account for the likely overbreak and challenges in supporting the TOPZ and TIGR. Increasing the mining height also enables installation of longer solid bar, which is beneficial in providing deeper anchorage of potentially stress fracture rock in the backs of the rooms.

Stable pillar dimensions are recommended by mining domain (Table 9). Note that the extraction ratio is higher in some cases than the historical mine achieved, but this is due to the lower mining height which increases the confinement in the pillar cores, therefore allowing higher extraction.

Table 9: Recommendations for Stable Pillar Dimensions for the WPN Project

Domain	Depth (m)	Mining Height (m)	Pillar Width (m)	Room Width (m)	Extraction Ratio (%)
A	400 to 750	2	8	8	75
B	400 to 750	2	8	8	75
C	750 to 900	2	10	8	69
D	750 to 900	2	10	8	69
E	900 to 1100	2.5	12	8	64
F	900 to 1150	2.5	14	8	60

10.3 Vent Raises

Due to the horizontal stresses at WPN, the deeper portions of the vent raises (approximately > 600 m depth) may be overstressed resulting in stress fracturing and spalling from the walls. The damage will be located on two opposite sides of the vent raises and will manifest in behind the cutter head. Based on WSP's experience and benchmarks of deep vent raises, the wall damage is not expected to be a critical flaw. The additional sloughed material will need to be mucked out, and the raise profile will be impacted (i.e., widened to a dog ear shape). Based on commentary in the historical reports around long term strength degradation due to exposure to moisture, it is recommended that the vent raises be lined with shotcrete. The main purpose of the shotcrete will be to provide a moisture barrier to prevent long term deepening of stress fracturing that could eventually compromise the raise serviceability and lead to collapse. The proposed vent raise locations require geotechnical holes to confirm that challenging ground conditions will not be encountered.

10.4 Boundary Pillar Size

A boundary pillar is required to separate WPN from the historical mining areas on the south boundary of the mining area. The boundary pillar is recommended to be at least 50 m wide, to prevent significant stress redistribution to the historic mining areas; the provided mine plan assumes a minimum 300 m offset.

As the structural model is improved during future studies, the boundary pillar thickness should be revisited. Criteria should also be developed during future study stages to evaluate acceptable levels of stress change in the historic mine.

10.5 Ground Support

Ground support recommendations are outlined in Table 10, Table 11, and Table 12.

Table 10: Ground Support Recommendations for Main Decline Accesses

Support Element Details		Domain						Notes
		A	B	C	D	E	F	
20mm Resin Bar Drift Back	Bolt Length (m)	2.4	2.4	2.4	3	3	3	1) Confirmation of ground conditions and support performance required during development. Additional support added as required based on actual conditions. 2) Additional cable support required for 3-way and 4-way intersections (see separate recommendations) 3) Resin bar should be 20mm min. 500 MPa yield strength. Threaded bolt heads to allow plates to be tightened to back. 4) Welded wire mesh should have at least 3 squares of overlap
	Bolt Spacing (m)	1.2	1.2	1.2	1.2	1.2	1.2	
	Ring Spacing (m)	1.2	1.2	1.2	1.2	1.2	1.2	
20mm Resin Bar Drift Ribs	Bolt Length (m)	1.8	1.8	1.8	2.4	2.4	2.4	
	Bolt Spacing (m)	1.2	1.2	1.2	1.2	1.2	1.2	
	Ring Spacing (m)	1.2	1.2	1.2	1.2	1.2	1.2	
	Distance from Lowest Rib Bolt to Floor (m)	0.5	0.5	0.5	0.5	0.5	0.5	
Welded Wire Mesh (mm)		5	5	5	5	5	5	
Shotcrete (mm)		50	50	50	50	50	50	

Support Element Details	Domain						Notes
	A	B	C	D	E	F	
							5) Shotcrete can be sprayed out of cycle and in campaigns

Table 11: Ground Support Recommendations for Panel Drifts

Drift	Location[(1)]	Mining Height (m)	Room Width (m)	Rebar Length (m)	Rebar spacing (m)	Welded Wire Mesh (mm)	Row of Bolts Into Shoulder?
Domain A Rooms	Back	2	8	1.6	1.2 x 1.2	5.0	Yes
Domain B Rooms	Back	2	8	1.6	1.2 x 1.2	5.0	Yes
Domain C Rooms	Back	2	8	1.6	1.2 x 1.2	5.0	Yes
Domain D Rooms	Back	2	8	1.6	1.2 x 1.2	5.0	Yes
Domain E Rooms	Back	2.5	8	2.1	1.2 x 1.2	5.0	Yes
Domain F Rooms	Back	2.5	8	2.1	1.2 x 1.2	5.0	Yes
Panel Accesses Drives and Panel Ventilation Drives	Back	3.2	8	3.0	1.2 x 1.2	5.0	Yes
	Ribs			2.0	1.2 x 1.2	5.0	-

Note:
(1) Rooms are not planned to have support in the ribs other than the row of bolts into the shoulder.

Table 12: Intersection Ground Support Recommendations

Intersection Type	Dimensions	Potential Damage Depth (m)	17.8 mm Cable Lengths (m)	Cable Spacing[(1)] (m)
3 or 4-way	6m x 6m	3	4	2.0
3 or 4-way	6m x 8m	4	5	2.0
3 or 4-way	8m x 8m	4	5	2.0

Note:
(1) Cables should extent two rings into each drift around the intersection.

Additional notes on ground support are provided below to address unique scenarios:

- Review of the fault model suggests that discrete fault structures will be present across the mining horizon. The intersecting core generally shows minimal damage (< 5 cm of gouge or increased core damage). 4 instances were identified with a 0.3 m to 0.5 m length of increased core damage without gouge.
- Where possible, drifts should be designed to cross major fault structures (identified in the fault model) at right angles. The main declines proposed in the mine plan generally follow this rule, the N-S cross-cuts accessing panels do not and should be adjusted accordingly. For the main declines:
  - The exposed length of rock damage, along a drift, from fault zones is assumed to generally be less than 1 round and therefore can be addressed with a localized increase in ground support
  - Assume that a fast application of 25 mm shotcrete is needed to prevent unravelling after taking a round and that ground support should otherwise follow what is recommended for Domains E and F. The declines cross only 14 fault zones, identified from the structural model.
- It is assumed that where two faults intersect, the local ground condition will be poor. One fault intersection occurs within 70 m of the declines. Considering the uncertainty of the fault locations at depth it is possible that the declines may have multiple rounds in poor ground in this area.
- The impacts of moisture on rock strength are assumed to be influential on mine stability for long term development near areas of ventilation (e.g., ventilation raises and areas close to the ventilation declines). Thus, shotcrete has been recommended in the declines and ventilation raises and may be necessary in other connecting drifts. The influence of moisture on rock strength should be assessed in greater detail during future study stages.
- If buckling becomes problematic at depth, higher strength rebar may be required (e.g., 24mm bar or higher steel grade) as well as thicker mesh.
- If the depth of damage exceeds 3m in the decline backs, cables may be required in addition to the recommended support. Currently, the maximum depth of damage has been forecast at 3m for the most unfavourable orientation of the declines relative to the maximum horizontal stress.
- The current recommendations have been developed assuming that moderate stress damage will need to be managed but that this damage will mainly manifest as gradual stress fracturing and spalling. Due to the uncertainties with the fault model and stress conditions at depth, rockburst potential has not been evaluated at this stage. However, the WPN project will be a deep mine and rockbursting and seismicity are possible which may require specific tactical and strategic approaches to manage during operation. Specifically, seismic conditions may have the following implications:
  - Re-entry time restrictions impacting cycle times
  - Requirements for enhanced ground support packages (e.g., dice bolt patterns, shotcrete, etc.)
  - Requirements for support at the advancing faces
  - Seismic monitoring
  - Hazard mapping and forecasting

11.0 CLOSURE

We trust that this report addresses your requirements for the WPN geotechnical study; if you have any questions or comments, please do not hesitate to contact the undersigned.

Sincerely,

WSP Canada Inc.

James Smith, P.Eng.
Principal Rock Mechanics Consultant,

Justin Roy Ph.D., P.Eng. (BC)
Principal Rock Mechanics Consultant,

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APPENDIX A:

LABORATORY TESTING REPORTS

Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens

Summary of Test Results

ASTM D7012 Method A

Project No.: US0043442.6551 Task 8000
Project: White Pine North
Location: Northern Michigan US
Client: White Pine LLC
Lab ID No: B25-089

Failure Modes

(1) Simple Extension (5) Single Shear
(2) Multiple Extension (6) Spalling
(3) Multiple Fracturing (7) Other
(4) Multiple Shear Note: (deg) measured from core axis

σ = differential failure stress
σ_1 = total failure stress
σ_3 = confining stress
Eq: $\sigma = \sigma_1 - \sigma_3$

No.	Borehole #	Sample #	Depth (m)	Dia (mm)	Ht (mm)	Area (cm^2)	Volume (cm^3)	Mass (g)	Wet Density (kg/m^3)	Moisture (%)	Dry Density (kg/m^3)	Stress σ_1 (MPa)	Stress σ_3 (MPa)	Stress σ (MPa)	Rock Type	Failure Criteria		
																Type	Mode	(deg)
1	WP592B	WP003541M	967.29 - 967.80	62.62	129.28	30.80	398.15	1089.20	2736	0.39	2725	239.01	10.00	229.01	UPSA	Combined	6/3	--
2	WP599B	WP003933M	770.03 - 770.60	63.39	131.03	31.56	413.53	1121.90	2713	0.69	2695	173.99	5.00	168.99	UPSA	Combined	3	--
3	WP596B	WP003748M	887.41 - 887.83	63.15	131.31	31.32	411.28	1139.60	2771	0.37	2761	197.09	5.00	192.09	UPSA	Intact	5	23
4	WP588B	WP003302M	996.31 - 996.79	62.33	130.27	30.51	397.49	1078.20	2713	0.65	2695	243.37	10.00	233.37	UPSA	Intact	5	21
5	WP601B	WP004147M	975.95 - 976.16	63.15	130.98	31.32	410.24	1137.50	2773	0.84	2750	208.42	10.00	198.42	UPSA	Intact	5	17
6	WP598B	WP003314M	1000.60 - 1000.10	62.28	130.00	30.46	396.03	1059.00	2674	1.38	2638	127.17	5.00	122.17	LWSA	Intact	5	30
7	WP593B	WP003710M	974.53 - 948.11	63.18	130.65	31.35	409.60	1077.00	2629	1.75	2584	95.98	2.50	93.48	LWSA	Intact	5	20
8	WP597B	WP003890M	887.00 - 887.50	60.63	130.62	28.87	377.12	1047.30	2777	1.28	2742	199.85	20.00	179.85	LWSA	Combined	5	29
9	WP597B	WP003891M	887.50 - 888.00	60.36	130.91	28.61	374.59	980.80	2618	1.61	2577	192.94	15.00	177.94	LWSA	Intact	5	22
10	WP601B	WP004153M	977.93 - 978.43	63.11	130.27	31.28	407.50	1081.50	2654	0.68	2636	236.79	10.00	226.79	LWSA	Intact	5	30
11	WP590B	WP003265M	873.13 - 873.58	62.85	130.63	31.02	405.27	1121.00	2766	0.35	2756	169.19	10.00	159.19	LTRA	Combined	6	--
12	WP591B	WP003386M	940.09 - 940.58	62.79	131.21	30.96	406.29	1135.80	2796	0.34	2786	101.08	5.00	96.08	DGMA	Discrete	5	23
13	WP600B	WP004093M	996.83 - 997.19	62.78	131.26	30.96	406.32	1122.50	2763	0.56	2747	164.95	5.00	159.95	TIGR	Intact	5/3	23
14	WP599B	WP003938M	772.11 - 772.57	63.33	130.97	31.50	412.55	1129.30	2737	0.37	2727	166.98	10.00	156.98	RMAS	Intact	6	--
15	WP593B	WP003694M	941.59 - 941.90	63.40	129.88	31.57	410.03	1122.40	2737	0.40	2726	182.74	5.00	177.74	BMAS	Intact	5	20
16	WP596B	WP003744M	886.45 - 886.80	63.12	130.93	31.29	409.70	1123.30	2742	0.26	2735	235.69	10.00	225.69	BMAS	Intact	5	32
17	WP598B	WP003294M	993.67 - 994.19	62.37	130.68	30.55	399.26	1100.10	2755	0.45	2743	139.70	5.00	134.70	RAGR	Combined	3/6	--
18	WP592B	WP003529M	962.59 - 967.09	62.72	130.24	30.90	402.39	1104.50	2745	0.34	2736	276.57	15.00	261.57	RAGR	Intact	5	13

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G. Patton	June 23, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens

Summary of Test Results

ASTM D7012 Method A

Project No.: US0043442.6551 Task 8000
Project: White Pine North
Location: Northern Michigan US
Client: White Pine LLC
Lab ID No: B25-089

Failure Modes

(1) Simple Extension (5) Single Shear
(2) Multiple Extension (6) Spalling
(3) Multiple Fracturing (7) Other
(4) Multiple Shear Note: (deg) measured from core axis

σ = differential failure stress
σ_1 = total failure stress
σ_3 = confining stress
Eq: $\sigma = \sigma_1 - \sigma_3$

No.	Borehole #	Sample #	Depth (m)	Dia (mm)	Ht (mm)	Area (cm²)	Volume (cm³)	Mass (g)	Wet Density (kg/m³)	Moisture (%)	Dry Density (kg/m³)	Stress σ_1 (MPa)	Stress σ_3 (MPa)	Stress σ (MPa)	Rock Type	Failure Criteria		
																Type	Mode	(deg)
19	WP597B	WP003869M	878.24 - 878.74	60.50	130.86	28.75	376.19	1038.80	2761	0.24	2755	230.56	10.00	220.56	RAGR	Intact	3/6	--
20	WP599B	WP003922M	765.70 - 766.20	63.32	131.15	31.49	412.99	1136.20	2751	0.34	2742	201.87	10.00	191.87	RAGR	Intact	5	25
21	WP597B	WP003875M	881.50 - 882.00	60.53	130.20	28.78	374.66	1030.90	2752	0.38	2741	201.52	10.00	191.52	UZVA	Intact	5	20
22	WP590B	WP003252M	868.49 - 868.79	62.86	130.18	31.03	404.00	1108.40	2744	0.44	2732	227.65	5.00	222.65	UZVA	Intact	6	--
23	WP592B	WP003533M	984.42 - 964.90	62.51	117.03	30.69	359.16	979.70	2728	0.38	2717	124.38	10.00	114.38	UZVA	Discrete	5	82
24	WP598B	WP004003M	911.71 - 912.19	59.80	131.15	28.09	368.35	1009.10	2740	0.26	2732	243.79	15.00	228.79	UZVA	Intact	3	--
25	WP597B	WP003871M	879.30 - 879.92	60.45	130.30	28.70	373.96	1028.80	2751	0.34	2742	222.47	10.00	212.47	WIDE	Intact	5	15
26	WP597B	WP003872M	879.92 - 880.50	60.50	130.38	28.75	374.81	1023.20	2730	0.58	2714	185.82	10.00	175.82	WIDE	Combined	5	25
27	WP591B	WP003370M	934.30 - 934.69	Sample broken - not suitable for testing											WIDE			
28	WP597B	WP003870M	878.74 - 879.30	60.49	130.50	28.74	375.03	1026.10	2736	0.47	2723	161.18	5.00	156.18	WIDE	Intact	5	11
29	WP599B	WP003923M	766.20 - 766.70	63.31	131.05	31.48	412.55	1130.00	2739	0.49	2726	157.05	5.00	152.05	WIDE	Intact	5	25
30	WP592B	WP003552M	972.05 - 972.55	62.74	129.93	30.92	401.69	1086.50	2705	0.97	2679	162.02	5.00	157.02	CCGL	Intact	5	28
31	WP592B	WP003553M (1	972.55 - 973.05	62.82	130.38	30.99	404.11	1110.80	2749	1.76	2701	161.12	10.00	151.12	CCGL	Intact	5	25
32	WP592B	WP003553M (2	972.55 - 973.05	62.76	130.48	30.94	403.65	1086.20	2691	1.49	2651	207.14	15.00	192.14	CCGL	Intact	5	24
33	WP594B	WP003576M	897.86 - 898.47	62.63	130.79	30.81	402.93	1103.30	2738	0.13	2735	243.03	20.00	223.03	CCGL	Intact	5	14
34	WP599B	WP003945M	774.42 - 774.92	63.44	160.88	31.61	508.53	1081.80	2127	0.20	2123	158.85	10.00	148.85	CCGL	Intact	6	--
35	WP601B	WP004155M	978.43 - 979.47	63.00	131.02	31.17	408.42	1098.90	2691	0.19	2685	177.37	15.00	162.37	CCGL	Intact	5	27
36	WP600B	WP004087M	994.60 - 994.77	62.83	130.92	31.00	405.91	1109.60	2734	0.05	2732	137.11	10.00	127.11	UTRA	Intact	5/6	36

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TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens

Summary of Test Results

ASTM D7012 Method A

Project No.: US0043442.6551 Task 8000
Project: White Pine North
Location: Northern Michigan US
Client: White Pine LLC
Lab ID No: B25-089

Failure Modes

(1) Simple Extension (5) Single Shear
(2) Multiple Extension (6) Spalling
(3) Multiple Fracturing (7) Other
(4) Multiple Shear Note: (deg) measured from core axis

σ = differential failure stress
σ_1 = total failure stress
σ_3 = confining stress
Eq: $\sigma = \sigma_1 - \sigma_3$

No.	Borehole #	Sample #	Depth (m)	Dia (mm)	Ht (mm)	Area (cm^2)	Volume (cm^3)	Mass (g)	Wet Density (kg/m^3)	Moisture (%)	Dry Density (kg/m^3)	Stress σ_1 (MPa)	Stress σ_3 (MPa)	Stress σ (MPa)	Rock Type	Failure Criteria		
																Type	Mode	(deg)
37	SRK Samples	WP003253M	868.88-869.13	62.90	129.07	31.07	401.07	1096.70	2734	0.27	2727	290.63	15.00	275.63	BMAS	Intact	5/6	22
38	SRK Samples	WP003406M	932.33-932.58	62.87	127.45	31.04	395.65	1065.60	2693	0.32	2685	344.99	30.00	314.99	UPSA	Intact	5	24
39	SRK Samples	WP003540M	967.06-967.29	62.61	127.88	30.79	393.71	1064.10	2703	0.44	2691	264.03	20.00	244.03	UPSA	Intact	5/6	22
40	SRK Samples	WP003750M	888.69-888.88	63.35	128.28	31.52	404.34	1092.00	2701	0.39	2690	255.33	15.00	240.33	UPSA	Intact	5/3	25
41	SRK Samples	WP003743M	886.00-886.24	62.89	127.06	31.06	394.70	1091.40	2765	0.28	2758	217.55	5.00	212.55	UZVA	Intact	5/6	25

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Pattom	June 23, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP592B
Project:	White Pine North	**Sample Number:**	WP003541M
Location:	Northern Michigan US	**Depth (m):**	967.29 - 967.80
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**736.10**
Stress σ_1 (MPa)	239.01
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	229.01
Pace Rate (kN/s)	1.25
Lithology	UPSA

Sample Measurements

Diameter	(mm)	62.62
Height	(mm)	129.28
Area	(cm^2)	30.80
Volume	(cm^3)	398.15
Mass	(g)	1089.20
Moisture Content	(%)	0.39
Wet Density	(kg/m^3)	2736
Dry Density	(kg/m^3)	2725

BEFORE TEST

Failure Mode

Type:	**Combined**
Mode:	**6/3**
Degrees:*	**--**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP599B
Project:	White Pine North	**Sample Number:**	WP003933M
Location:	Northern Michigan US	**Depth (m):**	770.03 - 770.60
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**549.10**
Stress σ_1 (MPa)	173.99
Stress σ_3 (MPa)	5.00
Stress σ (MPa)	168.99
Pace Rate (kN/s)	1.25
Lithology	UPSA

Sample Measurements

Diameter	(mm)	63.39
Height	(mm)	131.03
Area	(cm^2)	31.56
Volume	(cm^3)	413.53
Mass	(g)	1121.90
Moisture Content	(%)	0.69
Wet Density	(kg/m^3)	2713
Dry Density	(kg/m^3)	2695

BEFORE TEST

Failure Mode

Type:	**Combined**
Mode:	**3**
Degrees:*	**--**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP596B
Project:	White Pine North	**Sample Number:**	WP003748M
Location:	Northern Michigan US	**Depth (m):**	887.41 - 887.83
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**617.30**
Stress σ_1 (MPa)	197.09
Stress σ_3 (MPa)	5.00
Stress σ (MPa)	192.09
Pace Rate (kN/s)	1.25
Lithology	UPSA

Sample Measurements

Diameter	(mm)	63.15
Height	(mm)	131.31
Area	(cm^2)	31.32
Volume	(cm^3)	411.28
Mass	(g)	1139.60
Moisture Content	(%)	0.37
Wet Density	(kg/m^3)	2771
Dry Density	(kg/m^3)	2761

BEFORE TEST

Failure Mode

Type: **Intact**

Mode: **5**

Degrees:* **23**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP588B
Project:	White Pine North	**Sample Number:**	WP003302M
Location:	Northern Michigan US	**Depth (m):**	996.31 - 996.79
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**742.60**
Stress σ_1 (MPa)	243.37
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	233.37
Pace Rate (kN/s)	1.25
Lithology	UPSA

Sample Measurements

Diameter	(mm)	62.33
Height	(mm)	130.27
Area	(cm^2)	30.51
Volume	(cm^3)	397.49
Mass	(g)	1078.20
Moisture Content	(%)	0.65
Wet Density	(kg/m^3)	2713
Dry Density	(kg/m^3)	2695

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5**
Degrees:*	**21**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 23, 2025	D. Lim	April 28, 2025
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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

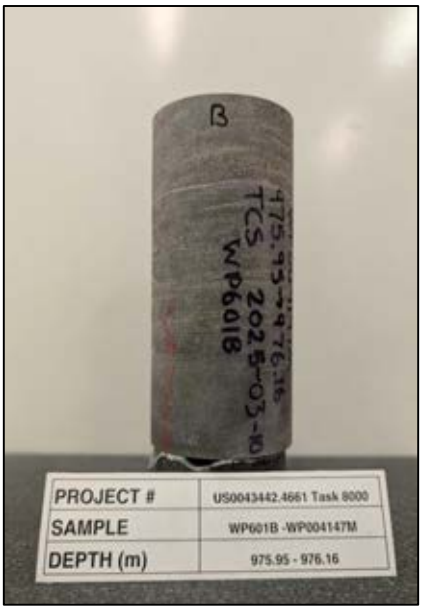
Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP601B
Project:	White Pine North	**Sample Number:**	WP004147M
Location:	Northern Michigan US	**Depth (m):**	975.95 - 976.16
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**652.80**
Stress σ_1 (MPa)	208.42
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	198.42
Pace Rate (kN/s)	1.25
Lithology	UPSA

Sample Measurements

Diameter	(mm)	63.15
Height	(mm)	130.98
Area	(cm^2)	31.32
Volume	(cm^3)	410.24
Mass	(g)	1137.50
Moisture Content	(%)	0.84
Wet Density	(kg/m^3)	2773
Dry Density	(kg/m^3)	2750



BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5**
Degrees:*	**17**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

σ1 = total failure stress (MPa)

σ3= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP598B
Project:	White Pine North	**Sample Number:**	WP003314M
Location:	Northern Michigan US	**Depth (m):**	1000.60 - 1000.10
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**387.40**
Stress σ_1 (MPa)	127.17
Stress σ_3 (MPa)	5.00
Stress σ (MPa)	122.17
Pace Rate (kN/s)	1.25
Lithology	LWSA

Sample Measurements

Diameter	(mm)	62.28
Height	(mm)	130.00
Area	(cm^2)	30.46
Volume	(cm^3)	396.03
Mass	(g)	1059.00
Moisture Content	(%)	1.38
Wet Density	(kg/m^3)	2674
Dry Density	(kg/m^3)	2638

BEFORE TEST

Failure Mode

Type: **Intact**

Mode: **5**

Degrees:* **30**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP593B
Project:	White Pine North	**Sample Number:**	WP003710M
Location:	Northern Michigan US	**Depth (m):**	974.53 - 948.11
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**300.90**
Stress σ_1 (MPa)	95.98
Stress σ_3 (MPa)	2.50
Stress σ (MPa)	93.48
Pace Rate (kN/s)	1.25
Lithology	LWSA

Sample Measurements

Diameter	(mm)	63.18
Height	(mm)	130.65
Area	(cm^2)	31.35
Volume	(cm^3)	409.60
Mass	(g)	1077.00
Moisture Content	(%)	1.75
Wet Density	(kg/m^3)	2629
Dry Density	(kg/m^3)	2584

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5**
Degrees:*	**20**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

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G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP597B
Project:	White Pine North	**Sample Number:**	WP003890M
Location:	Northern Michigan US	**Depth (m):**	887.00 - 887.50
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**577.00**
Stress σ_1 (MPa)	199.85
Stress σ_3 (MPa)	20.00
Stress σ (MPa)	179.85
Pace Rate (kN/s)	1.25
Lithology	LWSA

Sample Measurements

Diameter	(mm)	60.63
Height	(mm)	130.62
Area	(cm^2)	28.87
Volume	(cm^3)	377.12
Mass	(g)	1047.30
Moisture Content	(%)	1.28
Wet Density	(kg/m^3)	2777
Dry Density	(kg/m^3)	2742

BEFORE TEST

Failure Mode

Type:	**Combined**
Mode:	**5**
Degrees:*	**29**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada
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Tel: 604-412-6899 Fax: 604-412-6816

Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP597B
Project:	White Pine North	**Sample Number:**	WP003891M
Location:	Northern Michigan US	**Depth (m):**	887.50 - 888.00
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**552.10**
Stress σ_1 (MPa)	192.94
Stress σ_3 (MPa)	15.00
Stress σ (MPa)	177.94
Pace Rate (kN/s)	1.25
Lithology	LWSA

Sample Measurements

Diameter	(mm)	60.36
Height	(mm)	130.91
Area	(cm^2)	28.61
Volume	(cm^3)	374.59
Mass	(g)	980.80
Moisture Content	(%)	1.61
Wet Density	(kg/m^3)	2618
Dry Density	(kg/m^3)	2577

BEFORE TEST

Failure Mode

Type: **Intact**

Mode: **5**

Degrees:* **22**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

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G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP601B
Project:	White Pine North	**Sample Number:**	WP004153M
Location:	Northern Michigan US	**Depth (m):**	977.93 - 978.43
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**740.70**
Stress σ_1 (MPa)	236.79
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	226.79
Pace Rate (kN/s)	1.25
Lithology	LWSA

Sample Measurements

Diameter	(mm)	63.11
Height	(mm)	130.27
Area	(cm^2)	31.28
Volume	(cm^3)	407.50
Mass	(g)	1081.50
Moisture Content	(%)	0.68
Wet Density	(kg/m^3)	2654
Dry Density	(kg/m^3)	2636

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5**
Degrees:*	**30**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP590B
Project:	White Pine North	**Sample Number:**	WP003265M
Location:	Northern Michigan US	**Depth (m):**	873.13 - 873.58
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**524.90**
Stress σ_1 (MPa)	169.19
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	159.19
Pace Rate (kN/s)	1.25
Lithology	LTRA

Sample Measurements

Diameter	(mm)	62.85
Height	(mm)	130.63
Area	(cm^2)	31.02
Volume	(cm^3)	405.27
Mass	(g)	1121.00
Moisture Content	(%)	0.35
Wet Density	(kg/m^3)	2766
Dry Density	(kg/m^3)	2756

BEFORE TEST

Failure Mode

Type: **Combined**

Mode: **6**

Degrees:* **--**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

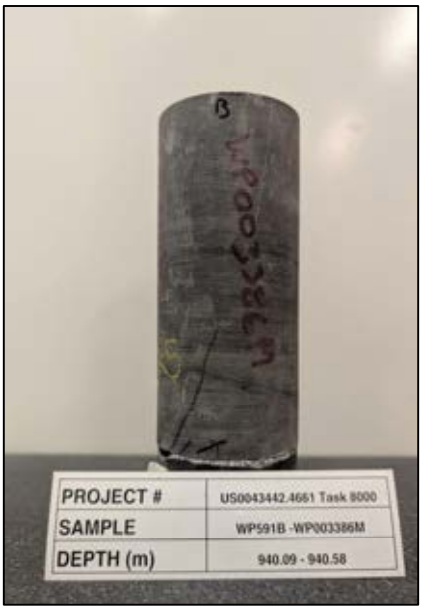
Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP591B
Project:	White Pine North	**Sample Number:**	WP003386M
Location:	Northern Michigan US	**Depth (m):**	940.09 - 940.58
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**313.00**
Stress σ_1 (MPa)	101.08
Stress σ_3 (MPa)	5.00
Stress σ (MPa)	96.08
Pace Rate (kN/s)	1.25
Lithology	DGMA

Sample Measurements

Diameter	(mm)	62.79
Height	(mm)	131.21
Area	(cm^2)	30.96
Volume	(cm^3)	406.29
Mass	(g)	1135.80
Moisture Content	(%)	0.34
Wet Density	(kg/m^3)	2796
Dry Density	(kg/m^3)	2786



BEFORE TEST

Failure Mode

Type: **Discrete**

Mode: **5**

Degrees:* **23**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

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G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP600B
Project:	White Pine North	**Sample Number:**	WP004093M
Location:	Northern Michigan US	**Depth (m):**	996.83 - 997.19
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**510.60**
Stress σ_1 (MPa)	164.95
Stress σ_3 (MPa)	5.00
Stress σ (MPa)	159.95
Pace Rate (kN/s)	1.25
Lithology	TIGR

Sample Measurements

Diameter	(mm)	62.78
Height	(mm)	131.26
Area	(cm^2)	30.96
Volume	(cm^3)	406.32
Mass	(g)	1122.50
Moisture Content	(%)	0.56
Wet Density	(kg/m^3)	2763
Dry Density	(kg/m^3)	2747

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5/3**
Degrees:*	**23**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP599B
Project:	White Pine North	**Sample Number:**	WP003938M
Location:	Northern Michigan US	**Depth (m):**	772.11 - 772.57
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**526.00**
Stress σ_1 (MPa)	166.98
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	156.98
Pace Rate (kN/s)	1.25
Lithology	RMAS

Sample Measurements

Diameter	(mm)	63.33
Height	(mm)	130.97
Area	(cm^2)	31.50
Volume	(cm^3)	412.55
Mass	(g)	1129.30
Moisture Content	(%)	0.37
Wet Density	(kg/m^3)	2737
Dry Density	(kg/m^3)	2727

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**6**
Degrees:*	**--**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

Minor oil contamination during removal from Hoek cell.

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

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G. Patton	April 23, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP593B
Project:	White Pine North	**Sample Number:**	WP003694M
Location:	Northern Michigan US	**Depth (m):**	941.59 - 941.90
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**576.90**
Stress σ_1 (MPa)	182.74
Stress σ_3 (MPa)	5.00
Stress σ (MPa)	177.74
Pace Rate (kN/s)	1.25
Lithology	BMAS

Sample Measurements

Diameter	(mm)	63.40
Height	(mm)	129.88
Area	(cm^2)	31.57
Volume	(cm^3)	410.03
Mass	(g)	1122.40
Moisture Content	(%)	0.40
Wet Density	(kg/m^3)	2737
Dry Density	(kg/m^3)	2726

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5**
Degrees:*	**20**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP596B
Project:	White Pine North	**Sample Number:**	WP003744M
Location:	Northern Michigan US	**Depth (m):**	886.45 - 886.80
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**737.50**
Stress σ_1 (MPa)	235.69
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	225.69
Pace Rate (kN/s)	1.25
Lithology	BMAS

Sample Measurements

Diameter	(mm)	63.12
Height	(mm)	130.93
Area	(cm^2)	31.29
Volume	(cm^3)	409.70
Mass	(g)	1123.30
Moisture Content	(%)	0.26
Wet Density	(kg/m^3)	2742
Dry Density	(kg/m^3)	2735

BEFORE TEST

Failure Mode

Type: **Intact**

Mode: **5**

Degrees:* **32**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

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S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP598B
Project:	White Pine North	**Sample Number:**	WP003294M
Location:	Northern Michigan US	**Depth (m):**	993.67 - 994.19
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**426.80**
Stress σ_1 (MPa)	139.70
Stress σ_3 (MPa)	5.00
Stress σ (MPa)	134.70
Pace Rate (kN/s)	1.25
Lithology	RAGR

Sample Measurements

Diameter	(mm)	62.37
Height	(mm)	130.68
Area	(cm^2)	30.55
Volume	(cm^3)	399.26
Mass	(g)	1100.10
Moisture Content	(%)	0.45
Wet Density	(kg/m^3)	2755
Dry Density	(kg/m^3)	2743

BEFORE TEST

Failure Mode

Type: **Combined**

Mode: **3/6**

Degrees:* **--**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP592B
Project:	White Pine North	**Sample Number:**	WP003529M
Location:	Northern Michigan US	**Depth (m):**	962.59 - 967.09
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**854.50**
Stress σ_1 (MPa)	276.57
Stress σ_3 (MPa)	15.00
Stress σ (MPa)	261.57
Pace Rate (kN/s)	1.25
Lithology	RAGR

Sample Measurements

Diameter	(mm)	62.72
Height	(mm)	130.24
Area	(cm^2)	30.90
Volume	(cm^3)	402.39
Mass	(g)	1104.50
Moisture Content	(%)	0.34
Wet Density	(kg/m^3)	2745
Dry Density	(kg/m^3)	2736

BEFORE TEST

Failure Mode

Type: **Intact**

Mode: **5**

Degrees:* **13**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

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S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP597B
Project:	White Pine North	**Sample Number:**	WP003869M
Location:	Northern Michigan US	**Depth (m):**	878.24 - 878.74
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**662.80**
Stress σ_1 (MPa)	230.56
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	220.56
Pace Rate (kN/s)	1.25
Lithology	RAGR

Sample Measurements

Diameter	(mm)	60.50
Height	(mm)	130.86
Area	(cm^2)	28.75
Volume	(cm^3)	376.19
Mass	(g)	1038.80
Moisture Content	(%)	0.24
Wet Density	(kg/m^3)	2761
Dry Density	(kg/m^3)	2755

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**3/6**
Degrees:*	**--**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP599B
Project:	White Pine North	**Sample Number:**	WP003922M
Location:	Northern Michigan US	**Depth (m):**	765.70 - 766.20
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**635.70**
Stress σ_1 (MPa)	201.87
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	191.87
Pace Rate (kN/s)	1.25
Lithology	RAGR

Sample Measurements

Diameter	(mm)	63.32
Height	(mm)	131.15
Area	(cm^2)	31.49
Volume	(cm^3)	412.99
Mass	(g)	1136.20
Moisture Content	(%)	0.34
Wet Density	(kg/m^3)	2751
Dry Density	(kg/m^3)	2742

BEFORE TEST

Failure Mode

Type: **Intact**

Mode: **5**

Degrees:* **25**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP597B
Project:	White Pine North	**Sample Number:**	WP003875M
Location:	Northern Michigan US	**Depth (m):**	881.50 - 882.00
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**579.90**
Stress σ_1 (MPa)	201.52
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	191.52
Pace Rate (kN/s)	1.25
Lithology	UZVA

Sample Measurements

Diameter	(mm)	60.53
Height	(mm)	130.20
Area	(cm^2)	28.78
Volume	(cm^3)	374.66
Mass	(g)	1030.90
Moisture Content	(%)	0.38
Wet Density	(kg/m^3)	2752
Dry Density	(kg/m^3)	2741

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5**
Degrees:*	**20**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP590B
Project:	White Pine North	**Sample Number:**	WP003252M
Location:	Northern Michigan US	**Depth (m):**	868.49 - 868.79
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**706.50**
Stress σ_1 (MPa)	227.65
Stress σ_3 (MPa)	5.00
Stress σ (MPa)	222.65
Pace Rate (kN/s)	1.25
Lithology	UZVA

Sample Measurements

Diameter	(mm)	62.86
Height	(mm)	130.18
Area	(cm^2)	31.03
Volume	(cm^3)	404.00
Mass	(g)	1108.40
Moisture Content	(%)	0.44
Wet Density	(kg/m^3)	2744
Dry Density	(kg/m^3)	2732

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**6**
Degrees:*	**--**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP592B
Project:	White Pine North	**Sample Number:**	WP003533M
Location:	Northern Michigan US	**Depth (m):**	984.42 - 964.90
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**381.70**
Stress σ_1 (MPa)	124.38
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	114.38
Pace Rate (kN/s)	1.25
Lithology	UZVA

Sample Measurements

Diameter	(mm)	62.51
Height	(mm)	117.03
Area	(cm^2)	30.69
Volume	(cm^3)	359.16
Mass	(g)	979.70
Moisture Content	(%)	0.38
Wet Density	(kg/m^3)	2728
Dry Density	(kg/m^3)	2717

BEFORE TEST

Failure Mode

Type: **Discrete**

Mode: **5**

Degrees:* **82**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

Sample does not meet ASTM length to diameter requirements.

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

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S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP598B
Project:	White Pine North	**Sample Number:**	WP004003M
Location:	Northern Michigan US	**Depth (m):**	911.71 - 912.19
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**684.70**
Stress σ_1 (MPa)	243.79
Stress σ_3 (MPa)	15.00
Stress σ (MPa)	228.79
Pace Rate (kN/s)	1.25
Lithology	UZVA

Sample Measurements

Diameter	(mm)	59.80
Height	(mm)	131.15
Area	(cm^2)	28.09
Volume	(cm^3)	368.35
Mass	(g)	1009.10
Moisture Content	(%)	0.26
Wet Density	(kg/m^3)	2740
Dry Density	(kg/m^3)	2732

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**3**
Degrees:*	**--**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP597B
Project:	White Pine North	**Sample Number:**	WP003871M
Location:	Northern Michigan US	**Depth (m):**	879.30 - 879.92
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**638.50**
Stress σ_1 (MPa)	222.47
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	212.47
Pace Rate (kN/s)	1.25
Lithology	WIDE

Sample Measurements

Diameter	(mm)	60.45
Height	(mm)	130.30
Area	(cm^2)	28.70
Volume	(cm^3)	373.96
Mass	(g)	1028.80
Moisture Content	(%)	0.34
Wet Density	(kg/m^3)	2751
Dry Density	(kg/m^3)	2742

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5**
Degrees:*	**15**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP597B
Project:	White Pine North	**Sample Number:**	WP003872M
Location:	Northern Michigan US	**Depth (m):**	879.92 - 880.50
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**534.20**
Stress σ_1 (MPa)	185.82
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	175.82
Pace Rate (kN/s)	1.25
Lithology	WIDE

Sample Measurements

Diameter	(mm)	60.50
Height	(mm)	130.38
Area	(cm^2)	28.75
Volume	(cm^3)	374.81
Mass	(g)	1023.20
Moisture Content	(%)	0.58
Wet Density	(kg/m^3)	2730
Dry Density	(kg/m^3)	2714

BEFORE TEST

Failure Mode

Type:	**Combined**
Mode:	**5**
Degrees:*	**25**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP597B
Project:	White Pine North	**Sample Number:**	WP003870M
Location:	Northern Michigan US	**Depth (m):**	878.74 - 879.30
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**463.20**
Stress σ_1 (MPa)	161.18
Stress σ_3 (MPa)	5.00
Stress σ (MPa)	156.18
Pace Rate (kN/s)	1.25
Lithology	WIDE

Sample Measurements

Diameter	(mm)	60.49
Height	(mm)	130.50
Area	(cm^2)	28.74
Volume	(cm^3)	375.03
Mass	(g)	1026.10
Moisture Content	(%)	0.47
Wet Density	(kg/m^3)	2736
Dry Density	(kg/m^3)	2723

BEFORE TEST

Failure Mode

Type: **Intact**

Mode: **5**

Degrees:* **11**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP599B
Project:	White Pine North	**Sample Number:**	WP003923M
Location:	Northern Michigan US	**Depth (m):**	766.20 - 766.70
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**494.40**
Stress σ_1 (MPa)	157.05
Stress σ_3 (MPa)	5.00
Stress σ (MPa)	152.05
Pace Rate (kN/s)	1.25
Lithology	WIDE

Sample Measurements

Diameter	(mm)	63.31
Height	(mm)	131.05
Area	(cm^2)	31.48
Volume	(cm^3)	412.55
Mass	(g)	1130.00
Moisture Content	(%)	0.49
Wet Density	(kg/m^3)	2739
Dry Density	(kg/m^3)	2726

BEFORE TEST

Failure Mode

Type: **Intact**

Mode: **5**

Degrees:* **25**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

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S. Mujib	May 8, 2025	D. Lim	June 23, 2025
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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP592B
Project:	White Pine North	**Sample Number:**	WP003552M
Location:	Northern Michigan US	**Depth (m):**	972.05 - 972.55
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**500.90**
Stress σ_1 (MPa)	162.02
Stress σ_3 (MPa)	5.00
Stress σ (MPa)	157.02
Pace Rate (kN/s)	1.25
Lithology	CCGL

Sample Measurements

Diameter	(mm)	62.74
Height	(mm)	129.93
Area	(cm^2)	30.92
Volume	(cm^3)	401.69
Mass	(g)	1086.50
Moisture Content	(%)	0.97
Wet Density	(kg/m^3)	2705
Dry Density	(kg/m^3)	2679

BEFORE TEST

Failure Mode

Type: **Intact**

Mode: **5**

Degrees:* **28**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

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B. Matheson	May 26, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP592B
Project:	White Pine North	**Sample Number:**	WP003553M (1)
Location:	Northern Michigan US	**Depth (m):**	972.55 - 973.05
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**499.40**
Stress σ_1 (MPa)	161.12
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	151.12
Pace Rate (kN/s)	1.25
Lithology	CCGL

Sample Measurements

Diameter	(mm)	62.82
Height	(mm)	130.38
Area	(cm^2)	30.99
Volume	(cm^3)	404.11
Mass	(g)	1110.80
Moisture Content	(%)	1.76
Wet Density	(kg/m^3)	2749
Dry Density	(kg/m^3)	2701

BEFORE TEST

Failure Mode

Type: **Intact**

Mode: **5**

Degrees:* **25**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

B. Matheson	May 26, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP592B
Project:	White Pine North	**Sample Number:**	WP003553M (2)
Location:	Northern Michigan US	**Depth (m):**	972.55 - 973.05
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**640.80**
Stress σ_1 (MPa)	207.14
Stress σ_3 (MPa)	15.00
Stress σ (MPa)	192.14
Pace Rate (kN/s)	1.25
Lithology	CCGL

Sample Measurements

Diameter	(mm)	62.76
Height	(mm)	130.48
Area	(cm^2)	30.94
Volume	(cm^3)	403.65
Mass	(g)	1086.20
Moisture Content	(%)	1.49
Wet Density	(kg/m^3)	2691
Dry Density	(kg/m^3)	2651

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5**
Degrees:*	**24**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

B. Matheson	May 26, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP594B
Project:	White Pine North	**Sample Number:**	WP003576M
Location:	Northern Michigan US	**Depth (m):**	897.86 - 898.47
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**748.70**
Stress σ_1 (MPa)	243.03
Stress σ_3 (MPa)	20.00
Stress σ (MPa)	223.03
Pace Rate (kN/s)	1.25
Lithology	CCGL

Sample Measurements

Diameter	(mm)	62.63
Height	(mm)	130.79
Area	(cm^2)	30.81
Volume	(cm^3)	402.93
Mass	(g)	1103.30
Moisture Content	(%)	0.13
Wet Density	(kg/m^3)	2738
Dry Density	(kg/m^3)	2735

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5**
Degrees:*	**14**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

B. Matheson	May 26, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP599B
Project:	White Pine North	**Sample Number:**	WP003945M
Location:	Northern Michigan US	**Depth (m):**	774.42 - 774.92
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**502.10**
Stress σ_1 (MPa)	158.85
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	148.85
Pace Rate (kN/s)	1.25
Lithology	CCGL

Sample Measurements

Diameter	(mm)	63.44
Height	(mm)	160.88
Area	(cm^2)	31.61
Volume	(cm^3)	508.53
Mass	(g)	1081.80
Moisture Content	(%)	0.20
Wet Density	(kg/m^3)	2127
Dry Density	(kg/m^3)	2123

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**6**
Degrees:*	**--**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

B. Matheson	May 26, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP601B
Project:	White Pine North	**Sample Number:**	WP004155M
Location:	Northern Michigan US	**Depth (m):**	978.43 - 979.47
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**552.90**
Stress σ_1 (MPa)	177.37
Stress σ_3 (MPa)	15.00
Stress σ (MPa)	162.37
Pace Rate (kN/s)	1.25
Lithology	CCGL

Sample Measurements

Diameter	(mm)	63.00
Height	(mm)	131.02
Area	(cm^2)	31.17
Volume	(cm^3)	408.42
Mass	(g)	1098.90
Moisture Content	(%)	0.19
Wet Density	(kg/m^3)	2691
Dry Density	(kg/m^3)	2685

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5**
Degrees:*	**27**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

B. Matheson	May 26, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP600B
Project:	White Pine North	**Sample Number:**	WP004087M
Location:	Northern Michigan US	**Depth (m):**	994.60 - 994.77
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**425.10**
Stress σ_1 (MPa)	137.11
Stress σ_3 (MPa)	10.00
Stress σ (MPa)	127.11
Pace Rate (kN/s)	1.25
Lithology	UTRA

Sample Measurements

Diameter	(mm)	62.83
Height	(mm)	130.92
Area	(cm^2)	31.00
Volume	(cm^3)	405.91
Mass	(g)	1109.60
Moisture Content	(%)	0.05
Wet Density	(kg/m^3)	2734
Dry Density	(kg/m^3)	2732

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5/6**
Degrees:*	**36**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

B. Matheson	May 26, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

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Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

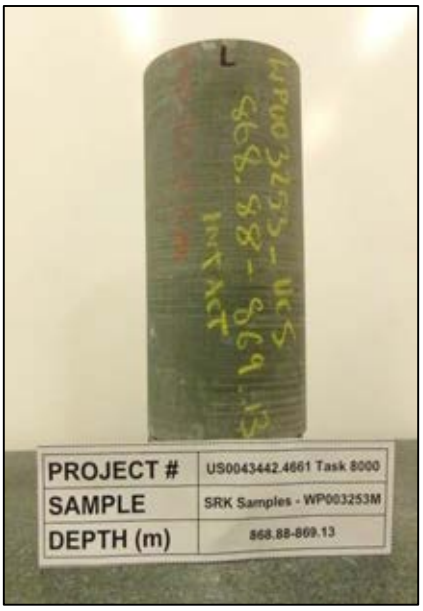
Project No.:	US0043442.6551 Task 8000	**Borehole:**	SRK Samples
Project:	White Pine North	**Sample Number:**	WP003253M
Location:	Northern Michigan US	**Depth (m):**	868.88-869.13
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**903.10**
Stress σ_1 (MPa)	290.63
Stress σ_3 (MPa)	15.00
Stress σ (MPa)	275.63
Pace Rate (kN/s)	1.25
Lithology	BMAS

Sample Measurements

Diameter	(mm)	62.90
Height	(mm)	129.07
Area	(cm^2)	31.07
Volume	(cm^3)	401.07
Mass	(g)	1096.70
Moisture Content	(%)	0.27
Wet Density	(kg/m^3)	2734
Dry Density	(kg/m^3)	2727



BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5/6**
Degrees:*	**22**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	SRK Samples
Project:	White Pine North	**Sample Number:**	WP003406M
Location:	Northern Michigan US	**Depth (m):**	932.33-932.58
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**1071.00**
Stress σ_1 (MPa)	344.99
Stress σ_3 (MPa)	30.00
Stress σ (MPa)	314.99
Pace Rate (kN/s)	1.25
Lithology	UPSA

Sample Measurements

Diameter	(mm)	62.87
Height	(mm)	127.45
Area	(cm^2)	31.04
Volume	(cm^3)	395.65
Mass	(g)	1065.60
Moisture Content	(%)	0.32
Wet Density	(kg/m^3)	2693
Dry Density	(kg/m^3)	2685

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5**
Degrees:*	**24**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	SRK Samples
Project:	White Pine North	**Sample Number:**	WP003540M
Location:	Northern Michigan US	**Depth (m):**	967.06-967.29
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**812.90**
Stress σ_1 (MPa)	264.03
Stress σ_3 (MPa)	20.00
Stress σ (MPa)	244.03
Pace Rate (kN/s)	1.25
Lithology	UPSA

Sample Measurements

Diameter	(mm)	62.61
Height	(mm)	127.88
Area	(cm^2)	30.79
Volume	(cm^3)	393.71
Mass	(g)	1064.10
Moisture Content	(%)	0.44
Wet Density	(kg/m^3)	2703
Dry Density	(kg/m^3)	2691

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5/6**
Degrees:*	**22**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

σ1 = total failure stress (MPa)

σ3= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	SRK Samples
Project:	White Pine North	**Sample Number:**	WP003750M
Location:	Northern Michigan US	**Depth (m):**	888.69-888.88
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**804.80**
Stress σ_1 (MPa)	255.33
Stress σ_3 (MPa)	15.00
Stress σ (MPa)	240.33
Pace Rate (kN/s)	1.25
Lithology	UPSA

Sample Measurements

Diameter	(mm)	63.35
Height	(mm)	128.28
Area	(cm^2)	31.52
Volume	(cm^3)	404.34
Mass	(g)	1092.00
Moisture Content	(%)	0.39
Wet Density	(kg/m^3)	2701
Dry Density	(kg/m^3)	2690

BEFORE TEST

Failure Mode

Type: **Intact**

Mode: **5/3**

Degrees:* **25**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Laboratory Determination of Triaxial Compressive Strength of Undrained Rock Core Specimens (Method A)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	SRK Samples
Project:	White Pine North	**Sample Number:**	WP003743M
Location:	Northern Michigan US	**Depth (m):**	886.00-886.24
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**675.80**
Stress σ_1 (MPa)	217.55
Stress σ_3 (MPa)	5.00
Stress σ (MPa)	212.55
Pace Rate (kN/s)	1.25
Lithology	UZVA

Sample Measurements

Diameter	(mm)	62.89
Height	(mm)	127.06
Area	(cm^2)	31.06
Volume	(cm^3)	394.70
Mass	(g)	1091.40
Moisture Content	(%)	0.28
Wet Density	(kg/m^3)	2765
Dry Density	(kg/m^3)	2758

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5/6**
Degrees:*	**25**

** Degrees measured with respect to core axis*

Notes

- Water content as received

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

Sample does not meet ASTM length to diameter requirements.

σ = differential stress (MPa)

$\sigma 1$ = total failure stress (MPa)

$\sigma 3$= confining stress (MPa)

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

S. Mujib	May 8, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Laboratory Determination of Uniaxial Compressive Strength of Intact Rock Core Specimens

Summary of Test Results

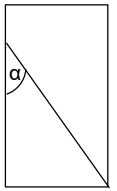
ASTM D7012 Method C

Project No.: US0043442.6551 Task 8000
Project: White Pine North
Location: Northern Michigan US
Client: White Pine LLC
Lab ID No: B25-089

Failure Modes
(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

* Wet density based on as received moisture

Note: Alpha angle, α, measured relative to the core axis



No.	Borehole #	Sample #	Depth (m)	Dia (mm)	Ht (mm)	Area (cm²)	Volume (cm³)	Mass (g)	Wet Density (kg/m³)	Moisture (%)	Dry Density (kg/m³)	Maximum Load (kN)	Stress σ_u (MPa)	Rock Type	Failure Criteria		
															Type	Mode	(deg)
1	WP594B	WP003567M	894.36 - 894.70	63.19	130.97	31.36	410.73	1113.70	2712	0.31	2703	349.70	111.51	UPSA	Intact	6	--
2	WP600B	WP004089M	995.07 - 995.53	62.82	131.08	30.99	406.28	1116.80	2749	0.40	2738	238.30	76.88	UPSA	Discrete	5	11
3	WP594B	WP003565M	893.38 - 893.91	63.25	130.98	31.42	411.54	1112.10	2702	0.29	2694	525.30	167.18	UPSA	Intact	3/6	--
4	WP594B	WP003575M	897.43 - 897.86	62.56	130.17	30.74	400.12	1057.70	2643	0.68	2626	183.00	59.53	LWSA	Intact	6	--
5	WP597B	VP003872M (2	879.92 - 880.00	60.49	130.25	28.74	374.31	1023.20	2734	0.45	2721	399.50	139.01	WIDE	Intact	3	--
6	SRK Samples	WP003257M	870.41-870.60	63.08	127.74	31.25	399.21	1075.80	2695	0.24	2688	462.10	147.86	UPSA	Intact	5	12

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	June 23, 2025	D. Lim	January 0, 1900
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Uniaxial Compressive Strength of Intact Rock Core Specimens (Method C)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP594B
Project:	White Pine North	**Sample Number:**	WP003567M
Location:	Northern Michigan US	**Depth (m):**	894.36 - 894.70
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**349.70**
Stress σ_u (MPa)	**111.5**
Pace Rate (kN/s)	1.25
Lithology	UPSA

Sample Measurements

Diameter	(mm)	63.19
Height	(mm)	130.97
Area	(cm²)	31.36
Volume	(cm³)	410.73
Mass	(g)	1113.70
Moisture Content	(%)	0.31
Wet Density	(kg/m³)	2712
Dry Density	(kg/m³)	2703

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**6**
α angle:	**--**

* Degrees measured with respect to core axis.

The impact of any pre-existing feature on the test results will be noted in the comments, if applicable.

Notes

- Water content as received
- Wet density based on as received moisture

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 14, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Uniaxial Compressive Strength of Intact Rock Core Specimens (Method C)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP600B
Project:	White Pine North	**Sample Number:**	WP004089M
Location:	Northern Michigan US	**Depth (m):**	995.07 - 995.53
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**238.30**
Stress σ_u (MPa)	**76.9**
Pace Rate (kN/s)	1.25
Lithology	UPSA

Sample Measurements

Diameter	(mm)	62.82
Height	(mm)	131.08
Area	(cm²)	30.99
Volume	(cm³)	406.28
Mass	(g)	1116.80
Moisture Content	(%)	0.40
Wet Density	(kg/m³)	2749
Dry Density	(kg/m³)	2738

BEFORE TEST

Failure Mode

Type: **Discrete**

Mode: **5**

α angle: **11**

* Degrees measured with respect to core axis.

The impact of any pre-existing feature on the test results will be noted in the comments, if applicable.

Notes

- Water content as received
- Wet density based on as received moisture

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 14, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Uniaxial Compressive Strength of Intact Rock Core Specimens (Method C)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP594B
Project:	White Pine North	**Sample Number:**	WP003565M
Location:	Northern Michigan US	**Depth (m):**	893.38 - 893.91
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**525.30**
Stress σ_u (MPa)	**167.2**
Pace Rate (kN/s)	1.25
Lithology	UPSA

Sample Measurements

Diameter	(mm)	63.25
Height	(mm)	130.98
Area	(cm²)	31.42
Volume	(cm³)	411.54
Mass	(g)	1112.10
Moisture Content	(%)	0.29
Wet Density	(kg/m³)	2702
Dry Density	(kg/m³)	2694

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**3/6**
α angle:	**--**

* Degrees measured with respect to core axis.

The impact of any pre-existing feature on the test results will be noted in the comments, if applicable.

Notes

- Water content as received
- Wet density based on as received moisture

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

Comments

AFTER TEST

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 14, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Uniaxial Compressive Strength of Intact Rock Core Specimens (Method C)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP594B
Project:	White Pine North	**Sample Number:**	WP003575M
Location:	Northern Michigan US	**Depth (m):**	897.43 - 897.86
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**183.00**
Stress σ_u (MPa)	**59.5**
Pace Rate (kN/s)	1.25
Lithology	LWSA

Sample Measurements

Diameter	(mm)	62.56
Height	(mm)	130.17
Area	(cm²)	30.74
Volume	(cm³)	400.12
Mass	(g)	1057.70
Moisture Content	(%)	0.68
Wet Density	(kg/m³)	2643
Dry Density	(kg/m³)	2626

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**6**
α angle:	**--**

* Degrees measured with respect to core axis.

The impact of any pre-existing feature on the test results will be noted in the comments, if applicable.

Notes

- Water content as received
- Wet density based on as received moisture

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 14, 2025	D. Lim	April 28, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Uniaxial Compressive Strength of Intact Rock Core Specimens (Method C)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	WP597B
Project:	White Pine North	**Sample Number:**	WP003872M (2)
Location:	Northern Michigan US	**Depth (m):**	879.92 - 880.00
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**399.50**
Stress σ_u (MPa)	**139.0**
Pace Rate (kN/s)	1.25
Lithology	WIDE

Sample Measurements

Diameter	(mm)	60.49
Height	(mm)	130.25
Area	(cm²)	28.74
Volume	(cm³)	374.31
Mass	(g)	1023.20
Moisture Content	(%)	0.45
Wet Density	(kg/m³)	2734
Dry Density	(kg/m³)	2721

BEFORE TEST

Failure Mode

Type: **Intact**

Mode: **3**

α angle: **--**

* Degrees measured with respect to core axis.

The impact of any pre-existing feature on the test results will be noted in the comments, if applicable.

Notes

- Water content as received
- Wet density based on as received moisture

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

Comments

AFTER TEST

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

G. Patton	April 29, 2025	D. Lim	May 1, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Uniaxial Compressive Strength of Intact Rock Core Specimens (Method C)

ASTM D7012

Project No.:	US0043442.6551 Task 8000	**Borehole:**	SRK Samples
Project:	White Pine North	**Sample Number:**	WP003257M
Location:	Northern Michigan US	**Depth (m):**	870.41-870.60
Client:	White Pine LLC	**Lab ID No:**	B25-089

Testing Results

Max Load (kN)	**462.10**
Stress σ_u (MPa)	**147.9**
Pace Rate (kN/s)	1.25
Lithology	UPSA

Sample Measurements

Diameter	(mm)	63.08
Height	(mm)	127.74
Area	(cm²)	31.25
Volume	(cm³)	399.21
Mass	(g)	1075.80
Moisture Content	(%)	0.24
Wet Density	(kg/m³)	2695
Dry Density	(kg/m³)	2688

BEFORE TEST

Failure Mode

Type:	**Intact**
Mode:	**5**
α angle:	**12**

* Degrees measured with respect to core axis.

The impact of any pre-existing feature on the test results will be noted in the comments, if applicable.

Notes

- Water content as received
- Wet density based on as received moisture

Mode:

(1) Simple Extension
(2) Multiple Extension
(3) Multiple Fracturing
(4) Multiple Shear
(5) Single Shear
(6) Spalling
(7) Other

AFTER TEST

Comments

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

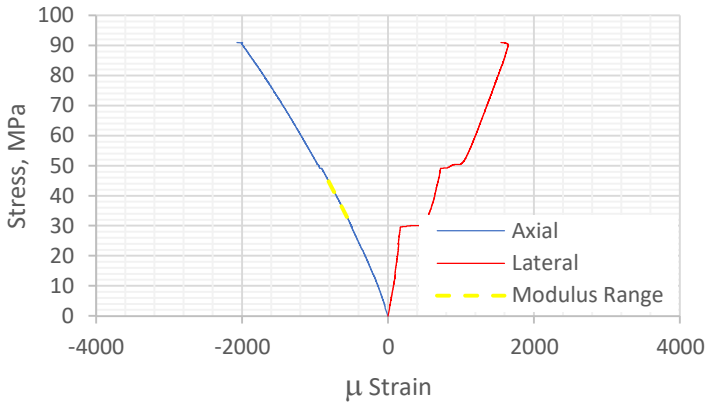
S. Mujib	June 17, 2025	D. Lim	June 23, 2025
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP590B	Sample ID No.:	WP003247M
Project:	White Pine North	Depth (m):	866.10-866.60		
Location:	Northern Michigan US	Rock Type:	RAGR		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	62.87	(mm)
Length	118.78	(mm)
Mass	1012.00	(g)
Area	31.04	(cm^2)
Volume	368.74	(cm^3)
ρ_{wet}	2744	(kg/m^3)
ρ_{dry}	2731	(kg/m^3)
W%	0.49	

Test Summary

Peak Stress	91.0	(MPa)
Young's Modulus	47.0	(GPa)
Poisson's Ratio	Invalid	
Failure Mode:	Multiple Fracturing	
Failure Type:	Intact	
Foliation Angle:	N/A	

Poisson's ratio lies outside range for normal materials (between 0 and +0.5)

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.
- Sample does not conform to ASTM length to diameter requirements and results may differ from those obtained from test specimens that meet requirements.

The test data given herein pertain to the sample provided only. This report constitutes a testing service

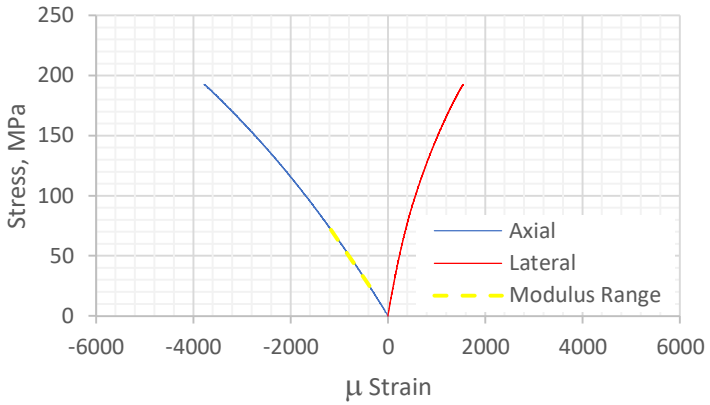
B. Matheson	May 5, 2025	G.Patton	May 14, 2025
TESTED	DATE	CHECKED	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP591B	Sample ID No.:	WP003368M
Project:	White Pine North	Depth (m):	933.51-933.93		
Location:	Northern Michigan US	Rock Type:	RAGR		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	62.07	(mm)
Length	130.36	(mm)
Mass	1085.60	(g)
Area	30.26	(cm^2)
Volume	394.46	(cm^3)
ρ_{wet}	2752	(kg/m^3)
ρ_{dry}	2746	(kg/m^3)
W%	0.22	

Test Summary

Peak Stress	192.6	(MPa)
Young's Modulus	59.0	(GPa)
Poisson's Ratio	0.32	
Failure Mode:	Multiple Extension / Multiple Fracturing	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

The test data given herein pertain to the sample provided only. This report constitutes a testing service

G. Patton	April 29, 2025	D. Lim	May 1, 2025
TESTED	DATE	CHECKED	DATE

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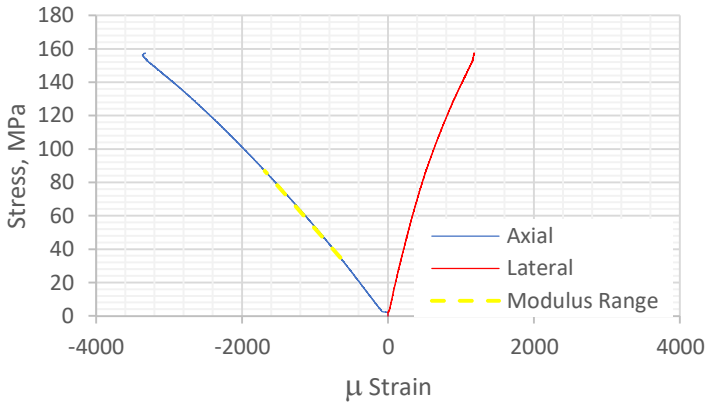
Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP591B	Sample ID No.:	WP003380M
Project:	White Pine North	Depth (m):	937.57-938.08		
Location:	Northern Michigan US	Rock Type:	UPSA		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	62.41	(mm)
Length	130.12	(mm)
Mass	1088.00	(g)
Area	30.59	(cm^2)
Volume	398.05	(cm^3)
ρ_{wet}	2733	(kg/m^3)
ρ_{dry}	2723	(kg/m^3)
W%	0.38	

Test Summary

Peak Stress	157.5	(MPa)
Young's Modulus	50.1	(GPa)
Poisson's Ratio	0.32	
Failure Mode:	Multiple Fracturing	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

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G. Patton	April 15, 2025	D. Lim	April 28, 2025
TESTED	DATE	CHECKED	DATE

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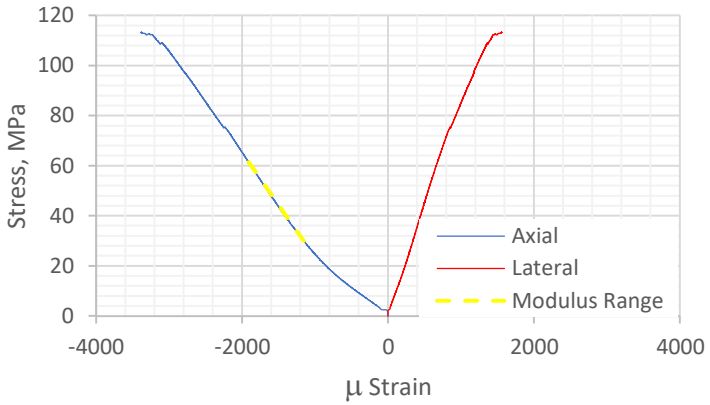
Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP591B	Sample ID No.:	WP003390M
Project:	White Pine North	Depth (m):	941.29-941.75		
Location:	Northern Michigan US	Rock Type:	LWSA		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	62.78	(mm)
Length	130.22	(mm)
Mass	1082.80	(g)
Area	30.96	(cm^2)
Volume	403.10	(cm^3)
ρ_{wet}	2686	(kg/m^3)
ρ_{dry}	2661	(kg/m^3)
W%	0.94	

Test Summary

Peak Stress	113.4	(MPa)
Young's Modulus	42.0	(GPa)
Poisson's Ratio	0.45	
Failure Mode:	Spalling / Single Shear 17°	
Failure Type:	Combined	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

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G. Patton	April 15, 2025	D. Lim	April 28, 2025
TESTED	DATE	CHECKED	DATE

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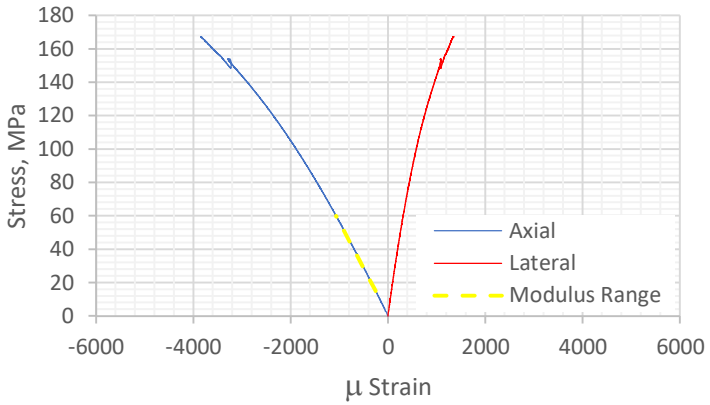
Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP594B	Sample ID No.:	WP003561M
Project:	White Pine North	Depth (m):	892.20-892.77		
Location:	Northern Michigan US	Rock Type:	BMAS		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	63.27	(mm)
Length	132.73	(mm)
Mass	1127.00	(g)
Area	31.44	(cm^2)
Volume	417.31	(cm^3)
ρ_{wet}	2701	(kg/m^3)
ρ_{dry}	2694	(kg/m^3)
W%	0.26	

Test Summary

Peak Stress	167.3	(MPa)
Young's Modulus	55.4	(GPa)
Poisson's Ratio	0.30	
Failure Mode:	Simple Extension / Multiple Fracturing	
Failure Type:	Combined	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

The test data given herein pertain to the sample provided only. This report constitutes a testing service

G. Patton	April 29, 2025	D. Lim	May 1, 2025
TESTED	DATE	CHECKED	DATE

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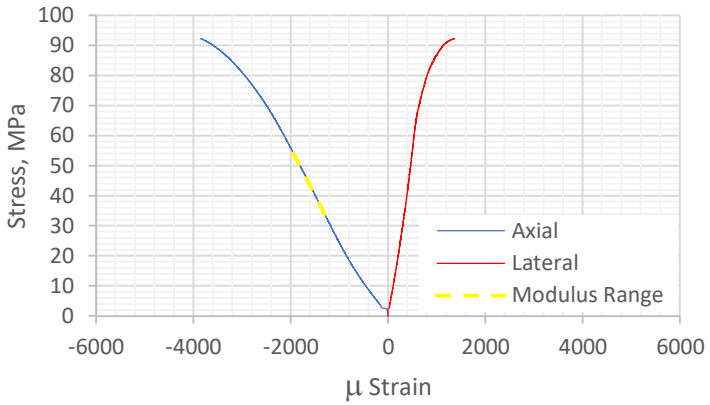
Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP594B	Sample ID No.:	WP003577M
Project:	White Pine North	Depth (m):	898.47-899.00		
Location:	Northern Michigan US	Rock Type:	CCGL		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	62.76	(mm)
Length	130.36	(mm)
Mass	1075.40	(g)
Area	30.94	(cm^2)
Volume	403.27	(cm^3)
ρ_{wet}	2667	(kg/m^3)
ρ_{dry}	2617	(kg/m^3)
W%	1.88	

Test Summary

Peak Stress	92.4	(MPa)
Young's Modulus	31.6	(GPa)
Poisson's Ratio	0.25	
Failure Mode:	Spalling / Single Shear 13°	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

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G. Patton	May 20, 2025	D. Lim	May 23, 2025
TESTED	DATE	CHECKED	DATE

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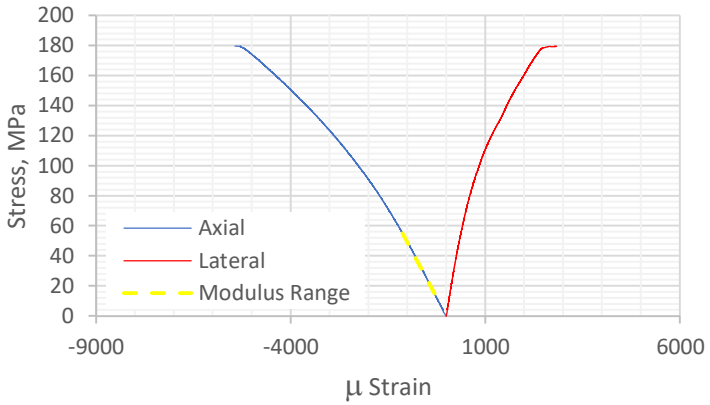
Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

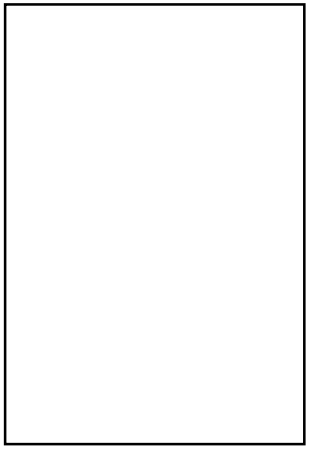
Project No.:	US0043442.4661 Task 8000	Borehole:	WP593B	Sample ID No.:	WP003689M
Project:	White Pine North	Depth (m):	939.37-939.89		
Location:	Northern Michigan US	Rock Type:	WIDE		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A



AFTER TESTING Side B

Sample Properties

Diameter	63.32	(mm)
Length	129.96	(mm)
Mass	1123.00	(g)
Area	31.49	(cm^2)
Volume	409.24	(cm^3)
ρ_{wet}	2744	(kg/m^3)
ρ_{dry}	2734	(kg/m^3)
W%	0.38	

Test Summary

Peak Stress	179.7	(MPa)
Young's Modulus	49.1	(GPa)
Poisson's Ratio	0.34	
Failure Mode:	Multiple Fracturing	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

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G. Patton	April 29, 2025	D. Lim	May 1, 2025
TESTED	DATE	CHECKED	DATE

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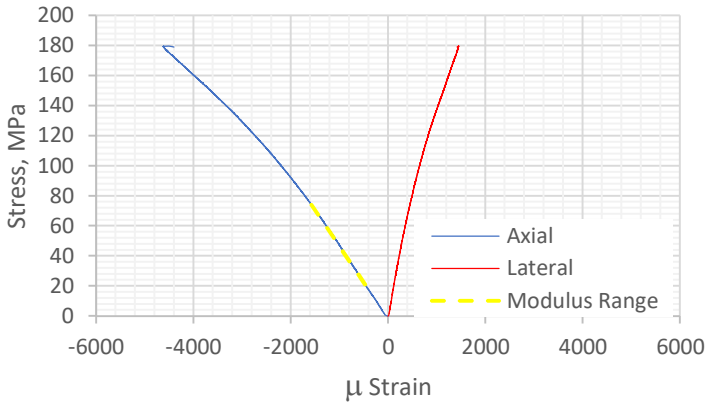
Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP593B	Sample ID No.:	WP003705M
Project:	White Pine North	Depth (m):	945.55-945.89		
Location:	Northern Michigan US	Rock Type:	DGMA		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	63.05	(mm)
Length	130.76	(mm)
Mass	1122.60	(g)
Area	31.22	(cm^2)
Volume	408.26	(cm^3)
ρ_{wet}	2750	(kg/m^3)
ρ_{dry}	2741	(kg/m^3)
W%	0.31	

Test Summary

Peak Stress	179.6	(MPa)
Young's Modulus	48.4	(GPa)
Poisson's Ratio	0.30	
Failure Mode:	Multiple Fracturing	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

The test data given herein pertain to the sample provided only. This report constitutes a testing service

G. Patton	April 24, 2025	D. Lim	April 28, 2025
TESTED	DATE	CHECKED	DATE

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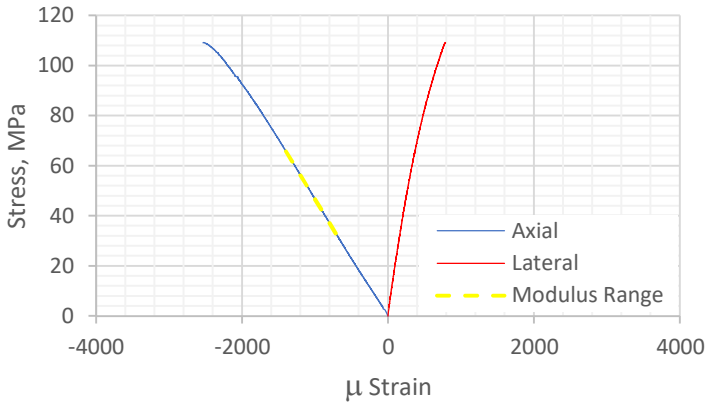
Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP593B	Sample ID No.:	WP003711M
Project:	White Pine North	Depth (m):	948.11 - 948.61		
Location:	Northern Michigan US	Rock Type:	CCGL		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	63.24	(mm)
Length	130.31	(mm)
Mass	1174.20	(g)
Area	31.41	(cm^2)
Volume	409.31	(cm^3)
ρ_{wet}	2869	(kg/m^3)
ρ_{dry}	2849	(kg/m^3)
W%	0.68	

Test Summary

Peak Stress	109.3	(MPa)
Young's Modulus	47.9	(GPa)
Poisson's Ratio	0.30	
Failure Mode:	Multiple Fracturing	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

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G. Patton	May 20, 2025	D. Lim	May 23, 2025
TESTED	DATE	CHECKED	DATE

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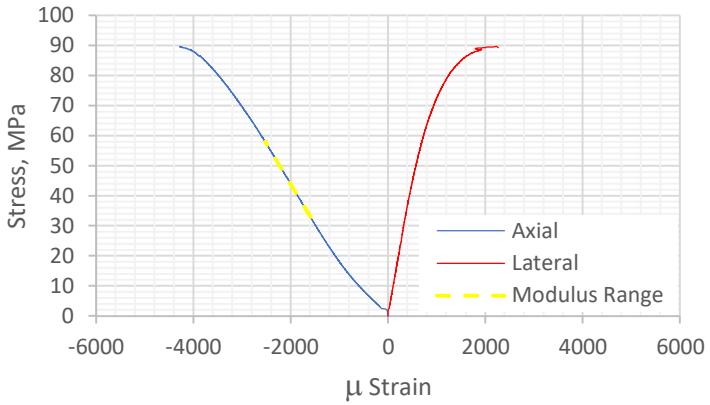
Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP596B	Sample ID No.:	WP003761M
Project:	White Pine North	Depth (m):	892.14-892.64		
Location:	Northern Michigan US	Rock Type:	CCGL		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	65.17	(mm)
Length	130.83	(mm)
Mass	1088.80	(g)
Area	33.36	(cm^2)
Volume	436.41	(cm^3)
ρ_{wet}	2495	(kg/m^3)
ρ_{dry}	2448	(kg/m^3)
W%	1.93	

Test Summary

Peak Stress	89.7	(MPa)
Young's Modulus	27.0	(GPa)
Poisson's Ratio	0.37	
Failure Mode:	Multiple Fracturing / Single Shear 23°	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

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G. Patton	May 20, 2025	D. Lim	May 23, 2025
TESTED	DATE	CHECKED	DATE

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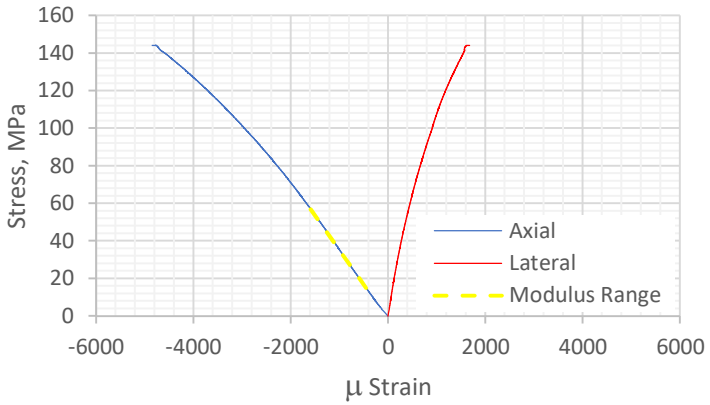
Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP597B	Sample ID No.:	WP003874M
Project:	White Pine North	Depth (m):	881.00-881.50		
Location:	Northern Michigan US	Rock Type:	UZVA		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	60.55	(mm)
Length	131.18	(mm)
Mass	1038.50	(g)
Area	28.80	(cm^2)
Volume	377.73	(cm^3)
ρ_{wet}	2749	(kg/m^3)
ρ_{dry}	2737	(kg/m^3)
W%	0.43	

Test Summary

Peak Stress	144.2	(MPa)
Young's Modulus	37.0	(GPa)
Poisson's Ratio	0.30	
Failure Mode:	Multiple Extension / Single Shear 23°	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

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G. Patton	April 29, 2025	D. Lim	May 1, 2025
TESTED	DATE	CHECKED	DATE

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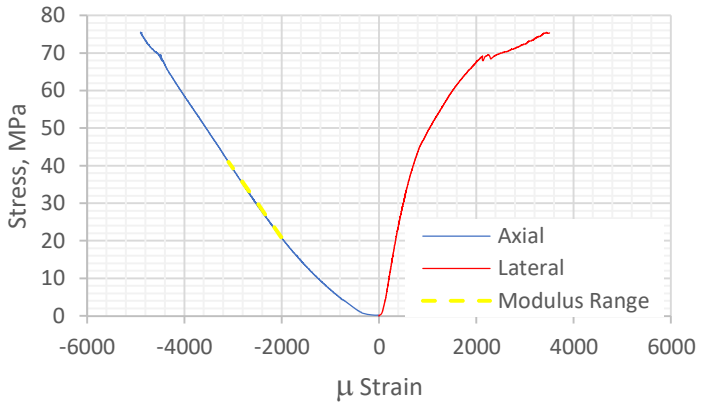
Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP597B	Sample ID No.:	WP003883M
Project:	White Pine North	Depth (m):	884.50-885.17		
Location:	Northern Michigan US	Rock Type:	UPSA		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	61.06	(mm)
Length	131.29	(mm)
Mass	1030.40	(g)
Area	29.28	(cm^2)
Volume	384.45	(cm^3)
ρ_{wet}	2680	(kg/m^3)
ρ_{dry}	2665	(kg/m^3)
W%	0.58	

Test Summary

Peak Stress	75.5	(MPa)
Young's Modulus	18.5	(GPa)
Poisson's Ratio	0.34	
Failure Mode:	Spalling / Simple Extension	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

The test data given herein pertain to the sample provided only. This report constitutes a testing service

G. Patton	April 15, 2025	D. Lim	April 28, 2025
TESTED	DATE	CHECKED	DATE

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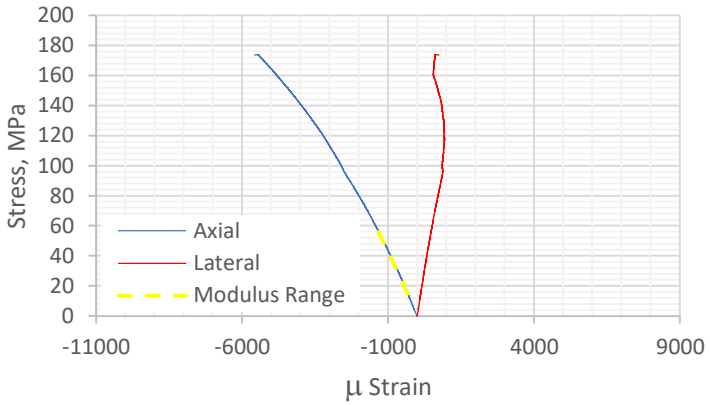
Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP597B	Sample ID No.:	WP003887M
Project:	White Pine North	Depth (m):	886.26-886.71		
Location:	Northern Michigan US	Rock Type:	RMAS		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	60.47	(mm)
Length	130.10	(mm)
Mass	1027.60	(g)
Area	28.72	(cm^2)
Volume	373.63	(cm^3)
ρ_{wet}	2750	(kg/m^3)
ρ_{dry}	2737	(kg/m^3)
W%	0.50	

Test Summary

Peak Stress	174.0	(MPa)
Young's Modulus	40.8	(GPa)
Poisson's Ratio	0.36	
Failure Mode:	Multiple Extension / Single Shear 22°	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

The test data given herein pertain to the sample provided only. This report constitutes a testing service

G. Patton	April 24, 2025	D. Lim	April 28, 2025
TESTED	DATE	CHECKED	DATE

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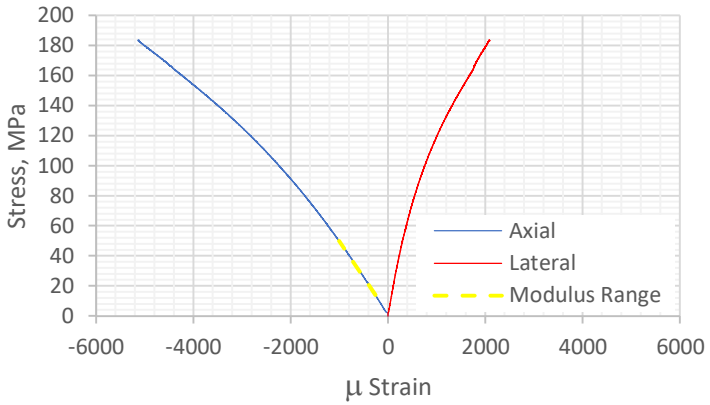
Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP599B	Sample ID No.:	WP003927M
Project:	White Pine North	Depth (m):	769.03-768.47		
Location:	Northern Michigan US	Rock Type:	UZVA		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	63.32	(mm)
Length	130.36	(mm)
Mass	1129.20	(g)
Area	31.49	(cm^2)
Volume	410.50	(cm^3)
ρ_{wet}	2751	(kg/m^3)
ρ_{dry}	2742	(kg/m^3)
W%	0.34	

Test Summary

Peak Stress	183.7	(MPa)
Young's Modulus	48.3	(GPa)
Poisson's Ratio	0.30	
Failure Mode:	Multiple Fracturing	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

The test data given herein pertain to the sample provided only. This report constitutes a testing service

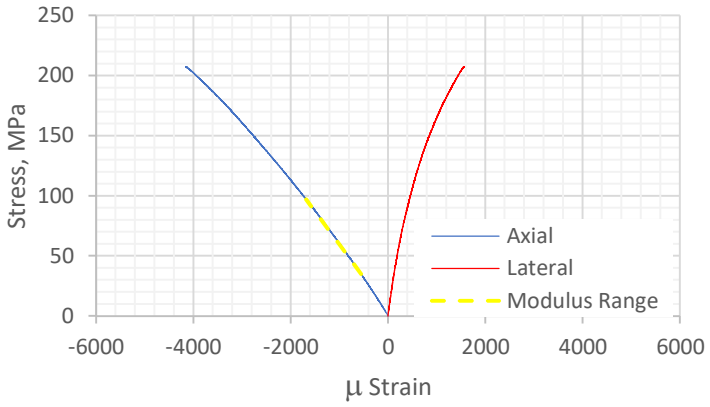
G. Patton	April 29, 2025	D. Lim	May 1, 2025
TESTED	DATE	CHECKED	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP599B	Sample ID No.:	WP003940M
Project:	White Pine North	Depth (m):	772.61-772.97		
Location:	Northern Michigan US	Rock Type:	LTRA		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	63.41	(mm)
Length	130.43	(mm)
Mass	1129.60	(g)
Area	31.58	(cm^2)
Volume	411.89	(cm^3)
ρ_{wet}	2742	(kg/m^3)
ρ_{dry}	2734	(kg/m^3)
W%	0.30	

Test Summary

Peak Stress	207.5	(MPa)
Young's Modulus	55.1	(GPa)
Poisson's Ratio	0.28	
Failure Mode:	Multiple Fracturing	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

The test data given herein pertain to the sample provided only. This report constitutes a testing service

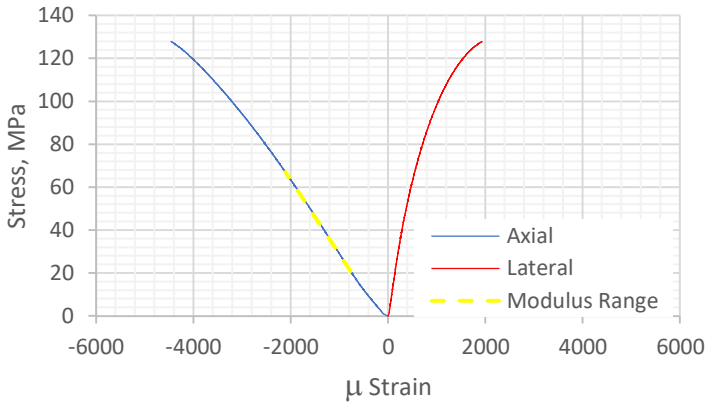
G. Patton	April 24, 2025	D. Lim	April 28, 2025
TESTED	DATE	CHECKED	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP599B	Sample ID No.:	WP003943M
Project:	White Pine North	Depth (m):	773.60-773.92		
Location:	Northern Michigan US	Rock Type:	LWSA		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	63.31	(mm)
Length	130.02	(mm)
Mass	1101.10	(g)
Area	31.48	(cm^2)
Volume	409.30	(cm^3)
ρ_{wet}	2690	(kg/m^3)
ρ_{dry}	2660	(kg/m^3)
W%	1.13	

Test Summary

Peak Stress	127.9	(MPa)
Young's Modulus	34.6	(GPa)
Poisson's Ratio	0.30	
Failure Mode:	Conical / Spalling	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

The test data given herein pertain to the sample provided only. This report constitutes a testing service

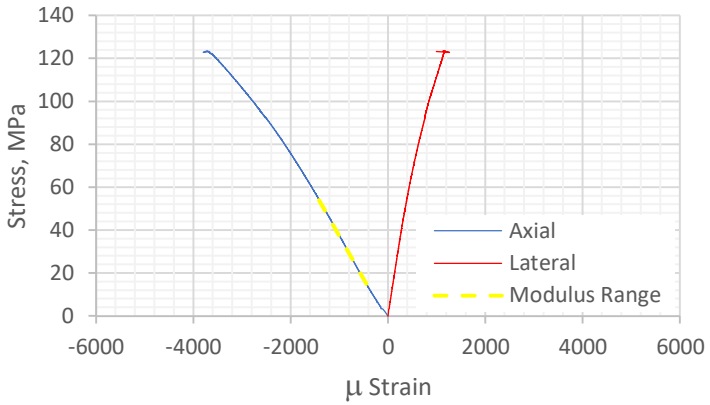
G. Patton	April 15, 2025	D. Lim	April 28, 2025
TESTED	DATE	CHECKED	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

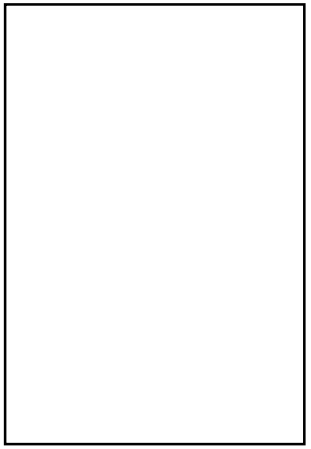
Project No.:	US0043442.4661 Task 8000	Borehole:	WP598B	Sample ID No.:	WP004000M
Project:	White Pine North	Depth (m):	910.39-910.79		
Location:	Northern Michigan US	Rock Type:	WIDE		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A



AFTER TESTING Side B

Sample Properties

Diameter	59.04	(mm)
Length	131.07	(mm)
Mass	983.50	(g)
Area	27.38	(cm^2)
Volume	358.83	(cm^3)
ρ_{wet}	2741	(kg/m^3)
ρ_{dry}	2728	(kg/m^3)
W%	0.46	

Test Summary

Peak Stress	123.5	(MPa)
Young's Modulus	40.6	(GPa)
Poisson's Ratio	0.30	
Failure Mode:	Multiple Fracturing	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

The test data given herein pertain to the sample provided only. This report constitutes a testing service

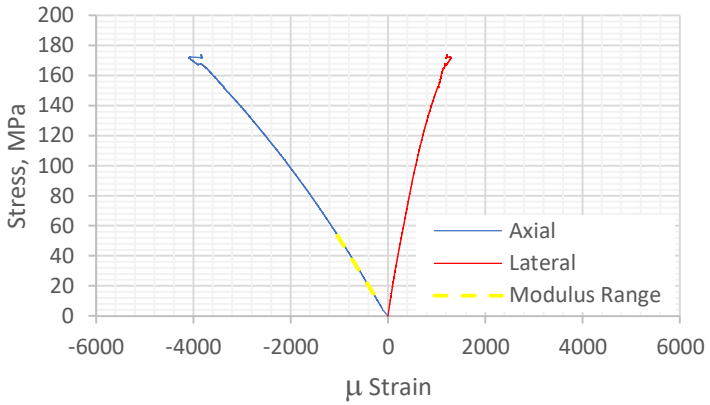
G. Patton	April 29, 2025	D. Lim	May 1, 2025
TESTED	DATE	CHECKED	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP598B	Sample ID No.:	WP004005M
Project:	White Pine North	Depth (m):	912.49-912.87		
Location:	Northern Michigan US	Rock Type:	BMAS		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	56.44	(mm)
Length	114.85	(mm)
Mass	788.30	(g)
Area	25.02	(cm^2)
Volume	287.34	(cm^3)
ρ_{wet}	2743	(kg/m^3)
ρ_{dry}	2742	(kg/m^3)
W%	0.04	

Test Summary

Peak Stress	174.1	(MPa)
Young's Modulus	52.5	(GPa)
Poisson's Ratio	0.30	
Failure Mode:	Multiple Fracturing	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

The test data given herein pertain to the sample provided only. This report constitutes a testing service

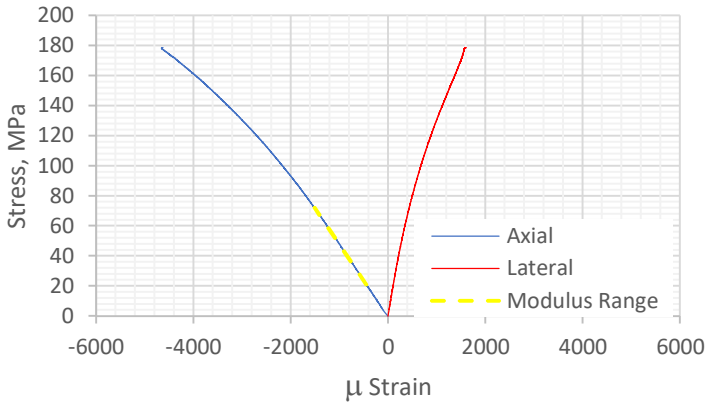
G. Patton	April 29, 2025	D. Lim	May 1, 2025
TESTED	DATE	CHECKED	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression

ASTM D7012 - Method D

Project No.:	US0043442.4661 Task 8000	Borehole:	WP598B	Sample ID No.:	WP004017M
Project:	White Pine North	Depth (m):	916.87-917.12		
Location:	Northern Michigan US	Rock Type:	RMAS		
Client:	White Pine LLC	Lab ID No.	B25-089		



BEFORE TESTING Side A

BEFORE TESTING Side B

AFTER TESTING Side A

AFTER TESTING Side B

Sample Properties

Diameter	63.22	(mm)
Length	129.18	(mm)
Mass	111.30	(g)
Area	31.39	(cm^2)
Volume	405.50	(cm^3)
ρ_{wet}	274	(kg/m^3)
ρ_{dry}	273	(kg/m^3)
W%	0.42	

Test Summary

Peak Stress	178.6	(MPa)
Young's Modulus	48.1	(GPa)
Poisson's Ratio	0.31	
Failure Mode:	Multiple Fracturing	
Failure Type:	Intact	
Foliation Angle:	N/A	

Notes

- Modulus data is obtained from the average of axially opposed strain gauges.
- Young's Modulus calculated using the average modulus method over the linear portion of axial strain data (highlighted in yellow on stress-strain plot). Poisson's Ratio is also calculated over the same range.
- Operating range of compression machine 13 - 2200kN, data below 13kN is truncated and excluded from calculations.

The test data given herein pertain to the sample provided only. This report constitutes a testing service

G. Patton	April 24, 2025	D. Lim	April 28, 2025
TESTED	DATE	CHECKED	DATE

WSP Canada
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816

APPENDIX B:

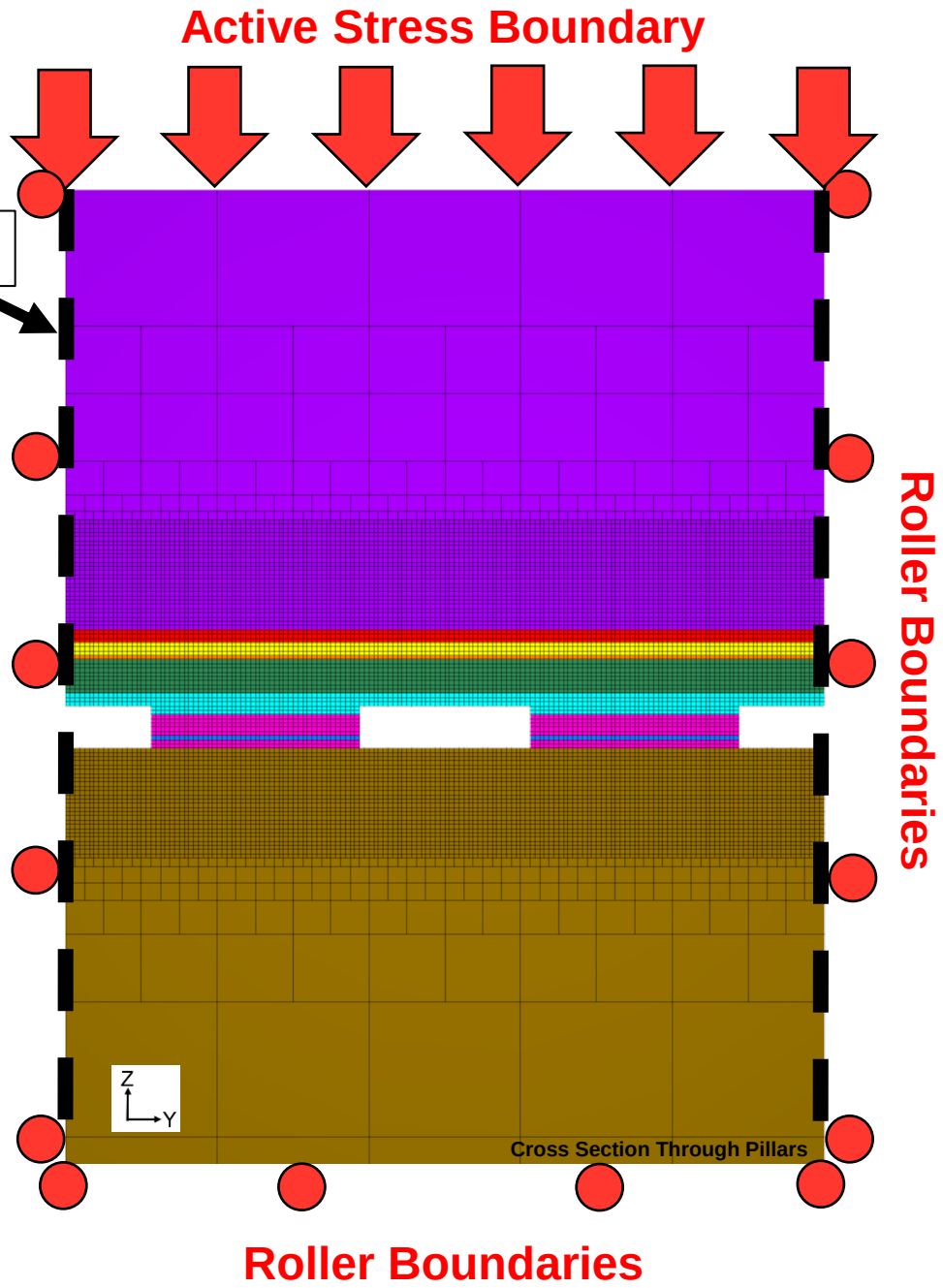
FLAC3D Model Construction

APPENDIX B

FLAC3D NUMERICAL STRESS MODELS

Boundary Conditions

- **Active Stress Boundary** on top to maintain the in-situ overburden stress in the model.
  - NOTE: the Panel-Scale model has a free boundary on top, as material is modeled to the ground surface.
- **Roller Boundary** (Reflective Boundary) on the sides and the bottom to prevent lateral movement; vertical movement is permitted.
  - The use of a reflective boundary allows for lines of symmetry to be used in the model
  - Reflective boundaries allow for a smaller window of the mine design to be modeled while considering repeating geometries

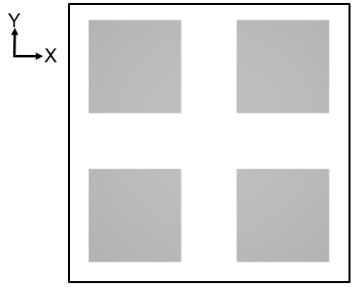


Stress Initialization

- **Full tributary loading is assumed**
- **Stresses in the FLAC3D models are initialized in the material based on the depth of the excavation modeled and the specified horizontal stress ratios (kH and kh)**
  - kH = major horizontal stress ratio
  - kh = minor horizontal stress ratio
- **kH is largely uncertain and can range between 2.0 and 2.5**
  - Each model was run twice with kH ratios of 2.0 and 2.5 to bound the range of possible behavior of the excavations
- **Major horizontal stress set to be the X direction and minor horizontal stress set to be the Y direction in the FLAC3D models**
  - Excavations in the models are oriented appropriately relative to the directions of major and minor horizontal stresses

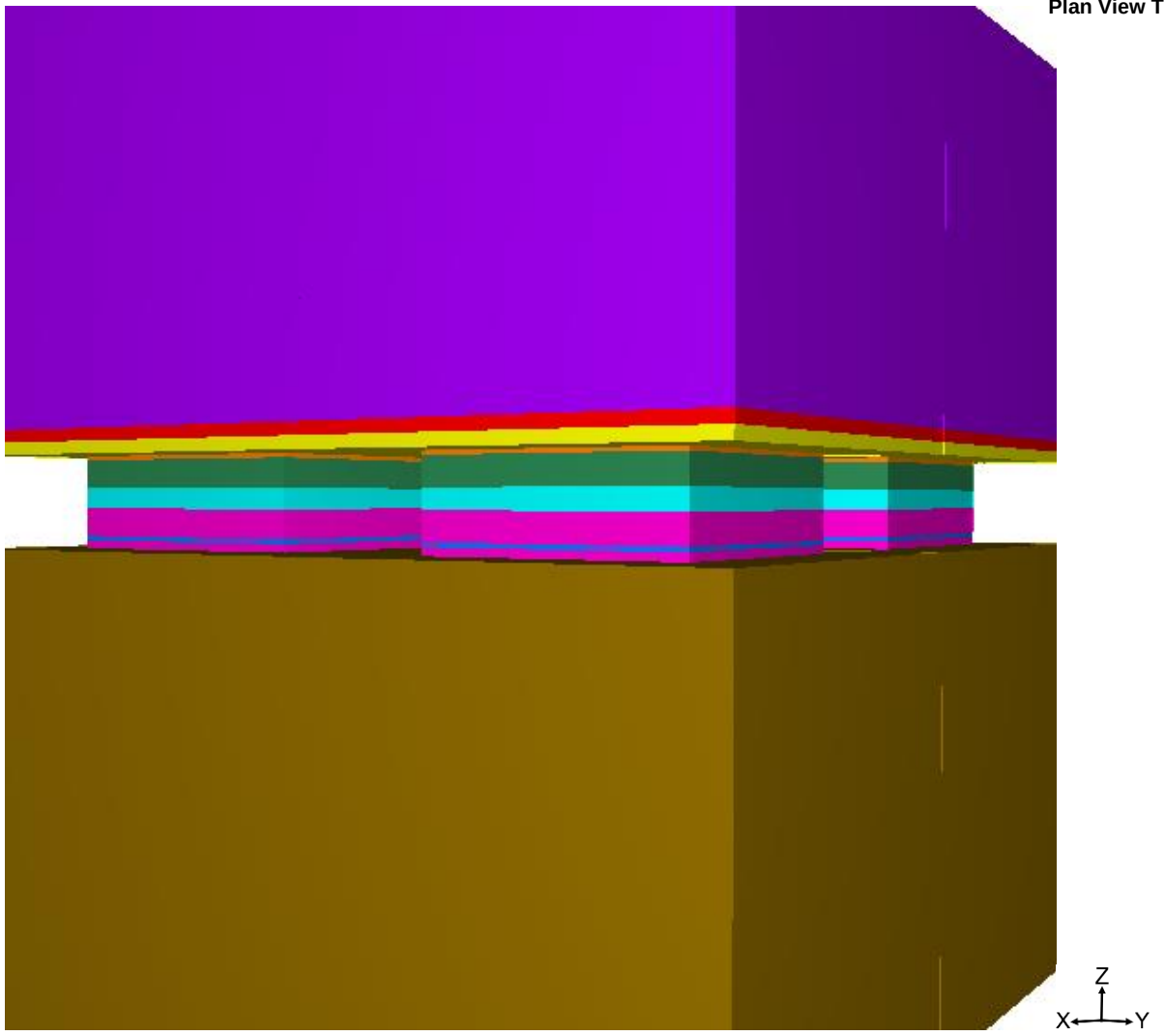
Stress	Cartesian Component	Stress Magnitude Calculation
Vertical	Z	Overburden Unit Weight * Depth of Model
Major Horizontal	X	Vertical Stress * kH
Minor Horizontal	Y	Vertical Stress * kh

Historic Mine Model: Geometry & Stratigraphy



Plan View Through Pillars

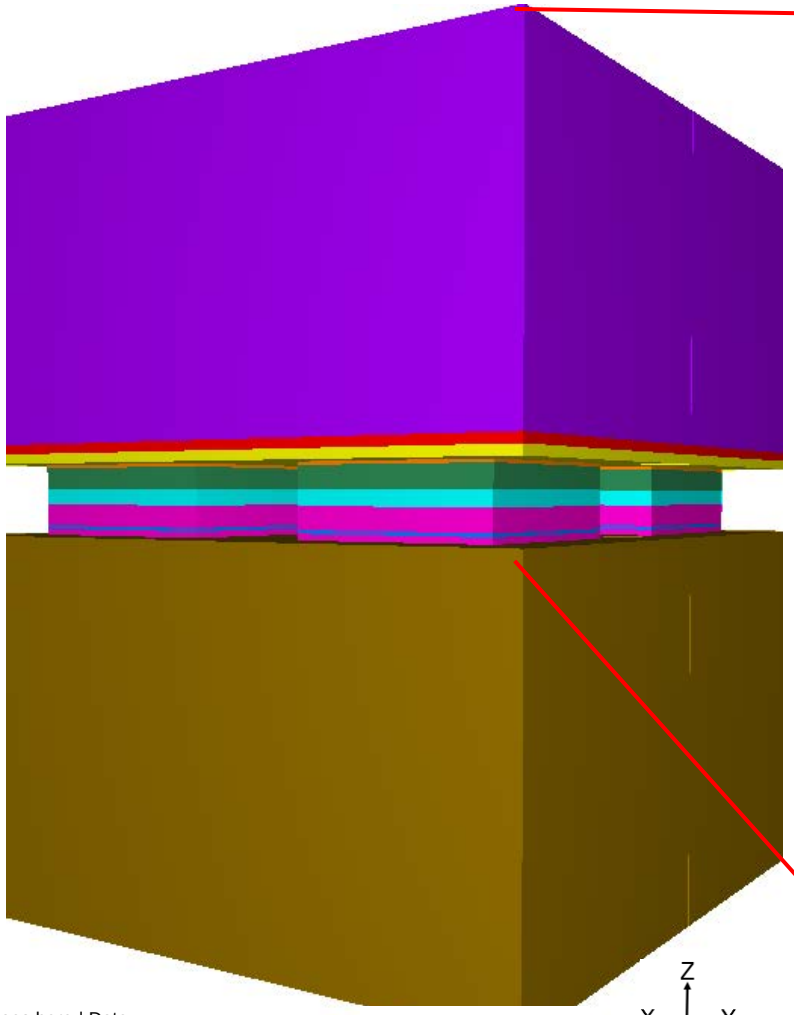
- WIDE (to top of model)
- UZVA (0.6 m thick)
- BMAS (0.6 m thick)
- THIN (0.2 m thick)
- UPSA (1.4 m thick)
- TOPZ-TIGR (1.0 m thick)
- ORE (2.0 m thick)
- DOMN (0.2 m thick)
- LWSA (to bottom of model)



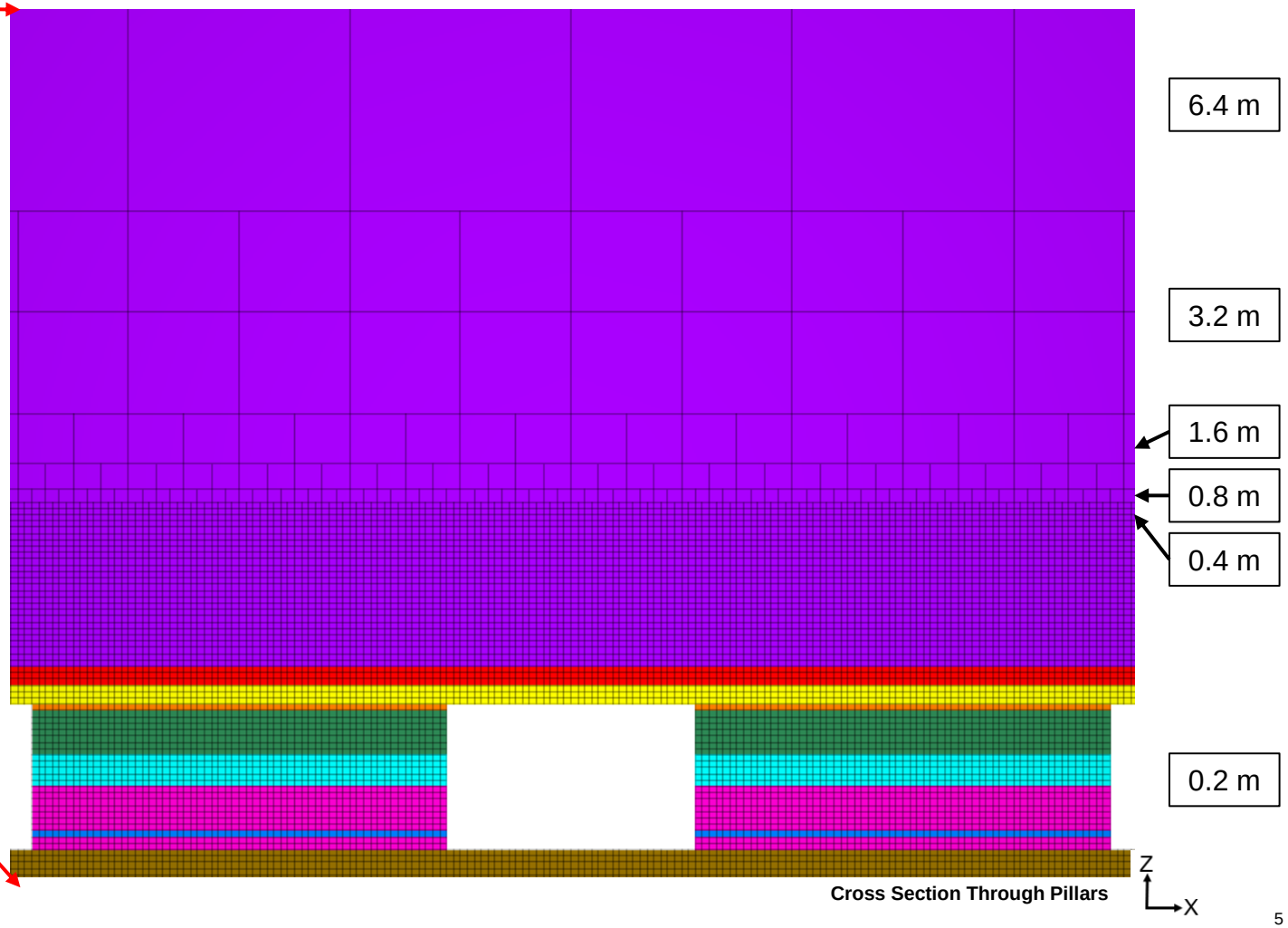
3D Isometric View

Historic Mine Model: Mesh

0.2 m mesh size near the excavation, transitions to 6.4 m near the top and bottom of the model to improve simulation runtime

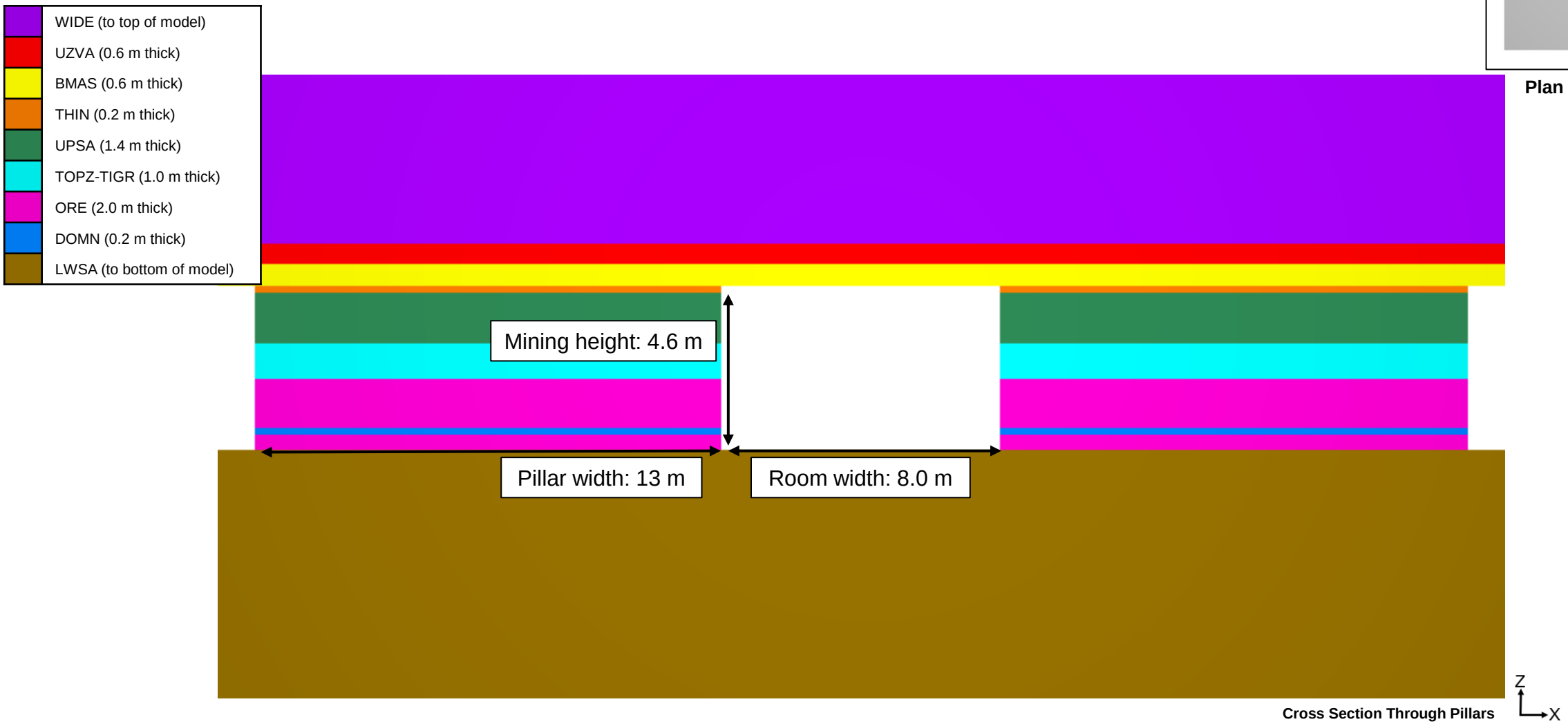


3D Isometric View

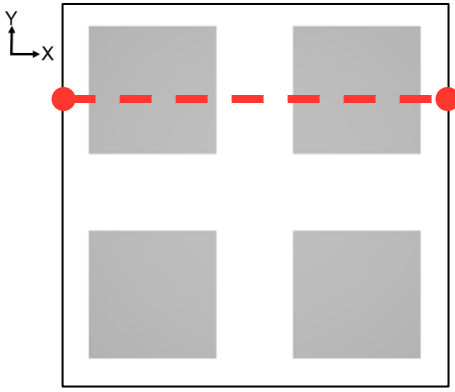


Cross Section Through Pillars

Historic Mine Model: Excavations

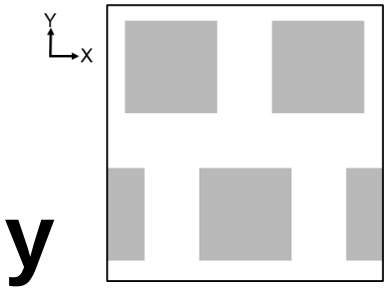


Cross Section Through Pillars



Plan View Through Pillars

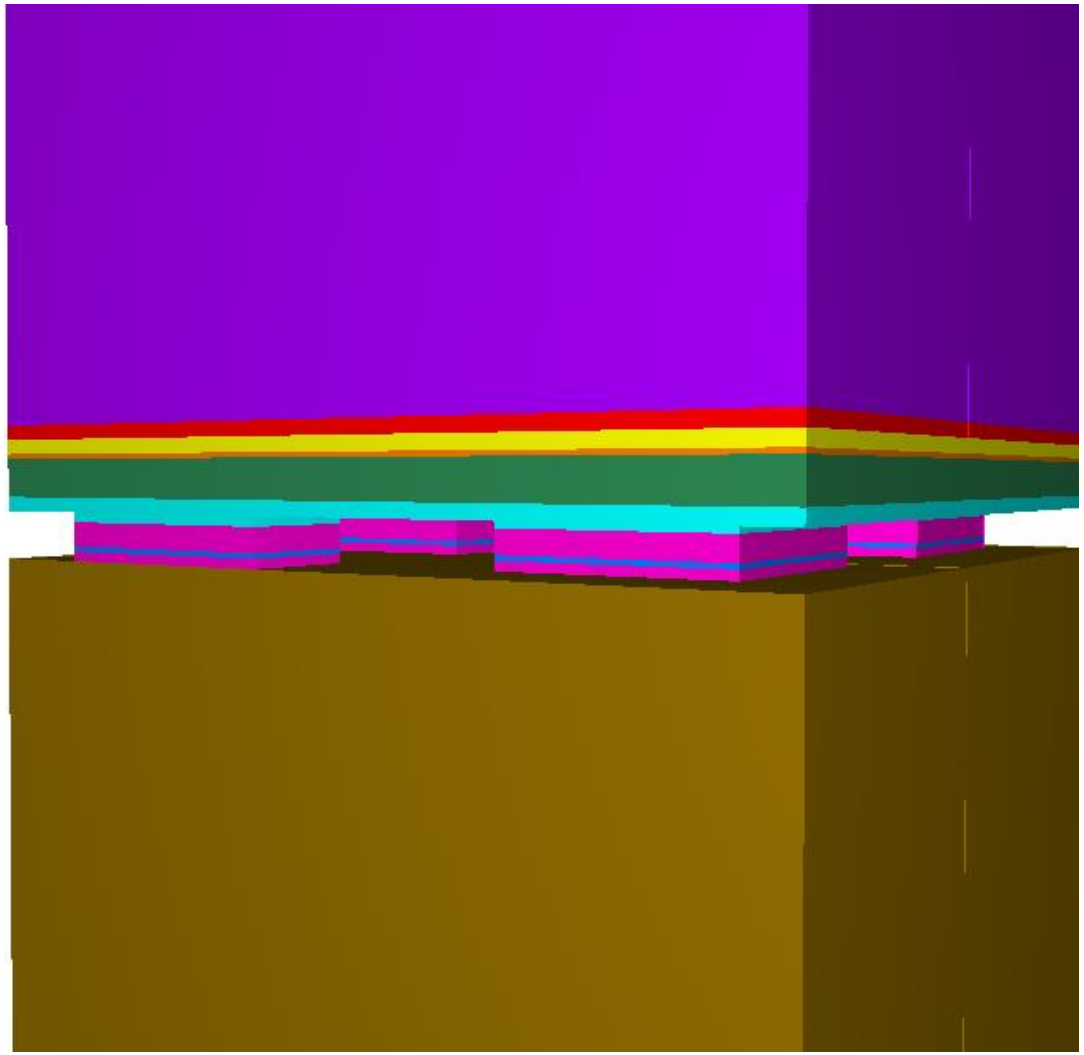
Retreat-Stubbing Model: Geometry & Stratigraphy



Plan View Through Pillars

- WIDE (to top of model)
- UZVA (0.6 m thick)
- BMAS (0.6 m thick)
- THIN (0.2 m thick)
- UPSA (1.6-2.6* m thick)
- TOPZ-TIGR (0.6-2.0* m thick)
- ORE (1.4-2.0* m thick)
- DOMN (0.2 m thick)
- LWSA (to bottom of model)

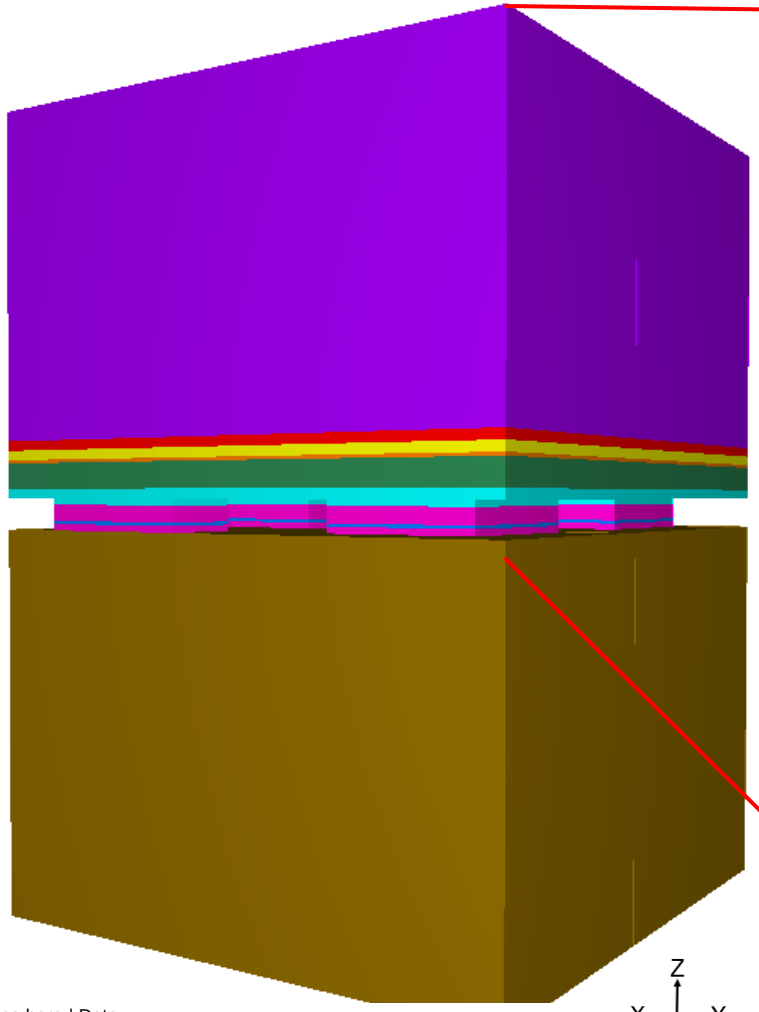
*Note: thicknesses of some units vary between stratigraphic domains



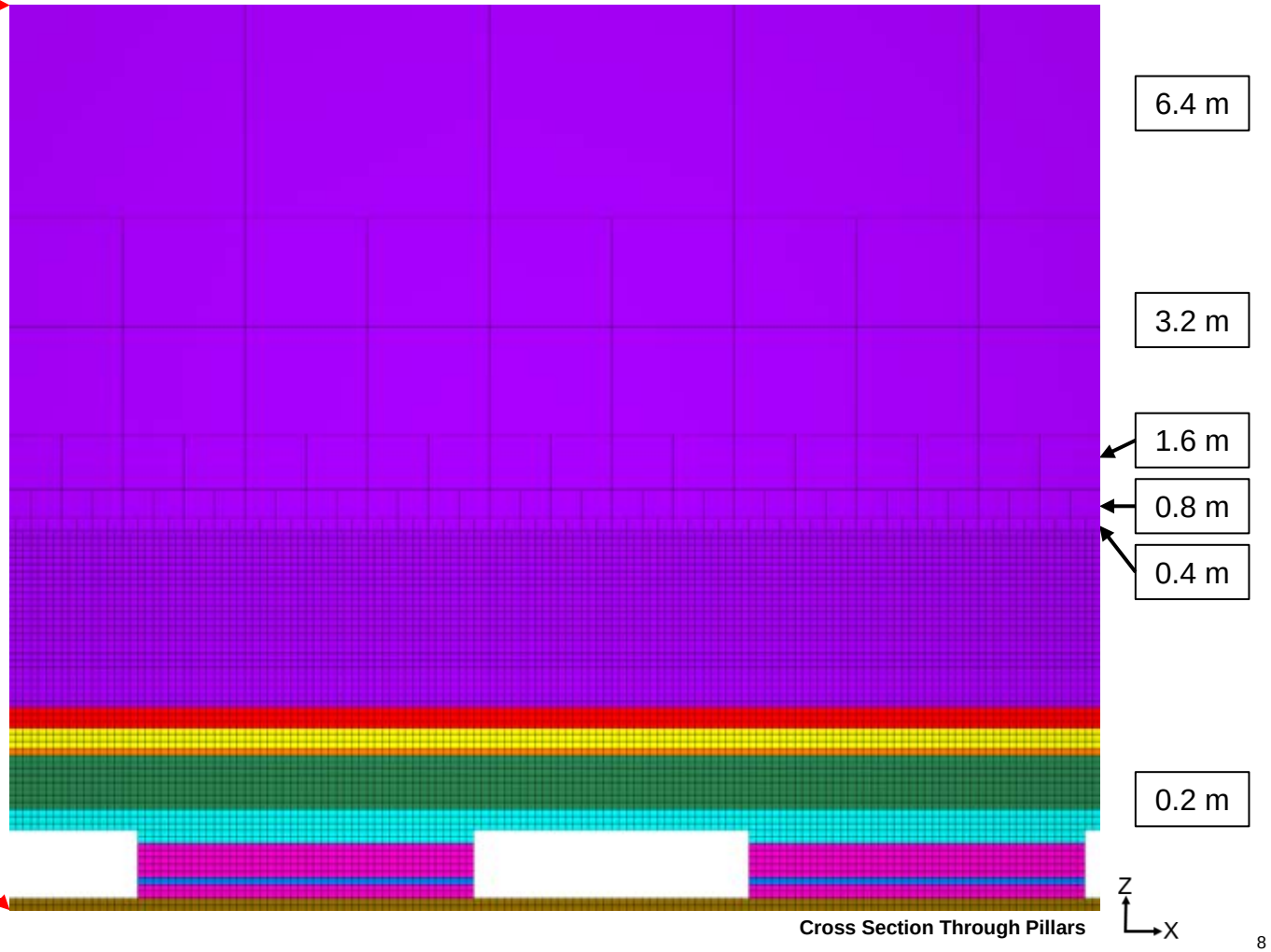
3D Isometric View

Retreat-Stubbing Model: Mesh

0.2 m mesh size near the excavation, transitions to 6.4 m near the top and bottom of the model to improve simulation runtime

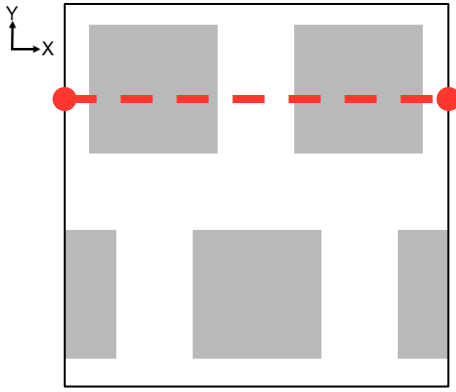


3D Isometric View



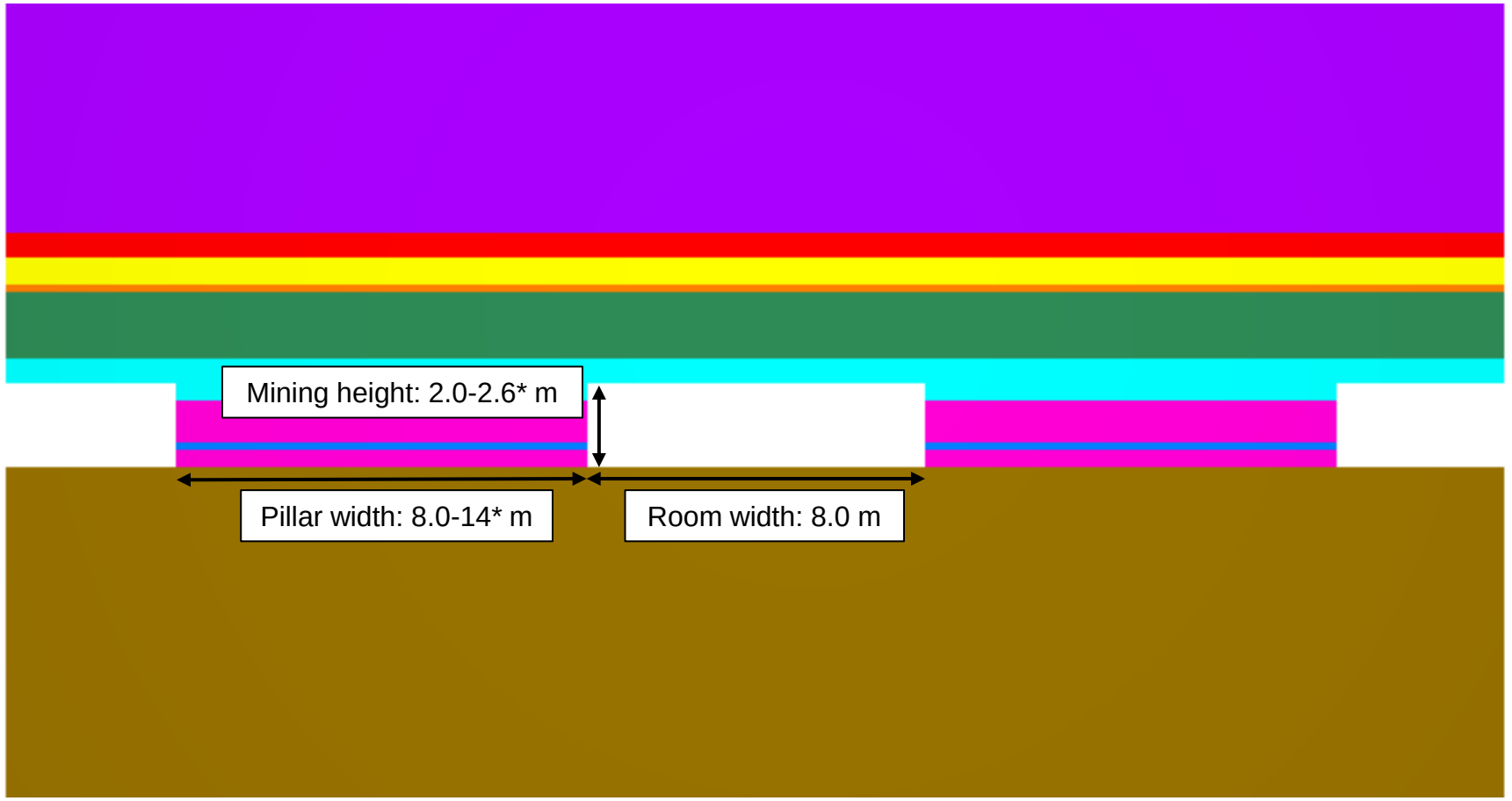
Cross Section Through Pillars

Retreat-Stubbing Model: Excavation



Plan View Through Pillars

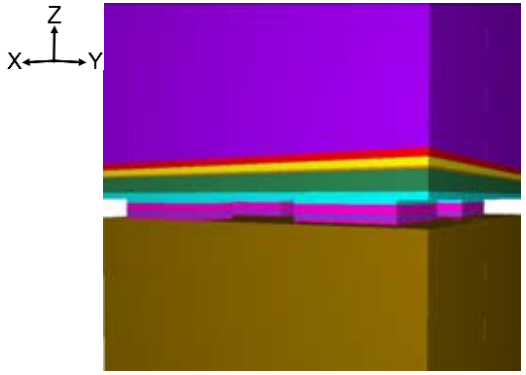
- WIDE (to top of model)
- UZVA (0.6 m thick)
- BMAS (0.6 m thick)
- THIN (0.2 m thick)
- UPSA (1.6-2.6 m thick)
- TOPZ-TIGR (0.6-2.0 m thick)
- ORE (1.4-2.0 m thick)
- DOMN (0.2 m thick)
- LWSA (to bottom of model)



Cross Section Through Pillars

*Note: pillar width and mining height vary between stratigraphic domains modeled

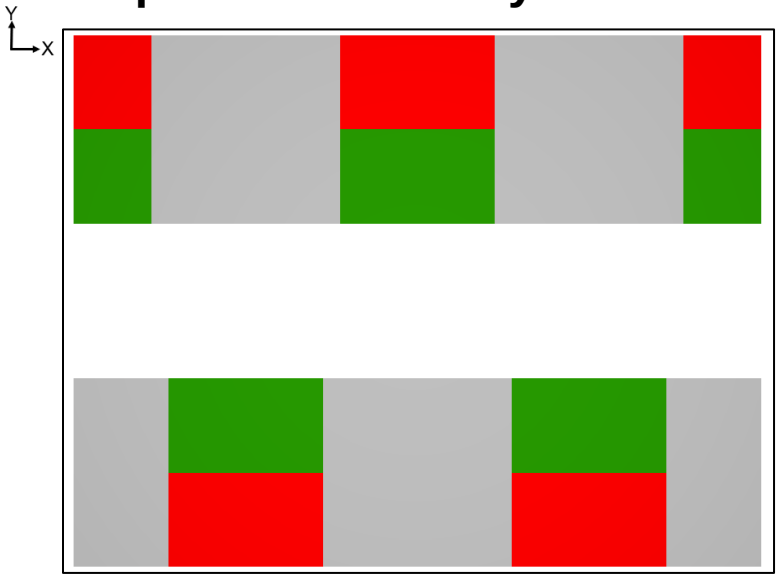
Retreat-Stubbing Model: Sequencing



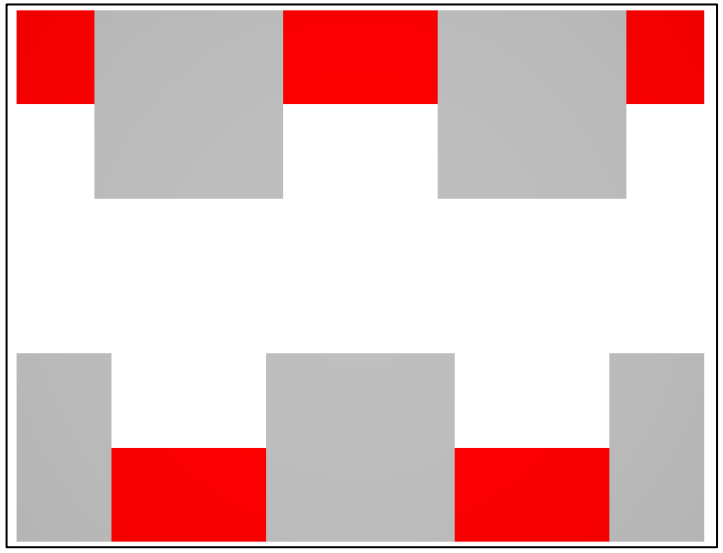
3D Isometric View

- Pillar
- Sequence 1 Excavation
- Sequence 2 Excavation

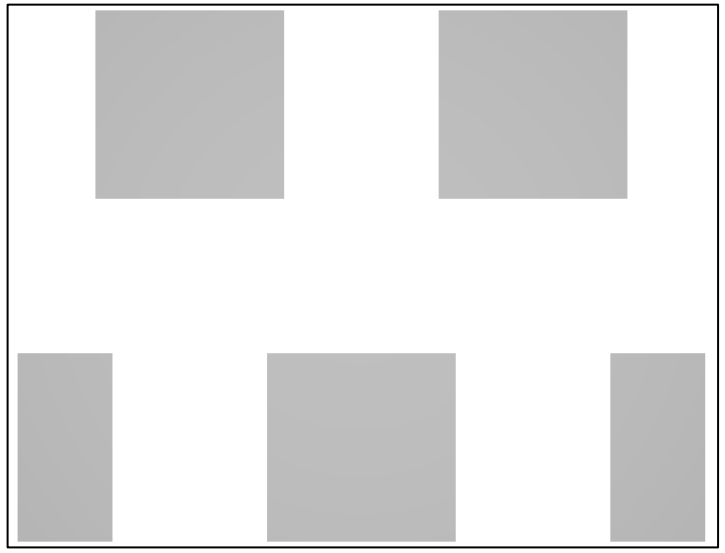
Sequence 1: Primary Ore Drive



Sequence 2: Stubbing



Sequence 3: Secondary Drive



Plan View Through Pillars

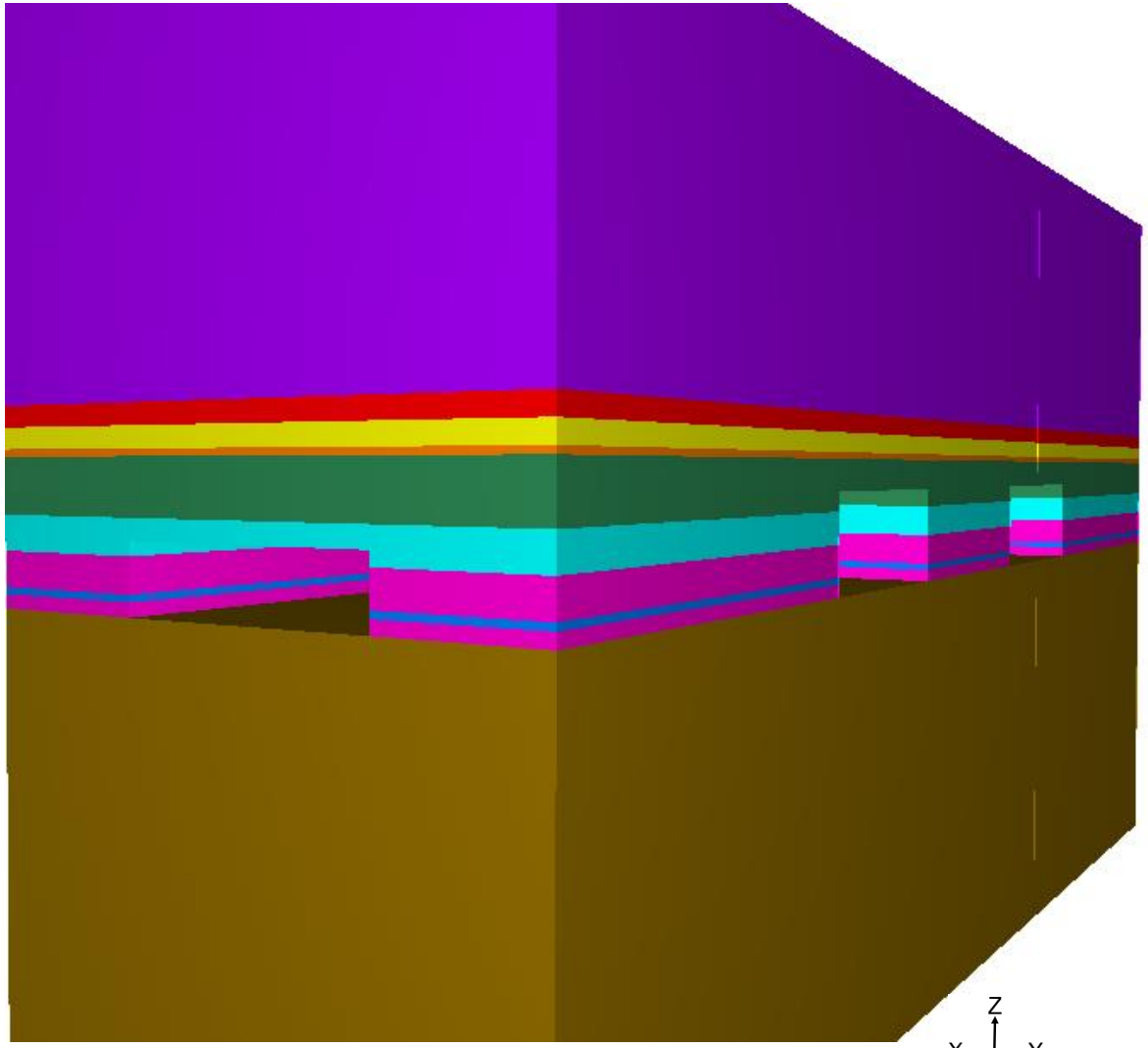
The excavation process is modeled as three sequences to match the retreat stubbing mining method that will be implemented at the White Pine North project.

Panel Access: Geometry & Stratigraphy

Plan View Through Pillars

- WIDE (to top of model)
- UZVA (0.6 m thick)
- BMAS (0.6 m thick)
- THIN (0.2 m thick)
- UPSA (1.6 or 2.6* m thick)
- TOPZ-TIGR (1.0 m thick)
- ORE (1.6 m thick)
- DOMN (0.2 m thick)
- LWSA (to bottom of model)

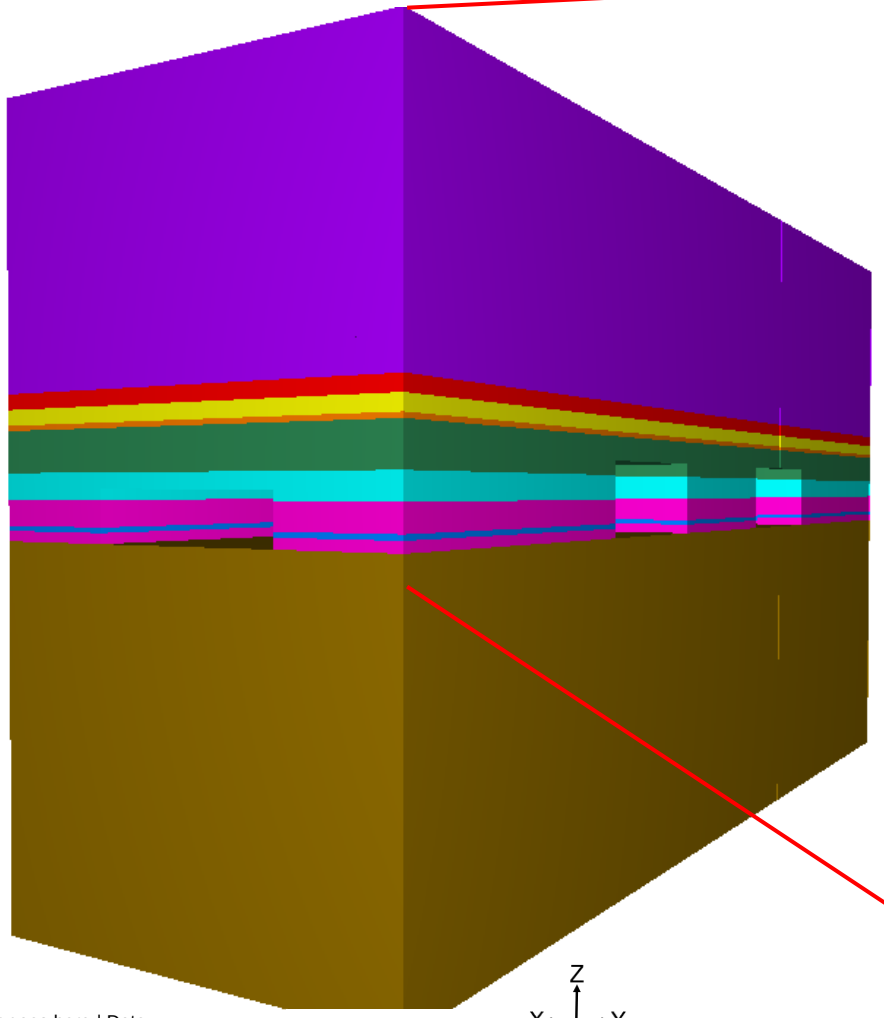
*Note: thicknesses of some units vary between stratigraphic domains



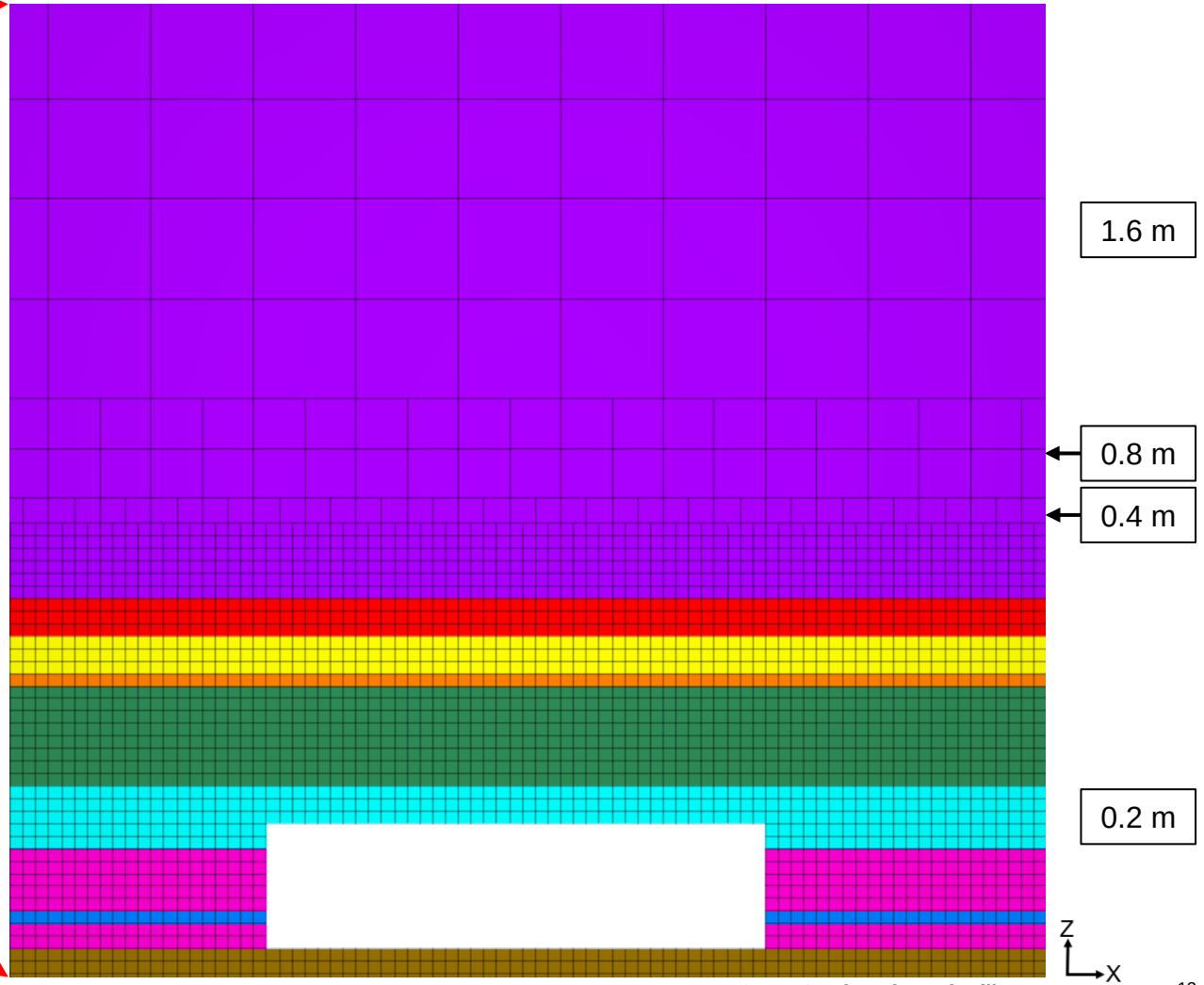
3D Isometric View

Panel Access: Meshing

0.2 m mesh size near the excavation, transitions to 1.6 m near the top and bottom of the model to improve simulation runtime

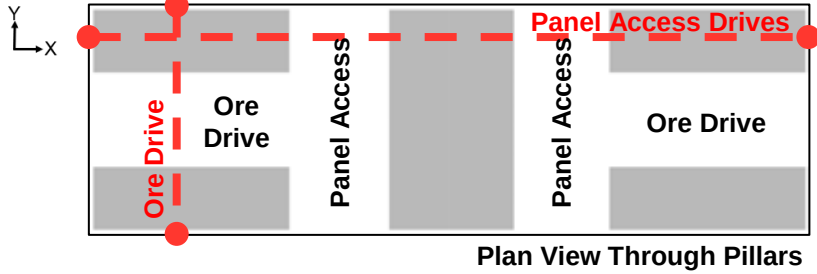


3D Isometric View



Cross Section Through Pillars

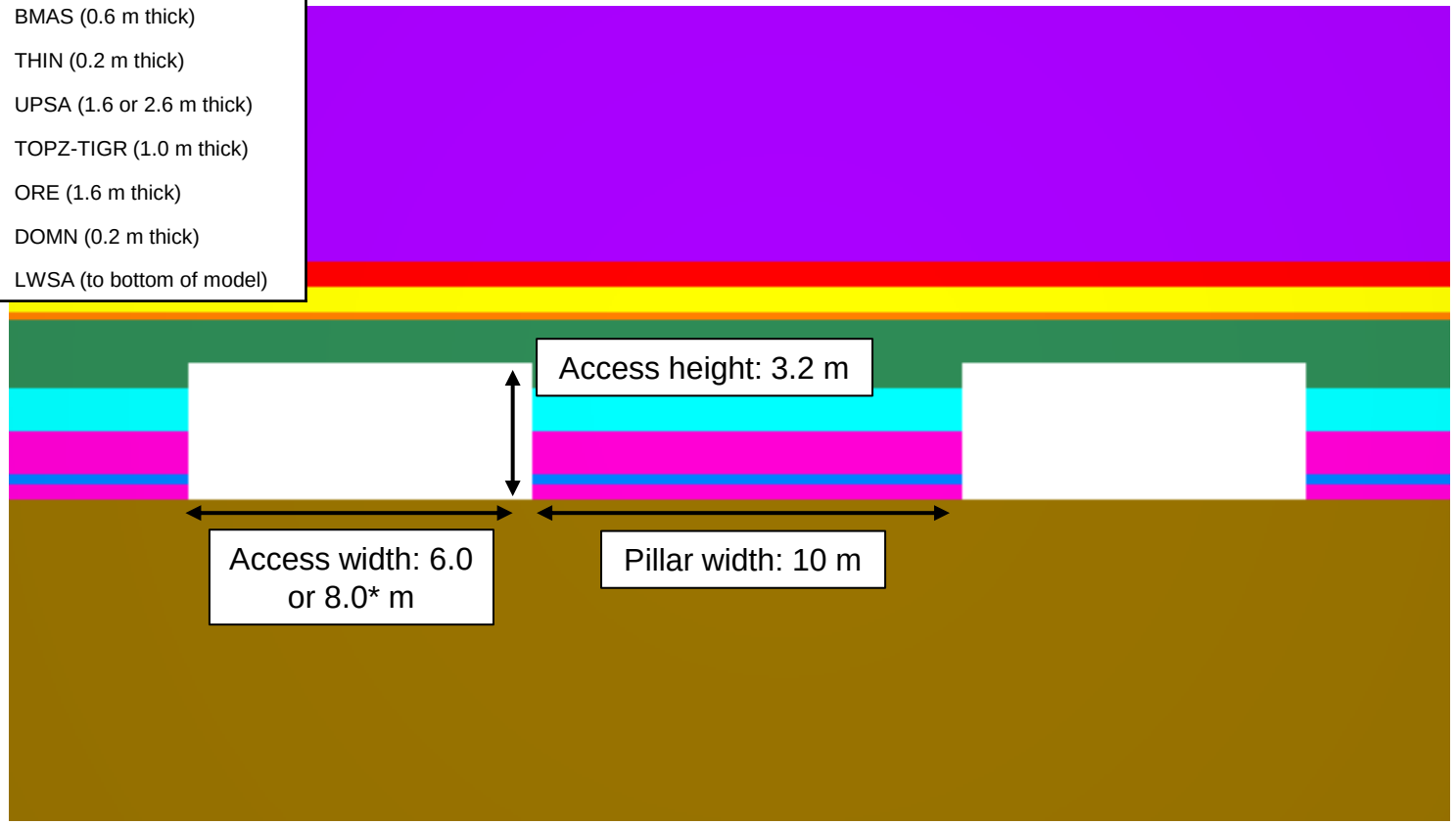
Model Excavation



Plan View Through Pillars

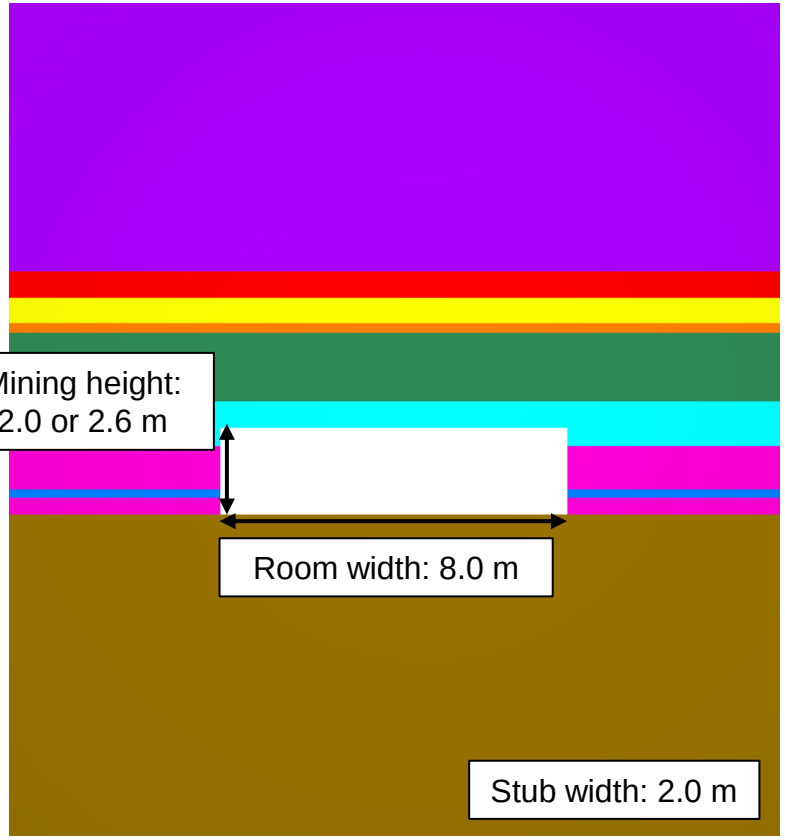
WIDE (to top of model)
UZVA (0.6 m thick)
BMAS (0.6 m thick)
THIN (0.2 m thick)
UPSA (1.6 or 2.6 m thick)
TOPZ-TIGR (1.0 m thick)
ORE (1.6 m thick)
DOMN (0.2 m thick)
LWSA (to bottom of model)

Panel Access Drives



Cross Section Through Pillars

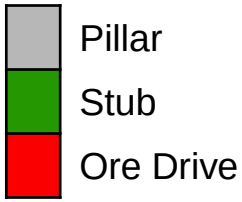
Ore Drive

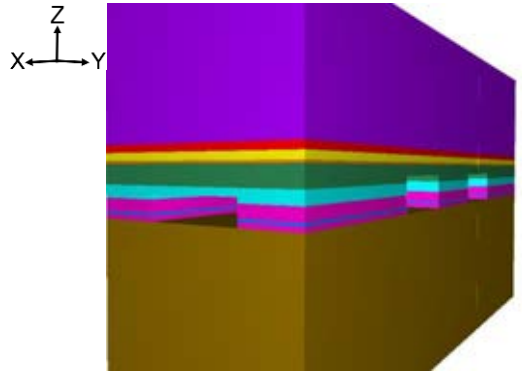


Cross Section Through Pillars

*Note: access drive width and mining height vary between stratigraphic domains modeled

Panel Access: Sequencing



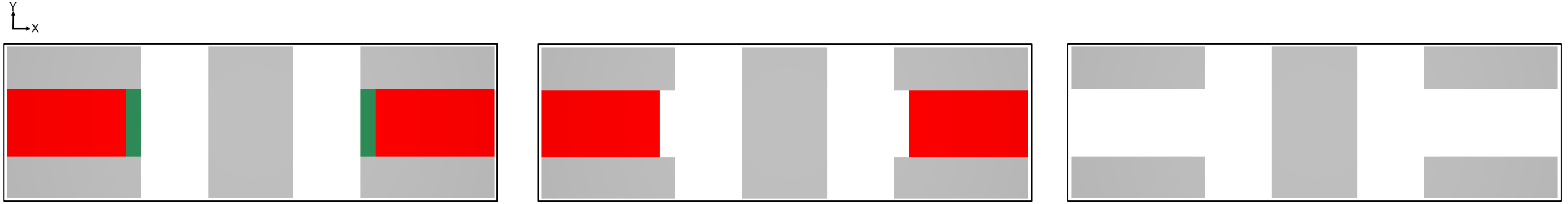


3D Isometric View

Sequence 1: Panel Access Drives

Sequence 2: Stubbing

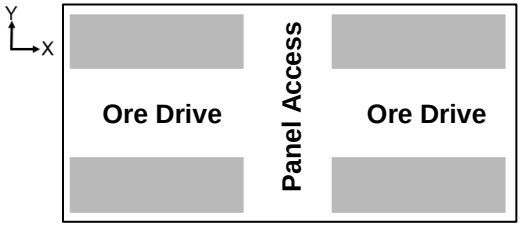
Sequence 3: Ore Drive



Plan View Through Pillars

The excavation process is modeled as three sequences to match the retreat stubbing mining method that will be implemented at the White Pine North project.

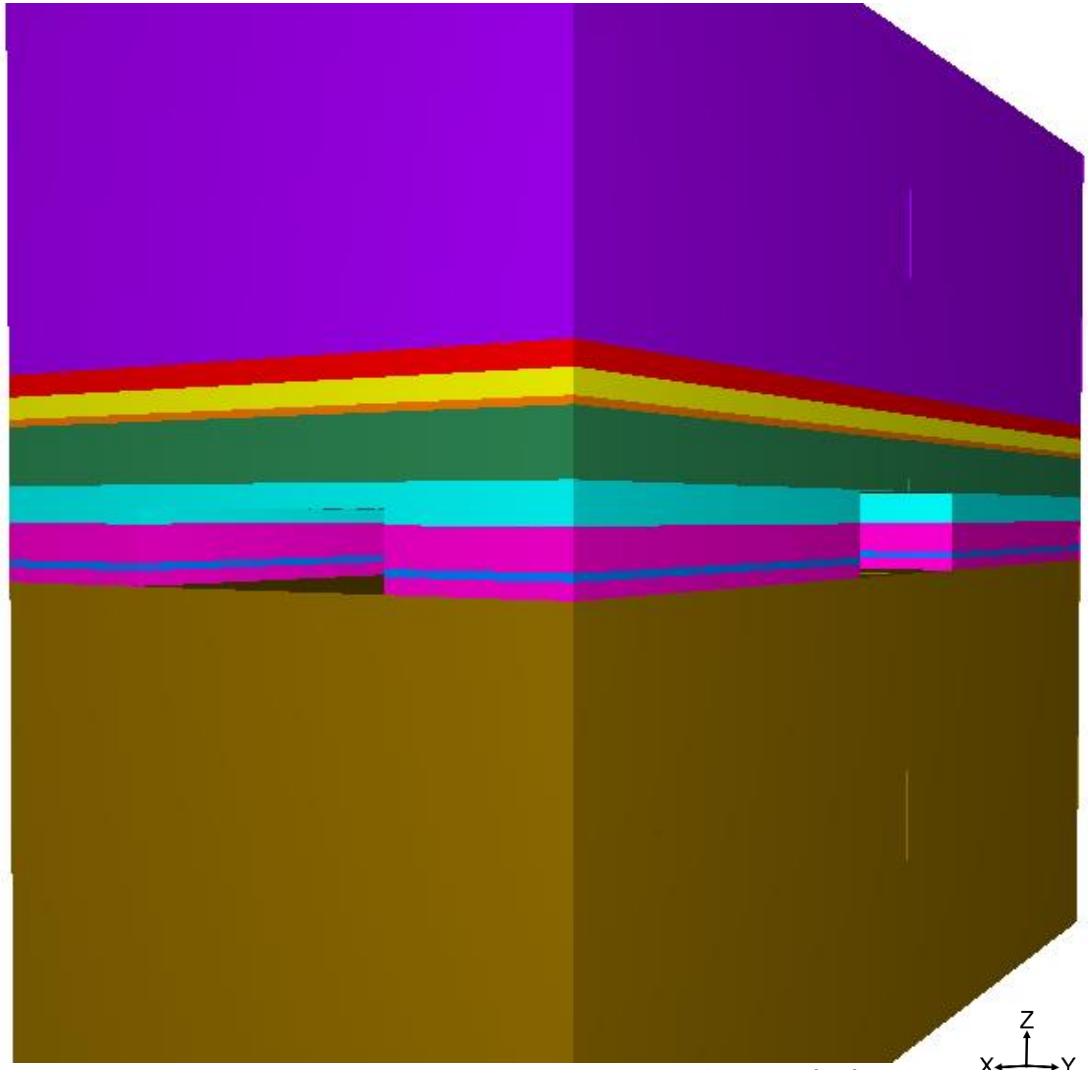
Panel Vent: Geometry & Stratigraphy



Plan View Through Pillars

- WIDE (to top of model)
- UZVA (0.6 m thick)
- BMAS (0.6 m thick)
- THIN (0.2 m thick)
- UPSA (1.6 or 2.6* m thick)
- TOPZ-TIGR (1.0 m thick)
- ORE (1.6 m thick)
- DOMN (0.2 m thick)
- LWSA (to bottom of model)

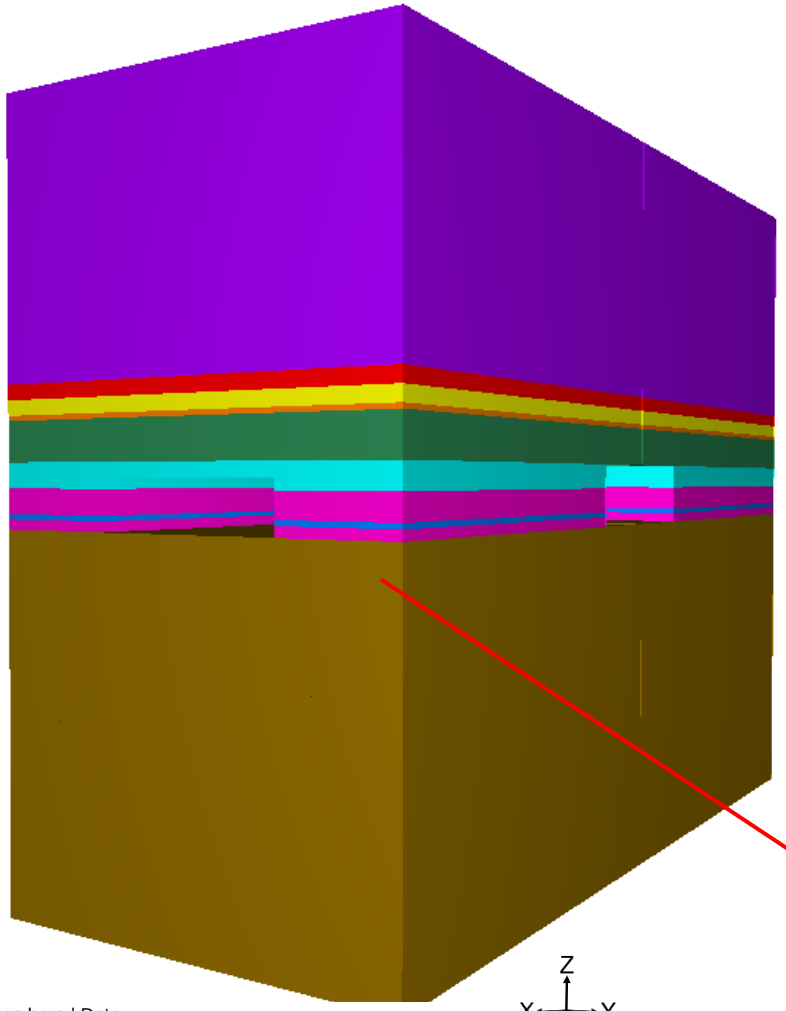
*Note: thicknesses of some units vary between stratigraphic domains



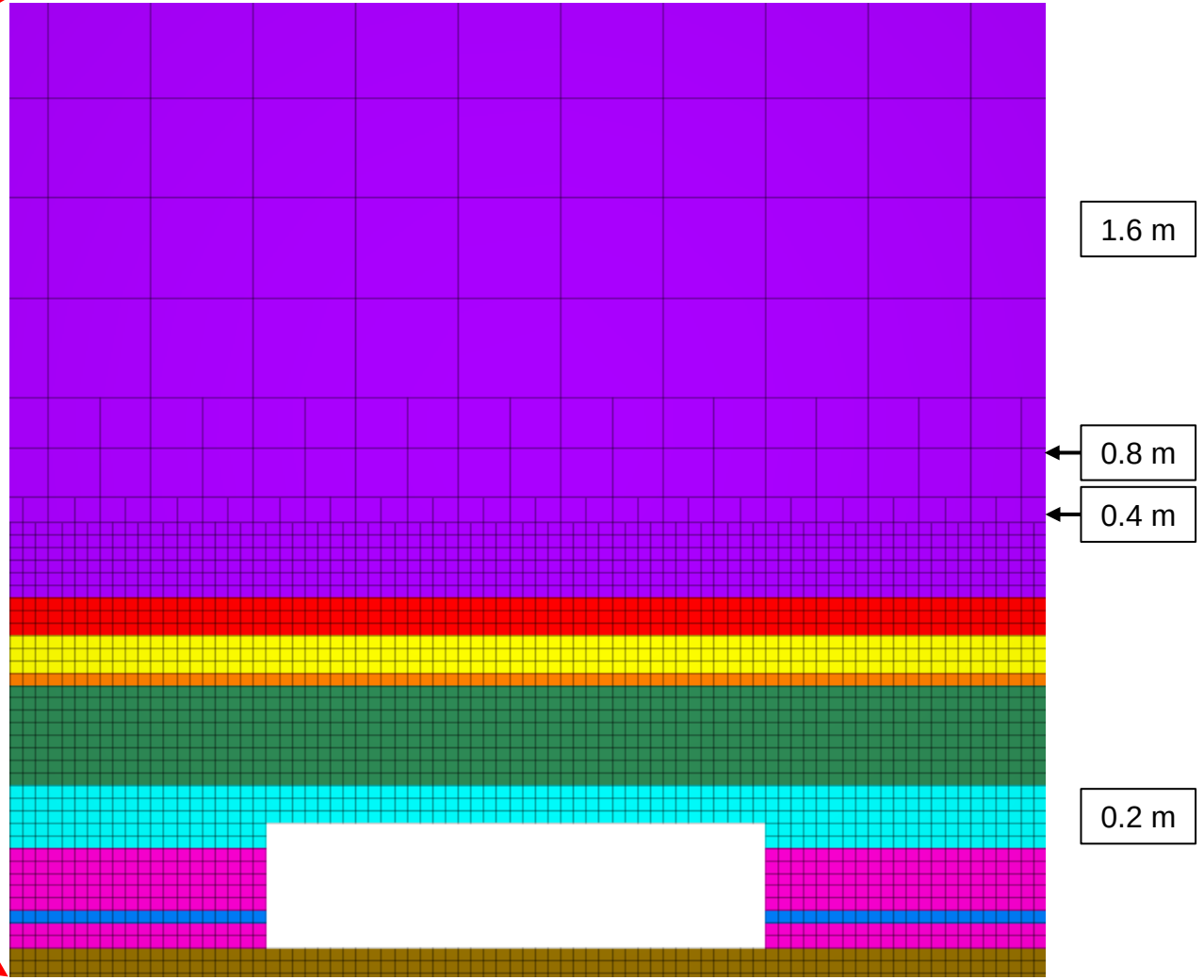
3D Isometric View

Panel Vent: Meshing

0.2 m mesh size near the excavation, transitions to 1.6 m near the top and bottom of the model to improve simulation runtime

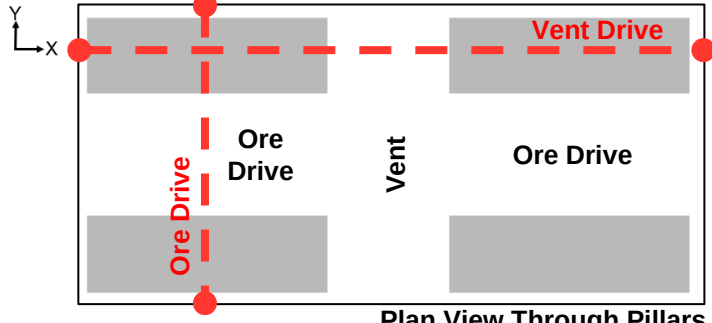


3D Isometric View

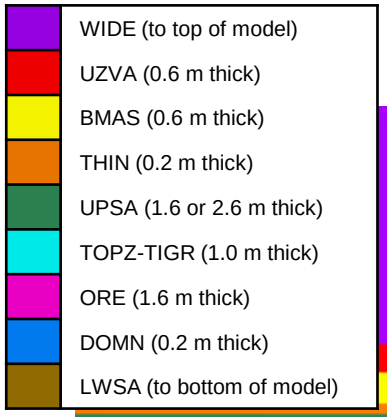


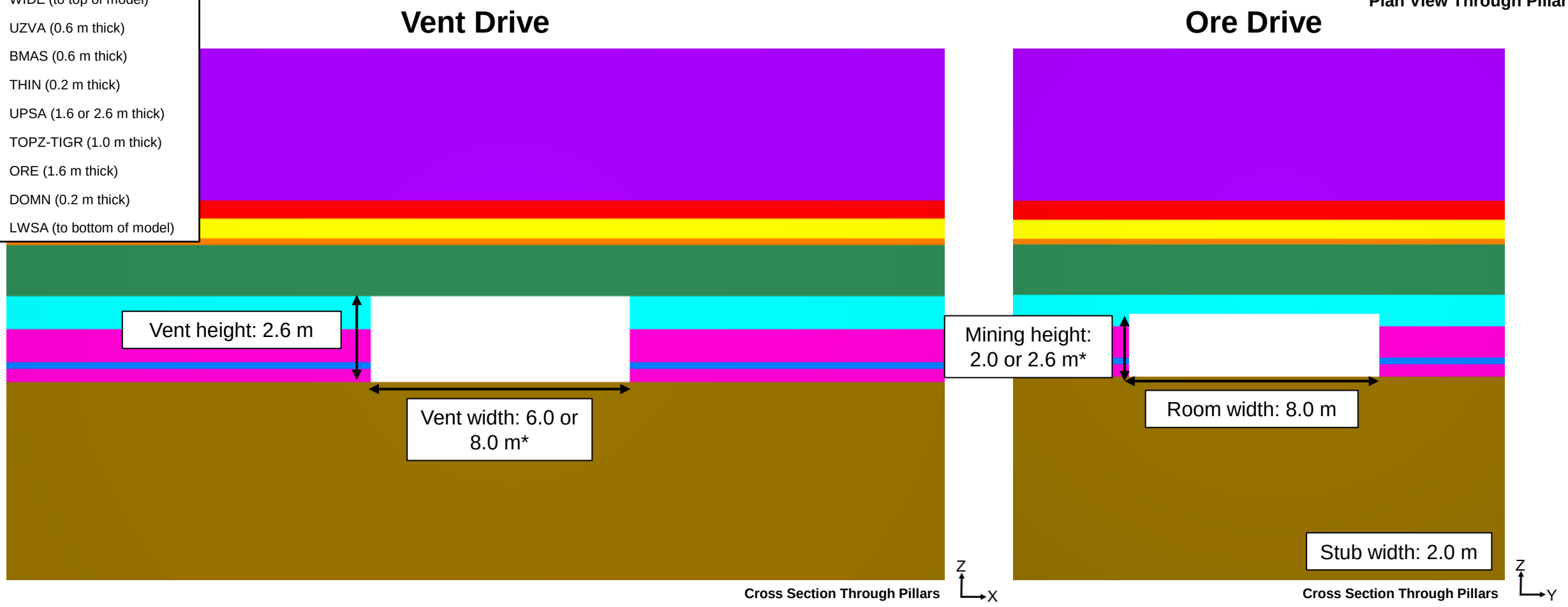
Cross Section Through Pillars

Panel Vent: Excavation



Plan View Through Pillars





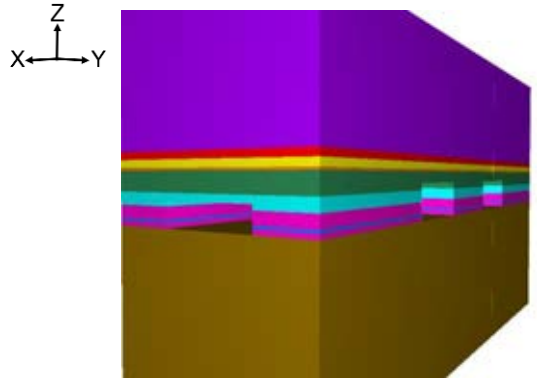
Cross Section Through Pillars

Cross Section Through Pillars

*Note: vent width and mining height vary between stratigraphic domains modeled

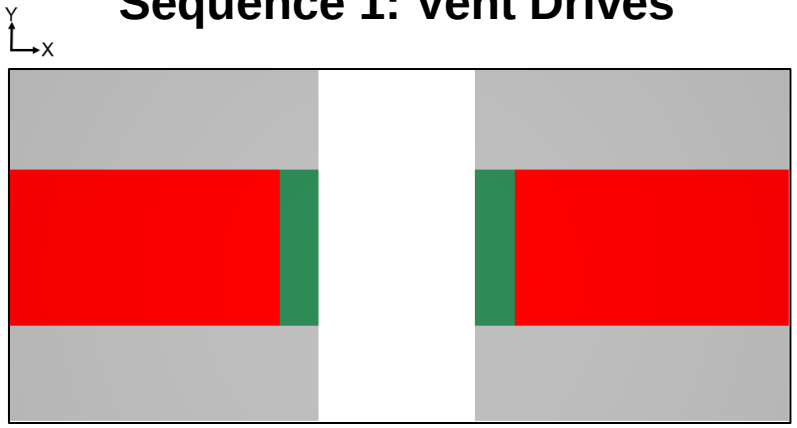
Panel Vent: Sequencing

Pillar
Stub
Ore Drive

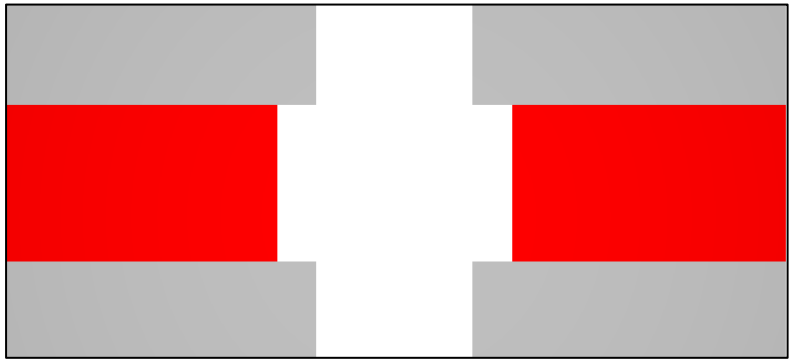


3D Isometric View

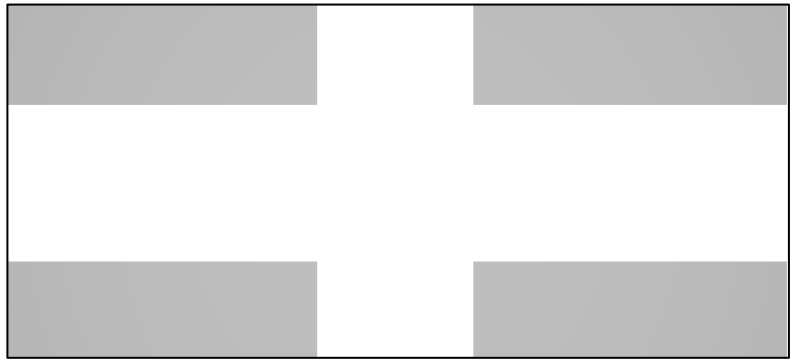
Sequence 1: Vent Drives



Sequence 2: Stubbing



Sequence 3: Ore Drive



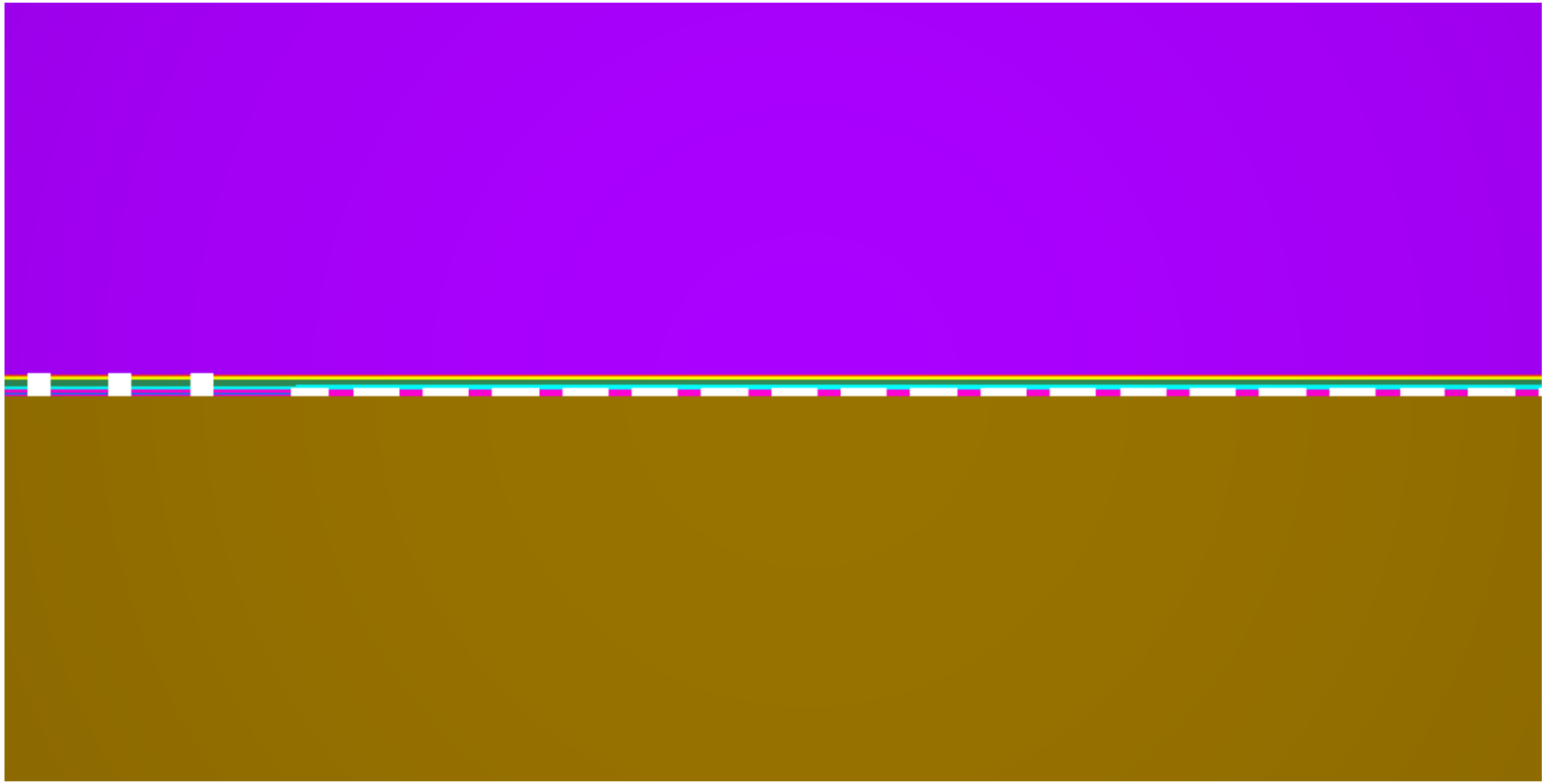
Plan View Through Pillars

The excavation process is modeled as three sequences to match the retreat stubbing mining method that will be implemented at the White Pine North project.

Decline: Geometry & Stratigraphy

- WIDE (to top of model)
- UZVA (0.6 m thick)
- BMAS (0.6 m thick)
- THIN (0.2 m thick)
- UPSA (1.6 or 2.6* m thick)
- TOPZ-TIGR (1.0 m thick)
- ORE (1.6 m thick)
- DOMN (0.2 m thick)
- LWSA (to bottom of model)

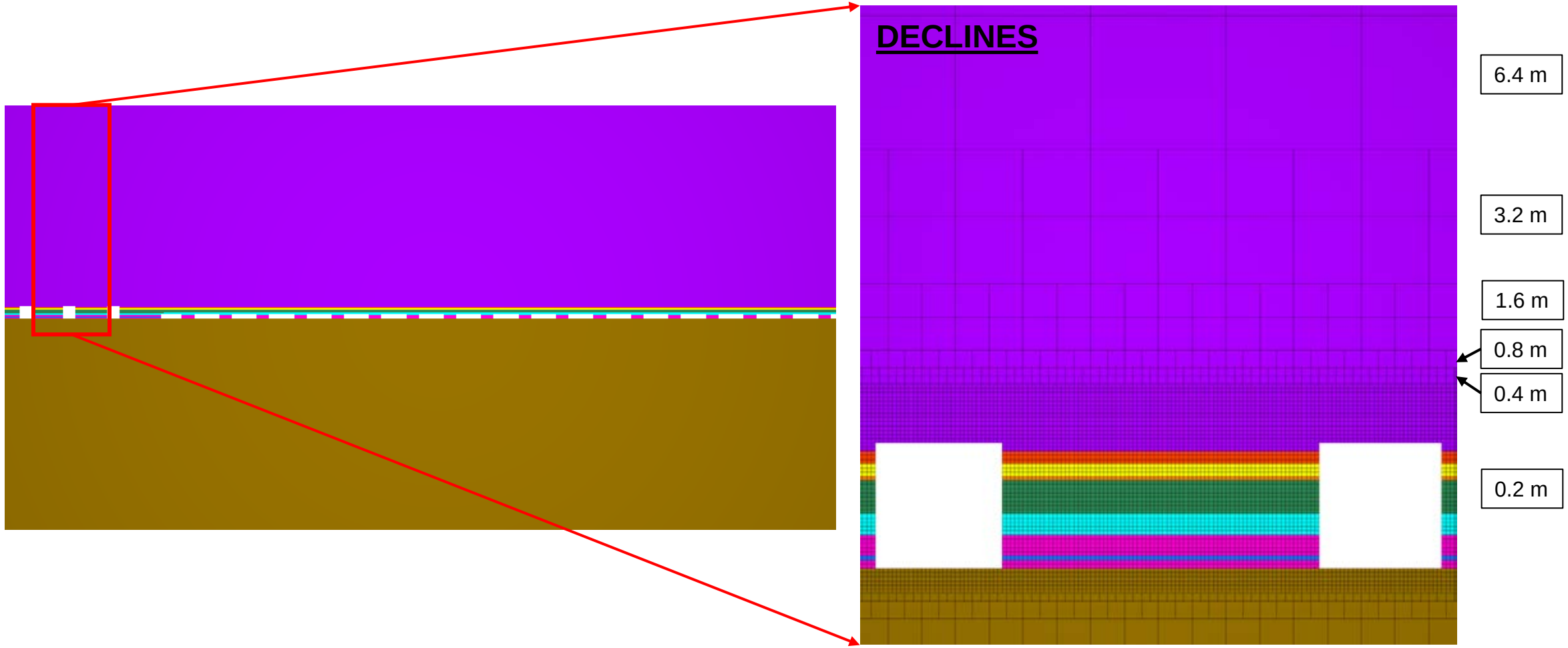
*Note: thicknesses of some units vary between stratigraphic domains



This model was developed in 2D only. Plane strain conditions are valid for long excavations (repeating pillars) and 2D models allow for shorter runtimes with higher resolution of results.

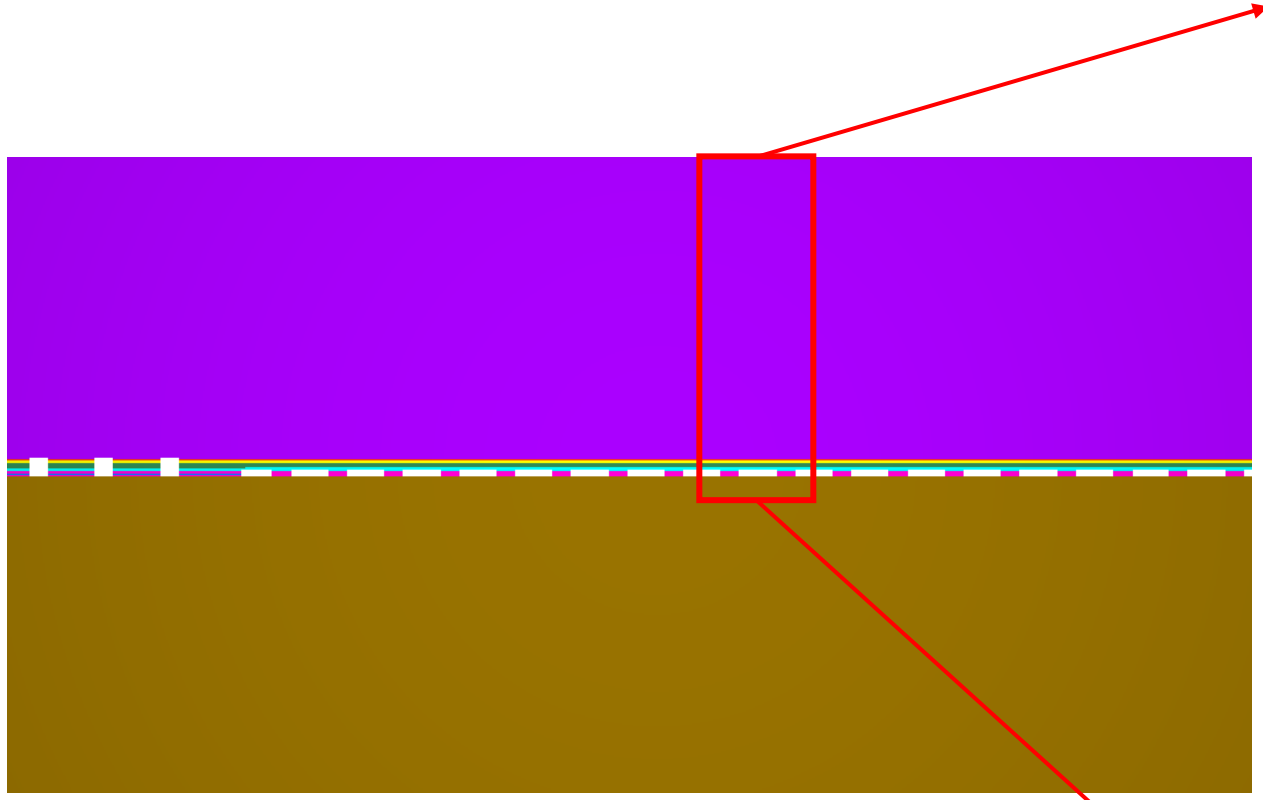
Decline 2D: Meshing

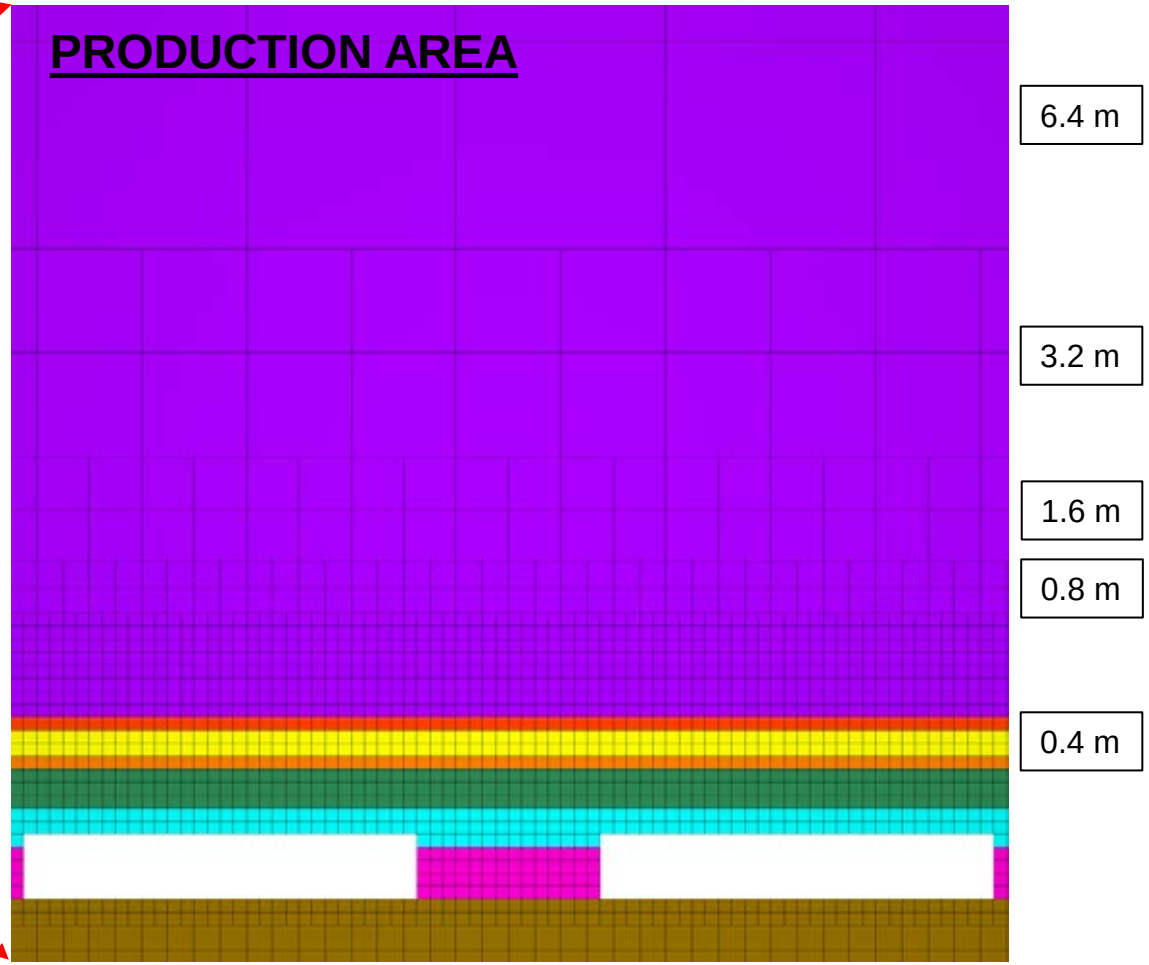
0.2 m mesh size near the excavation, transitions to 6.4 m near the top and bottom of the model to improve simulation runtime



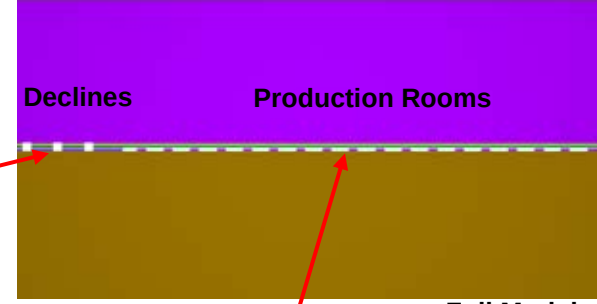
Decline 2D: Meshing

0.4 m mesh size near the excavation, transitions to 6.4 m near the top and bottom of the model to improve simulation runtime





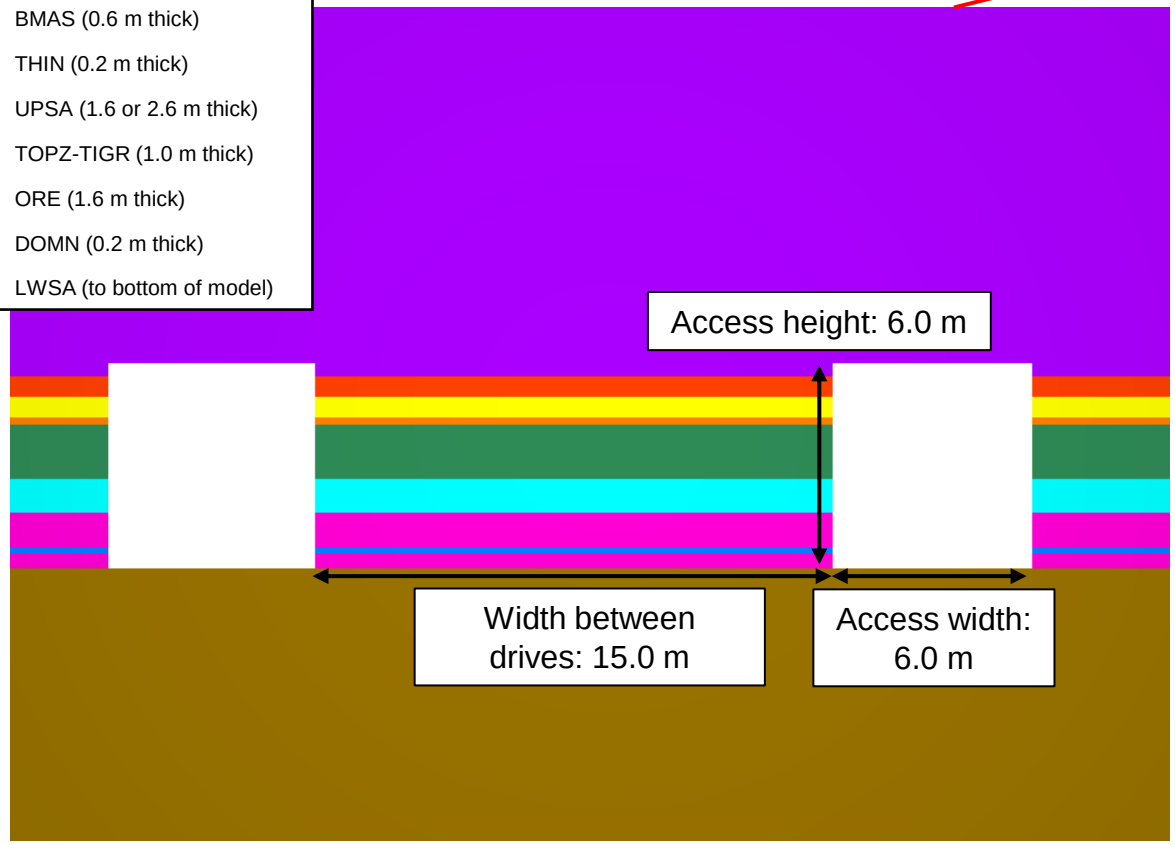
Decline 2D: Excavation



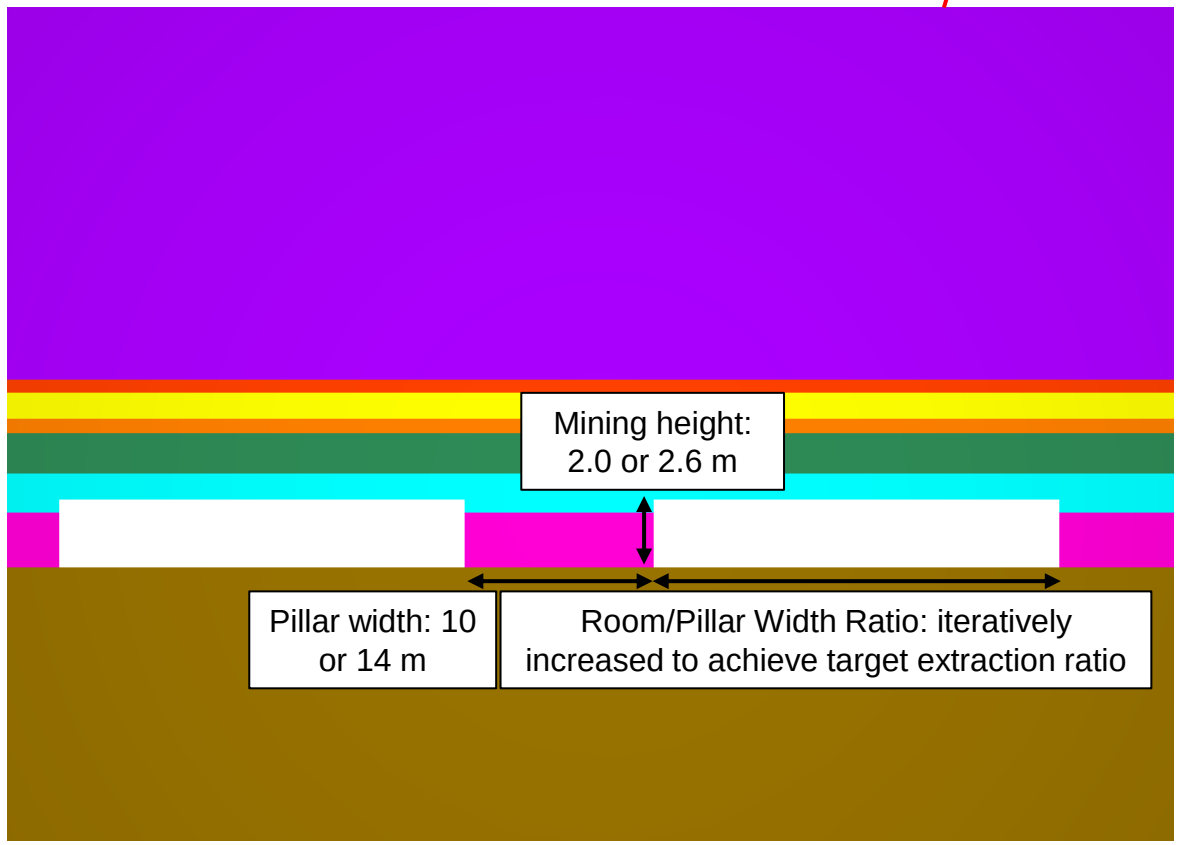
Full Model

WIDE (to top of model)
UZVA (0.6 m thick)
BMAS (0.6 m thick)
THIN (0.2 m thick)
UPSA (1.6 or 2.6 m thick)
TOPZ-TIGR (1.0 m thick)
ORE (1.6 m thick)
DOMN (0.2 m thick)
LWSA (to bottom of model)

Declines

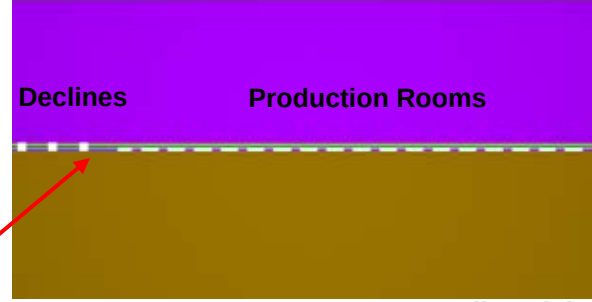


Production Rooms



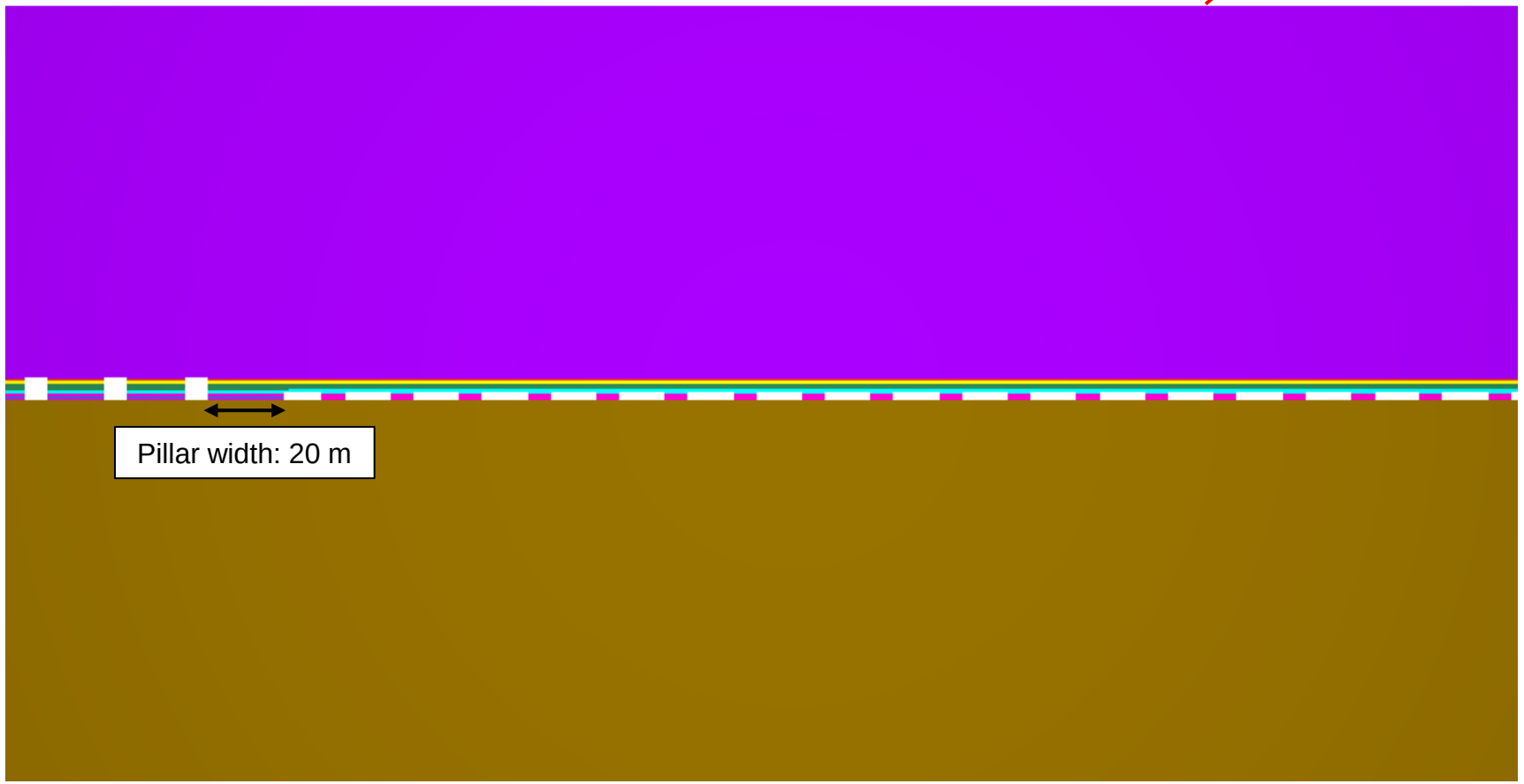
*Note: mining height varies between stratigraphic domains modeled

Decline 2D: Excavation



Full Model

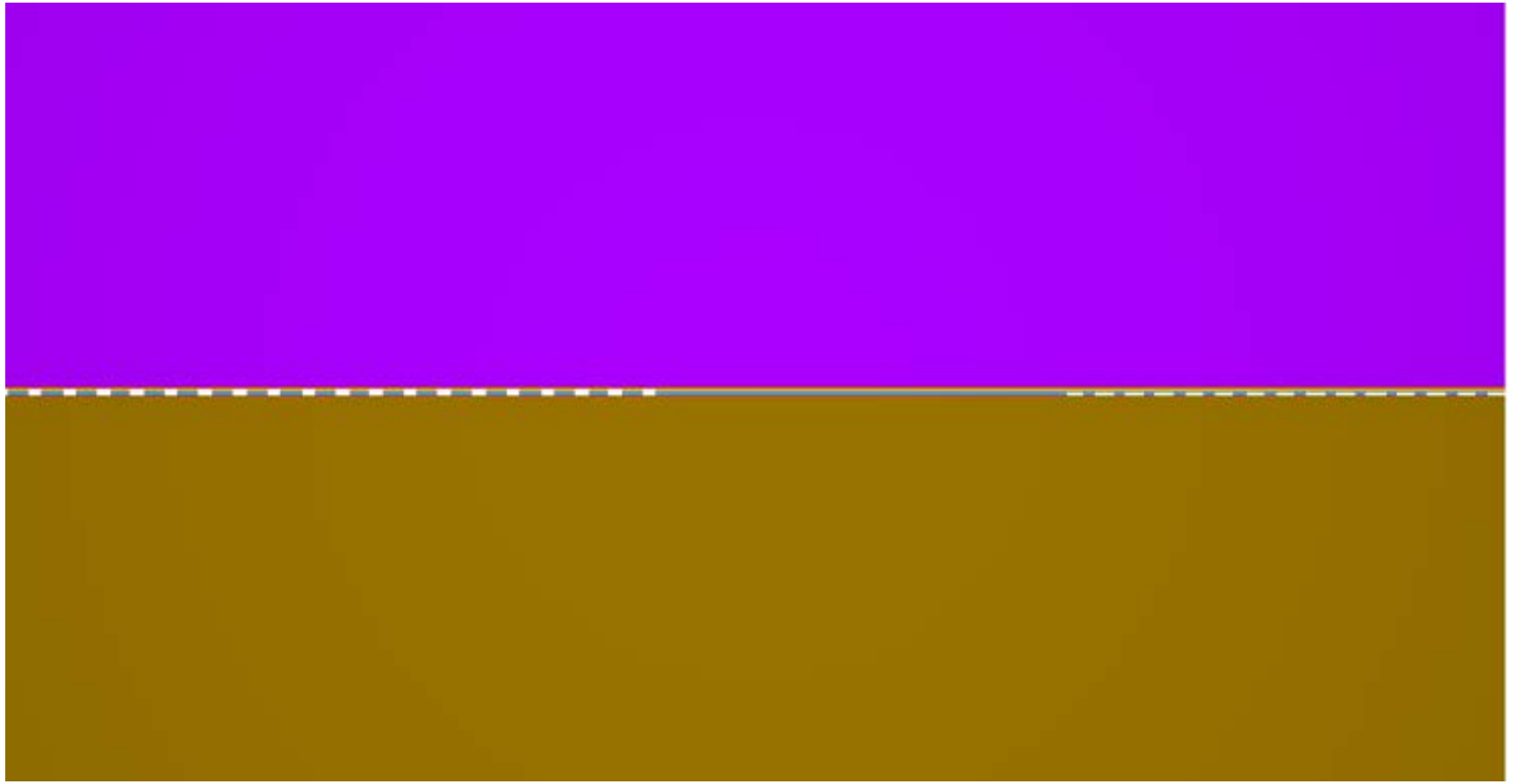
Protection Pillar



Barrier Pillar: Geometry & Stratigraphy

- WIDE (to top of model)
- UZVA (0.6 m thick)
- BMAS (0.6 m thick)
- THIN (0.2 m thick)
- UPSA (1.6 or 2.6* m thick)
- TOPZ-TIGR (1.0 m thick)
- ORE (1.6 m thick)
- DOMN (0.2 m thick)
- LWSA (to bottom of model)

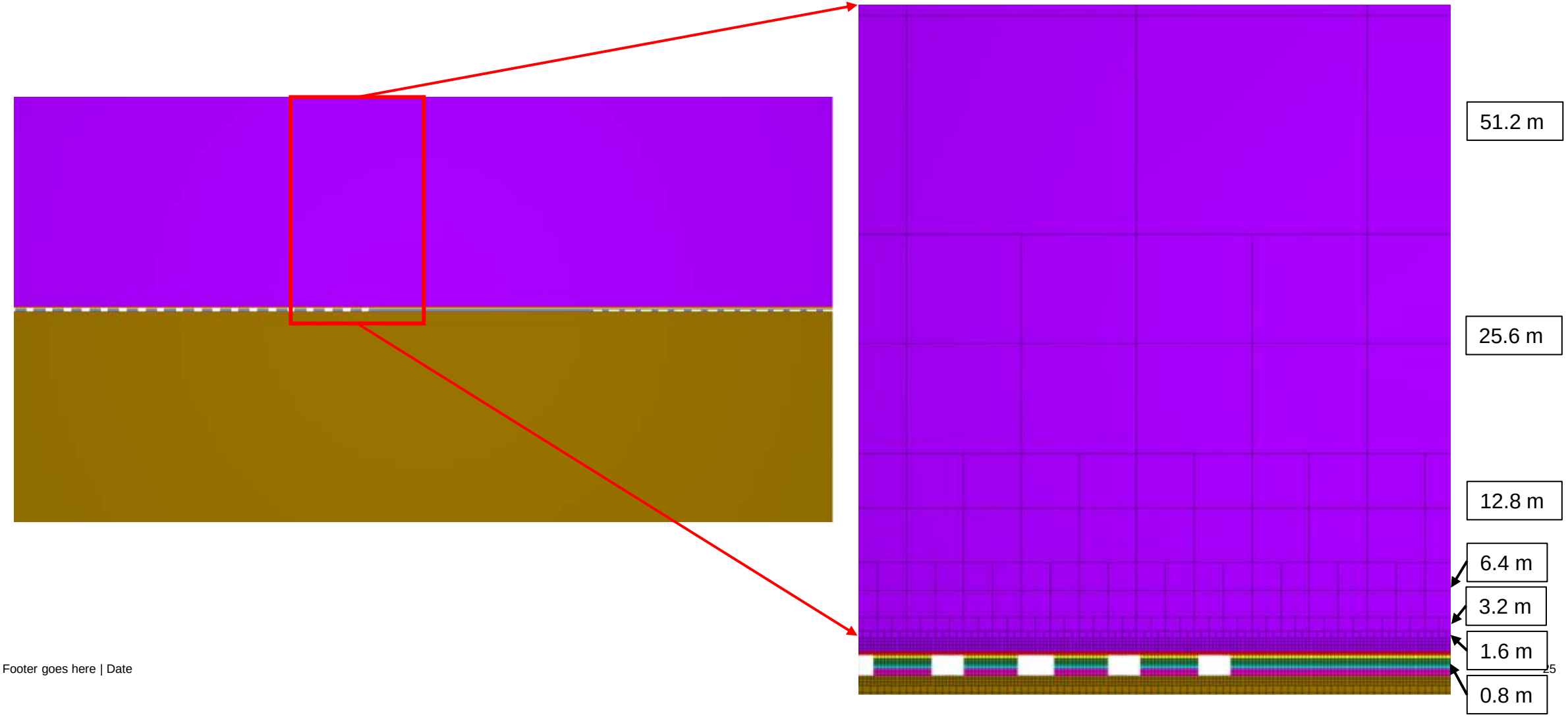
*Note: thicknesses of some units vary between stratigraphic domains



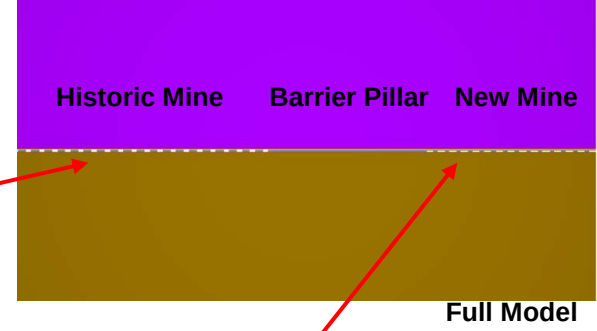
This model was developed in 2D only. Plane strain conditions are valid for long excavations (repeating pillars) and 2D models allow for shorter runtimes with higher resolution of results.

Barrier Pillar: Meshing

0.8 m mesh size near the excavation, transitions to 51.2 m near the top and bottom of the model to improve simulation runtime

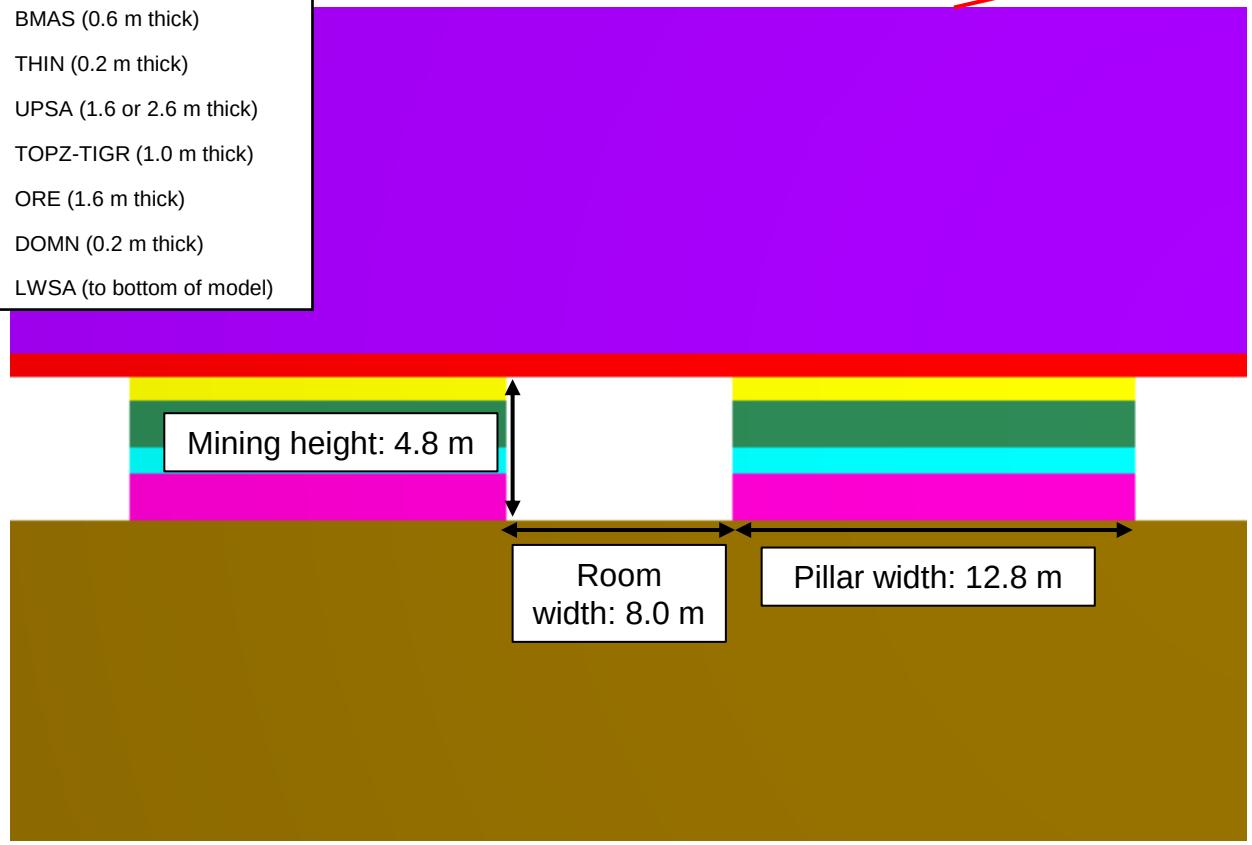


Barrier Pillar: Excavation

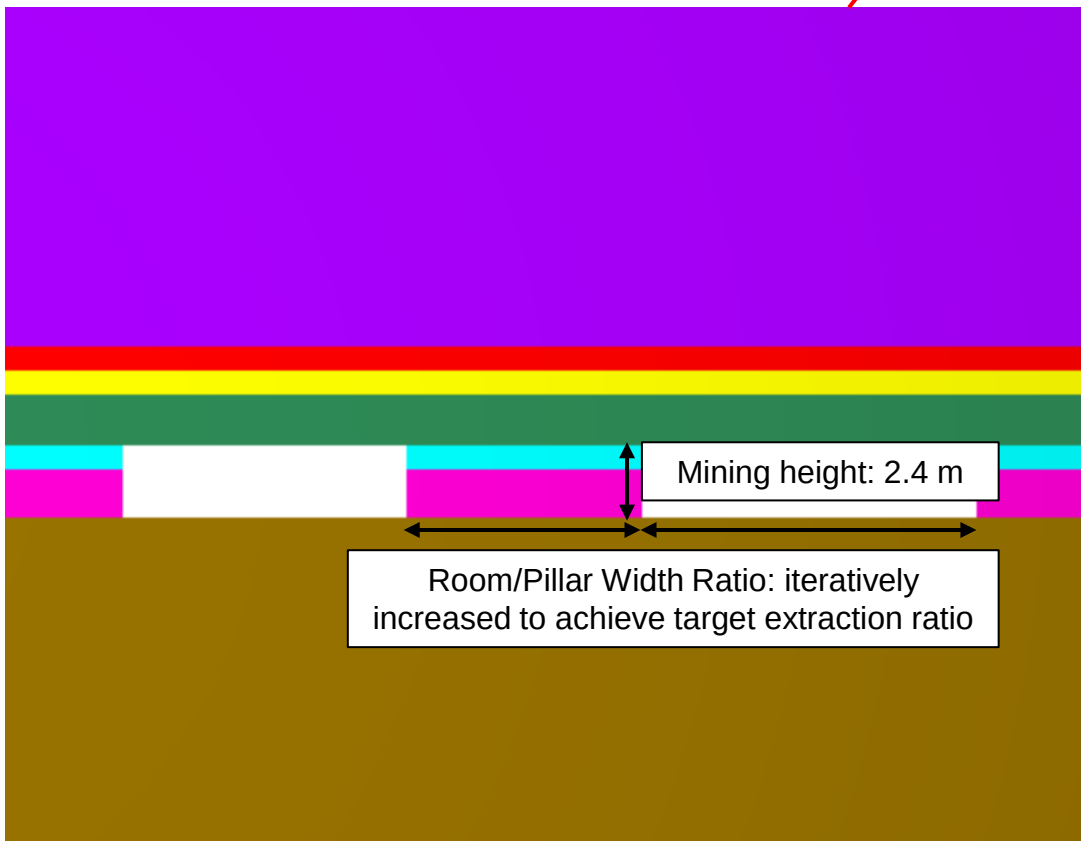


WIDE (to top of model)
UZVA (0.6 m thick)
BMAS (0.6 m thick)
THIN (0.2 m thick)
UPSA (1.6 or 2.6 m thick)
TOPZ-TIGR (1.0 m thick)
ORE (1.6 m thick)
DOMN (0.2 m thick)
LWSA (to bottom of model)

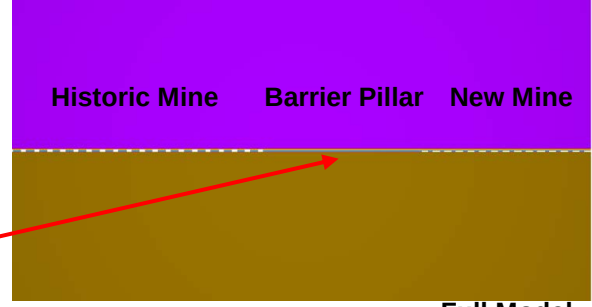
Historic Mine



New Mine



Barrier Pillar: Excavation



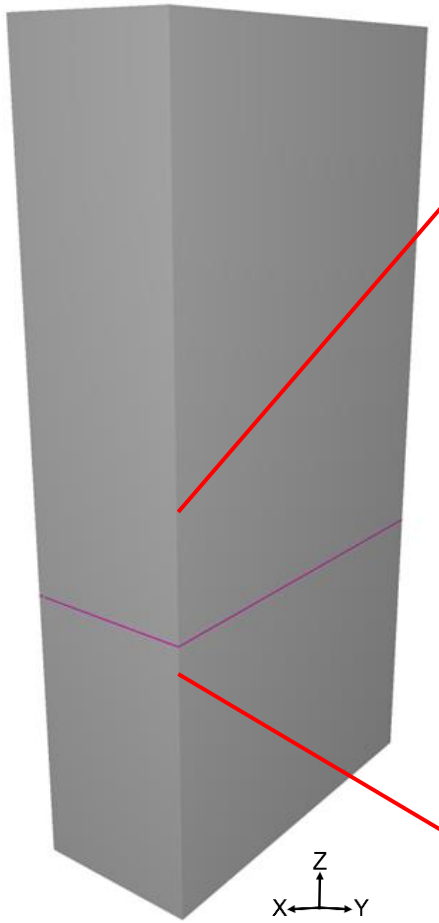
Full Model

Barrier Pillar

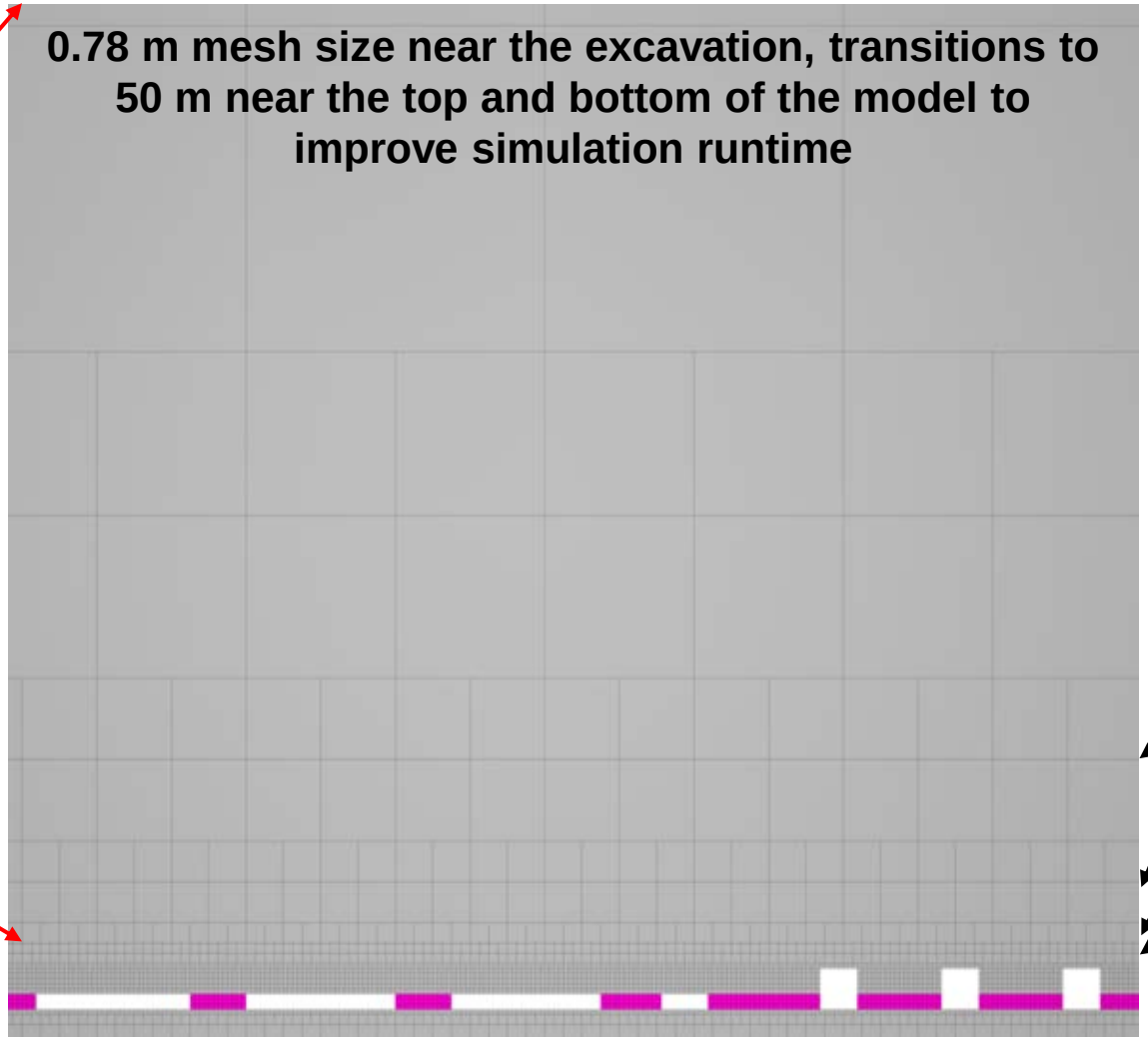
*Note: barrier pillar width varies between model runs

Panel Scale: Geometry & Meshing

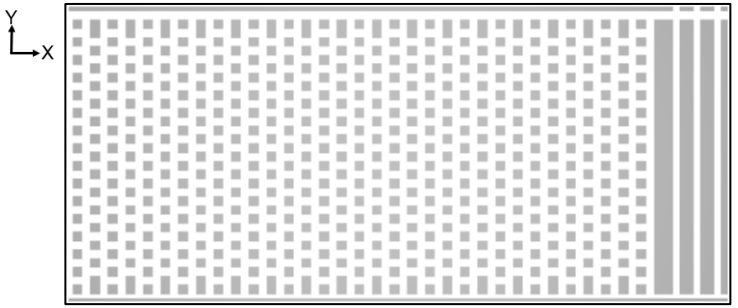
*Note: this model consists of only one material with average properties.



3D Isometric View

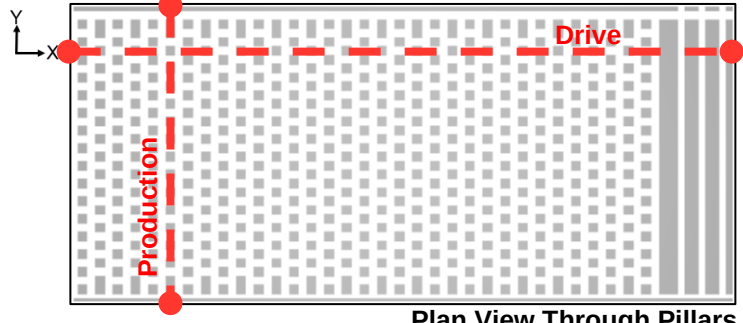


Cross Section Through Pillars



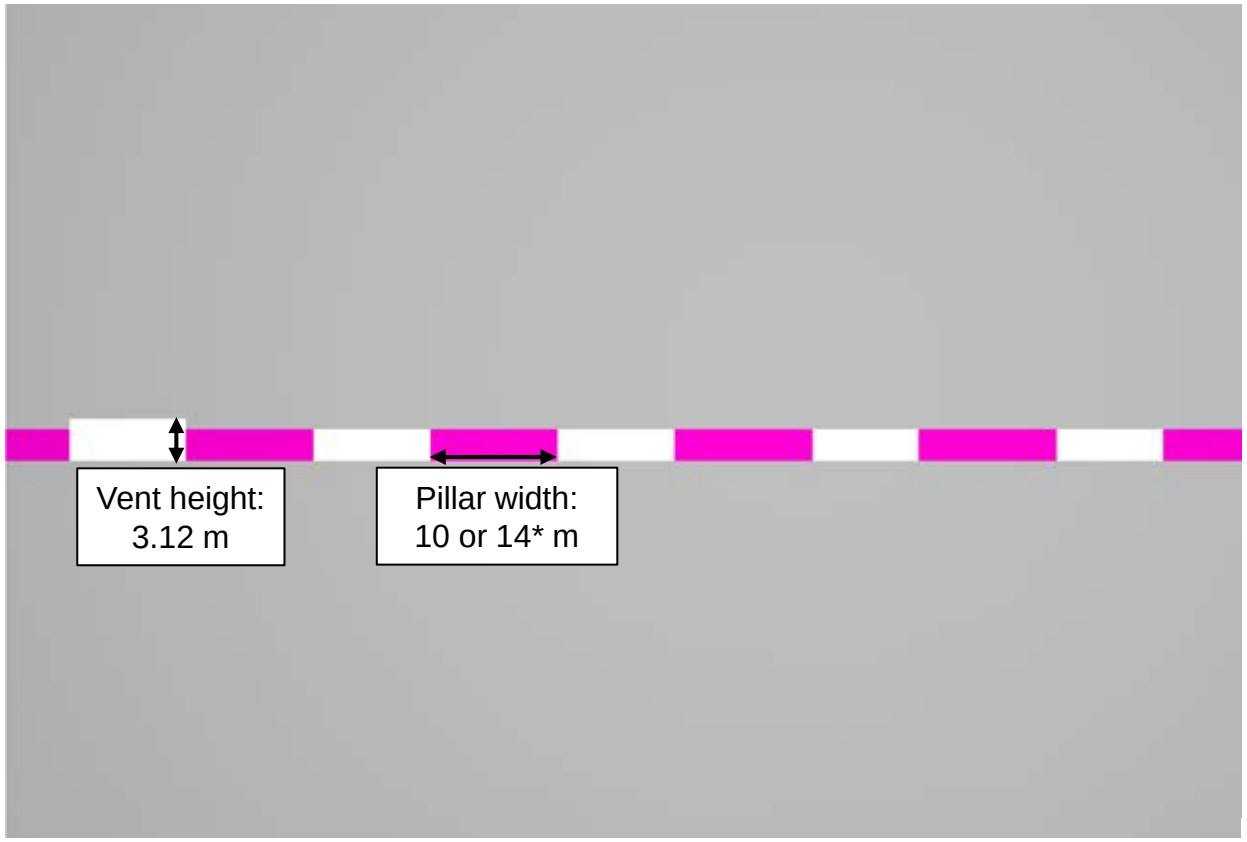
Plan View Through Pillars

Panel Scale: Excavation



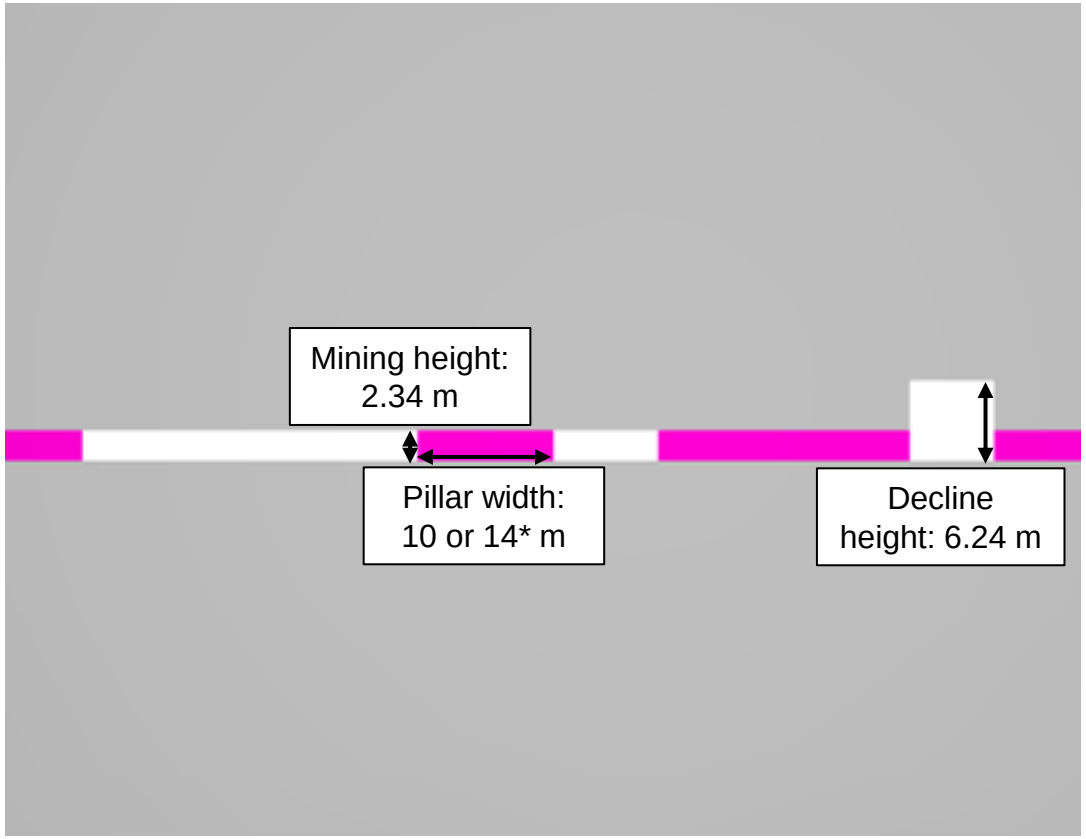
Plan View Through Pillars

Production



Cross Section Through Pillars

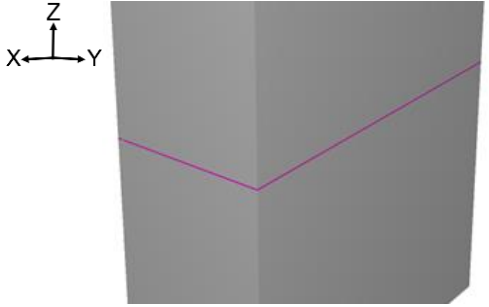
Drives



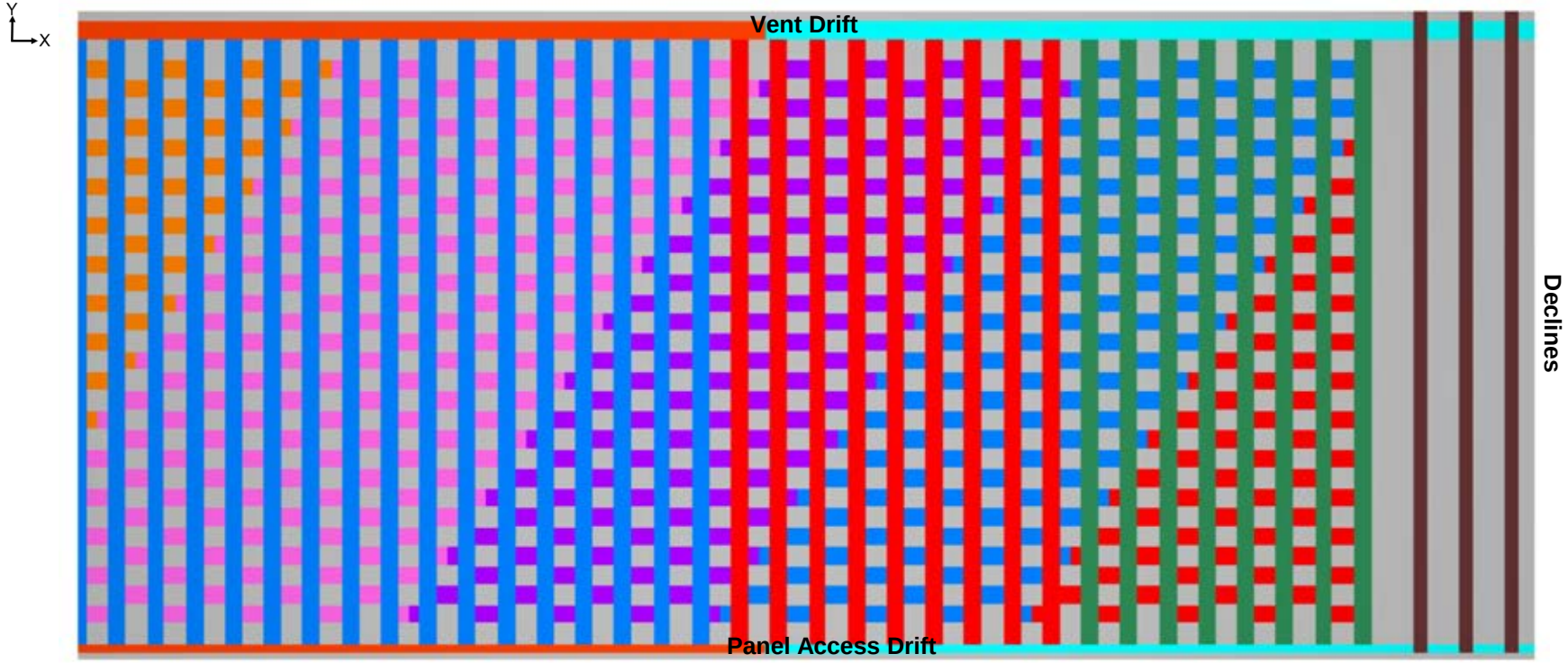
Cross Section Through Pillars

*Note: vent width and mining height vary between stratigraphic domains modeled

Panel Scale: Sequencing



3D Isometric View



Plan View Through Pillars

The excavation process is modeled as eight sequences to approximate the excavation sequencing of the declines followed by the retreat stubbing mining method in the production area that will be implemented at the White Pine North project.

wsp.com

C-2

White Pine North Project - Bedrock Deflection Assessment

TECHNICAL MEMORANDUM

DATE August 04, 2026 **Reference No.** US0055755.4008-001-TM-Rev0

TO Andrew Capes, CPG, C.Geol. – WSP USA Inc.

CC Richard Beddoes, Ross Hammett – WSP Canada Inc.

FROM James Smith and Justin Roy – WSP Canada Inc. **EMAIL james.smith@wsp.com justin.roy@wsp.com**

TECHNICAL MEMORANDUM: WHITE PINE NORTH PROJECT – BEDROCK DEFLECTION ASSESSMENT

1.0 INTRODUCTION

WSP Canada Inc. (WSP) has been retained by White Pine Copper LLC (WPC) to assess potential near-surface bedrock deflection associated with the proposed Life-of-Mine (LOM) plan for the White Pine North (WPN) Project. The proposed mining method for WPN is room and pillar. The design includes pillars that are sized to be stable in the long term, which inherently limits bedrock deflection. A separate document, titled, “White Pine North Geotechnical Study”, examines rock mechanics stability at WPN, contains the rock mass characteristics, available geotechnical data, and mine geometry recommendations, which provide input parameters for the bedrock deflection assessment presented in this Technical Memorandum (TM) (WSP, 2026).

2.0 MINING GEOMETRY

WSP was provided the proposed mine extents for the WPN Project. The mine extents were provided as a CAD file with the geometry of the declines and panel access drives explicitly digitized as wireframes. The room-and-pillar geometries were not available as explicitly digitized wireframes, but instead were provided as rectangular shapes, representing the extent and location of individual mining panels. The provided mining panels are typically represented by rectangles 300 metres (m) x 570 m; however, some are smaller based on where the economically mineable resource is located. The overall proposed mining footprint is approximately 9.6 kilometres (km) in the east-west direction and about 5.0 km in the north-south direction (Figure 1) and ranges in depth below ground surface (bgs) from ~300 m bgs up to ~1200 m bgs with part of the mining footprint beneath the existing tailings management facility (Figure 2).

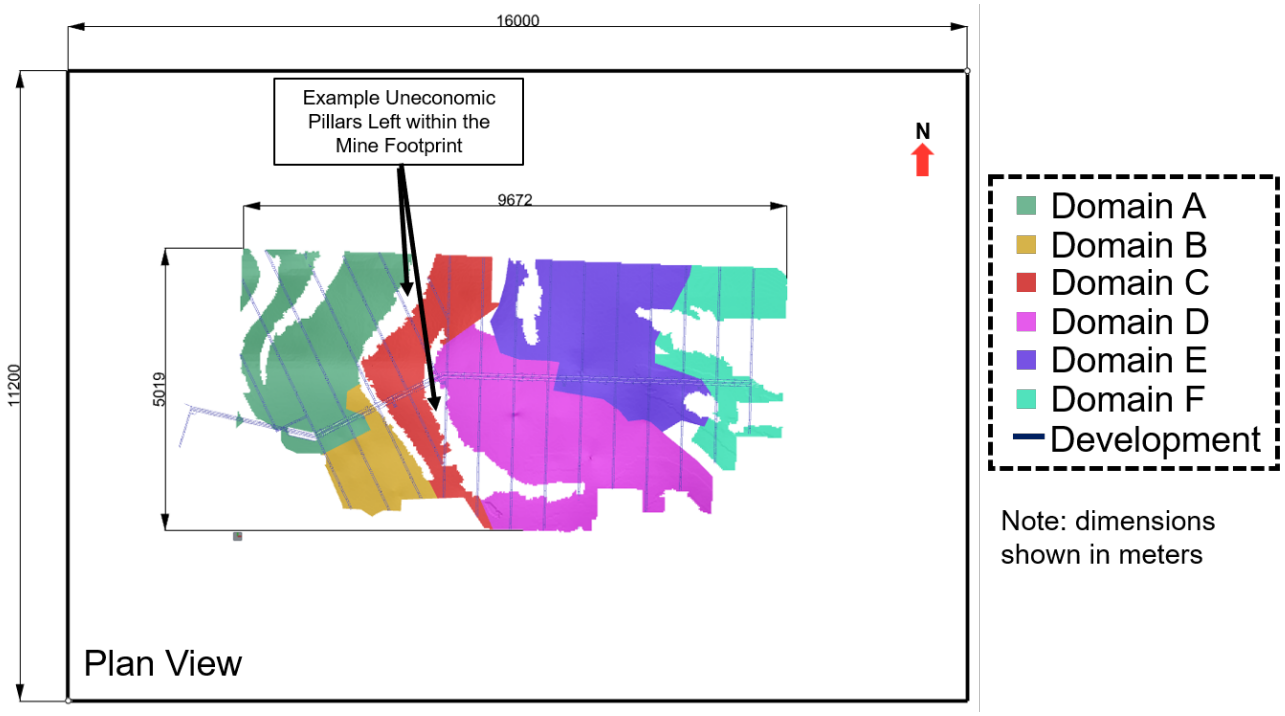


Figure 1: Plan View Showing the Extent of the Mine and Location of Each Geotechnical Domain

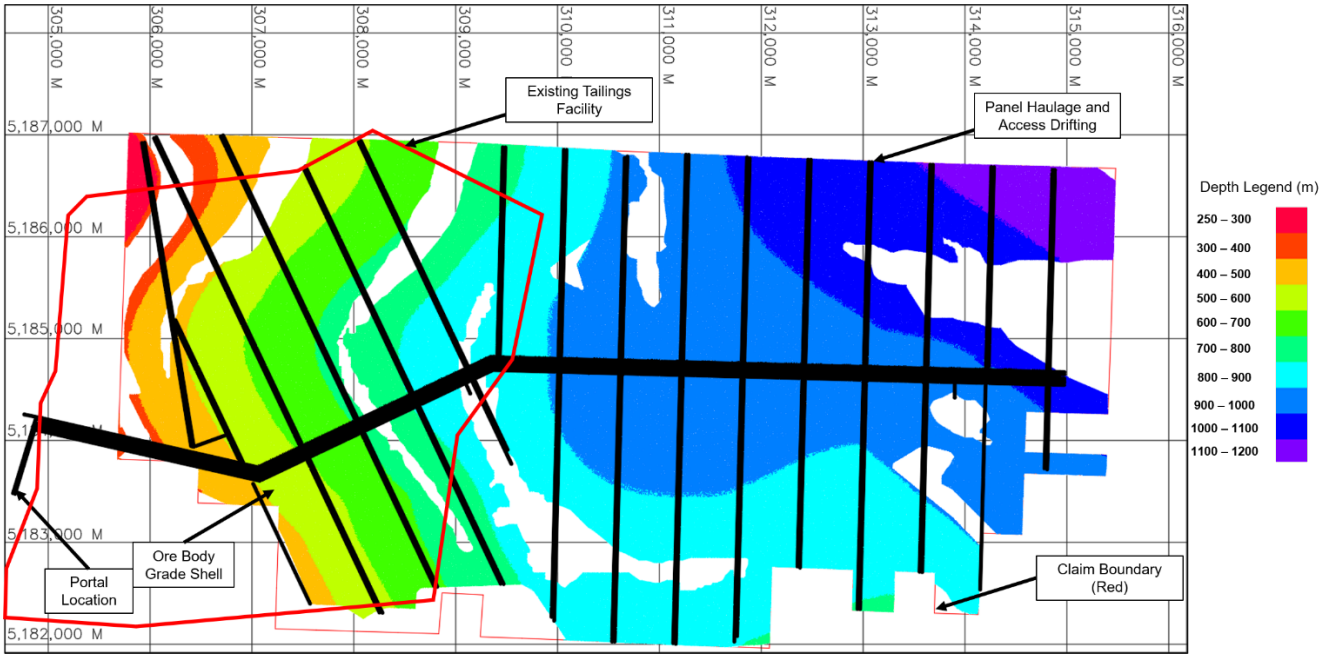


Figure 2: Plan View Showing the Grade Shell Contoured by Depth Increments (m bgs) and the Tailings Facility Footprint

The room-and-pillar geometries planned within each panel were defined to maintain stable pillars to facilitate ore recovery, mine access, and maintain long-term mine stability (WSP, 2026). The room-and-pillar geometries and associated extraction ratio (defined as the plan area extraction ratio) vary by geotechnical domains (Table 1). The

geotechnical domains are defined by the depth to the mining horizon, the dip of the ore body, and the thickness of certain lithological units within the backs and pillars of the room (WSP, 2026).

Figure 1 shows the spatial extent of the geotechnical domains and the size of the proposed mine extents. Figure 3 shows the location and size of individual panels with two panels highlighted to show examples of full panel sizes and truncated panel sizes.

Table 1. Proposed Room and Pillar Dimensions by Geotechnical Domain

Geotechnical Domain	Depth (m)	Mining Height (m)	Pillar width (m)	Room Width (m)	Plan Area Extraction Ratio (%)
A	400 to 750	2	8	8	75
B	400 to 750	2	8	8	75
C	750 to 900	2	10	8	69
D	750 to 900	2	10	8	69
E	900 to 1100	2.5	12	8	64
F	900 to 1150	2.5	14	8	60

Source: WSP, 2026

Note: Panels outlined in red represent examples of full and truncated panels.

Figure 3: Plan View Showing Individual Panels (Green / Brown) for the Entire Mine Extent

3.0 BEDROCK DEFLECTION ASSESSMENT

The bedrock deflection assessment approximates the vertical displacement of the rock mass overlying the mine in response to mining-induced loading using the numerical modelling software FLAC3D V7 (Itasca, 2019), a three-dimensional (3D), finite difference-based, stress-displacement software program. Bedrock deflection, for the purposes of this assessment, is defined as the vertical displacement at the bedrock / overburden contact, represented by the top of the model mesh. The soil overburden thickness (approximately ranging from 2 m to 20 m) is not included in the assessment.

Estimates of bedrock deflection presented in this section rely on parameters that are calculated by scaling the intact rock parameters measured in the laboratory using empirical relationships derived from rock mass quality observed in drill holes. These parameters are approximate. For the relatively small magnitudes of bedrock displacement expected at WPN (see Section 3.3.3 and 4.0), the uncertainty in the predictions is of a similar magnitude to the bedrock deflection itself.

Two numerical assessments were produced that provide an estimate of bedrock deflection. The first assessment is a non-elastic panel scale model that examines a 2.5-km-long panel at depths of 900 m and 1,100 m (WSP, 2026). This assessment was developed to examine pillar stability and produce an estimation of bedrock deflection (WSP, 2026). The second assessment is an elastic mine scale model that simulates the excavation of the entire mine following the proposed Life-of-Mine (LOM) sequence; this assessment was developed specifically for this bedrock deflection assessment.

3.1 In-situ Stress Scenarios

A range of magnitudes and orientations for the in-situ stress field was considered when completing stability assessments for the WPN Geotechnical Study (Table 2) (WSP, 2026). The in-situ stress scenarios were developed based upon regional datasets, stress testing at the historic White Pine mine, and local breakout data (WSP, 2026). The panel scale models considered scenarios where the ratio of the maximum horizonal stress to the vertical overburden stress (KH) is 2.0 and 2.5 and oriented parallel to the primary ore drives (WSP, 2026). The ratio of the minor horizontal stress to the vertical overburden stress (Kh) was assumed as 1.5 in each case. The mine scale models considered the four in-situ stress scenarios summarized in Table 2.

Table 2: In-situ Stress Scenarios Considered in the WPN Geotechnical Study

Stress Scenario	KH	Kh	KH Orientation
1	2.5	1.5	EW
2	2.5	1.5	NS
3	2.0	1.5	EW
4	2.0	1.5	NS

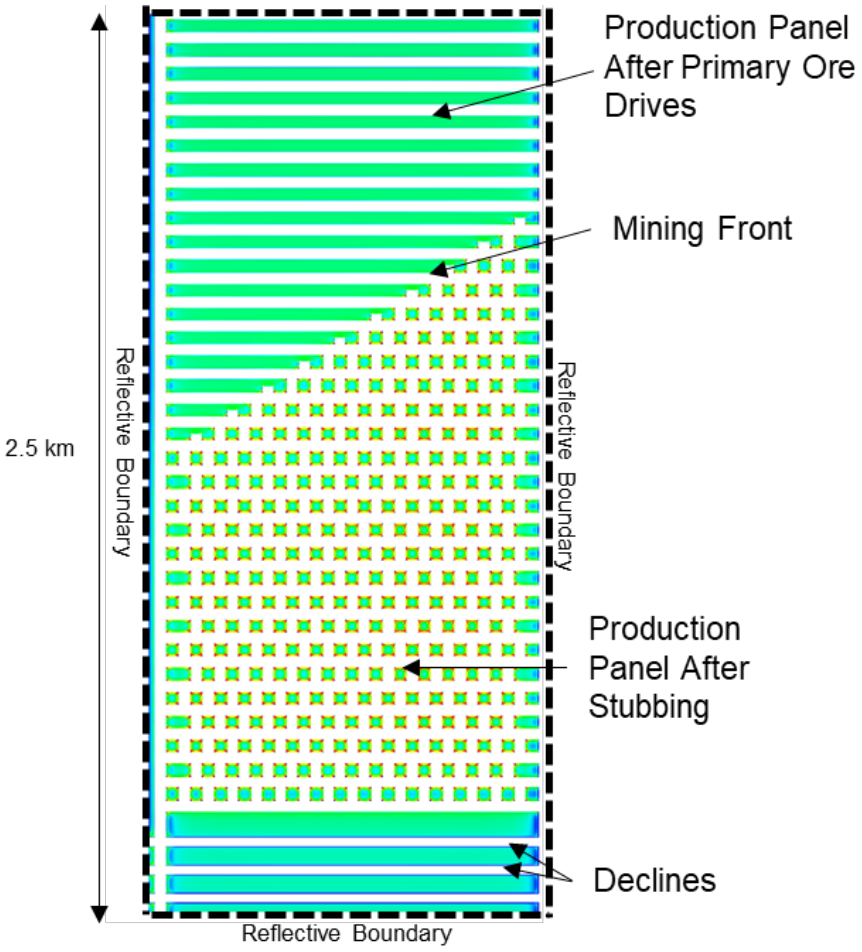
Source: (WSP, 2026)

Note: Minimum principal stress (S3) is assumed to be oriented vertically and defined by the lithostatic load (0.027 MPa/m).

3.2 Panel Scale Model

A panel scale model was assessed for geotechnical Domain D (at a depth of 900 m) and geotechnical Domain F (at a depth of 1,100 m) to examine pillar stability after mining, to investigate load redistribution that occurs near the mining front, and to estimate bedrock deflection. Figure 4 shows the setup of the panel scale models at an intermediate mining stage (i.e., before the panel is fully mined). The model considers a flat (horizontal) panel

spanning 2.5 km by 250 m with reflecting boundaries simulating a panel surrounded by other panels (also horizontal and at the same depth). Additional context and details on the assessment input parameters (in-situ stress, strength parameters, and elastic properties), model construction, and stability conclusions of the panel scale model are presented in the WPN Geotechnical Study (WSP, 2026). This Report section specifically discusses the bedrock deflection.

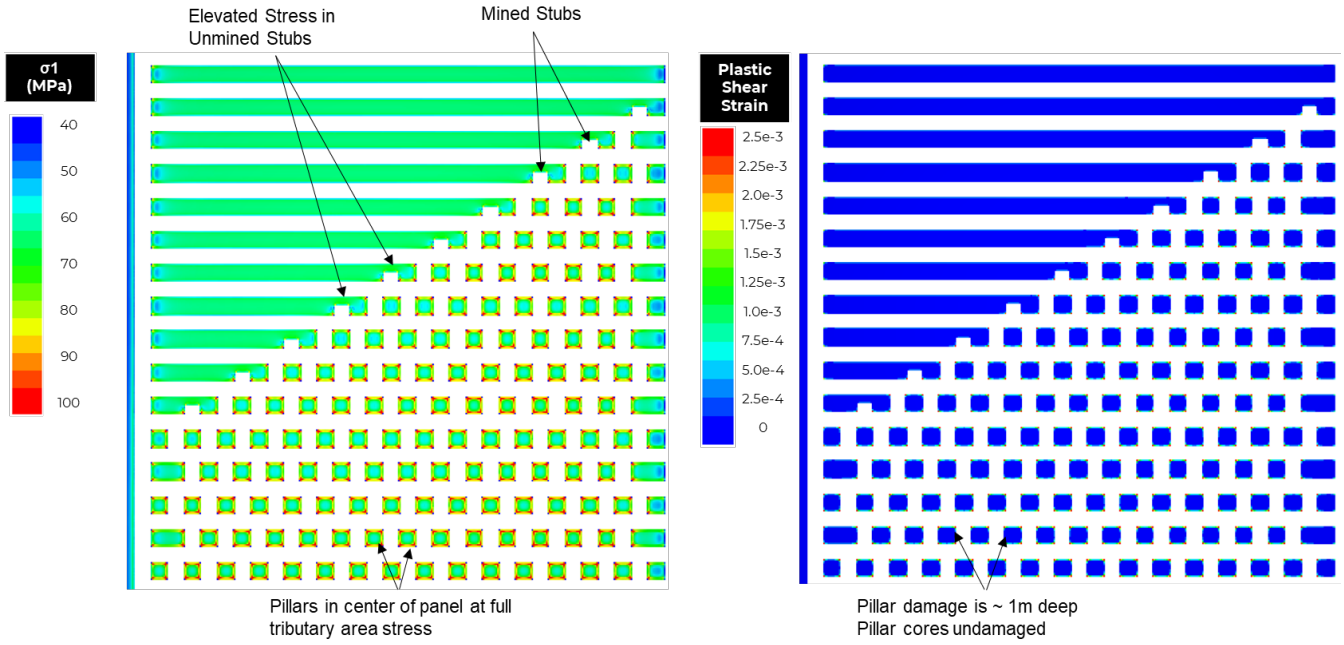


Note: An intermediate model step is shown (i.e., approximately 60% of the panel is extracted).

Figure 4: Example Setup of the Panel Scale Models Showing the Reflective Boundaries and the Mining Sequence

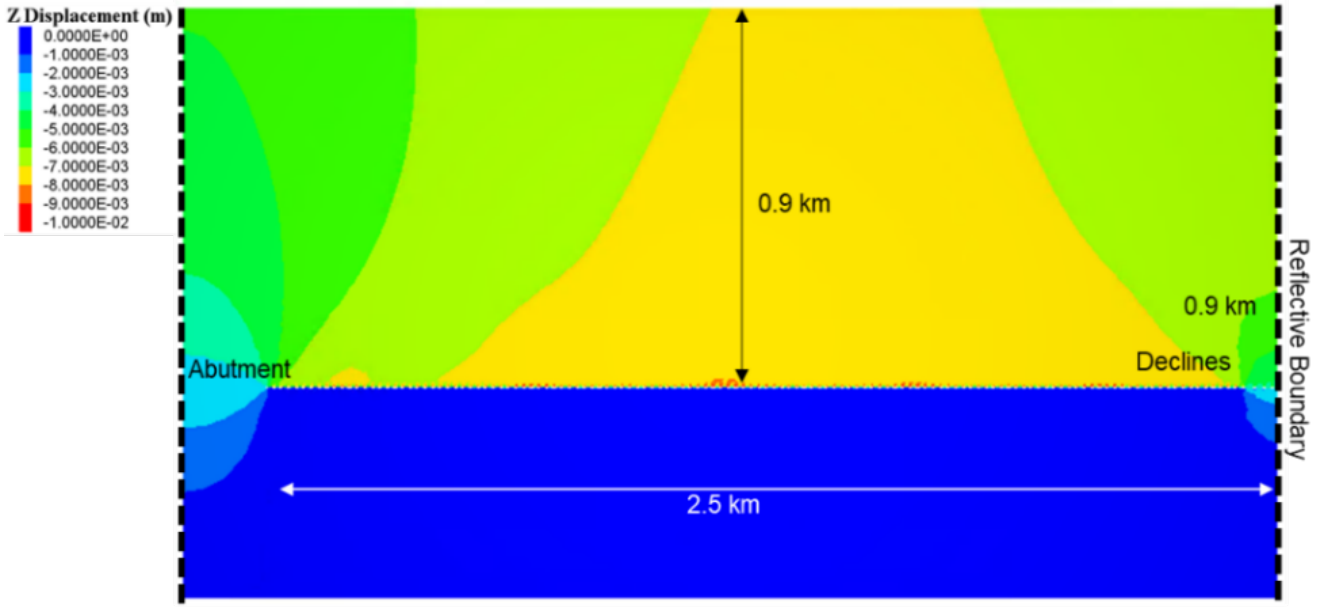
The panel scale model helps to understand pillar stability at a macro scale and how it relates to bedrock deflection (example for geotechnical Domain D shown in Figure 5). The panel scale models indicate that the pillars remain stable and only localized stress damage to pillar edges and room backs is expected (WSP, 2026). Bedrock deflection estimated from the panel scale models is limited to small magnitudes of elastic displacement (i.e., less than 1 centimetre (cm) of vertical displacement). These maximum vertical displacements will likely be experienced near the center of the panels and will gradually diminish toward the abutments (e.g., Figure 6 shows up to 8 millimetre [mm] of vertical displacement for the panel scale model of geotechnical Domain D after completion of mining and the left edge of the model shows reduced deformations over the abutment pillar). The rock mass input parameters and model construction techniques that produced this model are contained in the WPN Geotechnical Study (WSP, 2026) and are not repeated herein other than to highlight that the densities and mechanical

properties of rock masses may be influenced by slight changes in porosity due to dewatering. These effects have not been considered in the model but the impact on the model results is expected to be negligible.



Note: Showing the maximum stress and plastic shear strain contours.
An intermediate model step is shown (i.e., approximately 60% of the panel is extracted). KH is 2.5 oriented parallel to the primary ore drives.

Figure 5: Example Stress and Strain Output from the Panel Scale Model for Geotechnical Domain D



Note: Showing vertical displacements (negative = down).
Vertical displacements contours are shown at full panel extraction. KH is 2.5 oriented parallel to the primary ore drives.

Figure 6: Vertical Section through Geotechnical Domain D Panel Scale Model

3.3 Mine Scale Model

A mine scale numerical model was developed to simulate the entire LOM plan, considering the variable depth, dip, and areas planned for extraction. Due to the scale of the proposed mine plan (~10 km by ~5 km) the numerical model uses an octree mesh to allow for useable file sizes. An octree mesh represents the geometry using cubic zones with dimensions that increase as they retreat from the mining horizon. To simulate the full extent of the mine in a computationally efficient way, a minimum zone size of 2.5 m was used, regardless of room height; this results in a larger room height than proposed in the mine plan for geotechnical Domains A to D (proposed in the mine plan as a 2-m room height), which is conservative for the assessment of bedrock deflection.

The mine scale fault wireframes from the available structural model (SRK, 2025) were not included in the model due to the inability to correlate the wireframes with changes in the rock mass characteristics (as described in WSP, 2026). Deformation along structures not included in the model may locally impact the bedrock deflection forecasts.

Figure 7 shows an example of the room-and-pillar geometry simulated in the numerical model. Figure 7 (left) shows the excavations represented in section view. Figure 7 (right) shows the room-and-pillar geometry assigned in the model and represented using the cubic zones. The cubic zones result in irregular pillar shapes and a ‘staircase’ shape along areas where the ore body is dipping. The resulting irregular pillar shapes were reviewed for each panel to confirm that, at the panel scale, the planned extraction ratio (Table 1) is appropriately represented by the model geometry.

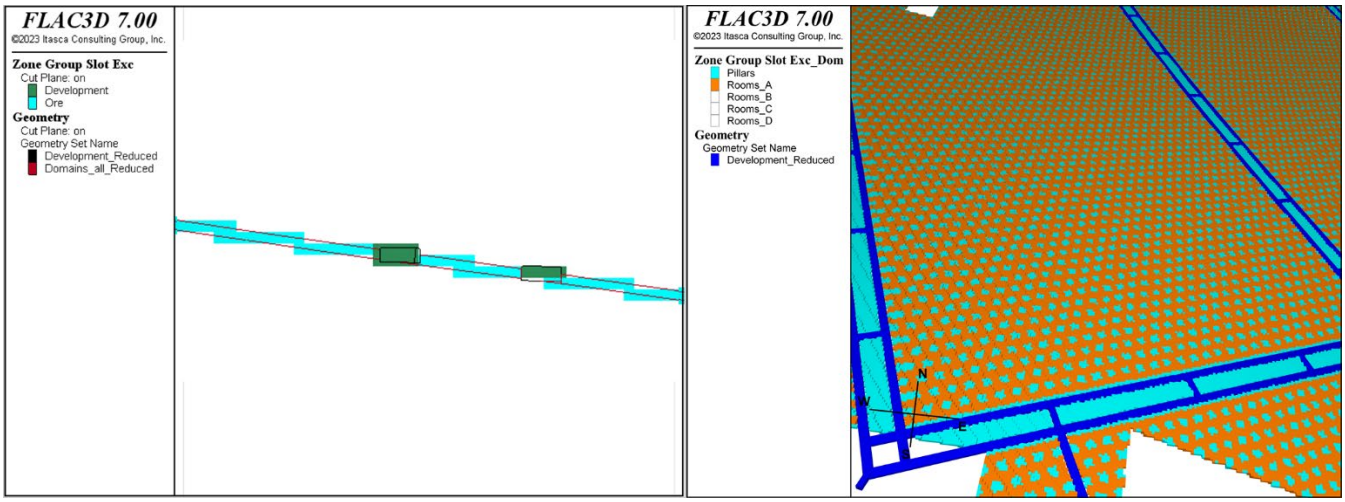


Figure 7: Modelled Geometry Showing a Cross-Section of the Dipping Ore Body (Left) and the Pillar Geometry in Perspective View (Right)

3.3.1 Sequence

The mining sequence was provided in monthly increments from October 2027 to December 2052. This sequence was simplified into six mining stages for the mine scale numerical model. The LOM sequence is presented in Figure 8 (left), where consecutive numbers from 1 to 240 represent the order in which the 240 panels are extracted. The simplified sequence used in the numerical model is presented in Figure 8 (right), representing the following time frames:

- Stage 1 = October 2027 to January 2031

- Stage 2 = January 2031 to May 2037
- Stage 3 = May 2037 to November 2041
- Stage 4 = November 2041 to November 2045
- Stage 5 = November 2045 to November 2048
- Stage 6 = November 2048 to December 2052

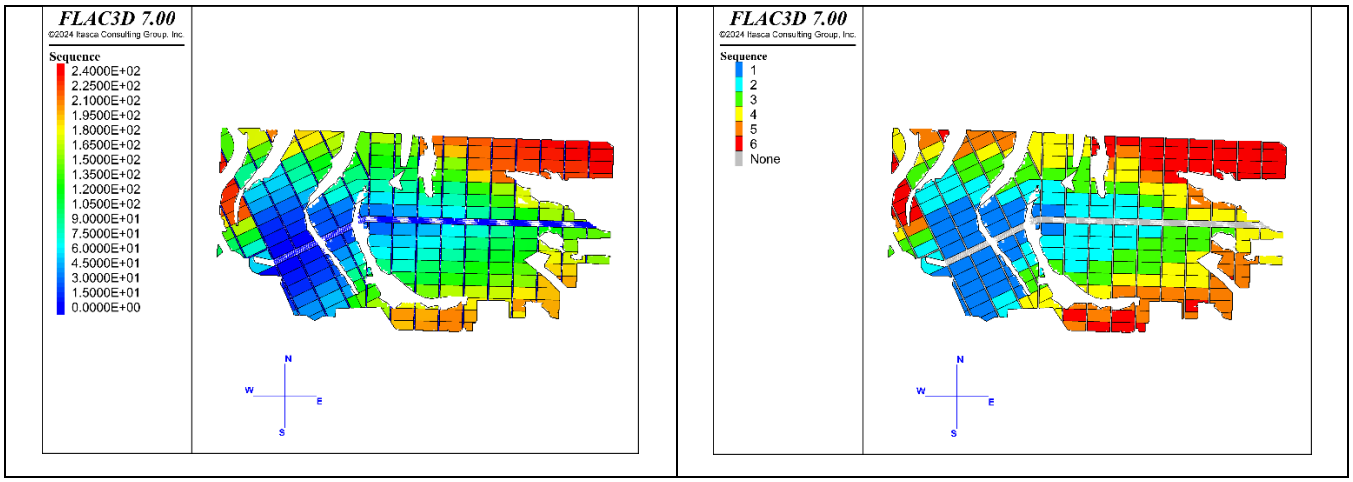


Figure 8: LOM Excavation Sequence, by Panel (Left) and Simplified Sequence Used in the Numerical Model (Right)

3.3.2 Elastic Properties

The mine scale model was simulated using an elastic approach and considers the crown rock mass as a single material. This approach is reasonable as the non-elastic panel scale models exhibited only small amounts of plastic strain localized within a few meters around the rooms, suggesting that most bedrock deflection response is likely to be elastic.

The available laboratory elastic modulus testing is located within and near the mining horizon and the lithologies tested (predominantly siltstones, shales, and sandstones) were interpreted to have similar Young's moduli, Poisson's ratios, and density values (WSP, 2026). No laboratory testing of rock elastic properties was completed within the crown rock masses at the time of this bedrock deflection assessment; although, the lithologies through the crown are understood to remain a mixture of siltstones, shales, and sandstones (WSP, 2026); and thus, can be expected to have similar elastic properties. Similar to the panel scale model, the influence of slight porosity changes due to dewatering were not considered in the model; the impact of porosity changes on the model results is expected to be negligible.

The mine scale model was assessed assuming a single set of elastic properties for the crown rock mass that are similar to those measured near the mining horizon. Two model scenarios were completed, one considering the intact Young's moduli and one considering the rock mass moduli. The latter accounts for the effects of natural fractures in the rock mass. The assumed intact elastic properties used in the assessment are presented in Table 3.

Table 3: Assumed Crown Rock Mass Elastic Properties and Density

	Young's Modulus (GPa)	Poisson Ratio	Density (kg/m³)
Intact Rock	45	0.3	2,700

The rock mass Young's modulus (E_{rm}) was estimated using the generalized Hoek and Diederichs (2006) formulation:

$$E_{rm} = E_i \left(0.02 + \frac{1 - \frac{D}{2}}{1 + e^{\left(\frac{60+15D-GSI}{11}\right)}} \right)$$

Where, Ei=Intact rock modulus

GSI=Geological Strength Index

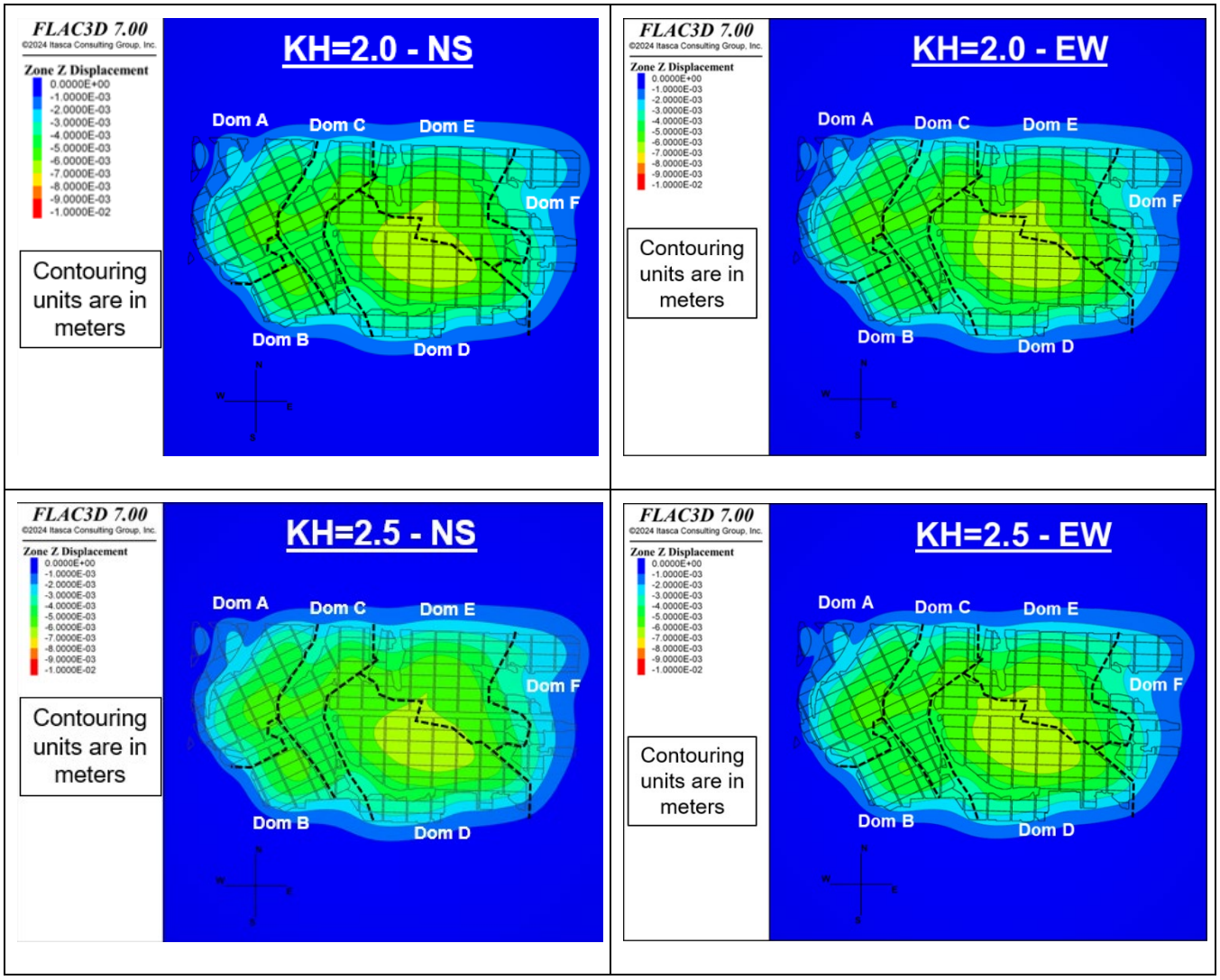
D=Disturbance Factor

Considering the intact Young's modulus (E_i) of 45 GPa, a GSI of 80 (broadly representative of the average crown rock mass character encountered during geotechnical logging), and no disturbance (D = 0), the estimated rock mass Young's modulus is approximately E_{rm} = 39.2 GPa. The close similarity between these values reflects the very limited number of fractures in the rock mass.

3.3.3 Model Results

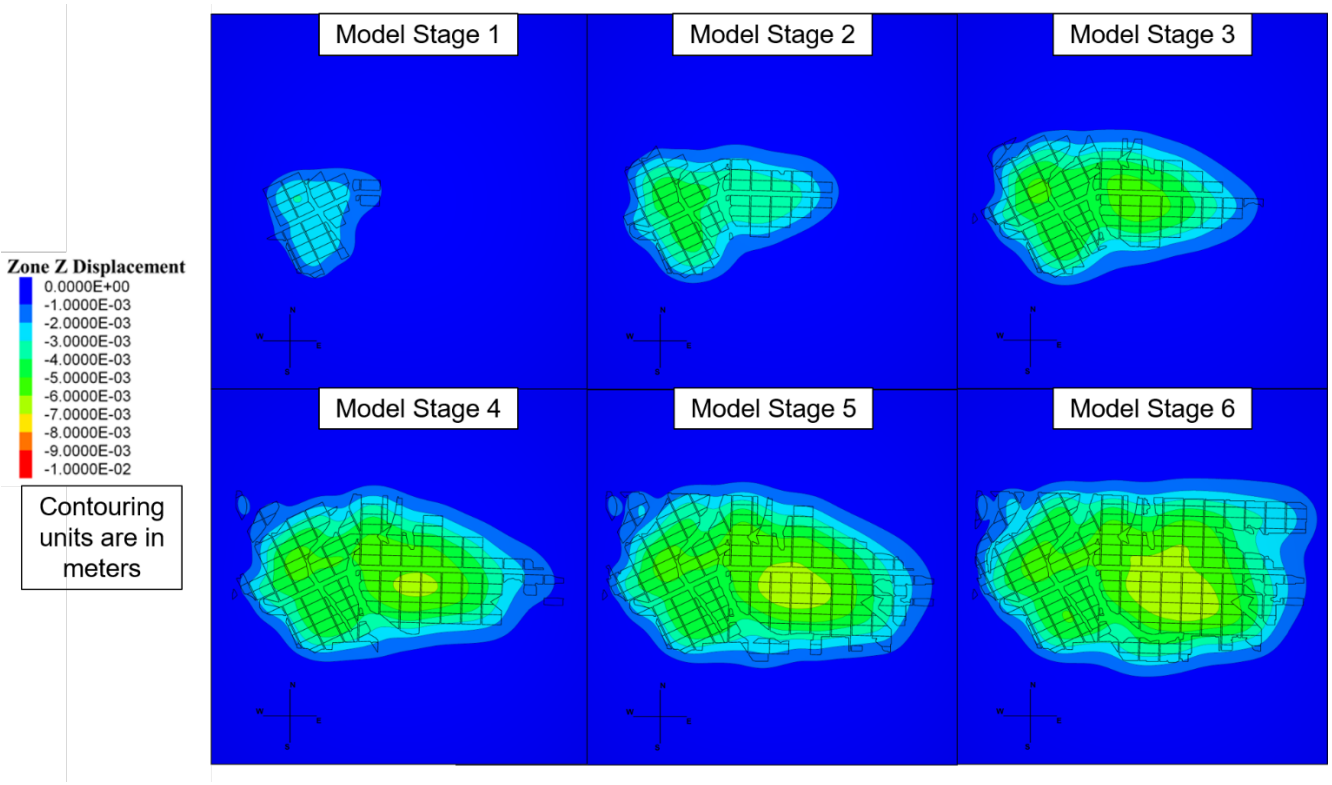
Due to the sparse natural fracturing in the rock masses, bedrock deflection was initially estimated considering the intact Young's modulus for the four in-situ stress scenarios from Section 3.1 (Table 3). From these scenarios, the maximum bedrock deflection (between 6 mm and 7 mm) is located above geotechnical Domains D and E, corresponding to the deeper eastern portion of the mine layout, the later stages of extraction, and at a depth consistent with the panel scale models. The simulated bedrock deflection (represented by contouring of vertical displacement) presented in this section is intended to represent the general pattern and magnitude of bedrock deflection across the Project area.

From Figure 9, it can be observed that the stress scenarios assessed show similar bedrock deflection patterns and comparable maximum displacement magnitudes over the mining footprint. Changing the orientation of the maximum horizontal stress from east-west to north-south produces localized differences in the distribution of displacement, mainly in the southwest portion of the model near Domain B. These differences do not materially change the maximum estimated bedrock deflection. An example (considering in-situ stress Scenario 1) of bedrock deflection during each model stage is shown in Figure 10.



Note: Intact Elastic Modulus Scenario Shown

Figure 9: Plan View Showing Estimated Bedrock Deflection (i.e., Vertical Displacement Contours) at Full Extraction for the Stress Scenarios Assessed

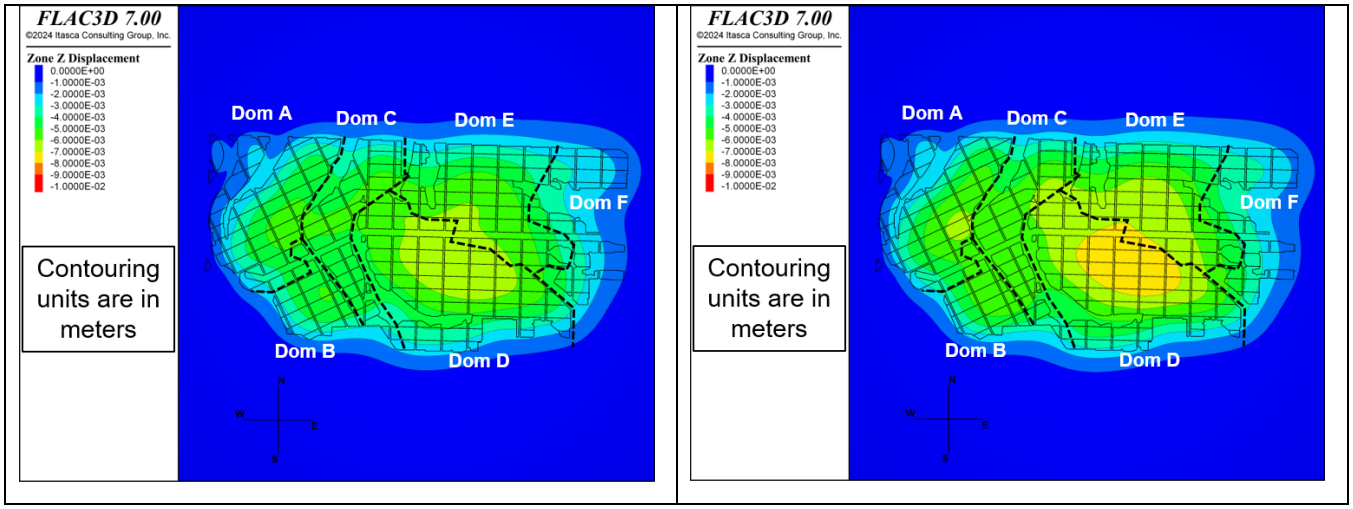


Note: Intact elastic modulus scenario and in-situ stress Scenario 1 shown.

Figure 10: Plan View Showing Estimated Bedrock Deflection (i.e., Vertical Displacement Contours) for Each Modelled Mining Stage

From the in-situ stress scenarios assessed with the intact Young's modulus, Scenario 1 (KH 2.5 oriented EW), showed the largest area of maximum bedrock deflection (between 6 mm and 7 mm). To evaluate the influence of the sparse natural fracturing in the crown pillar, in-situ stress Scenario 1 was assessed using a more conservative rock mass modulus. The rock mass modulus represents the elastic response of the rock mass, under induced stress, considering displacement associated with both intact rock blocks and open structure and will produce larger displacements than the intact Young's modulus. Due to the scarcity of naturally open structure observed in the crown pillar, particularly the lack of horizontally orientated fractures, the rock mass modulus case represents a conservative upper bound for bedrock deflection.

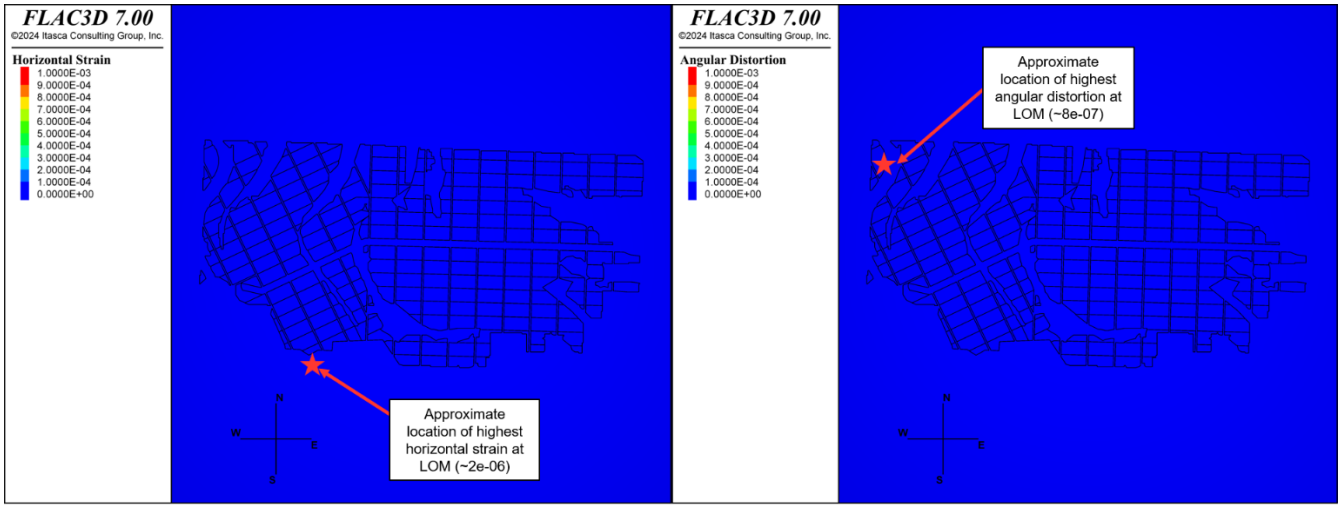
The results of the rock mass modulus model, shown at the end of mining in Figure 11, indicate that the bedrock deflection magnitude increases by approximately 1 mm relative to the assessments using the intact Young's modulus. The distribution and influence of the bedrock deflection is otherwise similar. For bedrock deflection interpretation at the WPN Project, the model scenario considering the rock mass modulus is considered the upper bound. Given current lack of detailed laboratory data and fracture characteristics for the full overburden rock mass, it can reasonably be concluded that the maximum vertical deformation will be less than 1 cm.



Intact Rock Elastic Modulus **Rock Mass Elastic Modulus**

Figure 11: Comparison of Modelled Bedrock Deflection (i.e., Vertical Displacement Contours) for the Intact Rock Modulus and Rock Mass Modulus Cases for In-Situ Stress Scenario 1

Relatively uniform vertical deflection is not problematic for structures; however, differential settlement can cause damage depending on its magnitude. In consideration of potential impacts to surface infrastructure, horizontal strain and angular distortion have also been calculated at the bedrock surface. For context, both parameters (angular distortion and horizontal strain) are unitless. The results of the models suggest that very minimal horizontal strain and angular distortion are forecast (less than 1e-05, Figure 12). The estimated magnitudes of strain and distortion are two orders of magnitude less than those typically associated with potential to cause damage to structures (typically > 1e-3; Harrison, 2011). Compared to seasonal fluctuations in near surface soils due to freeze / thaw and changes in saturation, the mining induced deflection is not expected to be observed.



Horizontal Strain **Angular Distortion**

Notes:

- Both angular distortion and horizontal strain are unitless parameters.
- Locations and magnitudes of maximum horizontal strain and angular distortion are shown on the figure. Both estimates are below 1e-05.

Figure 12: Plan View of Horizontal Strain (Left) and Angular Distortion (Right) at the End of Mine Life

3.3.4 Monitoring Considerations

The bedrock deflection forecast contained in this document is conditional on stable pillar cores at the mining horizon and is thus related to small magnitudes of elastic displacement of the pillars. Typical surface monitoring solutions (e.g., LiDAR surface scans) would be influenced by seasonal variations in the overburden elevation and would be unable to detect the mm scale bedrock deflection forecasted in this assessment.

As the bedrock deflection forecast is conditional to stable pillar cores and related to elastic deformation of the pillars at the mining level, typical operational monitoring of pillar performance can be used to determine that the bedrock deflection is likely to be within the forecasted range. If pillar performance differs from the interpreted range of pillar behaviour, mitigation options can be implemented to arrest further deterioration of the pillars; thus, mitigating risk of uncontrolled bedrock deflection.

Uncontrolled progressive ‘domino style’ collapse of pillars is not expected at WPN due to the progressive mining sequence and the squat pillar design which is not susceptible to rapid strength loss compared to mines employing slender pillars. Thus, the operations team will have time to respond to changes in pillar behaviour and to alter mining plans locally, as required, to maintain stable pillar cores (e.g., mitigation options include reducing extraction ratios locally, placement of waste rock around pillars, installation of additional ground support, etc.).

4.0 CONCLUSIONS

The numerical assessments presented in this document were developed considering rock mass characteristics and assumptions associated with the WPN Geotechnical Study (WSP, 2026). The results of the assessments suggest that the bedrock deflection at the WPN Project at the end of LOM is consists of small magnitudes of elastic displacement (i.e., less than 1 cm of vertical displacement), with the greatest deflection localized above Domains D and E, near the second half of the mining sequence. The in-situ stress scenarios evaluated produced localized differences in the distribution of bedrock deflection; however, the maximum estimated bedrock deflection remained similar.

5.0 CLOSING

WSP is grateful for the opportunity to provide this TM. Please contact the personnel with any questions or comments.

WSP Canada Inc.

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Principal Rock Mechanics Engineer

James Smith, P.Eng. (ON)
Principal Rock Mechanics Engineer

LS/JS/wk

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C-3

Evaluation of White Pine North Mine Subsidence on Tailings Storage Facility, Groundwater Wells, and Neighboring Surface Structures

TETRA TECH

Memo

To:	For File
From:	Chad Tomlinson and Guy Roemer
Date:	7/27/2026
Subject:	Evaluation of White Pine North Mine Subsidence on Tailings Storage Facility, Groundwater Wells, and Neighboring Surface Structures

Introduction

White Pine Copper, LLC (WPC) has engaged WSP to evaluate the potential extent of bedrock subsidence due to the White Pine North (WPN) mine workings. Based on a review of WSP's WPN Bedrock Deflection Assessment (WSP, 2026), subsidence at the top of the bedrock was modeled to be a maximum of ~6 to 7 mm (0.28 inches). The subsidence contouring based on WSP's modeling overlaid onto a site map of the mine and adjacent properties are presented in **Figure 1.** As shown in **Figure 1**, the mine infrastructure, including the tailings disposal facilities (TDFs) is outside of the modeled area of maximum subsidence.

WPC has requested that Tetra Tech evaluate the potential impacts from this subsidence on surface and near surface onsite (mine) and adjacent offsite infrastructure. A more conservative 2 cm (0.78 in) of subsidence was used in this analysis to assess the potential impacts to surface infrastructure including:

- TDFs.
- Neighboring offsite foundations.
- Groundwater wells.

Due to the de minimis estimated subsidence at the top of the bedrock (directly below the overburden), additional modeling was not conducted by Tetra Tech to predict how the estimated bedrock subsidence will propagate to the surface due to the limitations regarding accuracy of analytical methods and models. Although some attenuation of subsidence is expected, this evaluation assumes the full subsidence of 2 cm will propagate to the surface. To put the estimated surface subsidence into context, typical tolerances set forth by the United States Department of Agriculture (USDA) for earthworks allow for gradual changes of plus or minus 9.1 cm (3.6 inches) from plan and grade for finished fill slopes and plus 3 cm (1.2 inches) for top of fills for dams, dikes, and levees (USDA, 2025). These industry accepted tolerances are greater than the predicted subsidence 2 cm (0.78 inches).

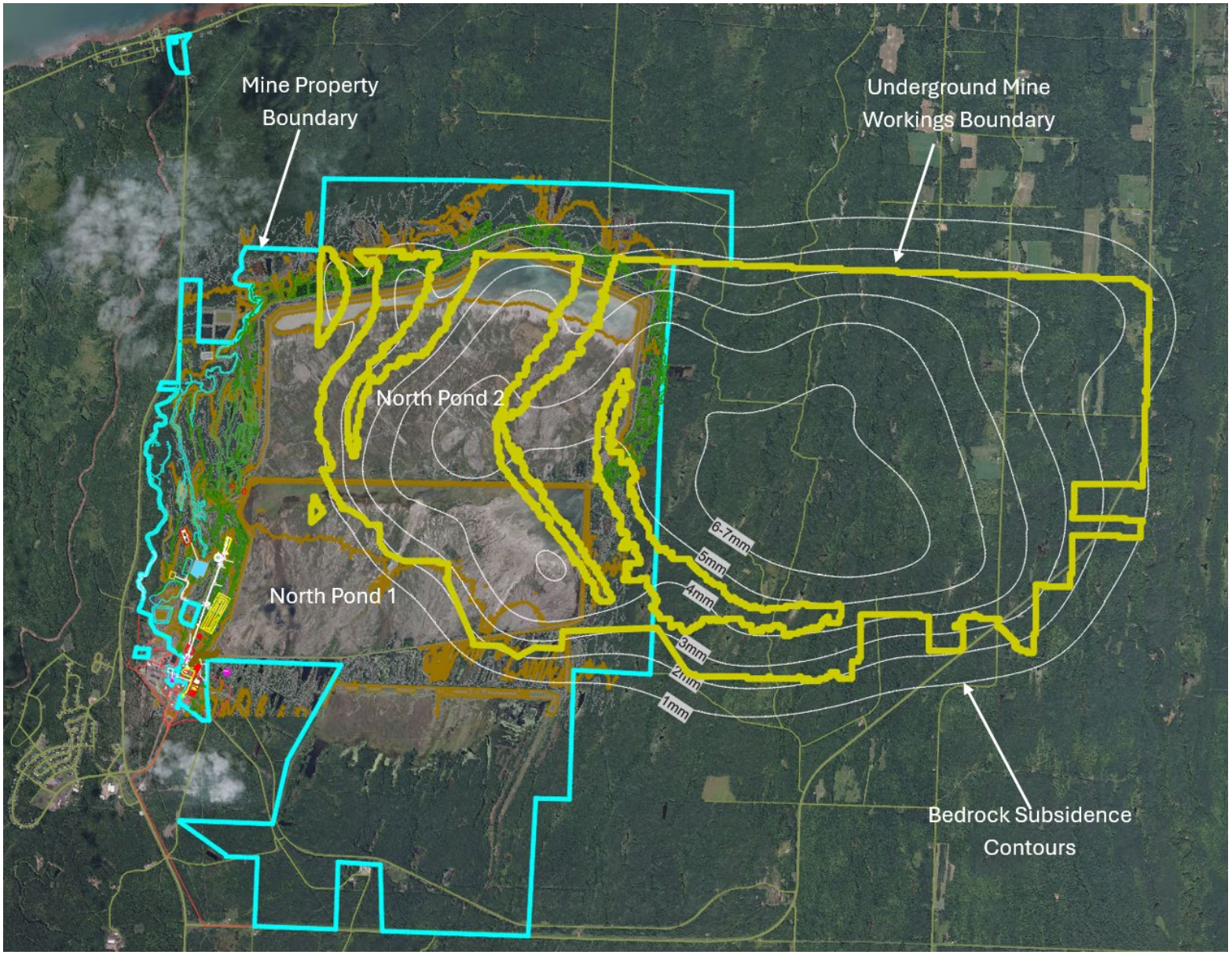

Figure 1: Site Map of Predict Subsidence Contours

Potential Impacts on Tailings Disposal Facilities

The impact/damage to the TDFs embankments caused by surface displacement can be viewed as analogous to the damage caused to an embankment as a result of embankment deformation caused by a seismic event. Based on embankment height of the TDFs ranging from 25 to 50 feet, the maximum predicted 2 cm (0.78 in) of surface displacement would equate to a relative crest settlement (based predicted subsidence divided by total embankment height) of between 0.26% to 0.13%. **Figure 2** below describes relative crest settlement between this range as between damage class 1 and 2, "Minor to Moderate" with a maximum longitudinal crack width between 10 to 80 mm (approximately 0.39 to 3.15 inches) and relative crest settlement of between 0.03 and 0.5 percent . Therefore, there is a minimal risk to the TDFs due to mine subsidence.

Damage Class		Maximum Longitudinal Crack Width (1) mm	Maximum Relative Crest Settlement (2) %
Number	Description		
0	No or Slight	< 10 mm	<0.03
1	Minor	10 - 30	0.03 - 0.2
2	Moderate	30 - 80	0.2 - 0.5
3	Major	80 - 150	0.5 - 1.5
4	Severe	150 - 500	1.5 - 5

(1) Maximum crack width is taken as the maximum width, in millimetres, of any longitudinal cracking that occurs.
(2) Maximum relative crest settlement is expressed as a percentage of the structural dam height.

Figure 2: Cracking and Settlement by Damage Class (Table 5.36 from Pells and Fell, 2003)

Potential Impacts on Surface Infrastructure

Differential settlement is the uneven settlement of a building's foundation, while uniform settlement is defined as settlement that occurs evenly over the foundation of a structure. Differential settlement and the associated angle distortion across a structure, opposed to the total settlement, is the primary concern when evaluating the potential settlement impact on a structure. In the case of the White Pine North Mine, if settlement, as a result of subsidence, were to occur to offsite structures, it is highly likely that it would occur uniformly across the structure; and therefore, would not result in appreciable differential settlement and therefore would not increase the stress conditions in the building. Although uniform settlement is expected to occur, the WSP WPN Bedrock Deflection Assessment (WSP, 2026), included modeling of horizontal strain (i.e. lateral stretching or compression) and angular distortion (i.e. twisting or tilting) at the bedrock surface. Based on the modeling results, WSP reported that very minimal horizontal strain and angular distortion are forecasted at the bedrock surface (< 1e-5 m).

Although the degree of settlement (both total and differential) that a structure can accommodate is dependent on foundation type and building materials used in the construction, the settlement criteria based on soil type in **Table 1** have been commonly referenced.

Table 1 – Recommendations of Permissible Settlement

Settlement (mm)	Sand		Clay
Maximum differential settlement	20[1/]	25[2/]	40[2/]
Maximum Settlement (isolated foundation)	25[1/]	40[2/]	65[2/]
Maximum Settlement (raft foundation)	50[1/]	40-65[2/]	65-100[2/]
Notes: [1/] Terzaghi and Peck (1948) [2/] Skempton and MacDonald (1956)			

Based on geotechnical investigations completed at the mine site, the overburden (soil) that neighboring structures would be constructed over are likely clayey sand. Based on the values provided in **Table 1**, the estimated subsidence (assumed to be the measured settlement at the surface) of 2 cm (0.78 in) is half the recommended maximum differential settlement and three times less than the maximum settlement assuming clay soil. Therefore, it is expected that the estimated subsidence will not result in damage to offsite structures.

Subsidence Monitoring

As part of the operations of the Tailings Disposal Facilities, WPC will develop an Operation, Maintenance, and Surveillance (OMS) Manual and an associated Trigger Action Response Plan (TARP). As part of the OMS Manual, settlement monuments will be installed on the embankments and on native ground and surveyed to monitor for settlement. The TARP will set trigger action levels and if the monitoring of the settlement monuments indicates that settlement is approaching the action level, the TARP will require additional actions, including increasing the frequency of settlement monument monitoring, inspection of the TDF embankments, and other engineered actions, as needed, to maintain the integrity of the tailings disposal facility.

Potential Impacts on Groundwater Wells

An estimated 2 cm (0.78 in) of subsidence at the base of the overburden is not expected to materially affect the structural integrity or performance of wells completed 6 to 18 meters (20 to 60 feet) below ground surface. At this magnitude, subsurface deformation would be anticipated to occur primarily as broadly uniform settlement of the soil column, resulting in minor vertical translation of the well casing and surrounding formation without inducing significant differential strain, bending, or shear stresses. Published geomechanical evaluations of subsidence effects indicate that well integrity impacts such as casing deformation, buckling, or shear failure occur under conditions of substantially greater displacement and/or pronounced differential movement that generate localized stress concentrations (Bruno, 1992).

Based on these considerations, the predicted subsidence magnitude is unlikely to cause material impacts to domestic wells or shallow monitoring wells. Any resulting change in top-of-casing or ground-surface elevation would be minor and would not be expected to compromise annular seals, screened intervals, hydraulic connectivity, groundwater level interpretation, or domestic well operation.

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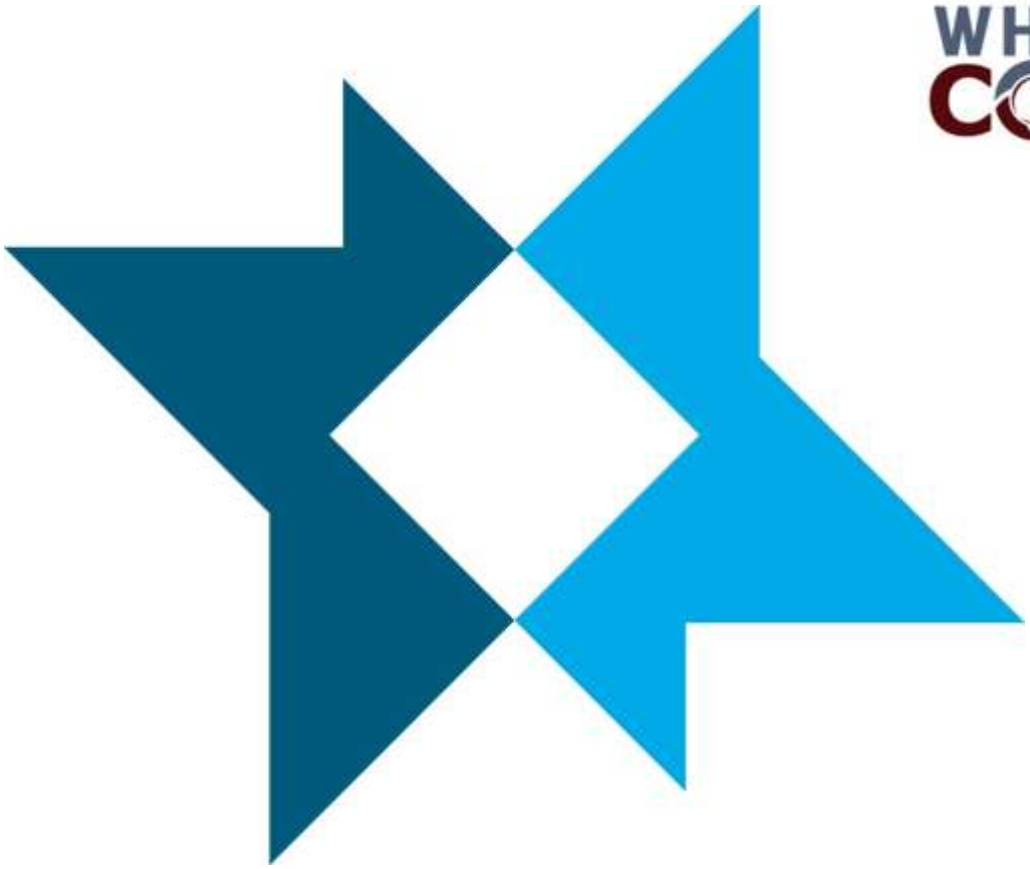
- WSP, 2026. Technical Memorandum: White Pine North Project - Bedrock Deflection Assessment, July 2026

Appendix D

Soil Erosion and Sedimentation Control Plan

Soil Erosion and Sediment Control Plan

White Pine North Project



White Pine Copper LLC

White Pine, Michigan

August 2026

Project ID: 23W200

Solving our clients' toughest science and engineering challenges.

Soil Erosion and Sediment Control Plan
White Pine North Project

Project ID: 23W200

Prepared for
White Pine Copper LLC
White Pine, Michigan

Prepared by
Foth Infrastructure & Environment, LLC

August 2026

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Soil Erosion and Sediment Control Plan
White Pine North Project

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Soil Erosion and Sediment Control Plan
White Pine North Project

Executive Summary

This Soil Erosion and Sedimentation Control Plan describes the erosion and sediment control measures to be implemented for the White Pine North Project during construction, operations, closure, and reclamation. The plan supports the Mining Permit Application by addressing the requirements of R 425.203(c)(xxii) and applicable Part 91 soil erosion and sedimentation control requirements. The plan summarizes site soils, existing topography and drainage, proposed contact and non-contact water routing, temporary construction Best Management Practices, permanent operational erosion controls, and inspection and maintenance procedures. Detailed drawings, figures, inspection forms, and supporting schedules are provided externally as part of the plan package.

List of Abbreviations, Acronyms, and Symbols

amsl	above mean sea level
BMP	Best Management Practices
CWP	Contact Water Pond
Foth	Foth Infrastructure & Environment, LLC
ft	feet/foot
m	meter
MDOT	Michigan Department of Transportation
NCWP	Non-Contact Water Pond
NRCS	Natural Resources Conservation Service
Project	White Pine North Project
SESC	Soil Erosion and Sedimentation Control
TDF	Tailings Disposal Facility
USDA	United States Department of Agriculture
WPM	White Pine Mine

1. Introduction

This Soil Erosion and Sedimentation Control (SESC) Plan has been prepared for the White Pine North Project (Project), owned by White Pine Copper, LLC, located in Carp Lake Township, Ontonagon County, Michigan. This SESC Plan is designed to support compliance with Michigan Administrative Code R 425.203(c)(xxii) and applicable Part 91 soil erosion and sedimentation control requirements administered by the Michigan Department of Environment, Great Lakes, and Energy.

The Project area is located within the following Public Land Survey System townships, ranges, and sections, as shown on Figure 2-1.

- Township: 50N Range: 42W Sections: 1-4
- Township: 51N Range: 40W Sections: 19, 30, 31
- Township: 51N Range: 41W Sections: 19-36
- Township: 51N Range: 42W Sections: 24, 25, 36

1.1 Purpose

The purpose of this SESC Plan is to describe the temporary and permanent erosion and sediment control measures that will be implemented to minimize soil erosion, control sediment transport, and support compliance with R 425.203(c)(xxii) and applicable Part 91 requirements.

1.2 Regulatory Requirements

This SESC Plan includes the minimum information required for Part 91 soil erosion and sedimentation control planning, including:

- A map with a scale of 1 inch = 200 feet (ft) or less, or an indication of exact distances between noted features on the site plan
- Legal description of the property, including township, range, section, and quarter-quarter section, where applicable
- Proximity of proposed earth changes to lakes, streams, wetlands, and other surface water features
- Predominant land features
- Slope descriptions or contour intervals
- Soil survey information or a written description of soil types in proposed exposed land areas
- Description and location of the physical limits of each proposed earth change
- Description and location of existing and proposed on-site drainage and dewatering facilities

- Timing and sequence of proposed earth changes
- Location and description of temporary SESC measures, including installation and removal procedures
- Description and location of proposed permanent SESC measures
- Maintenance program for permanent SESC measures, including designation of the person or role responsible for maintenance

This plan also supports R 425.203(c)(xxii) by describing erosion and sediment control measures during construction and operations; measures to limit the duration of land disturbance; measures to remove sediment before runoff leaves the mining area; controls to maintain non-erosive flow velocities; inspection, maintenance, and removal of temporary erosion controls; and stabilization with permanent erosion control measures as soon as practicable after completion of earthwork.

2. Background Information

The White Pine North Project is located at and adjacent to the historically mined White Pine Mine (WPM) site in Carp Lake Township, Ontonagon County, Michigan (Figure 2-1). The Project is proposed as an underground copper and silver mine with associated surface infrastructure, including a process plant, portal and box cut area, ore handling and stockpile areas, water management facilities, roads, support buildings, and reuse of existing tailings disposal areas depicted on Figure 2-2 and Figure 2-3. Because the site includes historically disturbed industrial areas, existing tailings facilities, wetlands, low-relief terrain, and constructed drainage features, erosion and sediment control will be integrated with Project stormwater and contact-water management systems.

3. Soil Erosion and Sediment Control Plan

This section describes existing conditions within the Project area and summarizes the erosion and sediment control measures that will be implemented during construction, operations, closure, and reclamation. The SESC Plan presents a high-level summary of soil characteristics, existing topography and drainage, planned disturbance, proposed drainage facilities, temporary construction Best Management Practices (BMPs), permanent operational controls, and inspection and maintenance procedures. Detailed erosion control drawings, inspection forms, and supporting schedules are provided externally as part of this plan package.

3.1 Soil Characteristics

Surface soils within the Project area have been influenced by both natural glacial deposition and historical mining-related activities associated with the former WPM. Soil mapping published by the United States Department of Agriculture Natural Resources Conservation Service (NRCS) indicates that soils across the Project area are variable and include both native glacial soils and anthropogenic mine spoil materials.

Much of the White Pine community area is mapped as Udorthents loams with slopes ranging from 0 to 2 percent. These soils consist primarily of loamy mine spoil fill extending to depths of approximately 2.0 meters (m) (80 inches) below ground surface and are generally moderately well drained with minimal runoff potential.

Additional prominent soil units within the Project area include the Big Iron–Flintsteel–Gull Point complexes, which consist of varying proportions of somewhat poorly drained fine loamy till, moderately well drained loamy till, and poorly drained alluvial soils. These soils generally consist of silt loam, loam, sandy loam, and clay loam textures with varying gravel contents and runoff characteristics.

The Main Facilities/Former Mining Operations Area is primarily mapped as “Dumps and Pits, Mine” and Slickens mine spoil fill associated with historical mining and tailings management activities. Areas planned for future mining-related infrastructure include soils mapped as Big Iron–Flintsteel–Gull Point complex, Udorthents–Alfic Udarents–Epiaquents complex, and Flintsteel–Watton–Gull Point complex. These soils range from moderately well drained upland tills to poorly drained organic and alluvial soils associated with wetlands and low-lying areas.

Field observations collected during the 2025 soil sampling program indicate that shallow soils generally consisted of silts and clays with varying percentages of sand, gravel, and organic matter. Organic-rich soils and peat were common in low-lying and wetland areas, while clay-rich fill and mine spoil materials were more common in historically disturbed portions of the Site.

Figure 3-1 presents the United States Department of Agriculture NRCS soil survey characterization for the Project area.

3.2 Existing Conditions

Existing topography of the Project area is shown on Figure 3-2. The Project area is located within the western Upper Peninsula of Michigan and is characterized by low-relief terrain formed by Late Pleistocene glaciation and subsequent glaciofluvial processes. The landscape is

dominated by a broad upland plain with limited topographic variability and numerous wetlands and shallow depressions.

Glacial landforms strongly influence the surface morphology of the Project area. The terrain consists primarily of gently undulating ground moraine and glaciofluvial deposits, with localized depressions that host wetlands and areas of seasonal ponding. These features reflect deposition and reworking of sediments by glacial ice and meltwater, as well as subsequent modification by surface water processes. In some areas, subtle ridges and shallow drainage swales are present, reflecting former meltwater channels and post-glacial drainage pathways.

Ground surface elevations within the Project area generally range from approximately 260 to 290 m (850 to 950 ft) above mean sea level (amsl), with the historical WPM area situated at approximately 266 m (873 ft) amsl. Topographic relief across the Project area is limited, with elevation differences typically less than 30 m (98 ft).

Slopes across the Project area are generally gentle to very gentle, reflecting glacial deposition and surface smoothing processes. Most ground surfaces exhibit slopes of less than 5 percent, with large areas having slopes in the 1 to 3 percent range based on USGS topographic mapping of the White Pine quadrangle. Steeper slopes are limited and typically occur:

- along local stream channels;
- at the margins of shallow depressions and wetland basins; and
- adjacent to man-made or legacy mining features.

Surface drainage within the Project area is moderately developed and strongly influenced by the low-gradient topography and glacial deposits. The gentle slopes promote short, low-energy streams, wetlands, and areas of seasonal saturation, particularly in forested lowlands and glacial depressions. Drainage patterns generally flow northward toward Lake Superior via small tributaries and stream systems, including those associated with the Mineral River watershed.

3.3 Proposed Drainage Facilities

Topography and drainage will be affected within the mining area by construction of several major structures and activities as shown on Figure 2-3. The following components will alter topography and drainage:

- Box Cut Area and Crusher Feed Conveyor including mine dewatering
- Temporary Ore Stockpile
- Crushed Ore Stockpile and Mill Feed Conveyor
- Process Plant Area including administration, maintenance, substation, and warehousing areas
- Existing Plant Area Contact Water Pond (CWP) (upgraded)
- Main CWP, upgraded from the former Bedell Pond, for contact stormwater storage
- Tailings Disposal Facility, including North Pond 1 and North Pond 2

- Industrial Wastewater Treatment Plant
- Main access roads
- Other existing and proposed facilities as shown on Figure 2-3

Other facilities shown on Figure 2-3 include mine laydown area, Village of Ontonagon Potable Water Treatment Plant, and minor access roads.

The site will be rough graded prior to excavation associated with construction. As part of the site grading, diversion berms and ditching will be created to separate the non-contact and contact water areas. Non-contact water areas are defined as areas with runoff that are not in contact with ore, processed ore products, or tailings. Contact water areas include the Process Plant, Box Cut Area and Crusher Feed Conveyor and Mill Feed Conveyor, Crushed Ore Stockpile, Temporary Ore Stockpile, Tailings Disposal Facility, Industrial Wastewater Treatment Plant, Contractor Facilities, Shotcrete Plant and Laydown, Fuel Station, Existing Waste Repository and Slag Pile, Mining Laydown Area, Explosive Magazine/Emulsion Silos, mine portal, haul roads, and CWPs. In addition, all other stormwater runoff that is within the Project area and not associated with non-contact water areas are included in the contact water area.

Figure 2-3 and Drawing No. 1 present the water management stormwater flow network.

Contact water from the Process Plant Area is routed to an upgraded, lined CWP via an existing storm sewer piping and manhole network and overland flow. Contact water contained in the Process Plant CWP is routed directly to the main CWP via a 36-inch high-density polyethylene pipeline. Water from the remaining contact areas (Box Cut Area and Crusher Feed Conveyor and Mill Feed Conveyor, Crushed ore Stockpile, Temporary Ore Stockpile, Tailings Disposal, Industrial Wastewater Treatment Plant, Contractor Facilities, Shotcrete Plant and Laydown, Fuel Station, Existing Waste Repository and Slag Pile, Mining Laydown Area, Explosive Magazine/Emulsion Silos, mine portal, and haul roads) is routed to the Main CWP through a series of gravity and pressure pipes and lined ditches constructed along the haul roads.

Non-contact water from the western and northeastern portions of the mine site is routed to the existing non-contact water ponds (NCWP) via overland flow. The first pond acts as the primary settling pond and the remaining ponds provide additional reduction in suspended solids. Collectively, the NCWP also functions as flow equalization ponds. Stormwater more than the design retention capacity (1.6 inches) will flow through a bypass spillway discharging to the receiving stream immediately below the main CWP as shown on Figure 2-3 and Drawing No. 1.

3.4 Erosion Control During Construction

Daily site erosion control features will be maintained during construction. Permanent soil erosion control measures for use during the operational phase will be established immediately following completion of the grading work to augment or replace the temporary measures installed during construction.

Drawing No. 1 illustrates the SESC Plan for the Project area that will be implemented during construction. Drawing No. 2 provides detailed drawings of specific erosion control features such as drainage ditches, tracking pads, silt fences, erosion mats, and straw wattles or straw bale barriers.

The following is a list of earthwork and other grading activities that are part of the construction phase. Each activity is followed by a description of the BMP that will be implemented to minimize soil erosion and site runoff. Where applicable, materials and methods specified in the Michigan Department of Transportation (MDOT) 2012 Standard Specification for Construction (MDOT, 2012) were used for specification of the materials proposed for soil erosion control.

Surface Water Diversion

Where the potential for run-on exists, runoff will be diverted away from surface facilities by earthen diversion berms.

Land Clearing and Grubbing

Land clearing is the removal of woody and herbaceous plant material and consists of two components: clearing, which is the cutting and removal of trees, and grubbing, which is the removal of stumps and roots. Marketable timber will be removed from the site and sold. Stumps or other woody material that cannot be chipped will be stockpiled and managed in accordance with applicable air quality, fire safety, solid waste, and site reclamation requirements. The soil and erosion control practices that are proposed during land clearing and grubbing include:

- Removing marketable and non-marketable trees in a single operation to minimize disturbance.
- Completing the grubbing and tree removal in a single operation to minimize disturbance.
- Construction of silt fence downstream of the areas to be cleared and grubbed prior to the start of earth disturbance. See Drawing No. 1 for the approximate locations where silt fence will be installed. Details of erosion control methods are provided on Drawing No. 2.

Topsoil Stripping and Stockpiling

Following land clearing and grubbing, topsoil will be stripped from new access road and the mine site area. Topsoil is defined as the A-horizon of the soil in which organic matter accumulates.

Prior to topsoil stripping, additional silt fence will be constructed downgradient of disturbed areas where needed. Topsoil and any excess soil will be stockpiled in piles with maximum side slopes of 3 horizontal to 1 vertical (3H:1V). Silt fence will also be constructed around the soil stockpiles. Topsoil and excess soil will be stockpiled with conventional earth-moving equipment. Prior to seeding the stockpile, the soil surface will be “track walked” (i.e., driving a bulldozer up and down the slope to leave a pattern of track imprints parallel to the slope contours) to create a rough surface. This rough surface will enhance the ability of the soil to withstand erosion until vegetation has developed. The stockpiles will be seeded with a seed mixture typically used on sandy soil for roadside restoration projects as specified in MDOT (2012).

The soil stockpiles will be seeded at a rate of approximately 250 kilograms per hectare. Seed mixtures will include temporary species such as oats or perennial rye grass that germinate quickly and act as a nurse crop until the perennial species germinate and develop.

Site Excavation and Grading

The site will be rough graded prior to excavation associated with construction of the process area, NCWP, and CWPs. As part of the site grading, diversion berms and ditching will be created to separate the contact and non-contact water areas as shown on Figure 2-3. Excess soil will be placed in soil stockpiles at the locations shown on Figure 2-3. Disturbed areas will be seeded as soon as possible. Stockpiles will be surrounded by silt fencing as shown on Drawing No. 1.

Non-Contact Water Ponds

Non-contact water from the western and northeastern portions of the mine site is routed to the two existing NCWPs via overland flow. The first pond acts as the primary settling pond and the remaining ponds provide additional reduction in suspended solids. Collectively, the NCWPs also function as flow equalization ponds. Stormwater more than the design retention capacity (1.6 inches) will flow through a bypass spillway discharging to the receiving stream immediately below the main CWP as shown on Figure 2-3 and Drawing No. 1.

During the construction phase of this Project, the NCWPs will act as sedimentation basins.

Contact Water Ponds

Contact water from the Process Plant Area is routed to an upgraded, lined CWP via an existing storm sewer piping and manhole network and overland flow. Contact water contained in the Process Plant CWP is routed directly to the main CWP via a 36-inch high-density polyethylene pipeline. Water from the remaining contact areas (Box Cut Area and Crusher Feed Conveyor and Mill Feed Conveyor, Crushed ore Stockpile, Temporary Ore Stockpile, Tailings Disposal Facilities, Industrial Wastewater Treatment Plant, Contractor Facilities, Shotcrete Plant and Laydown, Fuel Station, Existing Waste Repository and Slag Pile, Mining Laydown Area, Explosive Magazine/Emulsion Silos, mine portal, and haul roads) is routed to the Main CWP through a series of gravity and pressure pipes and lined ditches constructed along the haul roads.

During the construction phase of this Project, the CWP will act as a sedimentation basin.

Ditches

Lined ditches will be constructed along the haul roads to convey contact water to the CWP.

A bypass spillway discharging to the receiving stream immediately below the main CWP will be constructed to divert non-contact water from the contact water areas.

During construction, BMPs will be maintained daily and undergo formal inspection as outlined in Section 3.6.

3.4.1 Construction Schedule

The Project schedule includes an approximately three-year pre-production development period, followed by production ramp-up, steady-state operations, final production, and closure transition. Erosion control BMPs will be installed before land-disturbing activities commence and will be maintained through construction, operations, and reclamation as applicable. The start date of construction will depend on receipt of required permits and Project authorization. The detailed construction and erosion control schedule is provided in Appendix A.

3.5 Permanent Erosion Control Methods During Operation

After construction activities are complete, stormwater ditches, basins, culverts, outlet protection, stabilized road surfaces, and revegetated disturbed areas will be maintained as permanent soil erosion and sediment control measures during operations. Temporary controls such as silt fence, rock filters, erosion mats, and other construction-phase BMPs will be removed only after contributing areas have been stabilized and permanent controls are functioning as designed. Road surfaces, roadside ditches, culverts, basin embankments, spillways, and drainage outlets will be inspected and maintained for the life of the Project to limit erosion, maintain non-erosive flow velocities, and reduce sediment transport from operational areas.

3.6 Plan Oversight, Staffing, and Maintenance of Erosion Control Measures

White Pine Copper will ensure that technical staff responsible for implementation of the SESC Plan are trained and certified as construction and/or industrial stormwater operators, where required, SESC plans require a state certified stormwater operator to oversee and be responsible for the plan. A Certified Stormwater Operator will maintain the plan for the Project. This person will implement the plan in accordance with stormwater operator training, Michigan rules and guidance, and general industry practice. Any staff assigned to implement the SESC Plan will be directly supervised and trained by the Certified Stormwater Operator.

Maintenance of erosion control measures will be done through an inspection and maintenance program. Table 3-1 presents a list of facility structures that will be included in the inspections, the frequency of those inspections (daily, weekly, or precipitation event-triggered), and descriptions of the inspection and maintenance (contingent upon the results of the inspection).

3.6.1 Inspection and Maintenance of Erosion Controls

Inspections will be documented using the SESC Inspection and Maintenance Log shown in Appendix B. The form documents date, inspector name, reason for inspection, construction activity description, and recommended actions if needed. The form also documents the resolution of past actions and recommendations. Maintenance will be performed in accordance with details presented in this SESC Plan, Table 3-1, Michigan guidelines, and general industry practice. Inspection records will be retained as part of the Project compliance record and made available for review in accordance with applicable permit requirements.

4. Conclusions

This Soil Erosion and Sedimentation Control Plan has been prepared for the White Pine North Project as part of the Mine Permit Application. The plan describes temporary and permanent erosion and sediment control measures that will be used to minimize soil erosion, remove sediment from runoff before water leaves disturbed areas, maintain stable drainage conditions, and support compliance with R 425.203(c)(xxii) and applicable Part 91 requirements. Implementation of the plan, together with the Project Water Management Plan, construction sequencing, inspection program, and maintenance procedures, is intended to reduce sediment transport and protect surface water resources during construction, operations, closure, and reclamation.

5. Reference

Michigan Department of Transportation, 2012. *Standard Specifications for Construction*. Accessed July 2026 at https://mdotjboss.state.mi.us/SpecProv/specBookHome.htm.

Table

Table 3-1
Inspection and Maintenance of Soil Erosion Control Best Management Practices

Facility/Structure	Frequency	Description/Maintenance
Ponds[1]	Weekly[2]	1. Inspect for erosion/siltation 2. Repair erosion and remove siltation if present
Ditches/Culverts/Diversion Berms[3]	Weekly[2]	1. Inspect for erosion/siltation 2. Repair erosion and remove siltation if present 3. Add appropriate erosion control measures if needed
Silt Fencing	Weekly[2]	1. Inspect for integrity 2. Remove siltation if present 3. Repair or replace if damage is present
Access and Haul Roads and Paved Ditches[4]	Daily or as Required	1. Remove any material dropped from vehicles 2. Repair any cracking or other damage on roads

Notes:

Prepared by: BJS1

Checked by: CER2

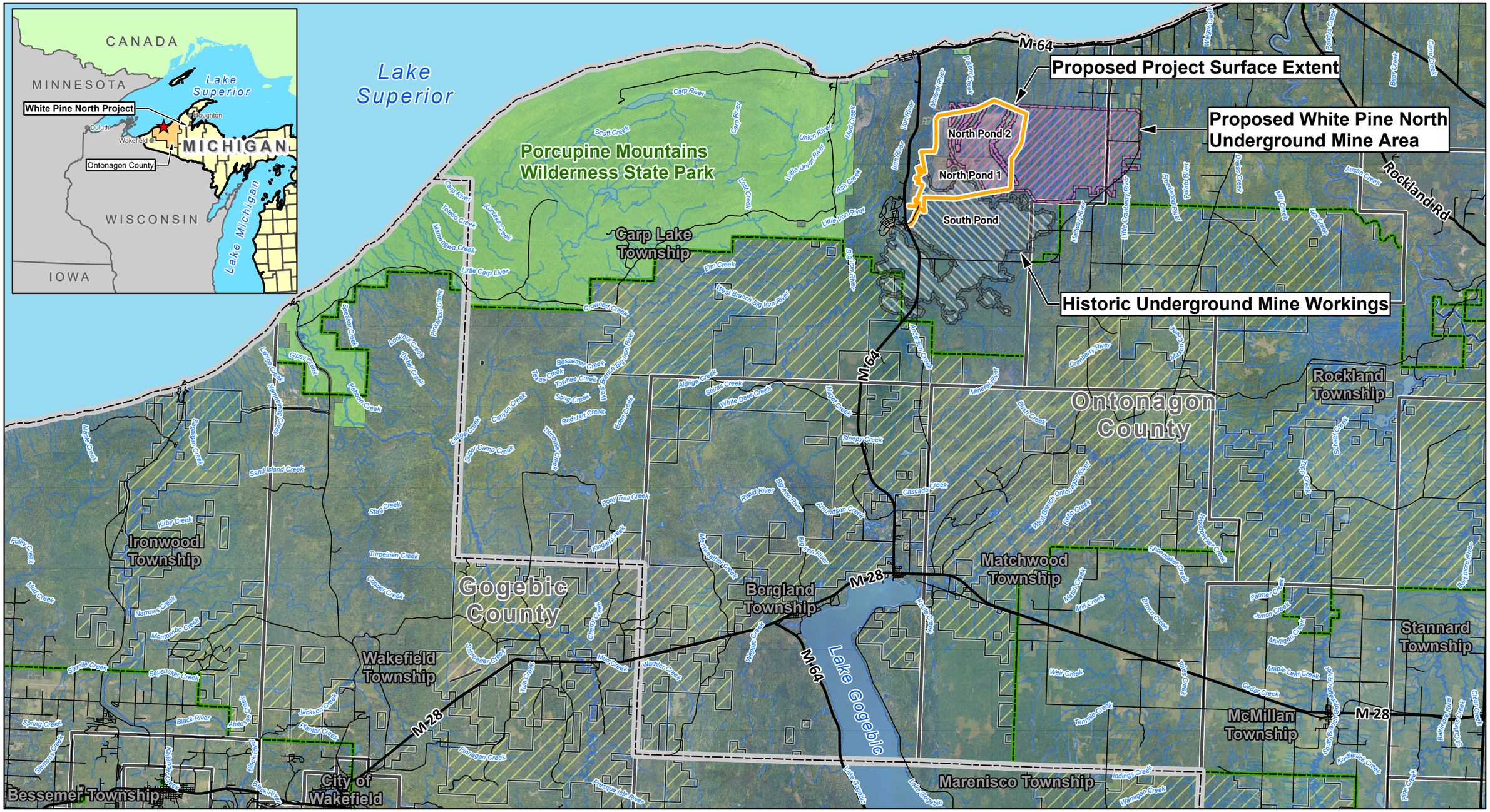
[1] Applies to contact and non-contact ponds

[2] Weekly or after a rain event exceeding 1/2 inch in a 24-hour period of following heavy snow melt runoff

[3] Applies to non-contact areas

[4] Applies to contact areas

Figures



NOTES:

1. 2024 - 18in resolution air photo from the USDA NAIP.
2. Horizontal coordinate system is NAD1983 UTM Zone, 16N, units in meters.
3. Road, boundary and hydrographic data from Michigan GIS Open Data.

LEGEND

- Major Road
- Minor Road
- River/Stream
- Lake/Pond
- Proposed Project Surface Extent
- Proposed White Pine North Underground Mine Area
- Historic Underground Mine Workings
- Municipal Boundary
- County Boundary
- Ottawa National Forest Boundary
- Porcupine Mountains Wilderness State Park
- USDA Forest Service Owned Property

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Foth Infrastructure & Environment, LLC			
REVISED	DATE	BY	DESCRIPTION
PREPARED BY:		DATE:	
REVIEWED BY:		DATE:	
APPROVED BY:		DATE:	

WHITE PINE COPPER LLC

FIGURE 2-1
PROJECT LOCATION
SOIL EROSION AND SEDIMENT CONTROL PLAN
WHITE PINE NORTH, ONTONAGON COUNTY, MI

Scale: 0 1.5 3 Miles

Date: JUNE 2026

Drafted By: DAT

Project: 25W006

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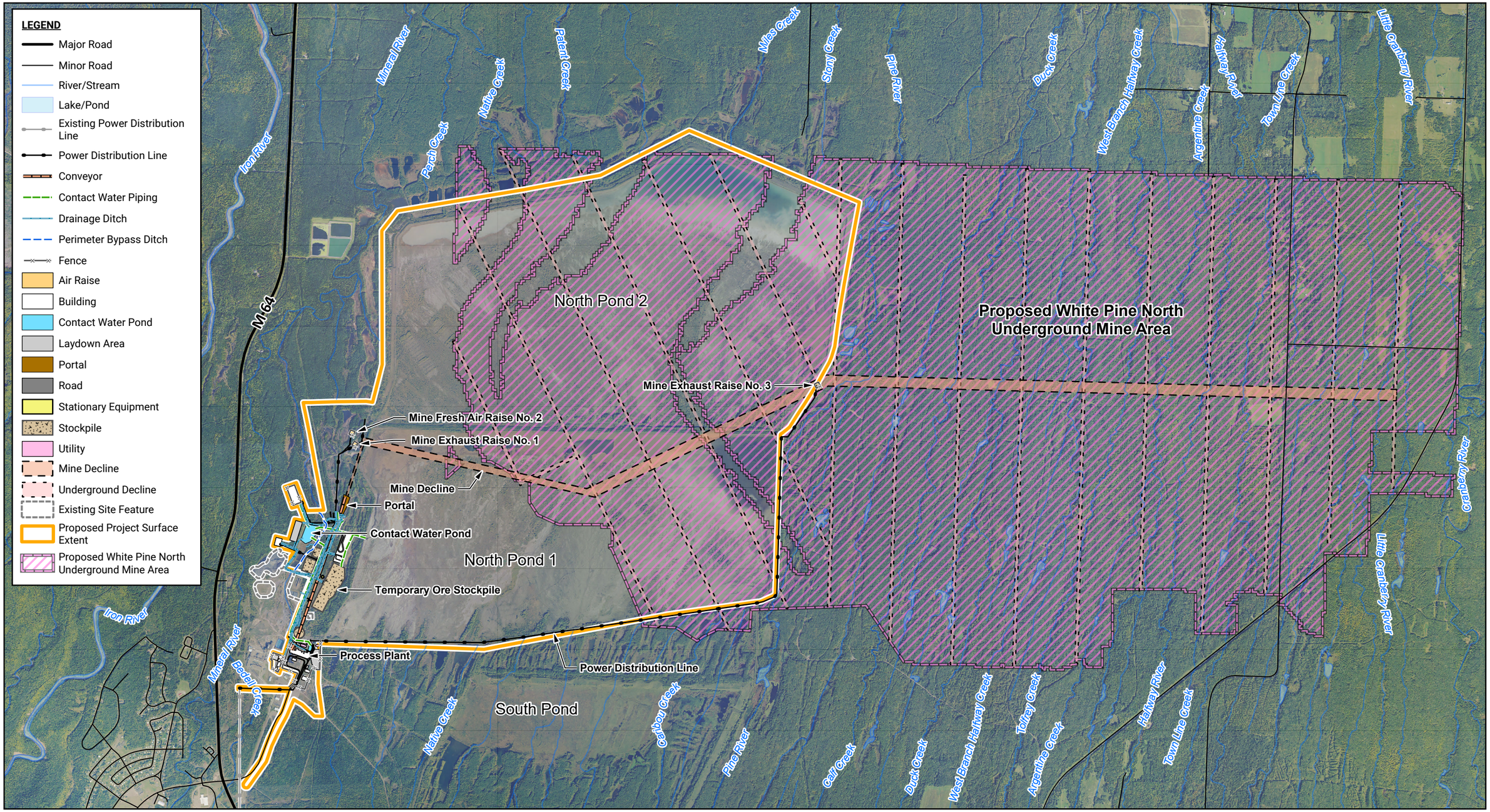

NOTES:

1. 2024 - 18in resolution air photo from the USDA NAIP.
2. Horizontal coordinate system is NAD1983 UTM Zone, 16N, units in meters.
3. Road, boundary and hydrographic data from Michigan GIS Open Data.
4. Site layout and infrastructure data from Ausenco, March 2026.

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REVISED	DATE	BY	DESCRIPTION

PREPARED BY:	DATE:
REVIEWED BY:	DATE:
APPROVED BY:	DATE:

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FIGURE 2-2
PROPOSED OVERALL SITE LAYOUT
SOIL EROSION AND SEDIMENT CONTROL PLAN
WHITE PINE NORTH, ONTONAGON COUNTY, MI

Scale: 0 1,500 3,000 Feet	Date: JUNE 2026
Drafted By: DAT	Project: 25W006

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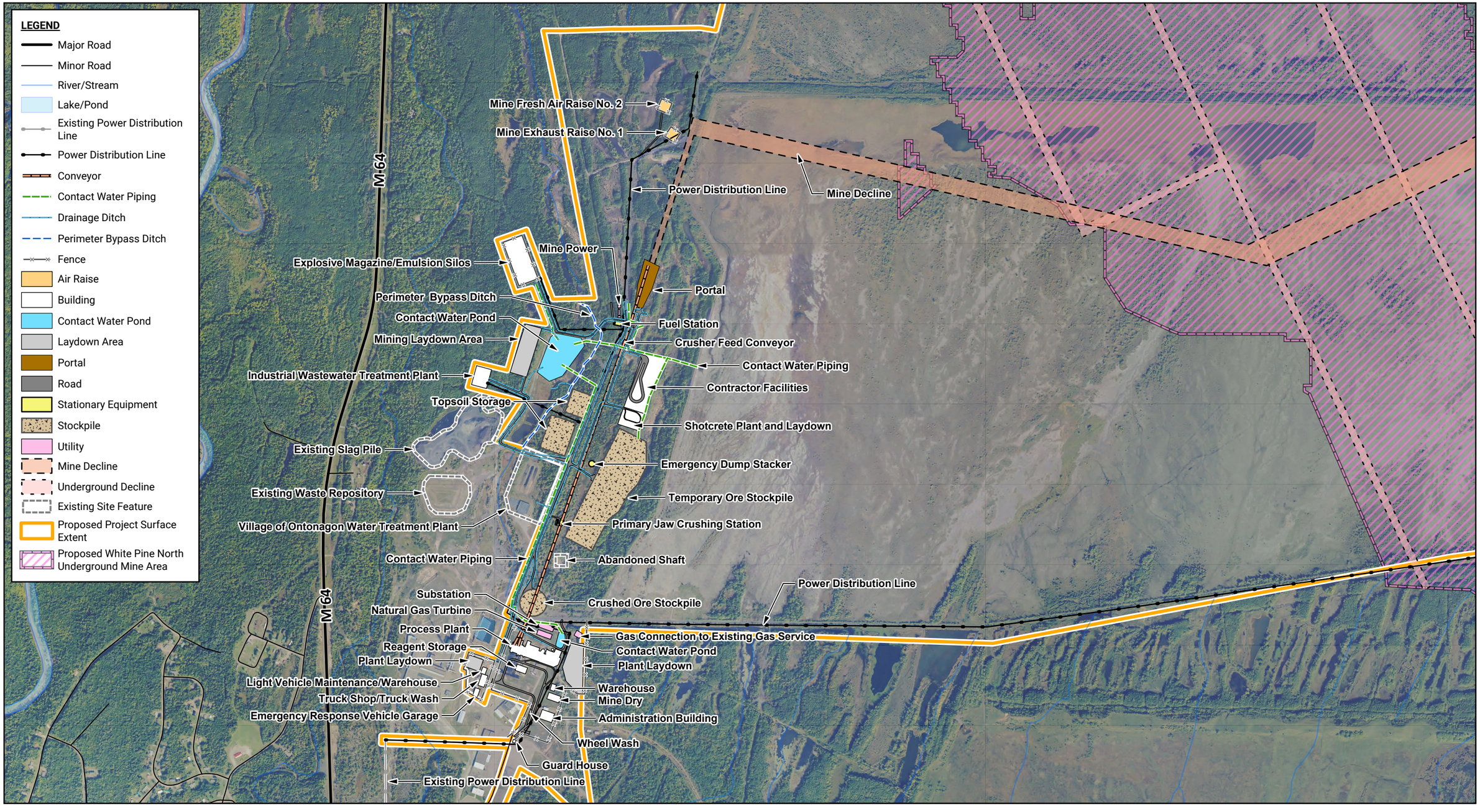

NOTES:

1. 2024 - 18in resolution air photo from the USDA NAIP.
2. Horizontal coordinate system is NAD1983 UTM Zone, 16N, units in meters.
3. Road, boundary and hydrographic data from Michigan GIS Open Data.
4. Site layout and infrastructure data from Ausenco, March 2026.

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REVISED	DATE	BY	DESCRIPTION

PREPARED BY:	DATE:
REVIEWED BY:	DATE:
APPROVED BY:	DATE:

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FIGURE 2-3
MINE SITE LAYOUT
SOIL EROSION AND SEDIMENT CONTROL PLAN
WHITE PINE NORTH, ONTONAGON COUNTY, MI

Scale: 0 600 1,200 Feet

Date: JUNE 2026

Drafted By: DAT

Project: 25W006

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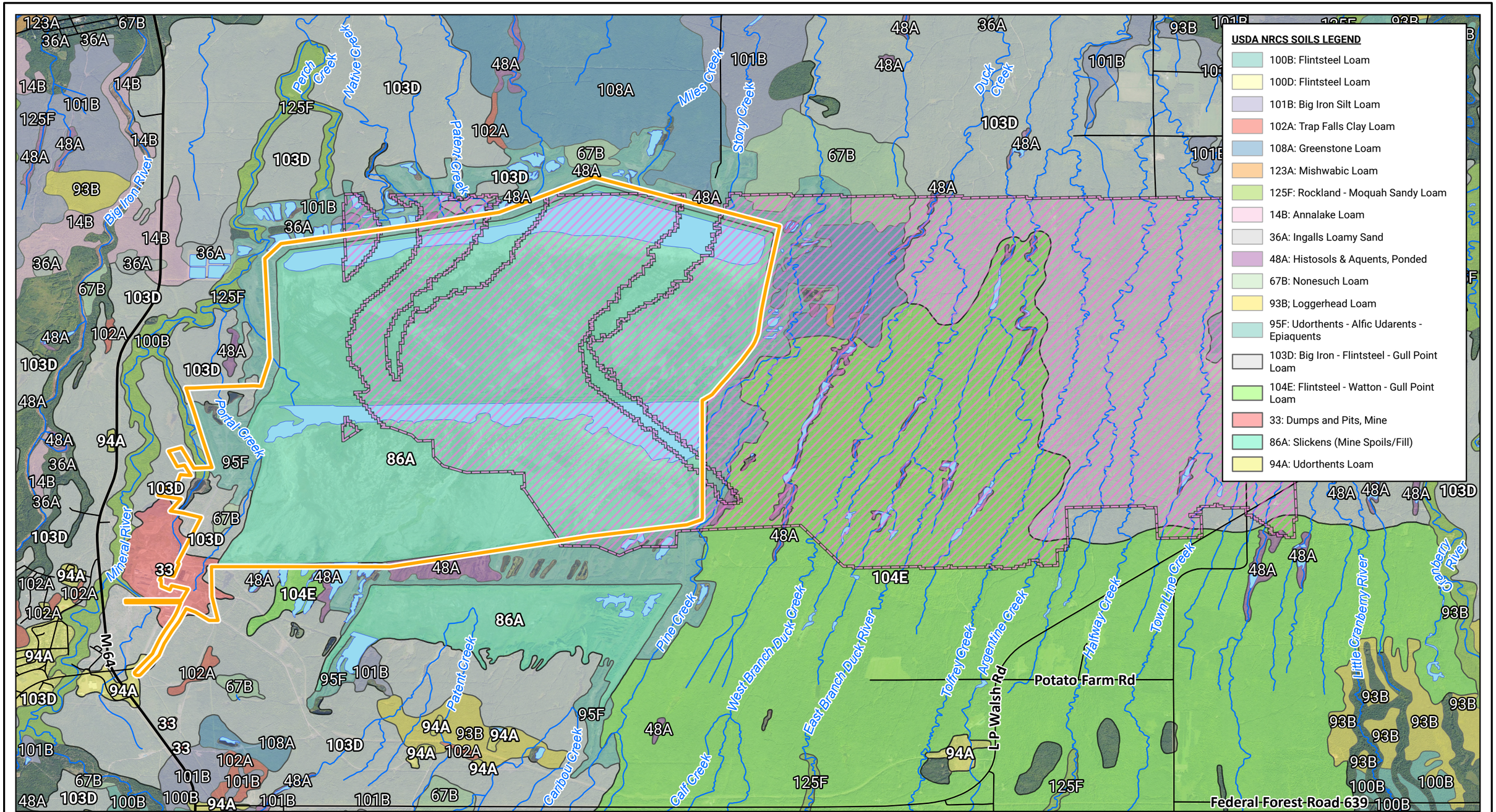

NOTES:

1. Base air photo from the USDA.
2. Horizontal coordinate system is NAD1983 UTM Zone, 16N, units in meters.
3. Hydrographic data from the USGS.
4. Road and boundary data from Michigan GIS Open Data and the USFS.
5. Site layout and infrastructure is based on 2019 White Pine North Copper Project PEA.
6. Soils data from USDA NRCS.

LEGEND

- Major Road
- Minor Road
- River/Stream
- Lake/Pond
- Proposed White Pine North Underground Mine Area
- Proposed Project Surface Extent

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Foth Infrastructure & Environment, LLC			
REVISED	DATE	BY	DESCRIPTION

PREPARED BY:	DATE:
REVIEWED BY:	DATE:
APPROVED BY:	DATE:

WHITE PINE COPPER LLC

FIGURE 3-1
USDA NRCS SOILS MAP
SOIL EROSION AND SEDIMENT CONTROL PLAN
WHITE PINE NORTH, ONTONAGON COUNTY, MI

Scale: 0 2,000 4,000 Feet

Date: JUNE 2026

Drafted By: DAT

Project: 25W006

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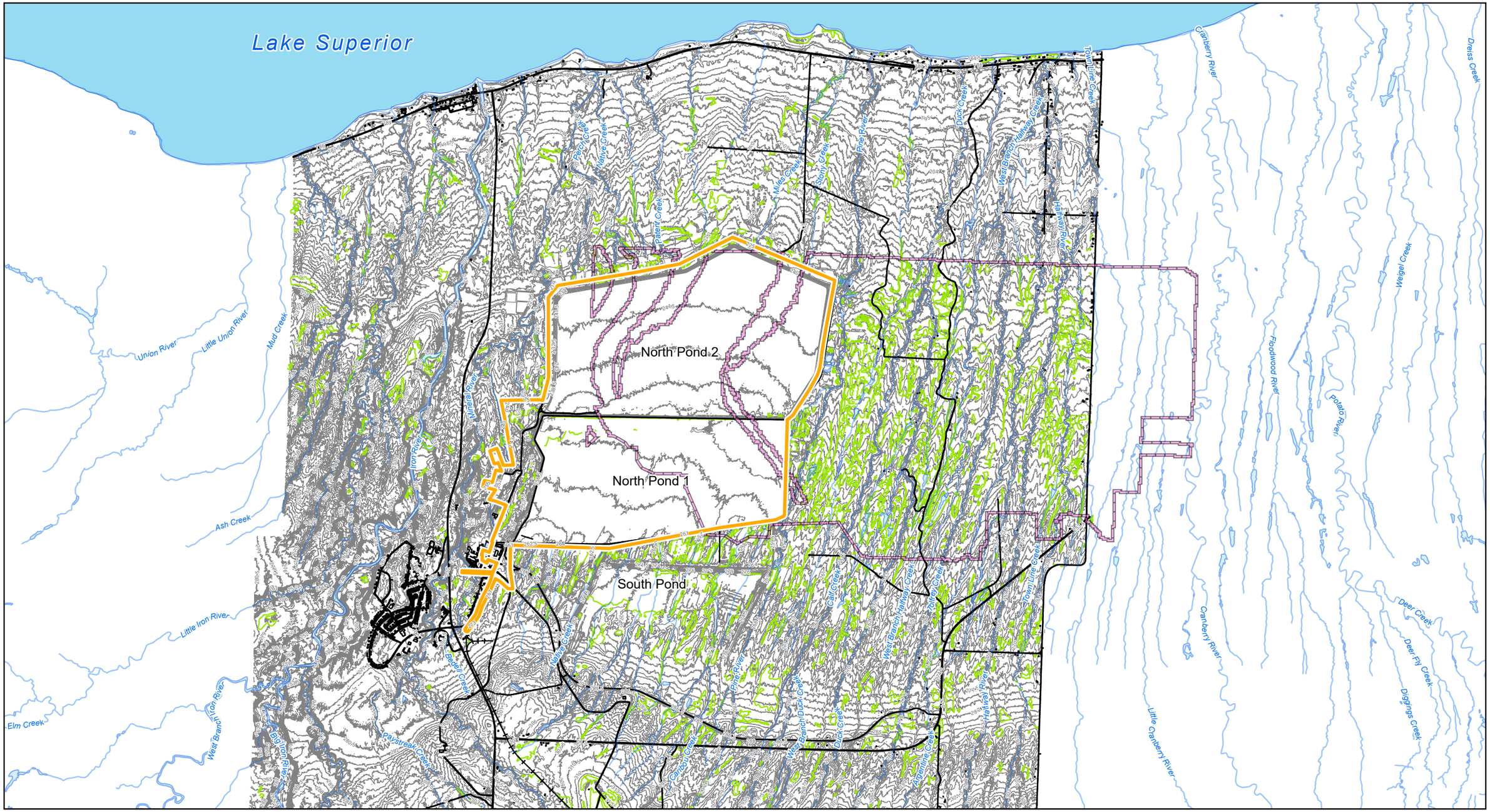


NOTES:

1. 2024 - 18in resolution air photo from the USDA NAIP.
2. Horizontal coordinate system is NAD1983 UTM Zone, 16N, units in meters.
3. Road, boundary and hydrographic data from Michigan GIS Open Data.
4. Topographic and planimetric data from Quantum Spatial, 2014.

LEGEND

- 1.5 Meter Index Contour
- Road, Building, Bridge, Slab, Silo, Pad, Etc.
- Railroad
- Wetland Feature
- River/Stream
- Lake/Pond
- Proposed Project Surface Extent

WHITE PINE COPPER

Foth

N

Foth Infrastructure & Environment, LLC			
REVISED	DATE	BY	DESCRIPTION
PREPARED BY:			DATE:
REVIEWED BY:			DATE:
APPROVED BY:			DATE:

WHITE PINE COPPER LLC

FIGURE 3-2
TOPOGRAPHIC MAP
SOIL EROSION AND SEDIMENT CONTROL PLAN
WHITE PINE NORTH, ONTONAGON COUNTY, MI

Scale: 0 2,500 5,000 Feet

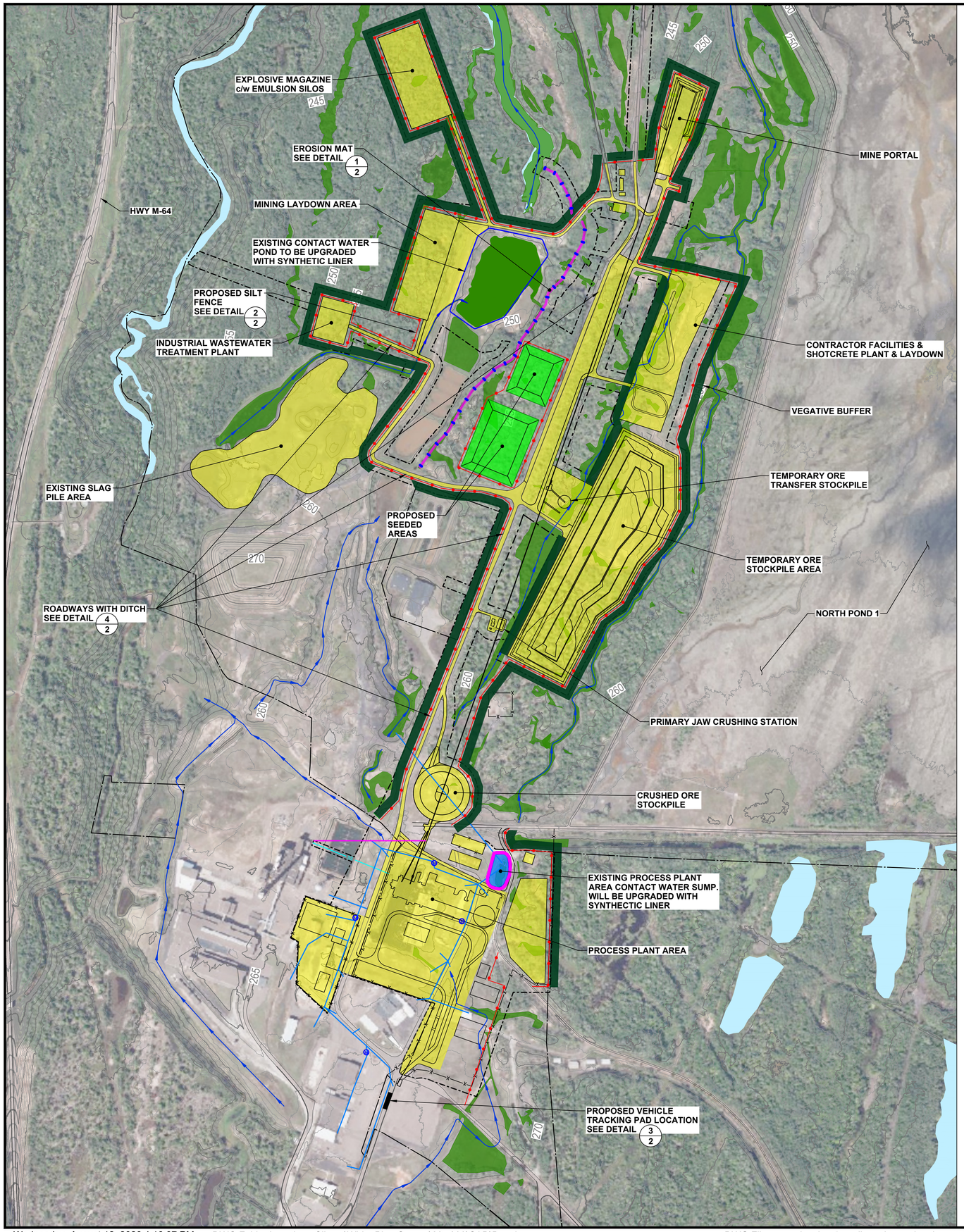
Date: JUNE 2026

Drafted By: DAT

Project: 25W006

Q:\White Pine Copper\23W200\GIS\aprx\2026 Soil Erosion and Sediment Control Plan.aprx 7/16/2026 3:22 PM

Drawings



LEGEND

- EXISTING INDEX CONTOUR
- EXISTING INTERMEDIATE CONTOUR
- APPROXIMATE PROPERTY LINE
- APPROXIMATE PROPERTY SUBAREA
- EXISTING ROADWAY
- EXISTING ELECTRIC
- EXISTING STORM WATER PIPE
- EXISTING CATCH BASIN
- EXISTING MANHOLE
- EXISTING CHANNEL FLOW
- EXISTING WATER
- EXISTING DELINEATED WETLAND
- CONTACT WATER AREA
- SEEDED AREA
- VEGETATIVE BUFFER (~15 M. WIDE) - SEE NOTE 5
- EROSION MAT
- SILT FENCE
- TEMPORARY DITCH CHECK
- TRACKING PAD

NOTES:

1. BASE AIR PHOTO FROM BING MAPS.
2. HORIZONTAL COORDINATE SYSTEM IS NAD 1983 UTM ZONE, 16N, UNITS IN METERS.
3. ROAD AND BOUNDARY DATA FROM MICHIGAN GIS OPEN DATA AND THE USFS.
4. TOPOGRAPHY BASE MAP OBTAINED FROM AUSENCO BASED ON LIDAR SURVEY. CONTOUR INTERVAL = 0.25 METERS.
5. THE VEGETATIVE BUFFER ZONE CONSISTS OF DENSE VEGETATION INTENDED TO SLOW RUNOFF AND TRAP SEDIMENT.

WHITE PINE COPPER

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SOIL EROSION AND SEDIMENT CONTROL PLAN

WHITE PINE COPPER
ONTONAGON COUNTY, MICHIGAN

REVISIONS			
NO.	DESCRIPTION	DATE	BY

DATE OF PREPARATION

	BY	DATE
SURVEYED		
DRAWN	JRB2	JULY 2026
DESIGNED	BDS1	JULY 2026
CHECKED	BDS1	JULY 2026

SHEET TITLE:

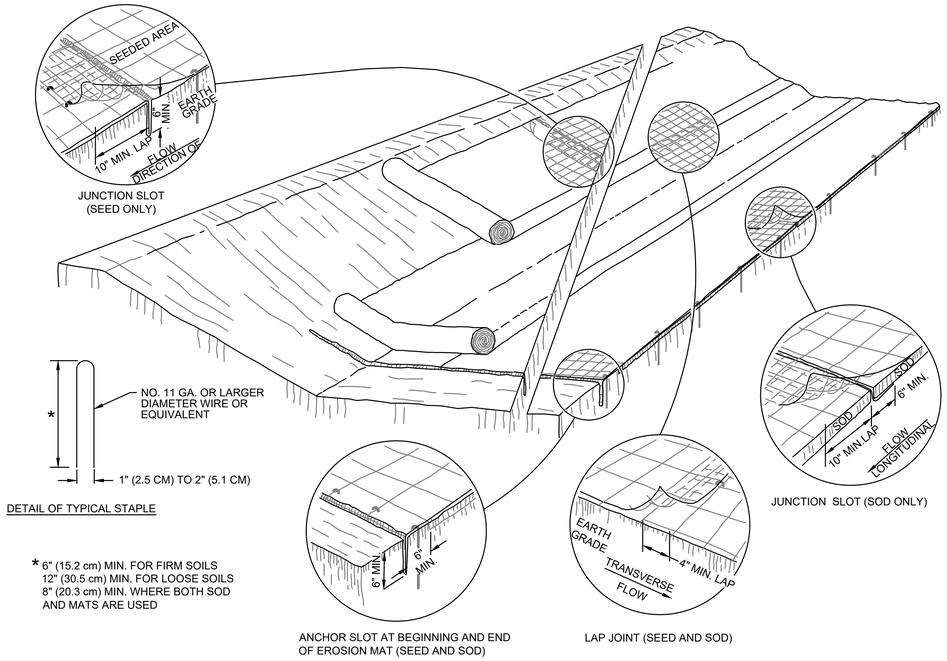
SOIL EROSION AND SEDIMENT CONTROL PLAN

PRELIMINARY DESIGN

PROJECT NO: 23W200

SHEET NUMBER

1



GENERAL NOTES

DETAILS OF CONSTRUCTION, MATERIALS AND WORKMANSHIP NOT SHOWN ON THIS DRAWING SHALL CONFORM TO THE PERTINENT REQUIREMENTS OF THE STANDARD SPECIFICATIONS AND THE APPLICABLE SPECIAL PROVISIONS.

VARIATIONS IN THE DIMENSIONS OR MATERIALS SHOWN HEREON SHALL BE PERMITTED IF THEY PROVIDE EQUIVALENT PROTECTION AND MATERIAL STRENGTH AND IF PRIOR APPROVAL OF THE ENGINEER IS OBTAINED.

LAP JOINTS SHALL NOT BE PLACED IN THE BOTTOM OF V-SHAPED DITCHES.

JUNCTION SLOTS ON ADJACENT STRIPS OF MATTING SHALL BE STAGGERED A MINIMUM OF 4 FEET (1.219 m) APART.

EDGES OF THE EROSION MAT SHALL BE IMPRESSED IN THE SOIL. EROSION MAT SHALL BE MEASURED AND PAYED FOR IN ACCORD-ANCE WITH THE STANDARD SPECIFICATIONS.

EROSION MAT OVER SOD

a. ONLY JUTE FABRIC WILL BE PERMITTED OVER SOD.

b. WOOD STAKES FOR SOD MAY BE OMITTED BY THE ENGINEER IF THE EXISTING SLOPE AND SOIL CONDITIONS SO WARRANT.

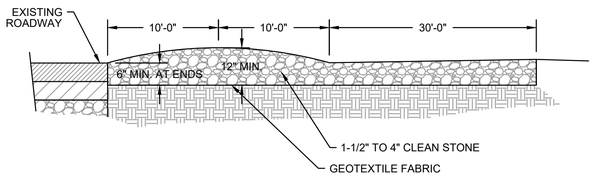
c. THE WIDTH OF THE EROSION MAT SHALL ALWAYS EQUAL THE SOD WIDTH.

d. SOD STRIPS MAY BE PLACED EITHER LONGITUDINALLY OR TRANSVERSELY TO THE FLOW LINE OF THE DITCH.

EROSION MAT OVER SEEDING

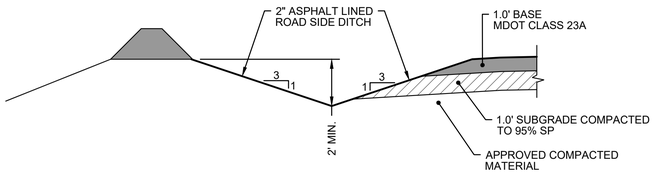
JUNCTION OR ANCHOR SLOTS SHALL BE AT MINIMUM INTERVALS OF 100 FEET (30.48 m) ON GRADES UP TO AND INCLUDING 3 PERCENT, AND 50 FEET (15.24 m) ON GRADES EXCEEDING 3 PERCENT.

1/2 **EROSION MAT DETAIL**
SCALE: NOT TO SCALE

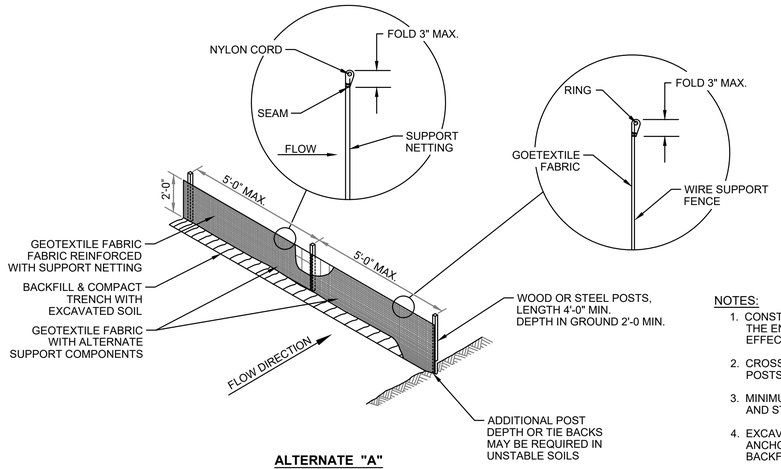


NOTE: PURPOSE IS TO REMOVE MUD FROM TIRES OF CONSTRUCTION VEHICLES. (12' WIDE MINIMUM)

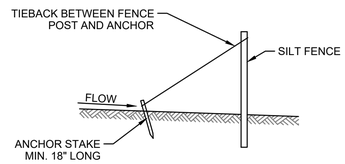
3/2 **TEMPORARY CONSTRUCTION VEHICLE TRACKING PAD**
SCALE: NOT TO SCALE



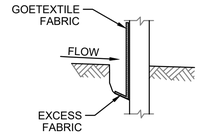
4/2 **ROAD DITCH (TYPICAL)**
SCALE: NOT TO SCALE



ALTERNATE "A"



GEOTEXTILE FENCE TIE BACK (WHEN REQUIRED BY THE ENGINEER

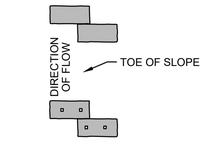


TRENCH DETAIL

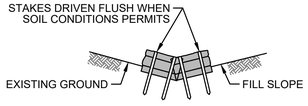
NOTES:

1. CONSTRUCT THE SILT FENCE IN AN ARC OR HORSESHOE SHAPE, WITH THE ENDS POINTING UPSLOPE TO MAXIMIZE BOTH STRENGTH AND EFFECTIVENESS.
2. CROSS BRACE WITH 2"x4" WOODEN FRAME OR EQUIVALENT AT TOP OF POSTS AS DIRECTED BY THE ENGINEER.
3. MINIMUM 14 GUAGE WIRE REQUIRED, FOLD FABRIC 3" OVER THE WIRE AND STAPLE OR PLACE WIRE RINGS ON 12" CENTERS.
4. EXCAVATE A TRENCH A MINIMUM OF 4" WIDE AND 6" DEEP TO BURY AND ANCHOR THE GEOTEXTILE FABRIC. FOLD MATERIAL TO FIT TRENCH AND BACKFILL AND COMPACT TRENCH WITH EXCAVATED SOIL.
5. USE 14 GUAGE MIIMUM WOVEN WIRE SUPPORT FENCE WITH A MAXIMUM MESH SPACING OF 6". SECURE TOP OF GEOTEXTILE FABRIC TO TOP OF FENCE WITH STAPLES OR WIRE RINGS AT 12" CENTERS.
6. GEOTEXTILE FABRIC WILL BE REINFORCED WITH AN INDUSTRIAL POLYPROPYLENE NETTING WITH A MAXIMUM MESH SPACING OF 1" OR EQUAL. A HEAVY DUTY NYLON TOP SUPPORT CORD OR EQUIVALENT IS REQUIRED.
7. STEEL POSTS WILL BE STUDDED "TEE" OR "U" TYPE WITH A MINIMUM WEIGHT OF 1.28 LBS/LINEAL FOOT (WITHOUT ANCHOR). FIN ANCHORS SUFFICIENT TO RESIST POST MOVEMENT ARE REQUIRED. WOOD POSTS SHALL BE A MINIMUM SIZE OF 4" DIA. OR 1"x3" EXCEPT WOOD POSTS FOR GEOTEXTILE FABRIC REINFORCED WITH NETTING SHALL BE A MINIMUM SIZE OF 1"x1" OAK OR HICKORY.
8. ALTERNATES A & B ARE EQUAL AND EITHER MAY BE USED.

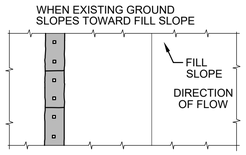
2/2 **SILT FENCE DETAIL**
SCALE: NOT TO SCALE



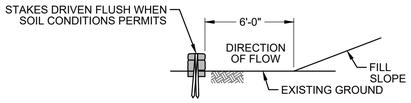
MIN. TWO BALES
PLAN VIEW



FRONT ELEVATION

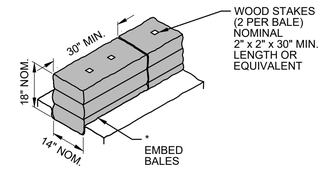


PLAN VIEW

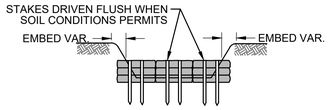


FRONT ELEVATION
WHEN GROUND SLOPES AWAY FROM FILL SLOPE
EROSION BALES AT TOE OF SLOPE

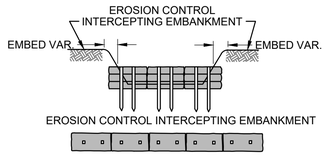
NOTE:
ALL DIMENSIONS ARE APPROXIMATE



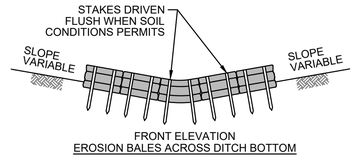
DETAIL OF EROSION BALE INSTALLATION



EROSION CONTROL CHANNEL CHANGE



PLAN VIEW
IF REQUIRED TWO ROWS MAY BE USED WITH STAGGERED JOINTS.



FRONT ELEVATION
EROSION BALES ACROSS DITCH BOTTOM

5/2 **HAY BALE DETAIL**
SCALE: NOT TO SCALE

WHITE PINE COPPER
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SOIL EROSION AND SEDIMENT CONTROL PLAN

WHITE PINE COPPER
ONTONAGON COUNTY, MICHIGAN

REVISIONS			
NO.	BY	DATE	DESCRIPTION

DATE OF PREPARATION

	BY	DATE
SURVEYED		
DRAWN	JRB2	JUNE 2026
DESIGNED	BDS1	JUNE 2026
CHECKED	BDS1	JUNE 2026

SHEET TITLE:

EROSION CONTROL PLAN DETAILS

PRELIMINARY DESIGN

PROJECT NO: 23W200

SHEET NUMBER

2

Appendix A

Project Timeline

Mine Year

	-3	-2	-1	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	47	48	49	50	51	52
	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042	2043	2044	2045	2046	2047	2048	2049	2050	2051	2052	2053	2054	2055	2056	2057	2058	2059	2060	2061	2062	2063	2064	2065	2066	2067	2068	2069	2070	2071	2072	2073	2074	2075	2076	2077	2078	2079	2080	2081

Construction
- Site Clearing and Grubbing
- Topsoil and Overburden Removal
- Water Management System
- Mine Portal
- Mine Decline
- Mine Ventilation
- Roads and utilities
- Bulk Earthworks
- Processing Plant
- Ancillary Buildings & Structures
- Industrial Water Treatment Plant (WWTP)
- Temporary Ore Stockpile Pad
- TDF NP1 Crest widening, Buttress Placement, Deposition Tower Install
- TDF NP1 Deposition Causeway Raises
- TDF NP2 Crest Widening, Ditching, Deposition Tower Install
- TDF NP2 Deposition Causeway Raises

Operations (26 yr LOM)
- Mine, Processign Plant, WWTP, Ancillary Features
- TDF NP1 Deposition
- TDF NP2 Deposition

Reclamation

Phase 1: Concurrent with Construction
- Restoration of temporarily disturbed areas

Phase 2: Concurrent with Operation
- Temporary Ore Stockpile Pad
- Revegetation of TDF NP1

Phase 3: Project Closure
- Removal of Non-Inert Mine Infrastructure & Equipment
- Installation of Bulkheads in Ventillation Raises & Decline
- Rapid flooding of Mine
- Installation of Concrete Plug at Portal Entrance
- Removal of Infrastructure on TDF NP1 & NP2 (not required for WWTP operation)
- Revegetation of TDF NP2
- Decomissioning of Processing Plant
- Removal of Ancillary Buildings, Temporary Ore Stockpile
- Removal of Infrastructure not required to operate WWTP
- Removal of Non-Permanent Stormwater Controls
- Grading and Re-use of Stockpiled Top Soil, Revegetation
- Removal of WWTP and Remaining Infrastructure
- Construction of Discharge Spillways TDF NP1 & NP2
- Removal of remaining Site Infrastructure (roadways, utilities, etc.)

Phase 4: Post Closure
- Monitoring and Maintenance (20 yr)

Notes
1. Timeline is approximate
2. TDF = Tailings Disposal Facility
3. NP1 = North Pond 1
4. NP2 = North Pond 2

Appendix B

Soil Erosion and Sedimentation Control Inspection and Maintenance Log

Soil Erosion and Sedimentation Control Inspection and Maintenance Log
White Pine Copper LLC

rea (s)of Inspection: _____ Inspector: _____

Certified Operator Name/Number: _____ Date/Time: _____

Reason for Inspection: _____Weekly Inspectior _____ Inspection After Stormwater Runoff

List Construction Activities Taking Place:

Inspection Results

Recommended Actions

Follow-up of Previous Recommendations

Date of Next Inspection: _____

Archive Inspection Form Until: _____ (5 yrs from date of inspection)

Appendix E

Tailings Management, Ore Storage, Water Management, and Water Treatment

E-1 Proposed White Pine Copper Water Treatment Plant Technical Memorandum
E-2 Treatment and Containment Plan
E-3 Tailings Management Design Report
E-4 Water Management Design Technical Report
E-5 Temporary Ore Stockpile Liner Design Criteria Technical Memorandum

E-1

Proposed White Pine Copper Water Treatment Plant

Technical Memorandum

Technical Memorandum

5117 West Terrace Drive, Suite 401
Madison, WI 53718
(608) 242-5900
foth.com

August 20, 2026

TO: Chada Wolahan, White Pine Copper LLC
Amanda Zeidler, White Pine Copper LLC

CC: Clinton Roberts, Foth Infrastructure & Environment, LLC

FR: Rob Brillhart, P.E., Foth Infrastructure & Environment, LLC

RE: Proposed White Pine Copper Water Treatment Plant

White Pine Copper LLC (WPC) is seeking to develop the White Pine North Project (Project) and planning to mine underground east of, and separate from, the historic underground mine workings of White Pine Mine. Mining operations will require treatment of wastewater generated during mining operations and run-off due to precipitation.

This Technical Memorandum (Technical Memo) presents the proposed 30% Basis of Design (BOD), discussion of the proposed process water treatment plant, and selected system sizing.

Process Wastewater Characterization

Process Wastewater Flow Estimates

Process water generation rates are likely to be variable from year to year due to mine dewatering rates, annual precipitation, and process water requirements. Table 1 provides the current water balance estimates for daily water treatment plant flows for the first 11 years of mining operations and beyond.

Table 1
Process Wastewater Flow Estimates

Years of Operation	Estimated Flow Rate (gpm)	Estimated Daily Flow (MGD)
1-11	2,250	3.25
12-26	4,500	6.48

gpm = gallons per minute
MGD = million gallons per day

Prepared by: RGB2
Checked by: KMB

Process Wastewater Quality Characteristics

Table 2 provides estimated process wastewater characteristics from the Tailings Disposal Facility (TDF) used to establish the proposed treatment train. The water quality data is based upon geochemical modeling completed by SRK Consulting.

Table 2
Estimated Process Wastewater Characteristics

Analyte	Units	Average	Peak
Alkalinity	mg/L	117	140
Aluminum	mg/L	0.0013	0.0017
Ammonia	mg/L	5.5	10.0
Antimony, Total Recoverable	mg/L	0.0018	0.0021
Arsenic, Total Recoverable	mg/L	0.0018	0.0020
Barium, Total Recoverable	mg/L	0.0467	0.0606
Beryllium, Total Recoverable	mg/L	0.0004	0.0006
Boron, Total Recoverable	mg/L	0.8281	0.9400
Bromide	mg/L	3.164	3.682
Cadmium, Total Recoverable	mg/L	0.0009	0.0015
Calcium	mg/L	108	139
Chloride	mg/L	305	385
Chromium, Total Recoverable	mg/L	0.0053	0.0060
Cobalt, Total Recoverable	mg/L	0.0088	0.0113
Copper, Total Recoverable	mg/L	0.0526	0.0732
Fluoride	mg/L	0.5430	0.6197
Iron	mg/L	0.0043	0.0081
Lead, Total Recoverable	mg/L	0.0028	0.0043
Lithium	mg/L	0.0724	0.0823
Magnesium	mg/L	11	14
Manganese	mg/L	0.1505	0.2256
Mercury, Total Recoverable	ng/L	3.1498	4.618
Molybdenum, Total Recoverable	µg/L	0.0348	0.0491
Nickel, Total Recoverable	µg/L	0.0089	0.0137
Nitrate	mg/L	0.0162	0.2355
pH	S.U.	7.0	7.3
Phosphorus, Total as P	mg/L	0.4182	0.6631
Potassium	mg/L	11	12
Selenium, Total Recoverable	mg/L	0.0011	0.0016
Silver, Total Recoverable	mg/L	0.0002	0.0002
Sodium	mg/L	126	155
Strontium, Total Recoverable	mg/L	1.4732	1.8278

Table 2 (continued)

Analyte	Units	Average	Peak
Sulfate	mg/L	77.6324	102.7830
Sulfide	mg/L	0.0000	0.0000
Thallium	mg/L	0.0004	0.0005
Tin	mg/L	0.0026	0.0030
Total Dissolved Solids	mg/L	768	960
Total Suspended Solids	mg/L	22(1)	120(1)
Uranium	mg/L	0.0044	0.0056
Vanadium, Total Recoverable	mg/L	0.0430	0.0684

mg/L = milligram per liter ng/L = nanogram per liter Prepared by: RGB2
µg/L = microgram per liter s.u. = standard unit Checked by: KMB
ND = not detected
Foth Infrastructure & Environment, LLC (Foth) used 650 mg/L as the average total suspended solids (TSS) concentration for the 30% Design based on input from WPC.

Water Treatment Plant Effluent Limits

Table 3 summarizes the estimated preliminary effluent limits (PELs) for the proposed Water Treatment Plant. The effluent limits below have been estimated in accordance with Michigan Department of Environment, Great Lakes, and Energy (EGLE) Part 4 standards and Mineral River samples, collected over the period of July 2024 to February 2026.

Table 3
Non-Domestic Wastewater – Preliminary Effluent Limits

Analyte	Units	Value
Ammonia	mg/L	pH and temperature dependent
Antimony, Total Recoverable	µg/L	130.35
Arsenic, Total Recoverable	µg/L	10.03
Barium, Total Recoverable	µg/L	299.88
Beryllium, Total Recoverable	µg/L	3.62
Boron, Total Recoverable	µg/L	7,219.7
Cadmium, Total Recoverable	µg/L	3.6
Chloride, Total[1]	mg/L	200
Chlorine, Total Residual	mg/L	0.019
Chromium, Total Recoverable	µg/L	82.75
Cobalt, Total Recoverable	µg/L	100.27
Copper, Total Recoverable	µg/L	9.85
Fluoride	mg/L	2.11
Lead, Total Recoverable	µg/L	83.27
Mercury, Total Recoverable	ng/L	1.30
Molybdenum, Total Recoverable	µg/L	3,208.77
Nickel, Total Recoverable	µg/L	42.16
pH	S.U.	6.5 – 9.0

Table 3 (continued)

Analyte	Units	Value
Phosphorus, Total as P	mg/L	1.0
Selenium, Total Recoverable	µg/L	5.01
Silver, Total Recoverable	µg/L	0.06
Strontium, Total Recoverable	µg/L	36,099
Total Dissolved Solids	mg/L	---[2]
Vanadium, Total Recoverable	µg/L	27.07
Zinc, Total	µg/L	172.19

[1] Site-specific limit requested instead of EGLE Part 4 Rule 57 criteria. Prepared by: RGB2
[2] Water quality value not provided in EGLE Part 4 Rule 57 criteria. Checked by: KMB

Proposed Treatment Train Basis of Design

The proposed treatment train BOD is based on the influent concentrations provided in Table 2, the anticipated effluent requirements outlined in Table 3, and an influent flow rate of 2,250 gpm during the first 11 years of mine operations, increasing to 4,500 gpm for subsequent years. The design assumes a minimum influent water temperature of 7.22°C.

Proposed Treatment Train

The proposed 30% design treatment train includes the following individual process units:

1. Influent pumping system
2. Intermediate Ultrafiltration (UF) equalization and pumping system
3. UF
4. Reverse osmosis (RO) buffer/RO influent equalization and pumping system
5. Building sump for collection of UF retentate, clean-in-place (CIP) wastes, and building floor drains
6. Primary solids settling and dewatering system – inclined plate clarification and filter press
7. Three-stage RO
8. Seawater Reverse Osmosis (SWRO) buffer/equalization and pumping system
9. SWRO for further concentrating process retentate pollutants
10. Evaporation/crystallization

The following briefly provides a summary of each major process technology proposed for the 30% design.

Ultrafiltration

UF is a cross-flow separation process that utilizes a membrane and pressure to separate high molecular-weight substances, suspended solids, colloidal materials, and organic and inorganic polymeric molecules from water. UF membranes typically have a pore size in the range of 0.1 to 0.001 micron. The feed stream flows tangentially along the membrane surface producing two separate streams called the permeate stream and the concentrate or retentate stream. Pressure is applied across the membrane surface promoting the passage of water and low molecular-weight substances and ions across the membrane. This stream is typically referred to as the permeate stream. The high molecular-weight substances, suspended solids, colloidal materials, and organic and inorganic polymeric substances larger than the pore size of the

membrane are retained in the feed stream. In wastewater applications, the retentate stream is recirculated through the UF unit concentrating the rejected materials and reducing the volume of waste requiring further processing or off-site disposal. In crossflow separation, the membrane does not physically collect materials desired for removal but rather acts as a barrier to these materials remaining in the “treated” permeate stream.

UF membranes come in a variety of configurations such as plate-and-frame, spiral-wound, and tubular. Each configuration has advantages and disadvantages depending upon the characteristics of the feed stream. Spiral-wound and hollow fiber configurations are generally used in low solids applications where solids will be less likely to plug the membrane. Tubular configurations are generally used when the feed stream has the potential to contain elevated levels of TSS or high levels of emulsified fats, oils, and grease which allow for more aggressive mechanical cleaning once the membrane becomes fouled. In all configurations, the optimum system design must take into consideration flow velocity, pressure drop, power consumption, membrane fouling, and module cost.

UF system sizing is based on the flux or permeation rate per unit area of membrane surface area. Membrane flux is defined as the amount of permeate produced per unit area of membrane surface per unit time expressed as gallons per square foot per day (GFD) or as cubic meters per square meters per day. UF systems generally operate at lower pressures than other membrane processes (i.e., RO) since the osmotic pressure across the membrane surface is negligible. Typical flux rates for UF membranes range between 50 and 200 GFD at an operating pressure of about 75 pounds per square inch gauge.

The performance of UF systems is affected by feed stream velocity, operating pressure, and feed stream temperature. Generally, as the velocity of the feed stream across the membrane surface increases, the permeation rate or flux rate increases.

The main benefit to utilizing a UF process in this application is the ability of the membrane to remove more than 99% of particulates and colloidal solids that would otherwise plug the downstream RO unit.

UF systems do require periodic operation and maintenance to maintain adequate permeate flows and system performance. Periodic cleaning cycles will be required to maintain system performance. In addition, UF membranes have a typical service life of 5 to 7 years at which time the membranes will require periodic replacement.

Clarification and Dewatering

An inclined plate clarifier, often referred to as a lamella clarifier, is a compact and highly efficient unit process used to separate suspended solids from water by directing flow through a closely spaced series of plates set at an angle of roughly 55 to 60 degrees. As water moves upward through the plate assembly, particles settle onto the upper surfaces of the plates below and then slide downward by gravity into a sludge hopper. This configuration provides a substantially larger effective settling area within a much smaller physical footprint than a conventional settling basin.

The process relies on counter-current movement, with solids traveling downward while clarified water rises between the plates. This arrangement increases settling efficiency and makes it possible to achieve up to an 80% reduction in required space compared to traditional clarifier

designs. The plate assemblies are commonly fabricated from materials such as polypropylene, PVC, or stainless steel, selected for durability and compatibility with specific wastewater characteristics.

In industrial wastewater treatment applications, inclined plate clarifiers are widely used where high solids loading demands effective particle removal within limited space. Their design supports reliable operation, simplified maintenance, and reduced capital cost relative to larger settling tanks, making them an advantageous clarification option for many process and treatment systems.

Solids removal is often enhanced with the addition of coagulants and polymers to increase the size and density of the solids. Provisions for the addition of coagulants and polymers has been provided in the 30% design. Clarified wastewater will discharge back to the UF buffer tank for reprocessing. Settled solids will be pumped to a settled solids collection tank for temporary storage prior to further dewatering using filter press technology.

A filter press is a heavy-duty, batch-operated solids-liquid separation system that uses pressure filtration to produce a clarified filtrate and a highly dewatered solids cake. During operation, slurry is pumped into a sealed pack of recessed or plate-and-frame assemblies, each lined with filter cloth. As pressure builds, liquid is forced through the cloth while solids accumulate within the plate chambers, gradually forming a dense filter cake. A hydraulic system maintains the required closing force on the plate pack to ensure the assembly remains sealed and capable of withstanding high internal pressures.

Once the chambers are filled and filtration is complete, hydraulic pressure is released and the plates are separated, allowing the formed cake to discharge either manually or through automated mechanisms. The process is widely applied across industries such as mineral processing, where it is used for dewatering tailings streams, and wastewater treatment, where it plays a key role in sludge dewatering.

Reverse Osmosis

RO is a water purification technology that uses a semipermeable membrane to remove ions, molecules, and larger particles from drinking water. Similar to UF, RO uses an applied pressure to overcome the osmotic pressure of the membrane to allow water to permeate across the membrane resulting in a permeate stream and a retentate stream. The main difference between an UF and RO membrane is the size of molecules that can be separated from water. In an RO application, low-molecular weight molecules and ions (heavy metals) can be removed with an RO membrane whereas these compounds will pass through an UF membrane.

RO is one the most effective technologies available for treating low concentrations of pollutants without generating large quantities of by-products that would require off-site disposal. Advantages of RO technology in mining applications include:

- High pollutant removal efficiency – up to 99% removal of dissolved salts and heavy metals producing a high-quality effluent.
- Ability to consistently meet compliance with stringent standards

- Compact and scalable systems – RO plants are modular and can be customized to handle variable flow rates.

Remineralization

Remineralization is a required post-treatment process for RO-treated effluent prior to discharge to surface waters. RO permeate is characterized by very low total dissolved solids, alkalinity, and hardness, and may exhibit depressed pH and chemically aggressive conditions. Without stabilization, such water may adversely affect conveyance infrastructure, outfall structures, and receiving water.

Remineralization restores alkalinity and hardness—primarily as calcium carbonate—bringing the effluent into compliance with applicable pH criteria. From an infrastructure perspective, failure to remineralize RO-treated water can result in accelerated corrosion of metallic piping and appurtenances, dissolution of concrete linings or structures, and mobilization of metals from downstream materials. From a water quality standpoint, discharge of unstable, low-alkalinity effluent may adversely affect aquatic organisms, particularly in receiving waters with limited buffering capacity.

Seawater Reverse Osmosis

SWRO systems differ fundamentally from standard RO units due to the extremely high-salinity feedwater and as such operate under substantially higher pressures and more demanding material conditions. To overcome the osmotic pressure associated with seawater—typically more than 35,000 mg/L total dissolved solids—SWRO units operate at pressures in the range of approximately 600 to 1,200 pounds per square inch (psi), compared to the 150 to 250 psi commonly used for brackish or tap-water applications. This higher operating pressure requires the use of specialized materials such as high-strength, corrosion-resistant alloys and/or corrosion-resistant plastics to withstand the aggressive nature of seawater.

The membranes themselves are also designed to provide extremely high salt rejection rates exceeding 99%. Treating seawater typically necessitates more extensive pretreatment to manage suspended solids, biological activity, and organic matter, which may include media filtration, ultrafiltration, or other conditioning processes. Energy consumption is correspondingly higher due to the elevated operating pressures. Due to high operating pressures and associated material and energy costs, a conventional or standard three-stage RO system is being provided to reduce the volume of concentrated retentate requiring processing with the SWRO unit.

Permeate from the SWRO will either be blended with the permeate from the preceding three-stage RO system or discharge back to the RO buffer tank for reconcentrating of pollutants with the conventional RO system.

Evaporation and Crystallization

Evaporation and crystallization processes are used to separate and recover solid materials from liquid solutions by removing the solvent, in this case water, and driving the solution to conditions where crystals can form. In the evaporation stage, the liquid is heated—often under reduced pressure—to vaporize the water and increase the dissolved solids concentration until the solution approaches saturation. Once sufficiently concentrated, the liquid enters the crystallization phase, where controlled cooling or further evaporation creates a supersaturated

environment that promotes the growth of crystals. The resulting solids are then separated from the remaining water through downstream solid-liquid separation equipment.

In advanced wastewater treatment, evaporation and crystallization are key components of zero-liquid-discharge systems, allowing facilities to recover clean water while converting high-salinity wastes into a manageable solid form. The same principles are applied to recover valuable salts and metals from industrial brines, including those used in energy and mining sectors. Because crystal size, shape, and distribution can be precisely managed through operating conditions, the process can be tailored to meet strict product quality requirements across a wide range of applications.

Proposed System Description

The following provides a high-level description and estimates of the power requirement for each process step of the proposed treatment train for the initial 2,250-gpm flow. See Attachment 1 for the proposed process flow diagram of the proposed system. Please note the below system descriptions/parameters are subject to change based on future updates to influent flow and wastewater characterization data.

Influent Pumping Station

Pumping rate per pump	2,250 gpm
Assumed pump total developed head	80 feet (ft)
Approximate horsepower (HP) per pump	100 HP

UF Buffer Tank/Equalization Tank and Pumping

Number of tanks	2
Tank Volume per tank	10,000 gallons
Number of transfer pumps	2
Pump total developed head	80 ft
Approximate HP per pump	60 HP

Ultrafiltration Process Unit

Gross Feed Rate	2,250 gpm
Membrane Type	Ultrafiltration, hollow-fiber
Number of Skidded Units	4 (3 operating, 1 stand-by)
Number of Membrane Modules per Unit	64 installed
Membrane Area per Unit	969 square feet (ft^2)
Flux Rate at Design Temp	19.4 GFD
Normalized Flux at 68°F	29.7 GFD
Water Recovery Rate	94%
Number of Feed Pumps	4 @ 30 HP (3 operating, 1 stand-by)
Number of Backwash Pumps	2 @ 30 HP
Number of Prefilters	4 (200 micron)
Skid Unit Dimensions	28’-10” L x 6’-9” W x 12’-4” H
CIP System Recirculation Pump	1 @ 15 HP
Total Operating Power Requirement	70-kW-hr

RO Buffer/EQ Tank and Pumping

Number of tanks	2
Tank Volume per tank	10,000 gallons

Number of transfer pumps	2
Pump total developed head	80-ft
Approximate HP per pump	60 HP

Inclined Plate Clarifier

Peak Flow Rate	400 gpm
Flash Mix Tank Volume	270 gallons
Flocculation Tank Volume	874 gallons
Sludge Capacity	960 gallons
Plate Area	1,634 ft^2
Projected Plate Area	937 ft^2
Design Solids Removal	up to 95%
Surface Loading Rate	0.43 gpm/ft^2 @ peak flow
Retention Time	15 minutes @ peak flow

Filter Press

Feed Pressure	up to 100 psig
Plate Size	2,000 millimeter (mm) x 2,000 mm
Cake Thickness	32 mm chambers
Press Capacity	460 ft^3
Filtration Area	9,011 ft^2
Filter Plates	Polypropylene, non-gasketed, recessed
Filter Cloths	Polypropylene, mono/multi-filament
Electrical	460 VAC, 3 phase, 60 hz, 20 amps

RO Process Unit

Gross Feed Rate	2,115 gpm
Membrane Type	RO, spiral wound
Number of Skidded Units	3 (no redundancy)
Array (number of stages)	16:8:4
Nominal Rejection Rate	95-99%
Number of Membrane Modules per Unit	168
Membrane Area per Element	400 ft^2
Element Diameter	8 inches
Flux Rate at Design Temp	12.1 GFD
Water Recovery Rate	90%
Number of High-Pressure Pumps	3 @ 200 HP
Number of Interstage Booster Pumps	3 @ 10 HP
Number of Chemical Pumps	3
Skid Unit Dimensions	24’-6” L x 8’-5” W x 11’ H
CIP System Recirculation Pump	1 @ 40 HP
Total Operating Power Requirement	450-kW-hr

Evaporation-Crystallizer Process Unit

Gross Feed Rate	210 gpm
Membrane Type	Brackish water
Permeate flow	128.2 gpm
Permeate total dissolved solids (TDS)	400 mg/L
Brine (concentrate) flow	60.3 gpm
Brine TDS	41,650 mg/L

Recovery	68%
Evaporator Crystallizer solids	0.6 TPH @ 10% moisture
Distillate Flow	59.3 gpm
Distillate TDS	300 mg/L
Recovery	98%
Total Operating Power Requirement	915-kW-hr

30% Design Water Treatment Plant Building Size and Power Consumption

The following section provides updated power consumption and building size for the pumping and transfer systems, as well as the UF, RO, inclined plate clarifier, filter press, SWRO and evaporation-crystallization units, based on the BOD stated above and vendor-supplied data.

The following are key assumptions used to develop power requirements and building size estimates:

1. Influent flow assumed to be an average flow of 2,250 gpm or 3.24 MGD.
2. Historical water quality data collected from North Pond 1 as detailed in this Technical Memo.
3. Effluent water quality requirements based on PELs developed in accordance with EGLE Part 4 Standards and Mineral River samples collected over the period of July 2024 to February 2026.
4. Building size based on initial 2,250 gpm treatment requirement and the full build out to accommodate treatment equipment to handle flows up to an average flow of 4,500 gpm or 6.48 MGD.
5. An influent water temperature of 7.22°C during winter operation and 18°C during summer operation.

The following are Project risks that may impact the power requirements and building size estimates presented below:

1. Updates or changes with the influent water quality data may impact the proposed treatment train including final costs, building size, power requirements, and solids generation rates requiring disposal.
2. Final effluent water quality limits may impact the proposed treatment train including need for additional processing units, costs, building size, power requirements, and solids generation rates requiring disposal.
3. Subsurface improvements (if required) to support bearing capacity of treatment building and equipment are not included in 30% design presented in this Technical Memo.
4. Electrical utility to bring the minimum required service to the proposed treatment building are not included in the 30% design presented in this Technical Memo.
5. Natural gas utility (if utilized) to bring the minimum required service to the proposed treatment building are not included in the 30% design presented in this Technical Memo.

Power Consumption Estimate

Estimated average power consumption estimate – 1,850-kW-hr at initial system start-up (2,250 gpm).

Total estimated average power consumption estimate – 3,770-kW-hr at full build out of system (4,500 gpm).

Building Footprint Estimate

Estimated building footprint – 200 ft x 200 ft x 18 ft clear height or approximately 40,000 ft^2.

Above estimated building footprint includes initial build out to accommodate an average influent flow of 2,250 gpm or 3.24 MGD, including all equipment and office space.

Additional estimated building footprint – 200 ft x 70 ft x 18 ft clear height or approximately 14,000 ft^2.

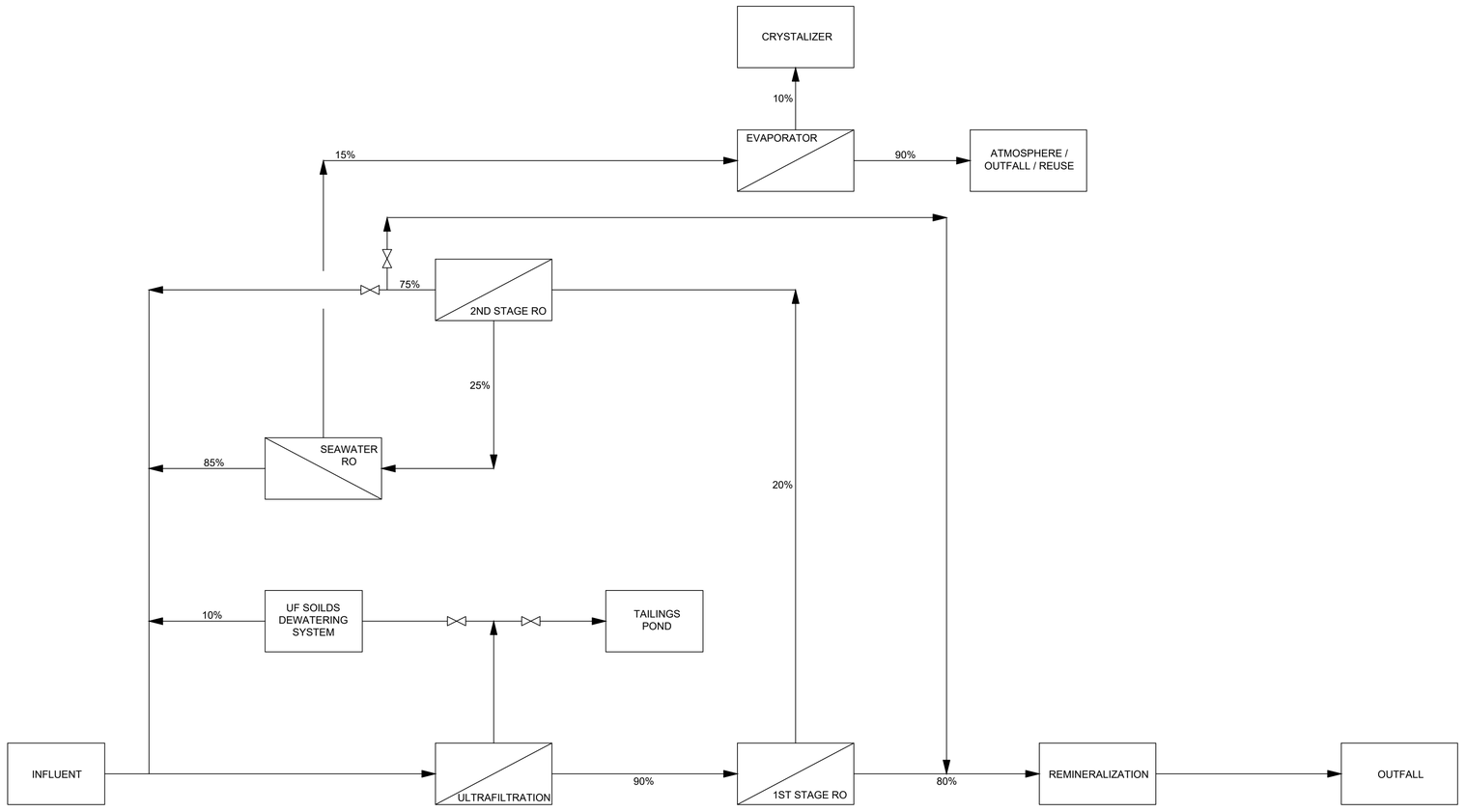
Above estimated building footprint includes the additional footprint required for the future full build out to accommodate an average influent flow of 4,500 gpm or 6.48 MGD, including all equipment above

Attachments:

Attachment 1	Proposed Mining Process Water Treatment System Process Flow Diagrams
Attachment 2	Proposed Equipment Layout for the Initial (2,250 gpm) and Future Flows (4,500 gpm)

Attachment 1

Proposed Mining Process Water Treatment System Process Flow Diagrams



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BLOCK FLOW DIAGRAM
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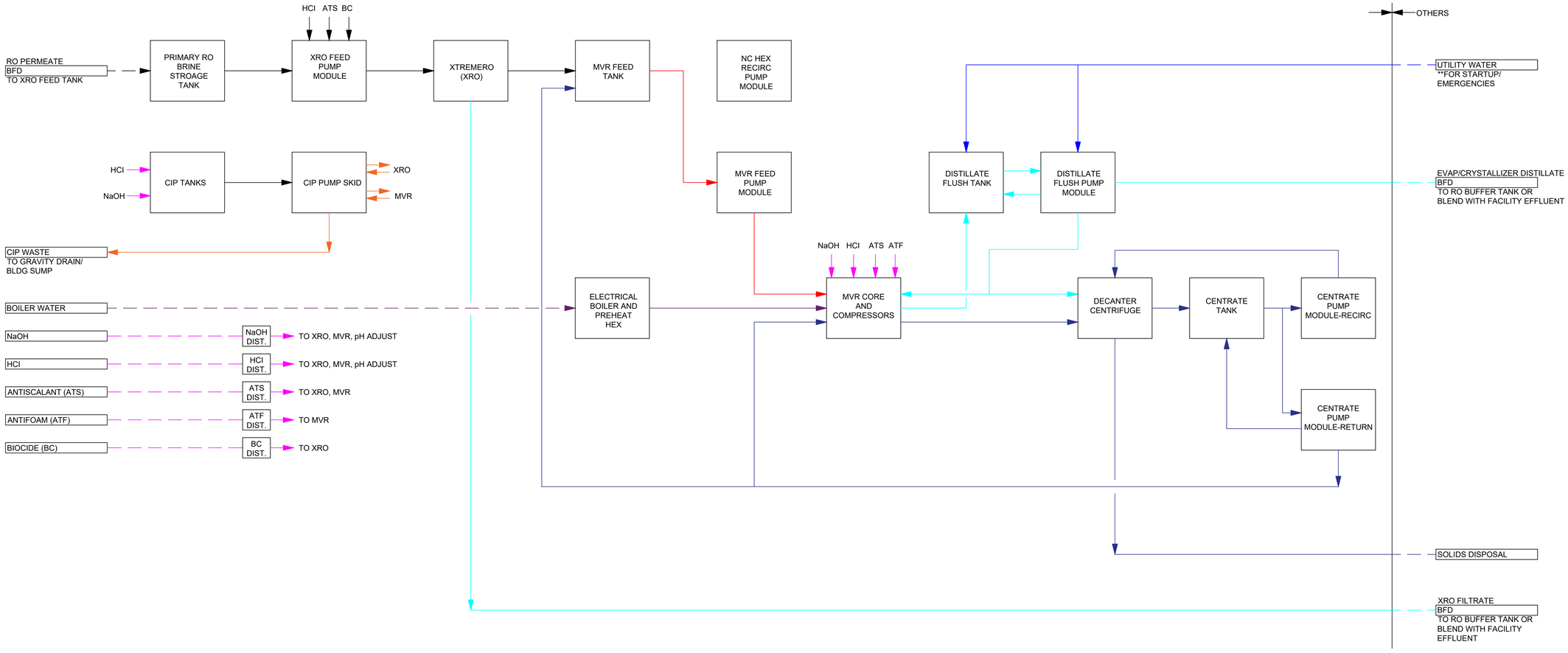

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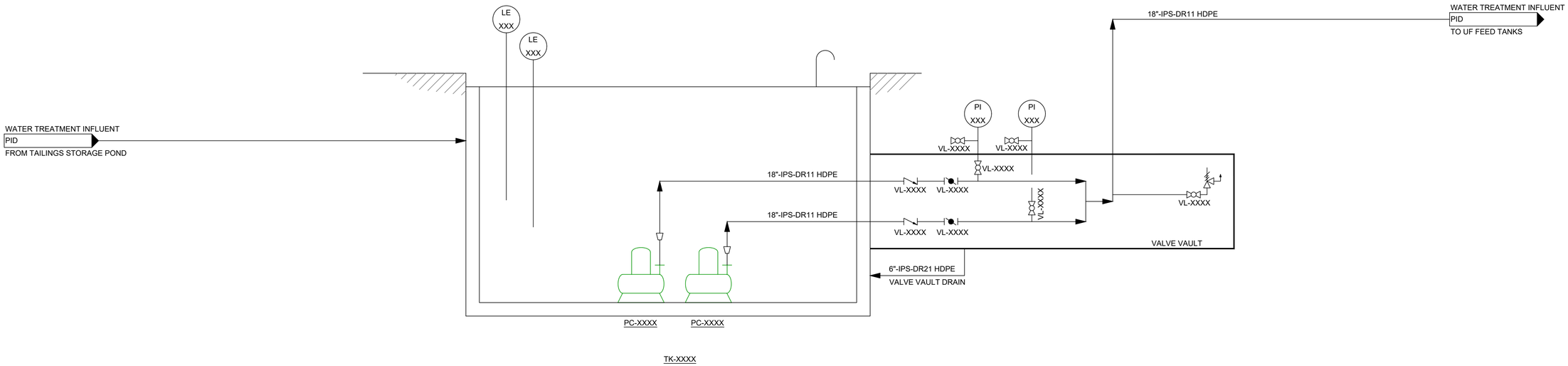

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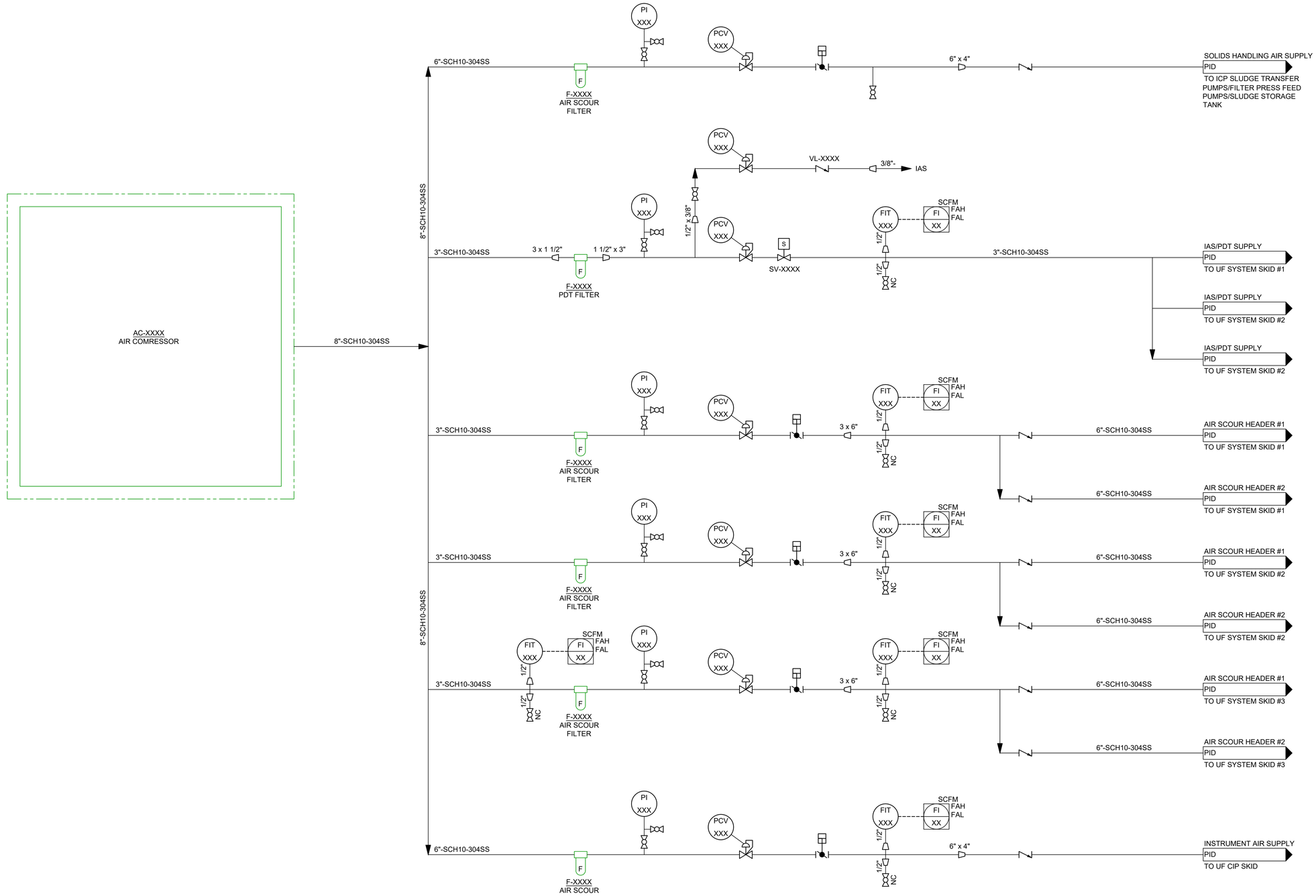

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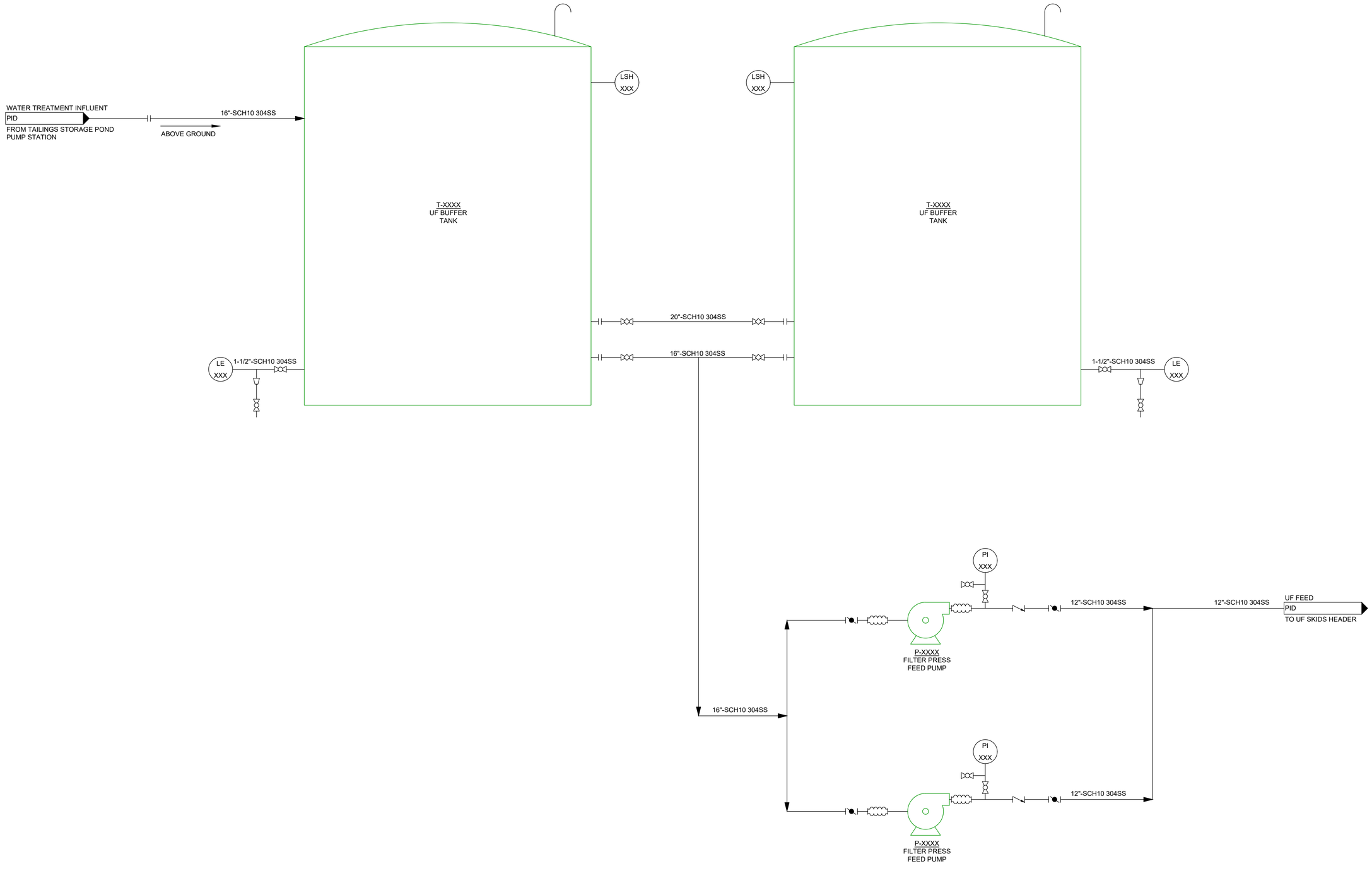

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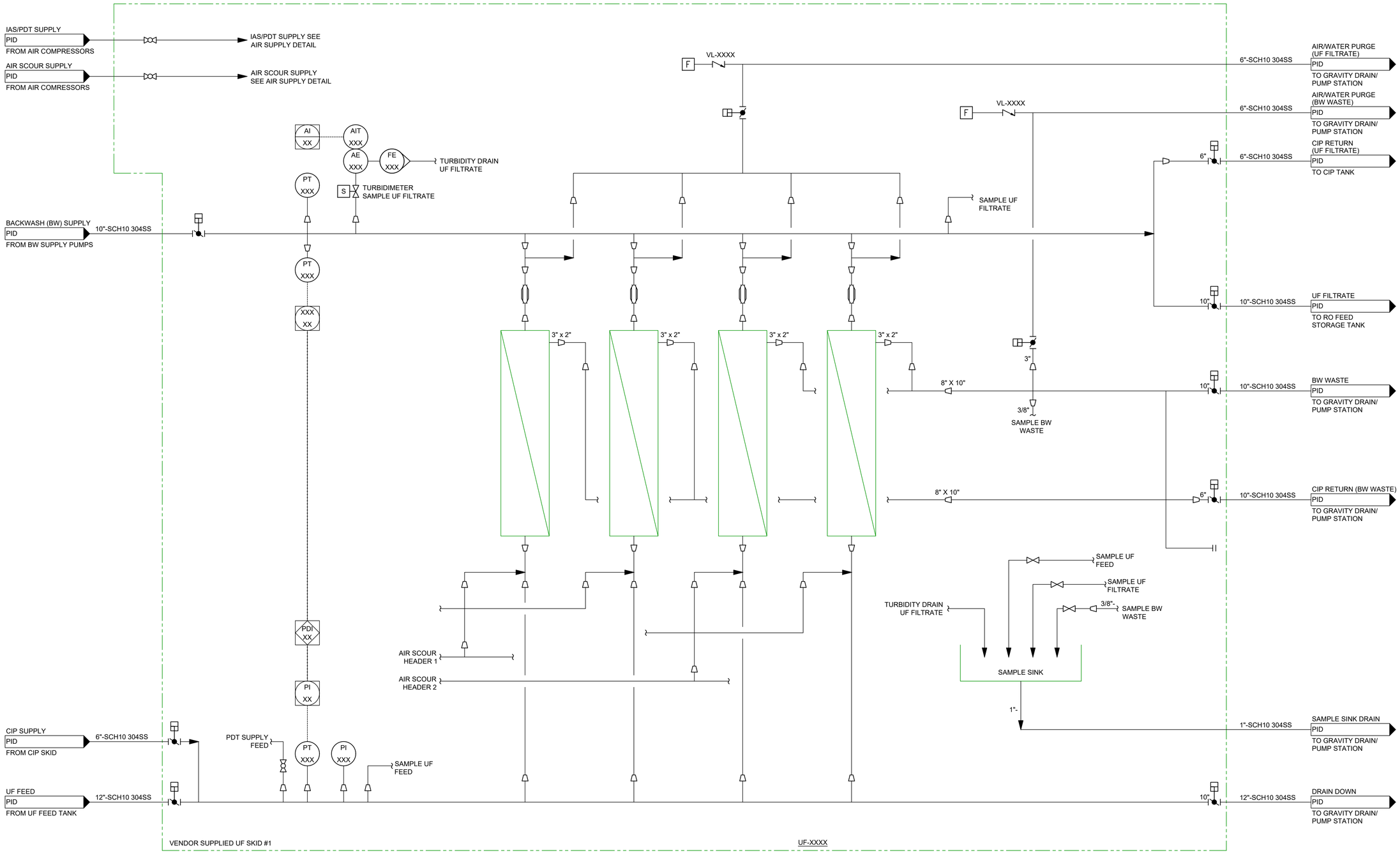

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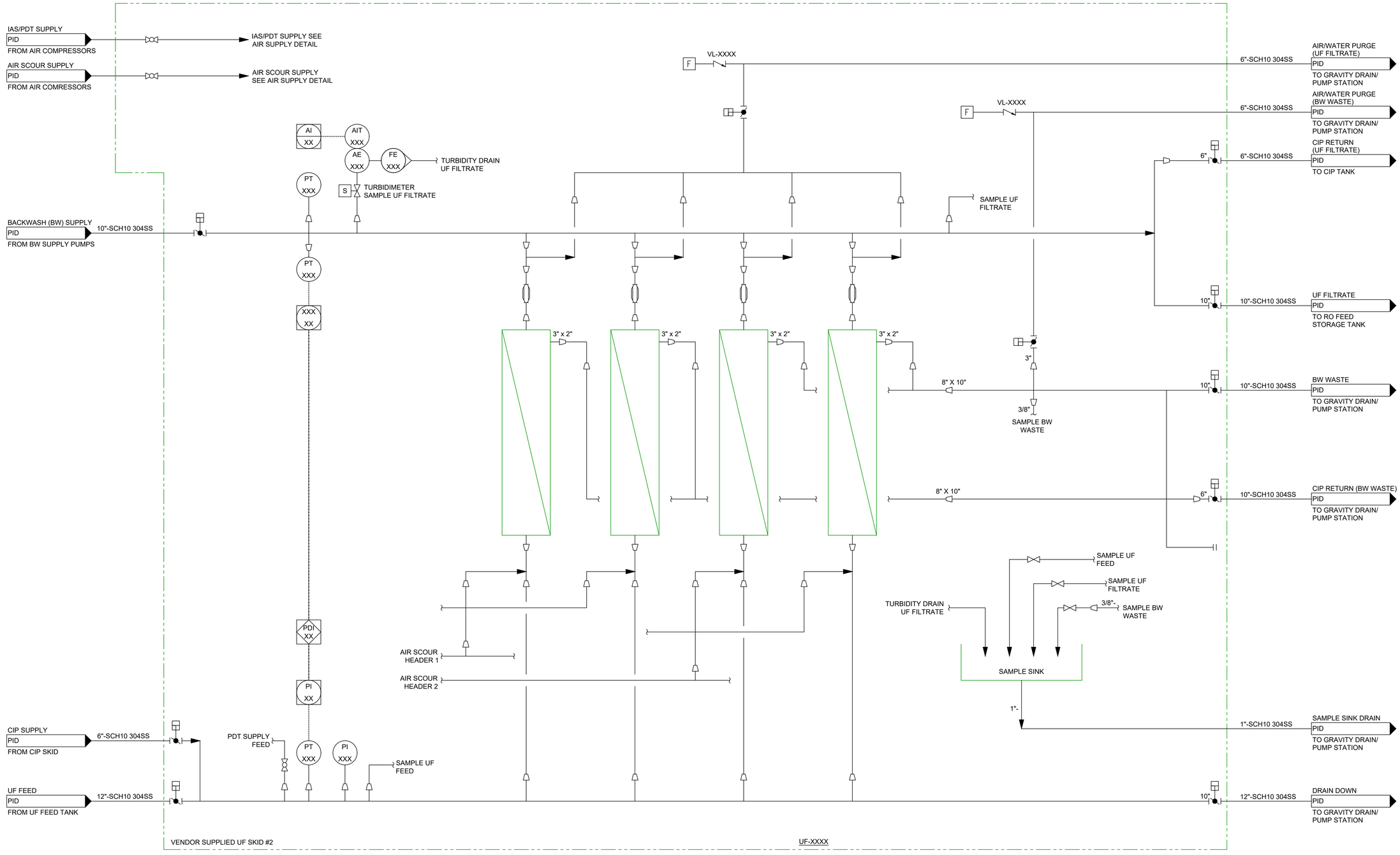

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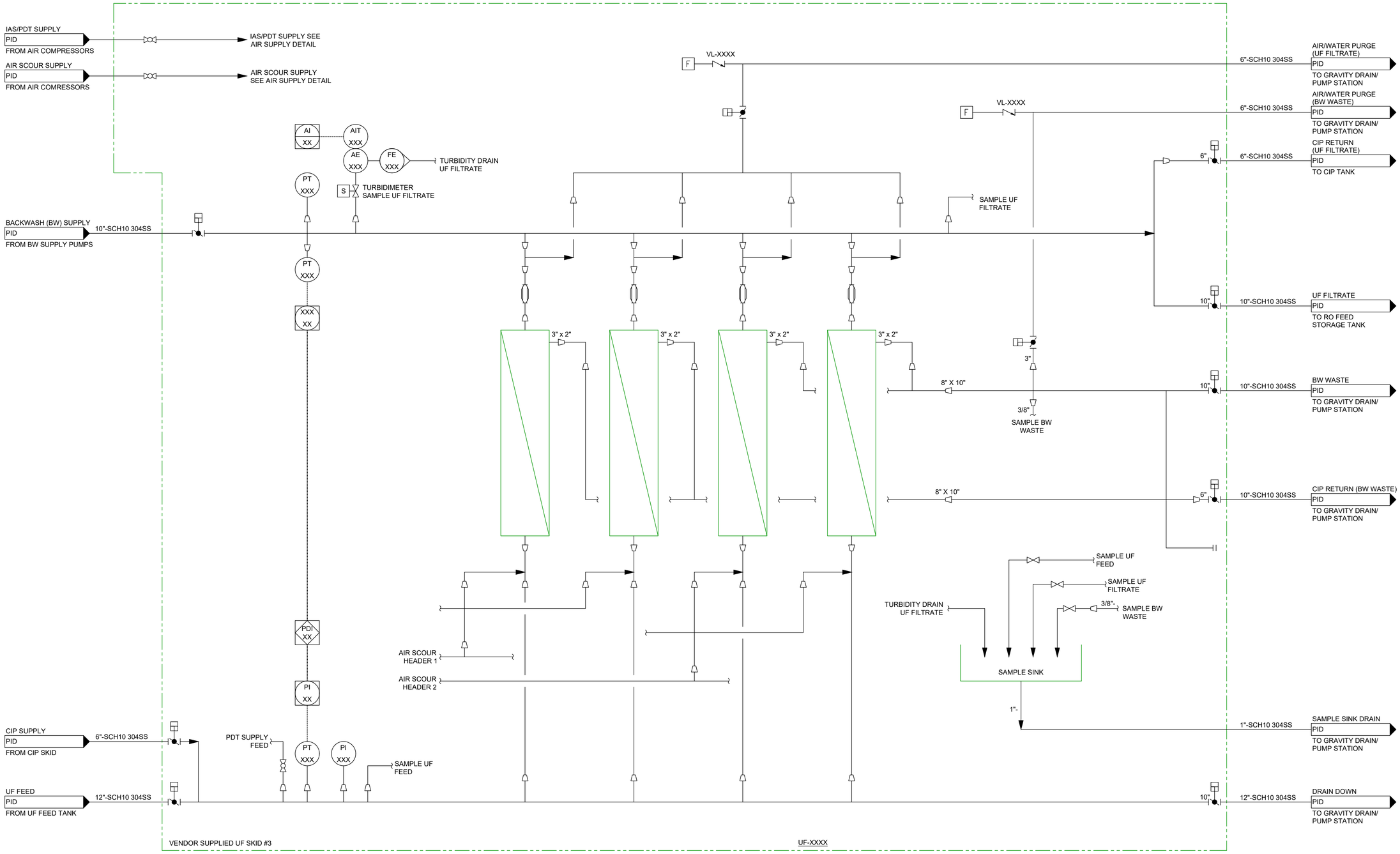

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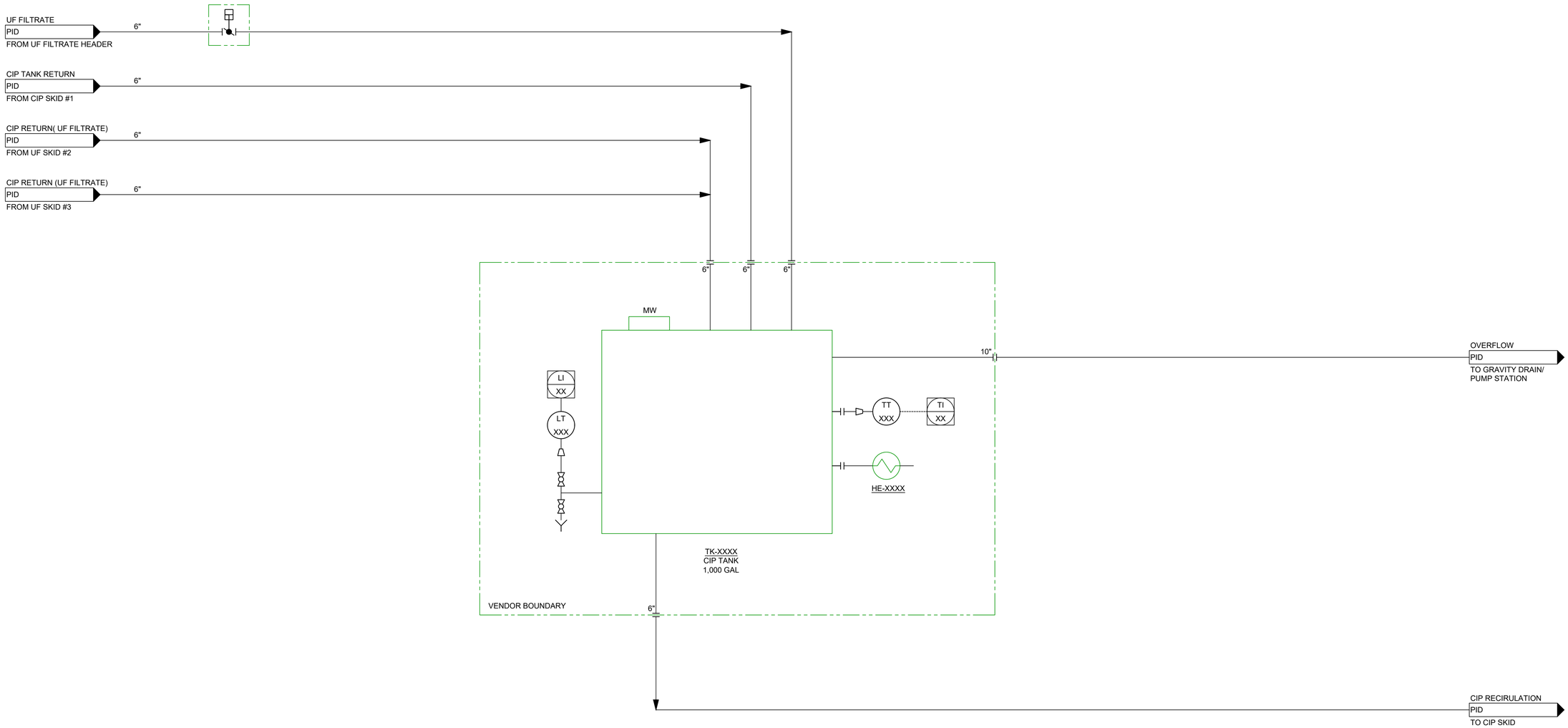

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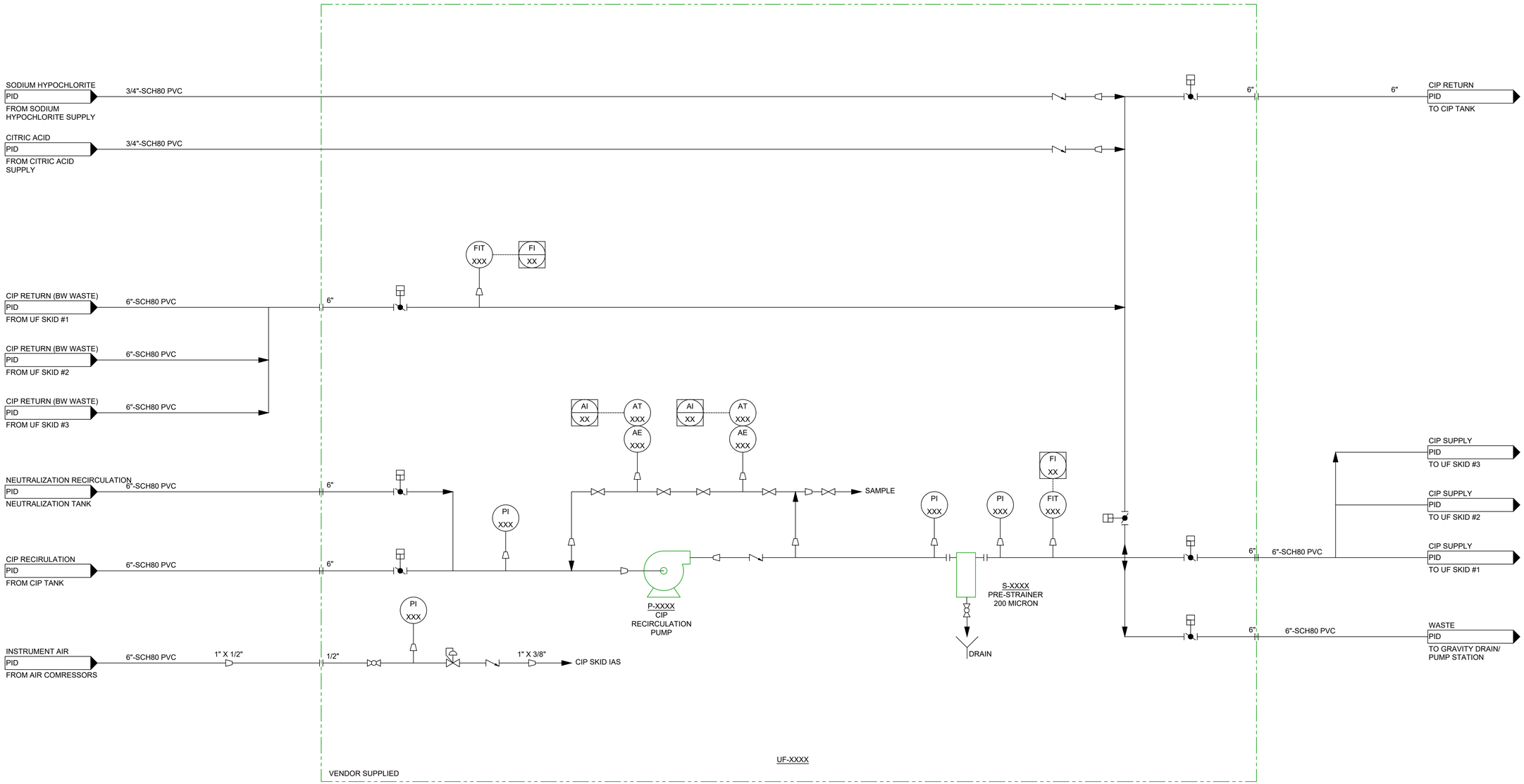

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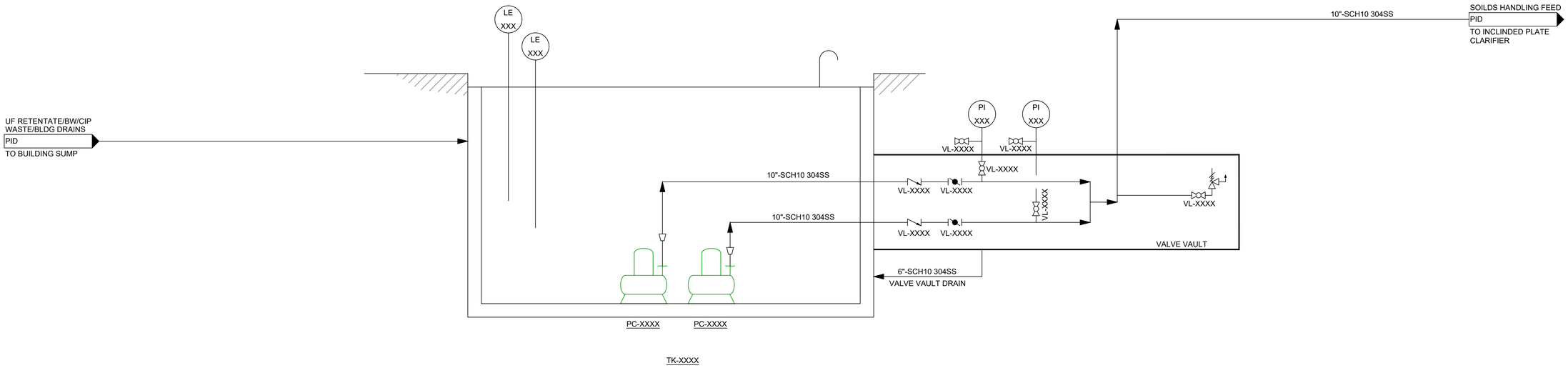

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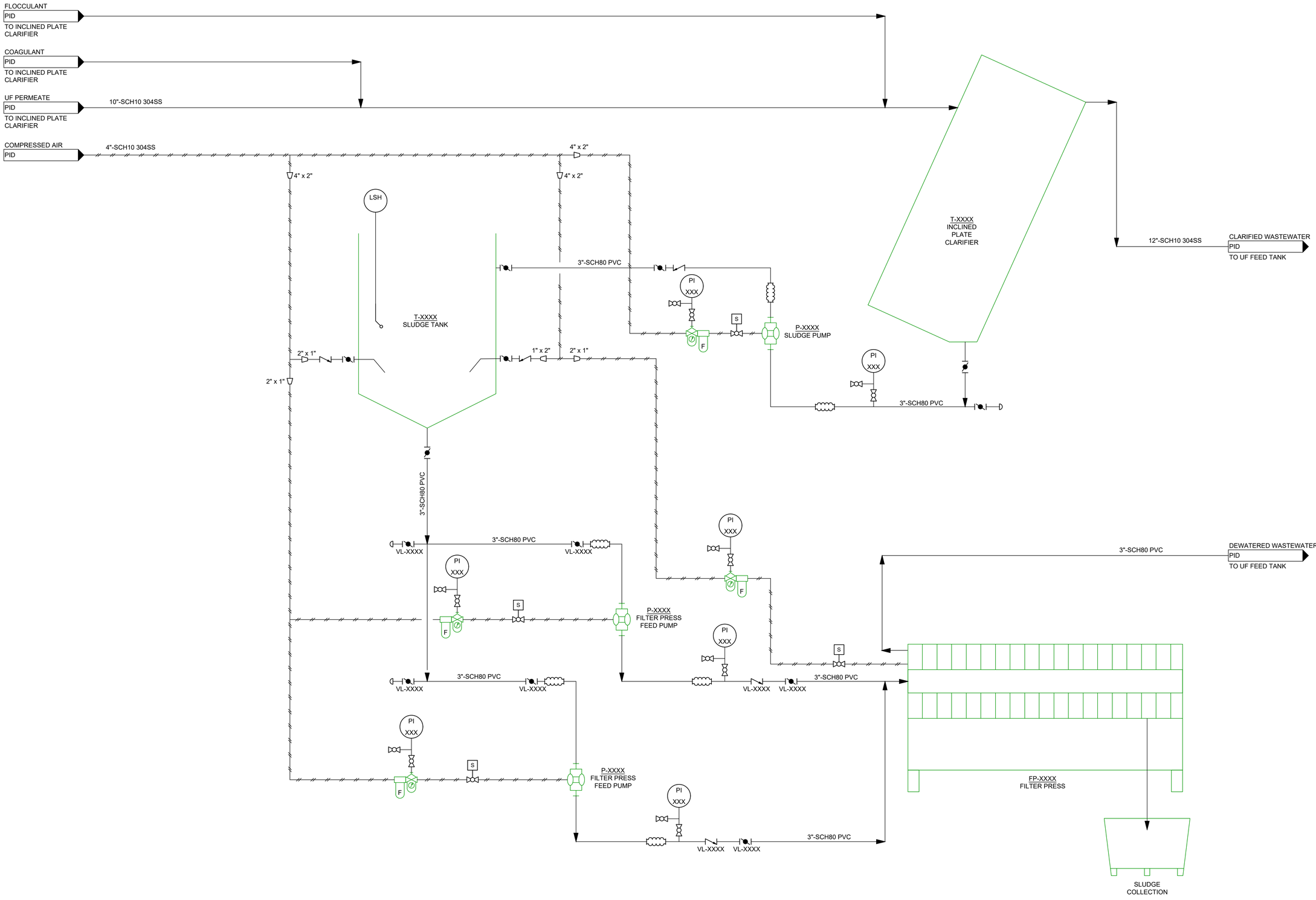

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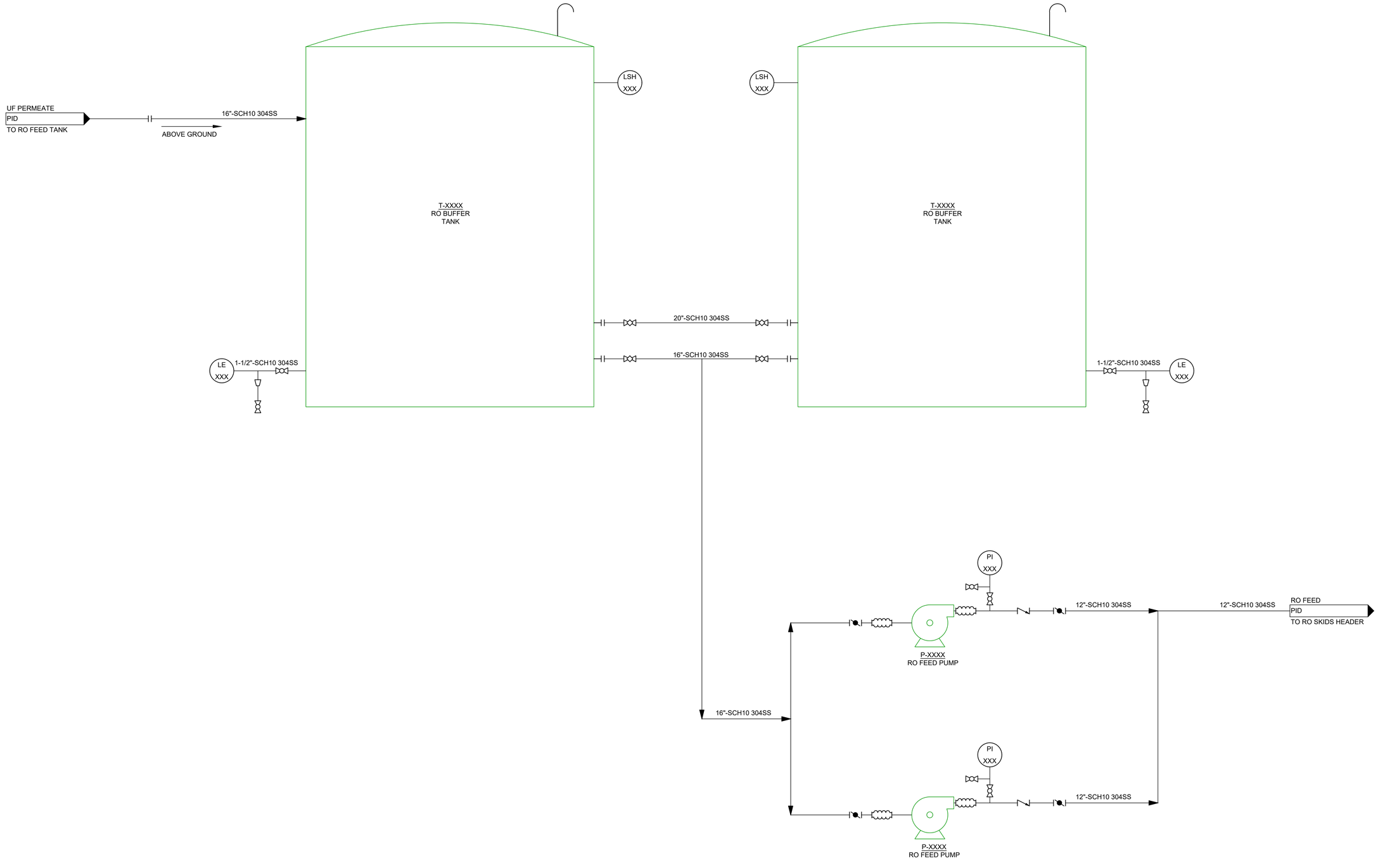

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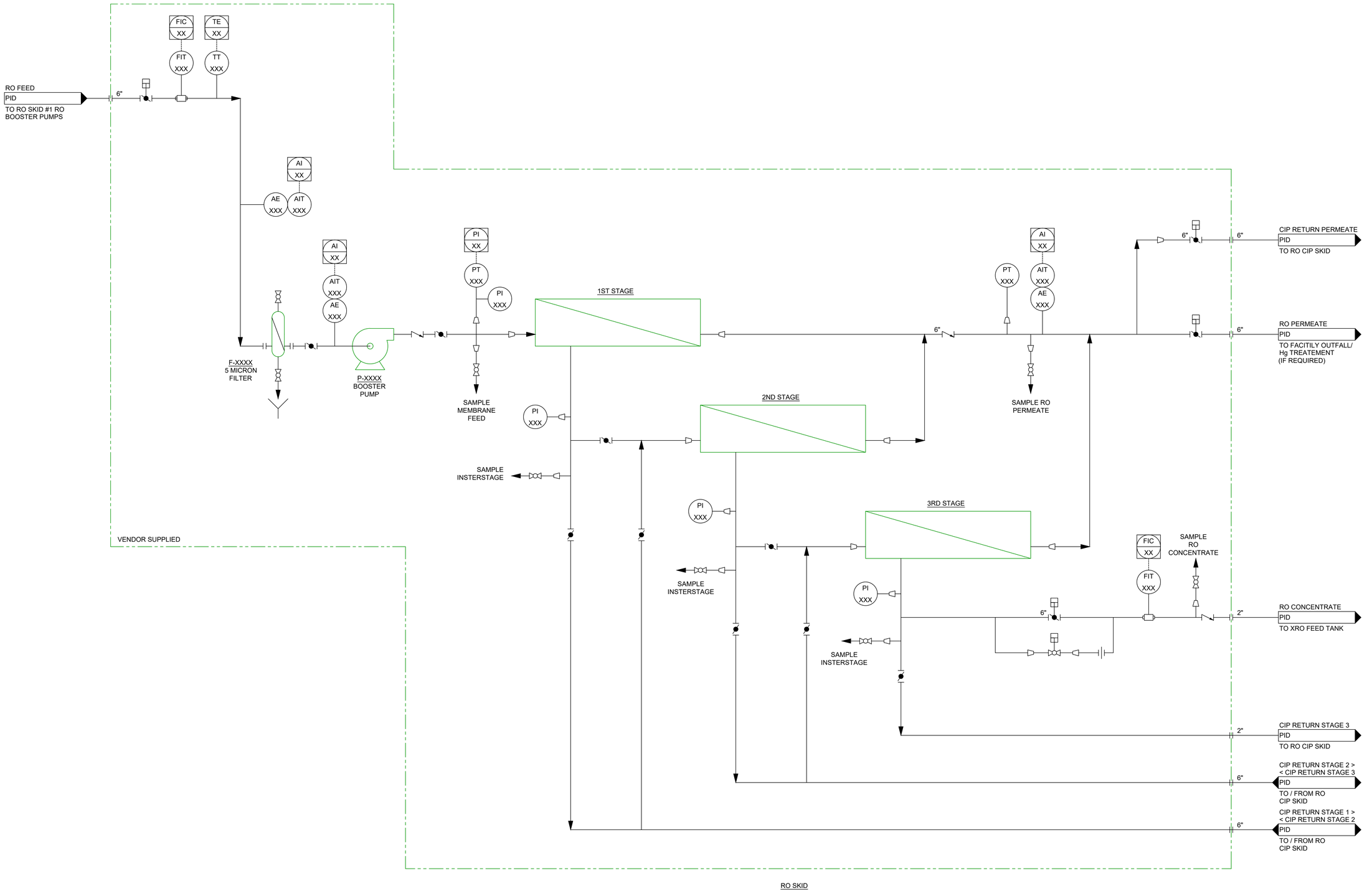

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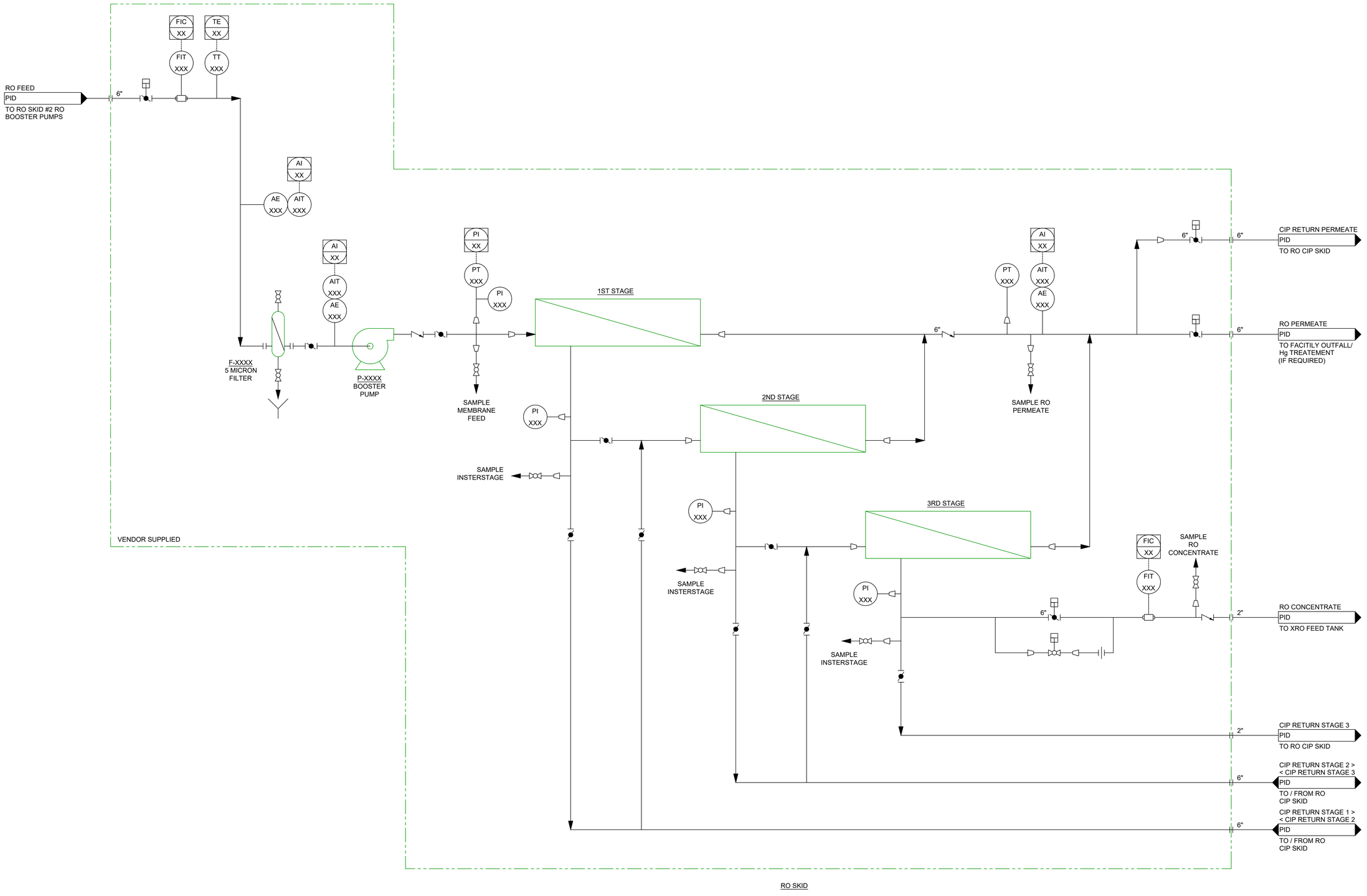

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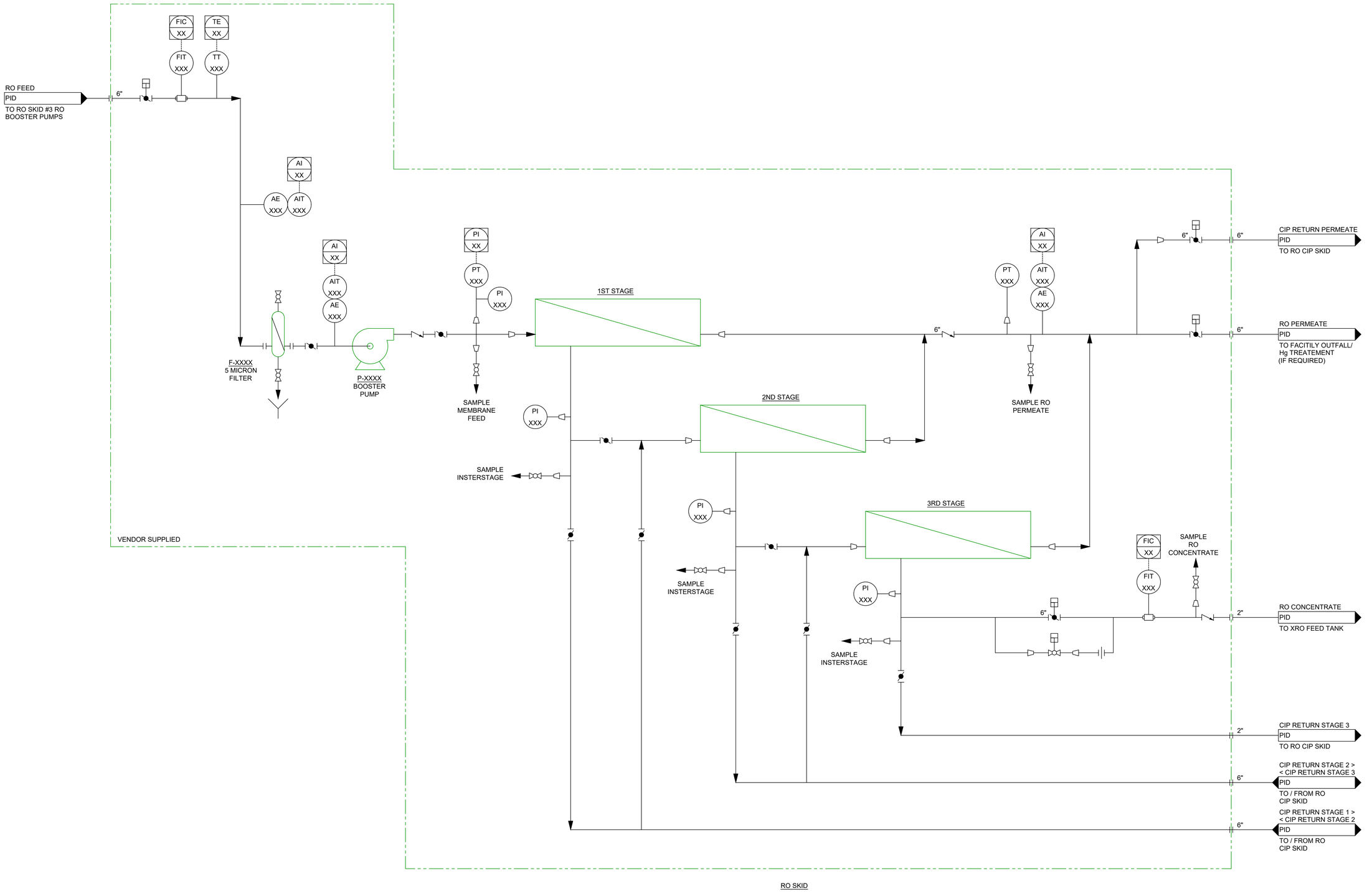

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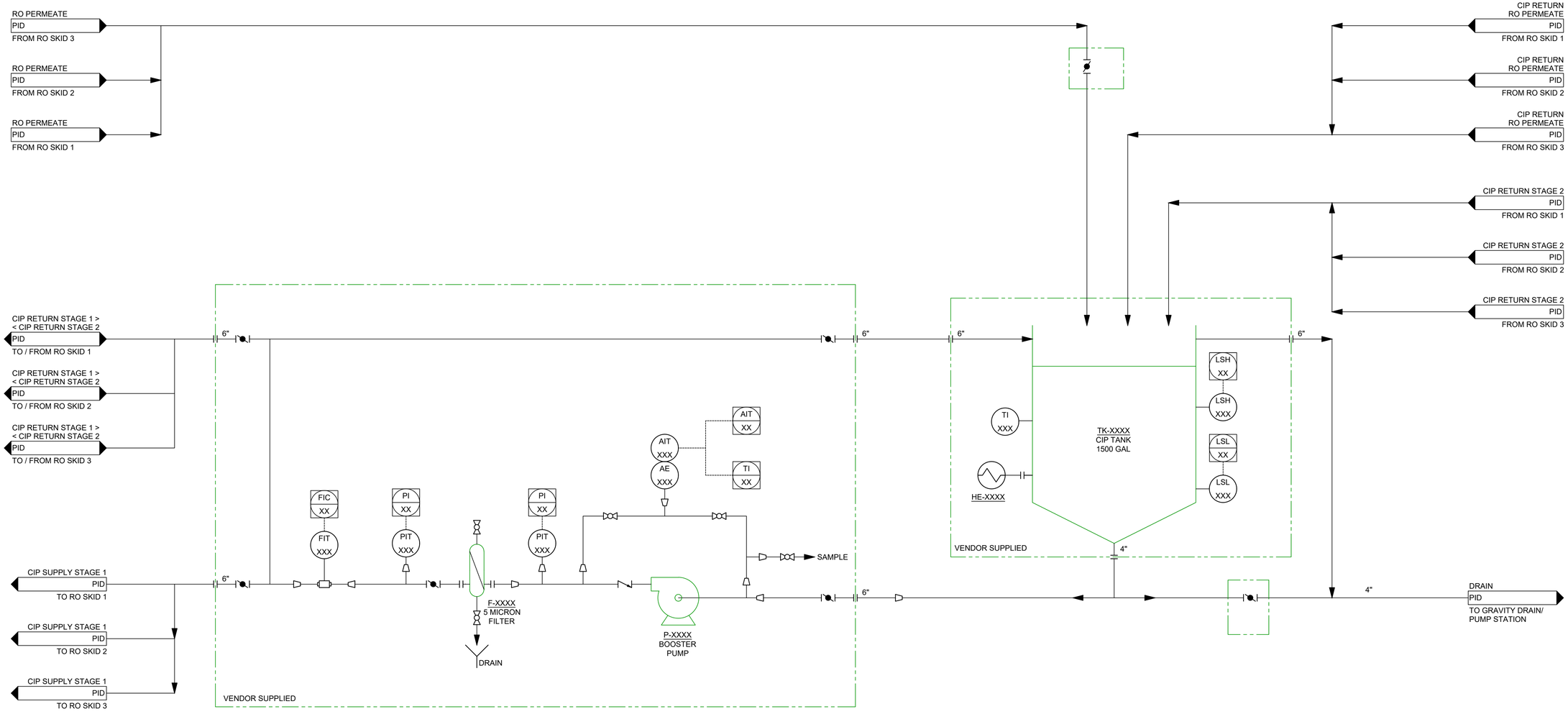

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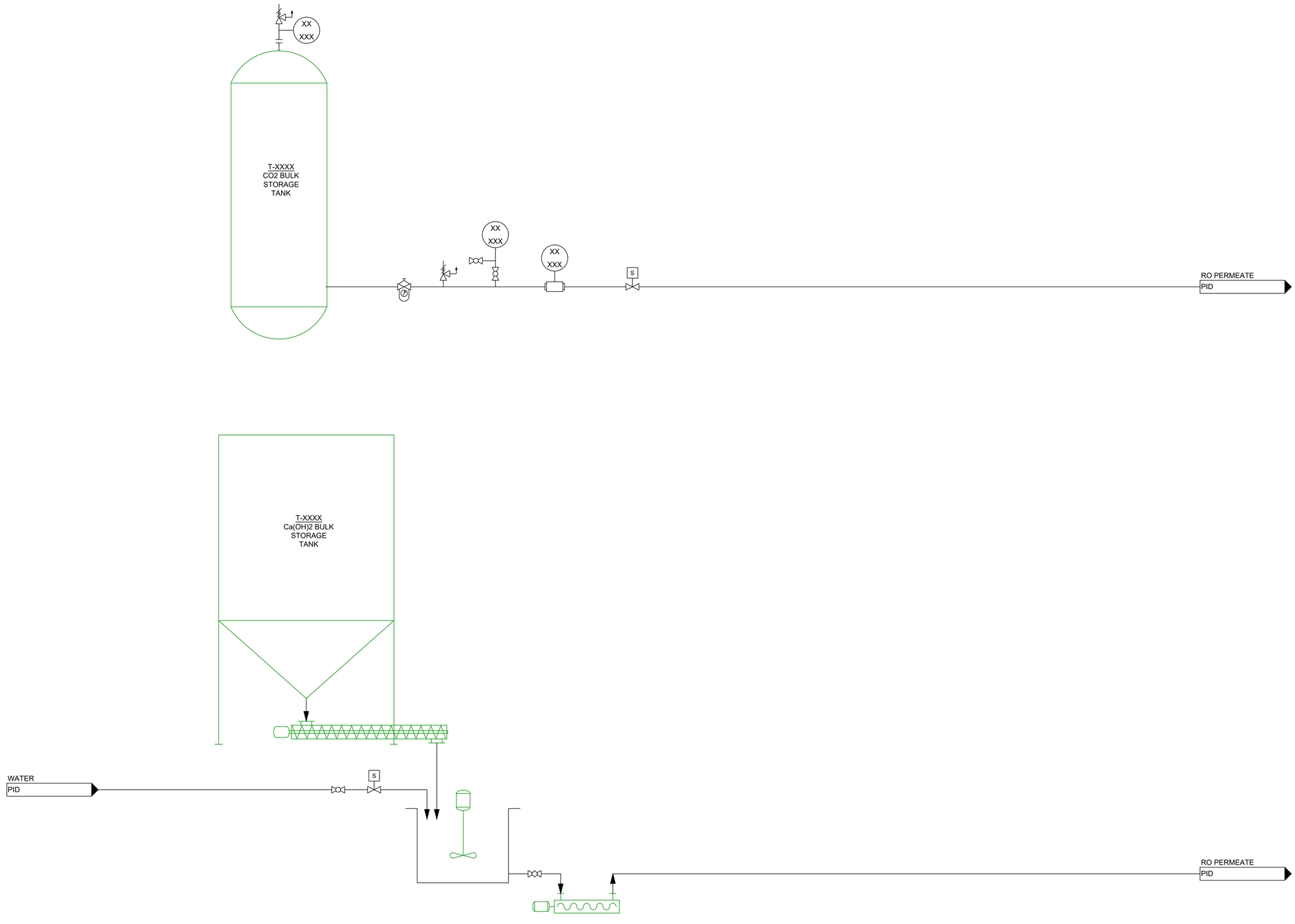

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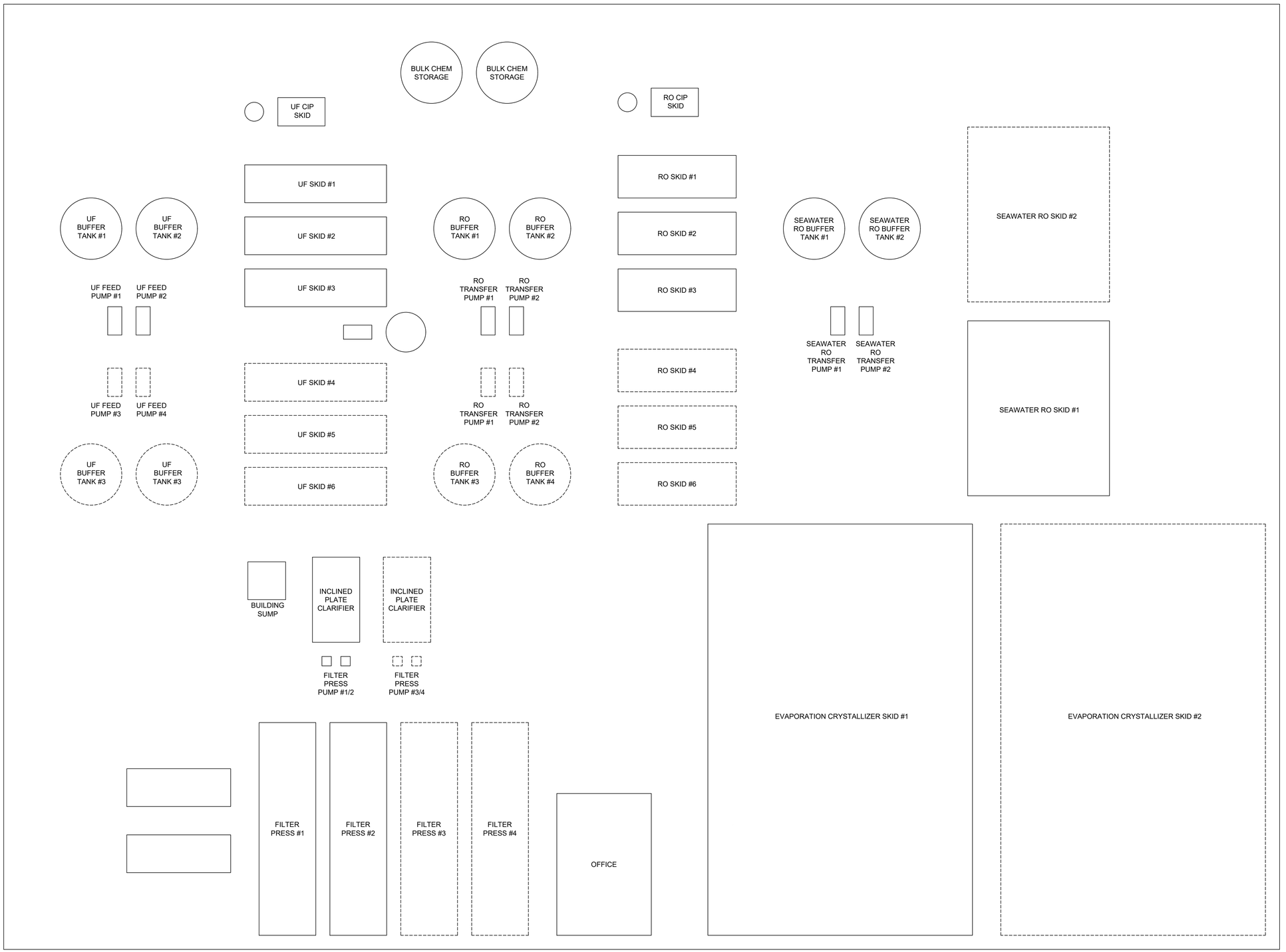
Attachment 2

Proposed Equipment Layout for the Initial (2,250 gpm) and Future Flows (4,500 gpm)



LEGEND

———— YEAR 1 EQUIPMENT
- - - - - - - - FUTURE EQUIPMENT

A/1 EQUIPMENT PLAN
SCALE: 3/32" = 1'

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PROPOSED EQUIPMENT LAYOUT
WATER TREATMENT SYSTEM
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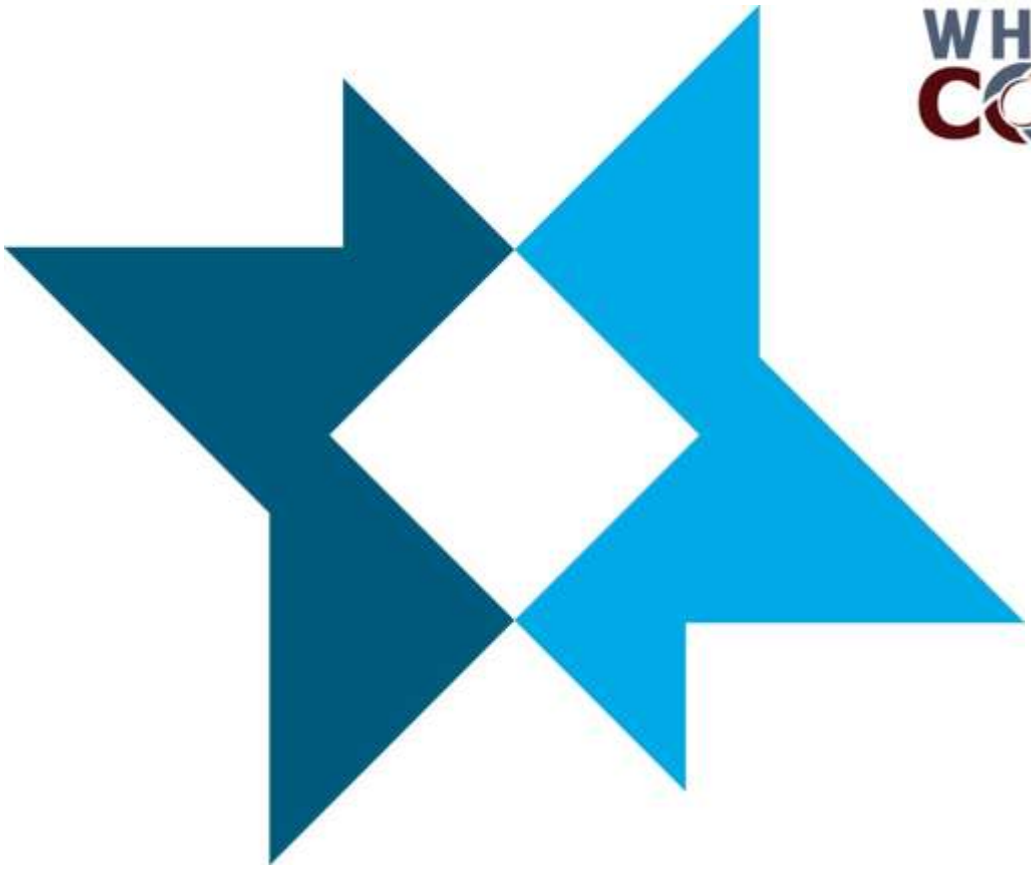
PLAN-001

E-2

Treatment and Containment Plan

Treatment and Containment Plan
White Pine North Project



White Pine Copper LLC
White Pine, Michigan

August 2026

Project ID: 23W200

Solving our clients' toughest science and engineering challenges.

Treatment and Containment Plan
White Pine North Project

Project ID: 23W200

Prepared for
White Pine Copper LLC
White Pine, Michigan

Prepared by
Foth Infrastructure & Environment, LLC

August 2026

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Treatment and Containment Plan
White Pine North Project

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List of Abbreviations, Acronyms, and Symbols

CQA	Construction Quality Assurance
EGLE	Michigan Department of Environment, Great Lakes, and Energy
Foth	Foth Infrastructure & Environment, LLC
GCL	Geosynthetic Clay Liner
Non-PAG	non-potentially acid generating
NPDES	National Pollutant Discharge Elimination System
NP1	North Pond 1
NP2	North Pond 2
NREPA	Natural Resources and Environmental Protection Act
Project	White Pine North Project
SMT	short tons
TDF	Tailings Disposal Facility

1. Introduction

This Treatment and Containment Plan (Plan) for Tailings and Other Reactive Mine-Related Materials has been prepared to demonstrate how tailings and other reactive materials generated by the White Pine North Project (Project) will be identified, managed, treated, and contained in a manner that protects surface water, groundwater, and other natural resources, and complies with Part 632 of the Michigan Natural Resources and Environmental Protection Act (NREPA), 1994 PA 451, as amended.

This Plan is prepared in accordance with R 425.203(h), which requires a treatment and containment plan for reactive mine-related materials, and R 425.409, which establishes design and performance requirements for the treatment and containment of tailings and other reactive materials at nonferrous metallic mineral mines. The Plan focuses on the placement of Project-generated reactive tailings within the existing Tailings Disposal Facility (TDF), consisting of North Pond 1 (NP1) and North Pond 2 (NP2), and describes the engineered containment framework, performance objectives, monitoring systems, and quality assurance measures used to ensure long-term isolation and control of reactive materials.

This document is intended to function as a stand-alone technical plan incorporated by reference into the Mining Permit Application submitted under Part 632. The Plan applies to mine-related materials that are determined to be reactive under R 425.203(c)(v) and describes measures to prevent contamination of groundwater and surface water from leachate containing acidic water or dissolved metals. Where the Project relies on legacy tailings basins that were constructed prior to modern composite liner requirements, the Plan documents how equivalent or better protection is achieved through an integrated containment approach, including engineered water management, seepage interception, monitoring, and response measures, consistent with R 425.203(h) and R 425.409.

1.1 Regulatory Basis

This Plan has been prepared pursuant to the following regulatory authorities and requirements:

- Michigan NREPA, 1994 PA 451, as amended
  - Part 632 – Nonferrous Metallic Mineral Mining, which governs the permitting, operation, and closure of nonferrous metallic mineral mines in Michigan.
- Michigan Administrative Code, Part 425 (Nonferrous Metallic Mineral Mining Rules), including:
  - R 425.203(h) – requiring submission of a treatment and containment plan for tailings and other reactive mine-related materials as part of a mining permit application.
  - R 425.409 – establishing design, performance, monitoring, and certification requirements for the treatment and containment of tailings and other reactive materials, including provisions for alternative liner system demonstrations where composite liners are not practicable.
- Part 315 – Dam Safety, to the extent applicable to the existing TDF embankments, which are regulated as dams under Michigan's Dam Safety Program.

- Part 31 – Water Resources Protection, and Michigan's authorized implementation of the National Pollutant Discharge Elimination System (NPDES), which governs treatment and discharge of water associated with tailings management, seepage, reclaim, and contact stormwater.

This Plan is also informed by guidance and standards commonly applied to tailings facilities, seepage control, and embankment stability, including dam safety guidance and industry best practices, as documented in supporting technical analyses prepared for the Project. Compliance with these requirements is demonstrated through the integrated design, operation, monitoring, and response measures described in this Plan and summarized in the Mining Permit Application, Foth, 2026a).

The integrated containment, water management, treatment, and monitoring measures described in this Plan are designed to minimize actual and potential adverse impacts to groundwater and surface water by preventing the leaching and runoff of acid-forming and other waste products from reactive materials, consistent with the requirements of R 425.409.

Table 1-1 summarizes the regulatory basis and section alignment.

2. Project and Site Context Summary

This section provides a summary of Project and site conditions relevant to the treatment and containment of tailings and other reactive mine-related materials. It is intended to establish context for the design and performance considerations described in this Plan. Detailed descriptions of site conditions, baseline environmental setting, and material characterization are provided in other Project permitting documents and supporting technical reports. Figure 2-1 depicts the Project location.

2.1 Project Location and Setting

The Project area is located in the western Upper Peninsula of Michigan within a landscape shaped by Late Pleistocene glaciation and subsequent glaciofluvial processes. The area is characterized by low-relief terrain, widespread wetlands, and shallow depressions that influence surface drainage and water management. Mining-related activities are confined to the footprint of proposed Project facilities and legacy mining features.

2.2 Topography and Surface Drainage

Existing topography within the Project area reflects glacial deposition and surface smoothing, resulting in a broad upland plain with limited topographic variability. Ground surface elevations generally range from approximately 260 to 290 meters (850 to 950 feet) above mean sea level, with total relief typically less than 30 meters (98 feet) across the Project footprint. Slopes are generally gentle to very gentle, with most areas exhibiting gradients of less than 5%, and large portions of the Project Area falling within the 1 to 3% slope range based on U.S. Geological Survey topographic mapping. Figure 2-2 is showing the mine site layout and topography.

Localized steeper slopes occur primarily:

- Along small stream channels
- At the margins of shallow depressions and wetland basins
- Adjacent to legacy mining and man-made features

Surface drainage is moderately developed and strongly influenced by the low-gradient terrain and glacial sediments. Drainage patterns generally flow northward toward Lake Superior, primarily through small tributaries and stream systems associated with the Mineral River watershed. The gentle topography promotes slow surface runoff, with infiltration and shallow groundwater interaction playing an important role in local hydrology. Fine-grained glacial tills and clay-rich sediments contribute to localized poor drainage, resulting in wetlands and areas of seasonal saturation.

2.3 Climate and Hydrologic Setting

The Project area experiences a cold, continental climate typical of Michigan's Upper Peninsula, with long winters, substantial snowfall, and pronounced spring snowmelt. Annual precipitation occurs throughout the year, with snow accumulation and melt representing an important driver of seasonal runoff and water management conditions.

State and regional assessments indicate that the Great Lakes region has experienced increases in total annual precipitation and the frequency of intense storm events over recent decades, with continued variability anticipated. These conditions are addressed through conservative

water management and containment practices, including controlled tailings deposition, reclaim pond management, spillway configuration, and active water treatment and discharge controls described in this Plan.

2.4 Seismic Setting

The Project Area is located within a region of low seismicity. Michigan, and particularly the Upper Peninsula, experiences infrequent and generally low-magnitude seismic events relative to tectonically active regions of the United States. Seismic loading is therefore considered a secondary design condition, rather than a governing factor, for tailings containment and embankment stability.

Where applicable, pseudo-static seismic loading conditions have been evaluated as part of embankment stability assessments consistent with dam safety guidance and industry practice, and the results support stability under applicable loading conditions.

3. Identification of Reactive Materials

This section identifies the mine-related materials generated by the Project and the facilities used to manage those materials. Geochemical characterization indicates that the materials expected to be mined from the mining column, Widely Laminated Siltstone, Upper Shale, Upper Sandstone, Parting Shale, and Lower Sandstone, are classified as non-potentially acid generating (Non-PAG). Fresh tailings generated from processing of these ore materials are also classified as Non-PAG. Although these materials are not expected to generate acid, they may still release dissolved metals and other constituents under certain conditions and therefore require controlled management of contact water. The treatment and containment measures described in this Plan are designed to address both the limited occurrence of potentially acid generating materials and the broader potential for dissolved constituent release.

Facilities that currently exist or are planned as part of the Project and that may store mine-related materials and/or manage contact water generated by mining operations are listed below and summarized in Table 3-1:

- NP1 and NP2
- Temporary Ore Stockpile
- Crushed Ore Stockpile
- Process Plant
- Contact Water Ponds
- Existing Slag Pile (historic)
- Existing Waste Repository (historic)

Historic facilities are identified for completeness and interface considerations but are not proposed to receive new Project-generated reactive mine-related materials unless explicitly stated.

Project geochemical characterization also indicates that all tested Till and Freda Formation waste rock samples are classified as Non-PAG. In contrast, Nonesuch Formation waste rock from above the mining column showed mixed geochemical behavior. Of the 17 Nonesuch waste rock samples tested, 11 samples were classified as Non-PAG, 1 sample was classified as uncertain PAG, and 5 samples were classified as PAG. This means that only a small fraction of the Nonesuch waste rock evaluated to date has been identified as potentially acid generating. At the same time, NAG leachate and related testing indicate that Nonesuch material may be comparatively more reactive than other waste rock types and may release elevated concentrations of constituents such as antimony, cadmium, copper, iron, lithium, and manganese under certain conditions. These constituents reflect the comparatively higher reactivity of Nonesuch waste rock. For the Project overall, geochemical characterization and kinetic testing identify copper and cadmium as the principal long-term constituents of potential concern associated with sulfide oxidation, primarily from the Upper Shale and Parting Shale, along with total dissolved solids, chloride, and other constituents associated with saline formation water, such as barium, beryllium, manganese, and strontium. These constituents inform the design of the Project water management and treatment systems.

Overall, the geochemical characterization indicates that the Project is not likely to generate acid mine drainage, but neutral mine drainage with circumneutral pH and elevated dissolved constituents, particularly copper, and in some cases cadmium and total dissolved solids, is likely to be an important water quality management consideration. This distinction is important

for the Project: most mined materials and tailings are not expected to generate acidity, but water in contact with those materials may still require collection, reuse, treatment, and discharge control.

Geochemical characterization indicates that Project ore, tailings, and mine-related rock are not significant sources of mercury. However, regional precipitation in the Lake Superior watershed contains mercury at concentrations that can exceed applicable Great Lakes discharge criteria. Accordingly, mercury deposited as precipitation on contact water surfaces, including the Contact Water Pond, NP1, and NP2, and in tailings-surface runoff, will be considered in water management and monitoring so that atmospheric mercury loading is not misattributed to mine materials.

This Plan applies to mine-related materials determined to be reactive in accordance with R 425.203(c)(v), with tailings as the primary reactive material addressed and other materials managed consistently with their characterized reactivity and applicable permit requirements. Rock material generated or exposed during underground mine development will be evaluated through the Project geochemical characterization program and managed in accordance with its reactivity. Where mine development rock is identified as having the potential to generate acidic drainage or leach dissolved metals, management measures will include minimizing exposure to precipitation, directing associated contact water to controlled collection, reuse, or treatment systems, and implementing monitoring and response actions as necessary to prevent impacts to groundwater and surface water. Rock determined to be non-reactive may be beneficially reused for approved Project construction purposes, consistent with design and material suitability requirements.

All mine development material brought to surface is currently planned to be reused within NP1 as construction material for the southern containment berm and associated approved earthworks. As a result, mine development rock will remain within the controlled footprint of the TDF, and any associated contact water will be contained within NP1 and managed in accordance with the Project water management framework.

The Temporary Ore Stockpile is intended for short-term use during the initial period of Project development, approximately the first 2 years, while the Process Plant is being constructed and commissioned. Because ore storage at this location is temporary and limited in duration, the potential for long-term water quality effects is reduced relative to a permanent ore storage facility. Any use of this area for temporary or emergency ore storage during operations would similarly be limited in duration and managed under the Project water management framework.

The Crushed Ore Stockpile and Process Plant are not managed as conventional open stockpile or lined storage facilities. These areas are located within enclosed structures, are constructed with concrete floors, and include sumps and drainage collection systems. Water reporting to these facilities is managed as contact water in accordance with the site water management plan and is routed to controlled collection, reuse, or treatment systems. Because these materials are contained within enclosed buildings and hard-surfaced operational areas, their management differs from that of external lined containment facilities.

Stockpiles and storage or disposal facilities used for ore, waste rock, overburden, and tailings are designed, constructed, and operated to manage materials in a manner that prevents the generation and migration of leachate and protects groundwater and surface water resources.

Design features, construction methods, and operational controls for these facilities, including the TDF and other contained contact-water management systems, are described in Sections 4 through 8 of this Plan. These measures collectively address the requirements of R 425.203(h)(i)(A).

4. Tailings Disposal Facility Description

Tailings generated by the Project originate from a conventional beneficiation process consisting of crushing, grinding, and flotation typical of sulfide ore processing. Following mineral recovery, the remaining tailings slurry is thickened to increase the percent solids by weight prior to conveyance to the TDF. Paste-thickened tailings are targeted to a solids concentration in the range of approximately 58% to 62% by weight (nominally about 62%) to support deposition objectives, including a target beach slope of approximately 2%. Water recovered during thickening and tailings deposition is managed under the Project's water management and treatment framework and is routed for reuse, treatment, or permitted discharge as appropriate.

The Project will utilize existing, legacy tailings disposal facilities, designated NP1 and NP2, for the placement of Project-generated tailings. These facilities were constructed and operated historically for tailings disposal and are being repurposed to receive new tailings under the treatment and containment framework described in this section. Use of the TDF to manage and contain tailings is subject to approval by the Michigan Department of Environment, Great Lakes, and Energy (EGLE) as part of the Mining Permit Application under Part 632. Figures 4-1 through 4-5 depict the NP1 and NP2 piping and discharge layouts and section profiles. Table 4-1 tailings design basis characteristics include (at minimum): solids percent, specific gravity, yield stress, beach slope, consolidated dry density.

NP1 was constructed and placed into service beginning in 1954 and received tailings through 1971, with approximately 91.6 million short tons (SMT) of tailings placed during historic operations. The impoundment consists of an irregular rectangular footprint encompassing approximately 1,870 acres, bounded by earthen embankments on the west, north, and east sides and a roadway embankment on the south. Embankments were constructed using locally borrowed native earth materials, primarily silty clay, using a downstream construction method. Embankment heights range from less than 10 feet to approximately 60 feet. Tailings were historically deposited as slurry, and a permanent pond remains along the northeastern portion of the impoundment. NP1 is currently classified as a "low" hazard dam under the Michigan Dam Safety Program (Part 315 of NREPA).

NP2 began receiving tailings in 1970 following construction of starter dams in 1968 and 1969 and continued operation until 1993. Approximately 92 million SMT of tailings were placed during historic operations, representing about 34% of the original design capacity. NP2 encompasses approximately 2,400 acres, with a permanent pond covering roughly 105 acres in the northeastern portion of the impoundment. The embankment system was constructed using a downstream method with low-permeability native clay starter dams and subsequent raises incorporating cyclone sand and compacted clay. Embankment heights range from less than 10 feet to approximately 55 to 70 feet, with a maximum structural height of approximately 80 feet in localized areas. NP2 is currently classified as a "significant" hazard dam under Part 315.

The Project has an estimated 26-year mine life, preceded by a pre-production period of 3 years and is anticipated to process approximately 15,000 dry metric tonnes of ore per day, generating a total of approximately 133.6 million metric tonnes of tailings. Tailings deposition has been designed to occur initially in NP1 and subsequently in NP2, with deposition configurations that achieve a target 2% beach slope and a conservative final consolidated dry density of approximately 1.46 tonnes per cubic meter. Based on these assumptions, the combined NP1

and NP2 facilities provide sufficient capacity to store the full Project tailings inventory without raising existing embankments.

Tailings will be deposited using a combination of central deposition towers and perimeter discharge points, with alternating deposition locations to promote drying and consolidation and to maintain operational flexibility. A portion of NP1 capacity is reserved to provide contingent storage in the event of deposition system disruptions during operations in NP2. Earthworks associated with the Project are limited to internal berms, access features, and water management separations necessary to support deposition geometry, contact and non-contact water segregation, and maintenance access; no raises to existing embankments are required.

Water associated with tailings deposition, reclaim ponds, and contact areas within NP1 and NP2 is managed in accordance with the Project's Water Management Plan. Contact water generated within the TDF is collected and routed for reuse in processing, treatment, or discharge under applicable permits, while non-contact water is segregated to the extent practicable. The integration of tailings placement with water management and treatment systems ensures that reactive tailings remain contained and that water quality protections are maintained throughout operations and into closure.

5. Composite Liner System Design

This section addresses the applicability of composite liner system requirements under R 425.409 in the context of the Project's proposed use of the existing, historic TDF, consisting of NP1 and NP2, together with the design of new lined reactive material management facilities constructed as part of the Project. The Project does not propose construction of a new tailings impoundment or installation of a composite liner beneath the existing TDF footprint. Instead, Project-generated tailings will be deposited directly on top of legacy tailings within the legacy tailings basins that were designed and constructed prior to promulgation of modern environmental liner requirements. By contrast, new Project-constructed facilities used to store ore or manage contact water outside the TDF footprint will incorporate engineered liner systems where appropriate and practicable.

5.1 Historic Tailings Disposal Facility Construction and Liner Configuration

NP1 and NP2 were designed and constructed during the 1950s through 1970s, at a time when synthetic liners and composite liner systems were not required under Michigan or federal environmental regulations. As a result, the historic TDF does not include a geomembrane liner, compacted clay liner, or composite liner system beneath the original tailings deposits, and no such liner exists below the historic tailings footprint. Project-generated tailings will therefore be placed on top of legacy tailings within facilities that were originally developed without modern liner systems.

The historic TDF relies on rolled-earth embankments keyed into native materials and bedrock, constructed primarily of locally borrowed silty clay and earth fill. In the case of NP1, the embankment is understood to function as a solid clay embankment and does not incorporate underdrains or a perimeter seepage drainage ditch. In contrast, NP2 includes seepage-control features associated with its clay-and-sand embankment configuration. Historical design information for NP2 describes the foundation area as dense red clay with scattered rock fragments and limited permeability, which contributed to the original containment concept for that facility.

Historic seepage-control features documented for NP2 include a series of underdrains, or toe drains, located at the toe of the embankment and discharging to the perimeter channel. These drains are intended to intercept seepage, relieve pore water pressures, and support embankment stability. These design elements reflect the engineering practice in effect at the time of construction and continue to function as part of the existing TDF seepage management system.

5.2 Applicability of Composite Liner Requirements

Because the Project proposes to place new tailings within an existing, previously constructed and operated TDF and does not involve construction of a new tailings disposal area, installation of a composite liner beneath NP1 or NP2 is not practicable and is not proposed. The absence of a composite liner beneath the historic TDF is explicitly recognized in this Plan and is addressed through the alternative containment measures described in Section 6. In lieu of the composite liner requirements of R 425.409(a)(i)(A), containment performance for the TDF is achieved through the alternative liner system and containment approach described in Section 6, consistent with R 425.409(a)(i)(B) and R 425.409(a)(ii).

Although a composite liner system is not present beneath the historic TDF, the Project incorporates lined containment systems for other reactive material management facilities where appropriate and practicable. These facilities include, as applicable:

- the Temporary Ore Stockpile, where liner systems and contact water controls are used to limit infiltration and facilitate collection of contact water;
- Contact Water Ponds, sumps, and related water management system components, which are designed with liner systems to support controlled collection, treatment, and discharge of contact water; and
- other Project-constructed containment features where new reactive materials are managed outside the historic TDF footprint.

For the Temporary Ore Stockpile, and other applicable lined reactive material management facilities, the containment system will be designed to meet the composite liner intent of R 425.409(a)(i). The liner configuration will include, from subgrade upward, compacted subgrade, a cushioning geotextile, a secondary geomembrane not less than 1.0 millimeters (40 mil), a leak detection and drainage layer, a geosynthetic clay liner (GCL) or a minimum 0.91 meter (3 foot) compacted soil layer with a maximum hydraulic conductivity of 1.0×10^{-7} centimeters per second, a primary geomembrane not less than 1.5 millimeters (60 mil) thick, and protective cover materials as required by the final design. These systems will also incorporate leachate or contact water collection and leak detection features, as applicable, to limit hydraulic head and support monitoring and response.

The GCL layer consists of granular bentonite clay encapsulated between two geotextile fabrics. The GCL is proposed as an equivalent substitute for the minimum 0.9 meter (3 foot) compacted clay layer prescribed under R 425.409(a)(i)(A) and is used to meet the composite liner intent of R 425.409(a)(i)(A) rather than as a separate alternative liner system. A GCL is proposed for the composite liner design primarily because suitable low-permeability clay soils are not reliably available on site; approximately 55,800 cubic meters of conforming clay would otherwise be required.

Geosynthetic clay liners are a well-established equivalent to compacted clay for containment applications and provide equal or superior hydraulic performance. EGLE permits the use of GCLs in solid waste disposal facilities under Part 115, where liners may contact more concentrated leachate than is anticipated for the Temporary Ore Stockpile. The use of GCLs in mining and landfill containment applications in Michigan is documented in state permitting records. On this basis, the GCL is considered to provide containment performance equivalent to, or better than, the prescribed compacted soil layer.

Detailed Temporary Ore Stockpile liner design criteria, including the composite liner configuration, leachate collection and leak detection design, HELP model seepage analysis, and construction quality assurance requirements, are provided in the Temporary Ore Stockpile Liner Design Criteria (Appendix D-5).

For lined Contact Water Ponds, sumps, and related water management facilities, the current design basis includes a GCL, a 1.5 millimeter (60 mil) high-density polyethylene geomembrane, geotextile, and a protective sand layer. Final liner details may vary by facility depending on its function, subgrade conditions, and final engineering, but these facilities will be designed to

provide controlled containment of contact water and compatibility with the Project water management system. Where required, the final design will incorporate leak detection, underdrainage, collection piping, and sump configuration appropriate to the intended service of each facility.

The manner in which containment performance is achieved for Project-generated tailings within NP1 and NP2, in the absence of a composite liner, is addressed in the Alternative Liner System Demonstration in Section 6.

6. Alternative Liner System Demonstration

This section provides the Alternative Liner System Demonstration required under R 425.409(a)(i)(B) for the continued use of the existing, historic TDF, consisting of NP1 and NP2, which do not contain a composite liner beneath the tailings footprint. Because the Project proposes to place new tailings within legacy tailings basins constructed prior to modern liner requirements, environmental protection is achieved through a system-based containment approach rather than through installation of a subsurface liner.

The alternative plan for the TDF incorporates measures to collect and treat contact water prior to discharge and to hydraulically isolate reactive tailings from groundwater and surface water through controlled water management and containment systems, consistent with R 425.409(a)(ii)(B) and (D).

6.1 Basis for Alternative Containment Approach

NP1 and NP2 were constructed in the mid-20th century using rolled-earth embankments keyed into native materials and bedrock. NP1 functions as a solid clay embankment, while seepage control in NP2 is supported by an underdrain system and associated perimeter drainage features. While these facilities do not include synthetic or compacted clay liner systems, they are founded on low-permeability native materials and have historically functioned as controlled tailings impoundments with defined seepage pathways and engineered discharge points.

The Project's alternative liner demonstration is based on the integrated performance of the following systems:

- The existing embankments in NP1 and NP2 and the seepage control systems present in NP2
- Controlled tailings deposition practices that manage beach slopes, consolidation, and reclaim pond location
- Active water management, treatment, and discharge controls regulated under the NPDES program
- Monitoring, inspection, and response measures designed to identify and manage seepage or abnormal conditions

Together, these elements provide containment through hydraulic control, seepage interception, and treatment prior to discharge. The TDF does not employ a low-permeability cover system designed to isolate reactive materials from precipitation and air. Instead, containment is achieved through controlled water management and treatment systems, which limit the migration of constituents and maintain protection of surface water and groundwater resources. This approach constitutes an alternative plan in accordance with R 425.409(a)(ii), in lieu of the cover requirements of R 425.409(a)(i)(F).

The combined containment measures described herein provide a level of protection that is equivalent to or greater than that achieved by a conventional composite liner system, consistent with the requirements of R 425.409(a)(i)(B). This equivalency is achieved through active management of water and seepage, including interception, controlled conveyance, treatment prior to discharge, and ongoing monitoring to verify system performance.

This Plan accounts for the volume, rate, and movement of leachate and contact water generated from tailings placement and associated contact areas. Leachate and contact water generation is influenced by precipitation, snow accumulation and melt, evaporation, and seasonal frozen-ground conditions. The water management system is designed to maintain controlled hydraulic head within reclaim compartments and to provide storage and conveyance capacity for severe precipitation and snowmelt events through a combination of reclaim pumping, spillway capacity, and controlled routing to treatment or permitted discharge. These measures collectively limit the potential for uncontrolled release and support the overall containment performance of the TDF.

Quantitative evaluation of the generation, volume, rate, and movement of leachate and contact water within the TDF, including precipitation inputs, runoff, tailings bleed water, and process-related flows, is provided in the Project water balance and hydrologic modeling analyses. These analyses, presented in the Project Water Management Strategy Memorandum and supporting tailings design reports (Appendix D-4), evaluate storage capacity, conveyance, and discharge under a range of operating and design storm conditions. The integrated results of these analyses form the basis for the water management, containment, and treatment systems described in this Plan and demonstrate compliance with R 425.203(h)(ii).

6.2 Integration with NPDES-Regulated Water Management

The TDF manages water by segregating non-contact water associated with legacy conditions from contact water generated by Project operations. Non-contact water includes precipitation and runoff that contacts legacy tailings and other pre-existing sources and is routed to designated non-contact compartments for discharge through the existing permitted outfall. Contact water includes Project-generated sources such as mine dewatering, process plant contact water (including runoff routed through the Contact Water Pond), precipitation falling on active tailings deposition areas, and bleed water released from newly deposited tailings.

Contact water is contained within designated reclaim compartments and is actively managed through a combination of reuse, treatment, and controlled discharge. Reclaim water is pumped for reuse in mineral processing and, as necessary, conveyed to the water treatment plant for treatment prior to discharge under applicable permits. Project-generated contact water from the underground mine, process plant, contact stormwater systems, and active tailings deposition areas is collected, managed, and treated in accordance with the Project Water Management Design Technical Report (Appendix D-3) and discharged only after meeting applicable NPDES permit limits.

By maintaining operational separation between legacy and Project-generated water systems, the Project ensures that contact water associated with reactive tailings is independently collected, controlled, and regulated. This system-based approach prevents uncontrolled migration of constituents by intercepting, treating, and monitoring water prior to discharge. The controlled collection, treatment, and discharge of contact water under enforceable permit conditions serves as a primary containment mechanism and contributes to the overall level of protection achieved by the TDF in the absence of a subsurface liner.

6.3 Containment Performance Without a Composite Liner

In lieu of a composite liner, containment of Project-generated reactive tailings within NP1 and NP2 is achieved through the combined hydraulic and operational controls of the TDF system. Key elements include:

- water balance management that limits hydraulic head within the tailings mass through reclaim, evaporation, and controlled discharge;
- engineered seepage interception features in NP2, including underdrain systems that direct seepage to defined collection and discharge pathways;
- active treatment of contact water prior to discharge, ensuring compliance with surface water quality standards; and
- monitoring and inspection programs that provide early detection of changes in seepage quantity or quality.

Containment performance is also supported by the Project's use of paste-thickened tailings. By recovering the majority of process water in the tailings thickeners within the process plant prior to deposition, the volume of tailings-associated water managed within the TDF is substantially reduced. This reduces the volume of potentially impacted process water requiring storage and management in the TDF and correspondingly reduces health, safety, and environmental risks associated with storage of contact water on the facility.

The higher solids content of paste-thickened tailings also reduces the amount of pore water available to bleed from deposited tailings over time and promotes faster consolidation of the deposited tailings mass. These characteristics improve water management efficiency during operations, reduce the volume of contact water requiring active management, and support more efficient stabilization and closure of the TDF at the end of mine life.

Together, these measures ensure that seepage and tailings-associated water are not allowed to migrate uncontrolled into the surrounding environment, and that any water exiting the TDF is managed under enforceable permit conditions.

6.4 Relationship to Other Treatment and Containment Measures

The alternative liner system demonstration described in this section is supported by additional treatment and containment measures presented elsewhere in this Plan, including:

- Leachate and seepage collection systems (Section 7)
- Leak detection, monitoring, and response triggers (Section 8)
- Construction quality assurance and documentation for new Project-related containment features (Section 9)
- Cover systems and isolation measures implemented during operations and closure (Section 10)

In combination, these measures provide a redundant and layered containment strategy that achieves the protective objectives of R 425.409 for a legacy tailings facility where installation of a composite liner is not practicable.

7. Leachate and Seepage Collection System

Because the existing TDF, consisting of NP1 and NP 2, does not include a composite liner, leachate and seepage management relies on engineered drainage features, hydraulic head control, reclaim pumping, and active water management, rather than on subsurface containment. This section describes the layout and function of seepage collection and reclaim systems, the basis for hydraulic head control, and integration with treatment and discharge systems.

7.1 System Layout and Components

Seepage control within NP2 is achieved through a combination of historic engineered drainage features and Project-installed reclaim and conveyance infrastructure. Historic seepage control elements include:

- A series of underdrains (i.e., toe drains) located at the toe of NP2 that extend from beneath the embankment and discharge into the perimeter channel.
- Perimeter drainage channels and controlled discharge structures that route collected seepage and surface water to defined collection points.

Routine inspections of downstream embankment slopes and toes of NP2 have historically observed low seepage rates, predominantly limited to toe areas, with clear seepage indicating that internal erosion has not occurred during the operational life of the TDF.

To actively manage water within the TDF, the Project includes reclaim pumping systems that remove water from reclaim ponds and convey it for reuse or treatment. A reclaim pumping system will be installed initially in the northeast corner of NP1, followed by a second system in later years in the northwest corner of the NP2 reclaim basin, as tailings deposition progresses.

Each reclaim location will be equipped with two positive-suction (self-priming) pumps, operating in a one-duty/one-standby configuration to provide redundancy. The pumps are currently sized for a combined flow rate of up to approximately 4,000 gallons per minute and are capable of 15 feet of lift. Sizing may be adjusted as designs progress. Suction piping will consist of 16-inch SDR 9 pipe, ballasted with concrete blocks to maintain submergence and prevent air entrainment. Pumps will be installed within climate-controlled enclosures to ensure reliable, year-round operation. In advance of tailings deposition in NP2, internal berms and/or trenches will be constructed to separate contact (reclaim) and non-contact (stormwater) compartments, consistent with the TDF water management approach.

In the event of a leak, breakage, or blockage in the NP1 tailings conveyance pipeline, paste-thickened tailings can be temporarily discharged to the southwest corner of NP1 using a dedicated tailings emergency upset discharge. In addition, approximately 10% of NP1 capacity will remain available as contingent storage to provide operational flexibility during upset conditions, such as tailings pipeline disruptions during deposition in NP2.

Grading, berming, and perimeter drainage controls are incorporated into stockpiles and storage or disposal facilities to ensure that surface water is directed away from reactive material storage and disposal areas, except where intentionally captured and managed as contact water.

7.2 Hydraulic Head Control Design

Hydraulic head within the TDF is controlled through tailings deposition practices, reclaim pond management, spillway configuration, and active water management as described in this Plan and the Project Tailings Management Study Report (Appendix D-2).

Seepage and embankment stability were evaluated using site-specific numerical modeling, with critical embankment sections selected based on downstream slope geometry and topographic survey data. Modeled phreatic surfaces were compared to measured piezometer water levels and to observed seepage locations. Modeled depths to water closely matched measured values, confirming that hydraulic gradients and seepage behavior are well represented for both NP1 and NP2. Calculated exit hydraulic gradients under future flood pool conditions were below critical hydraulic gradients, resulting in factors of safety exceeding recommended minimum values against piping and internal erosion.

Where modeling identified localized areas with reduced factors of safety under future loading conditions, targeted mitigation measures, such as downstream buttressing, have been incorporated into the Project design to restore minimum factors of safety. These measures do not require raising existing embankments and further reduce seepage-related risks.

7.3 Integration with Treatment and Discharge Systems

Water collected in reclaim ponds and contact water management areas associated with NP1 and NP2 is managed in accordance with the Project water management framework. Reclaim pumps discharge to the Project reclaim water system for reuse in mineral processing, and excess reclaim water is directed to treatment and managed for reuse or permitted surface water discharge as needed. Figure 7-1 depicts the conceptual water balance. Table 7-1 summarizes contact and non-contact waters.

Consistent with the Project water management design, contact water from the Process Plant Area and other contact stormwater collection areas is routed through the Contact Water Pond and then to the appropriate TDF compartment in accordance with the operating-period water balance and tailings deposition sequence. Legacy water associated with the historic White Pine Mine and portions of the TDF not used for Project tailings continues to discharge through the existing permitted structure. Project-generated contact water is collected separately, treated, and either recycled within the Project or discharged under applicable NPDES permit conditions only after meeting permit limits.

By integrating reclaim pumping, contact water routing, active treatment, and controlled discharge under enforceable permits, the Project provides effective leachate and seepage management in the absence of a composite liner, consistent with the alternative liner system demonstration presented in Section 6.

8. Leak Detection and Seepage Response System

This section describes the leak detection, seepage monitoring, and response approach for facilities addressed by this Plan. Because the Project includes both new Project-constructed lined containment features and the existing unlined legacy TDF, leak detection and response mechanisms differ by facility type and are described accordingly. For facilities that incorporate dedicated leak detection systems, monitoring is focused on identification of liquid migration through or beneath the liner system. For facilities that do not include a separate leak detection layer, performance is verified through inspections, operational monitoring, and observation of seepage and water management conditions.

8.1 Leak Detection Systems and Performance Verification

Leak detection requirements apply differently depending on facility type. Certain lined reactive material management facilities, such as the Temporary Ore Stockpile and other applicable lined ore storage areas, are planned to include dedicated leak detection and drainage layers in accordance with the composite liner system design described in Section 5. For those facilities, any liquid migrating past the primary liner will be conveyed to a dedicated detection sump through a drainage layer and associated piping. Detection sumps and piping will be accessible for inspection and monitoring, and any liquid detected will be managed as contact water and evaluated for source identification and corrective action.

Not all lined water management facilities require a distinct leak detection system as part of their design. Contact Water Ponds, sumps, and related water management components are designed as lined containment and collection facilities, but their performance is generally verified through facility inspections, operating water balances, and periodic leak detection surveys by a qualified third party, rather than through a separate dedicated leak detection layer and detection sump. These facilities are part of the active contact water collection system and are managed under the Project Water Management Design Technical Report (Appendix D-3).

For lined facilities that do include leachate collection and leak detection systems, those systems are designed and constructed to limit hydraulic head to not more than 0.30 meter (1 foot), excluding the collection sump, in accordance with R 425.409(a)(i)(C).

8.2 Monitoring Approach

The monitoring approach described in this section establishes the methods and parameters used to verify the performance of leak detection and seepage control systems for both lined facilities and the unlined TDF. Monitoring is intended to identify deviations from expected conditions and support timely implementation of response actions to prevent potential impacts to groundwater and surface water. Figures 8-2 and 8-3 depict groundwater and surface water monitoring locations relevant to this monitoring framework.

8.2.1 Lined Facilities

For lined facilities that include dedicated leak detection systems, such as the Temporary Ore Stockpile and other applicable lined ore storage features, leak detection sumps will be monitored routinely as part of facility inspections. Monitoring parameters may include the presence or absence of liquid, accumulation rates, and changes relative to baseline or expected operating conditions. Under normal operating conditions, liquid accumulation in the leak detection system is not expected.

For lined Contact Water Ponds, sumps, and related water management facilities that do not include a formal leak detection layer, monitoring will focus on facility performance verification through routine inspections, operating observations, and periodic leak detection surveys conducted by a qualified third party. Monitoring may include visual inspection of embankments and subgrade conditions, review of pond levels and water balances, inspection of liner-accessible areas and appurtenances, and follow-up evaluation if unexplained water loss or other abnormal conditions are observed.

In addition to direct facility inspections, routine monitoring of downgradient groundwater wells and relevant surface water monitoring locations may provide indication of changes in site conditions that could be associated with seepage or loss of containment from lined facilities. Monitoring locations, parameters, and response thresholds will be implemented consistent with the approved groundwater and surface water monitoring plan and the site water management framework.

8.2.2 Unlined Legacy Tailings Disposal Facility

Because NP1 and NP2 do not include a composite liner, leak detection for these facilities is based on seepage monitoring and hydraulic performance indicators rather than detection of liner breaches. Monitoring elements include:

- routine visual inspections of downstream embankment slopes, toes, and drainage features for evidence of seepage, erosion, turbidity, or changes in flow conditions;
- observation of seepage clarity and quantity at known seepage locations, where clear, low-volume seepage is indicative of stable conditions;
- instrumentation data, including piezometers installed within embankments, used to track phreatic surface elevations and changes in hydraulic gradients; and
- monitoring of reclaim pond levels, spillway performance, and drainage conveyance to confirm that hydraulic head within the TDF remains within design expectations.

In addition, water exiting the TDF through permitted discharge points is monitored under the applicable NPDES permit, providing an additional line of performance verification for water management and containment. Routine monitoring of downgradient groundwater wells and relevant surface water monitoring locations may also provide indication of changes in water quality or hydraulic conditions that could be associated with seepage from the TDF.

Monitoring locations, parameters, and response thresholds will be implemented consistent with the approved groundwater and surface water monitoring plan required under R 425.203(g) and R 425.406, including leachate and compliance monitoring as applicable.

8.3 Response Triggers and Operational Response Actions

Response triggers vary by facility type and are intended to identify conditions that deviate from expected performance.

8.3.1 Lined Facilities

For lined facilities with dedicated leak detection systems, potential response triggers include:

- detection of liquid within a leak detection sump;
- sustained or increasing liquid accumulation rates relative to baseline or expected conditions; and
- evidence of damage to the primary liner or associated appurtenances identified during inspection.

For lined Contact Water Ponds, sumps, and related water management facilities without a dedicated leak detection system, response triggers may include unexplained water loss, evidence of seepage or wet areas outside the intended containment footprint, abnormal changes in pond operating levels that are not explained by inflow, evaporation, or discharge, and inspection findings indicating potential compromise of liner integrity or associated conveyance components.

Operational response actions for lined facilities may include investigation of the source, temporary cessation of material placement or water routing to the affected facility, repair or replacement of compromised liner or conveyance components, removal and management of collected liquids as contact water, and increased inspection or monitoring frequency as warranted.

8.3.2 Unlined Legacy Tailings Disposal Facility

For NP1 and NP2, response triggers are based on changes in seepage behavior or hydraulic conditions, such as:

- new or increased seepage at embankment toes or slopes;
- changes in seepage clarity, including sediment-laden flow;
- unexpected increases in piezometric water levels;
- overtopping or near-overtopping of drainage or conveyance features; and
- changes in reclaim pond levels inconsistent with operational controls.

Operational response actions may include adjustment of reclaim pumping rates, modification of deposition practices, inspection and maintenance of drainage features, implementation of localized mitigation measures such as buttressing or drainage improvements, and increased monitoring frequency.

8.4 Linkage to Contingency Planning

Conditions indicating a leak, loss of containment, or uncontrolled release from lined facilities, or seepage conditions that could threaten containment performance at the legacy TDF, will be evaluated to determine whether they constitute a contingency under the Project Contingency Plan (Appendix H) provisions addressing leaks from containment systems or storage or disposal facilities.

Where response thresholds defined in the Contingency Plan are met, appropriate notification, escalation, and corrective action procedures will be implemented in accordance with that Plan

and applicable regulatory requirements, including reporting to the Pollution Emergency Alerting System where warranted.

This leak detection and seepage response framework will be refined and updated as final facility designs are completed and additional monitoring information becomes available, and prior to commencement of operations, to ensure that detection thresholds and response actions remain appropriate for site-specific conditions.

9. Construction Quality Assurance and Pre-Operation Testing

Construction quality assurance (CQA) and pre-operation testing are implemented to verify that Project-constructed treatment and containment features are installed in accordance with design intent and are functional prior to receipt of reactive materials or contact water. For this Project, CQA is implemented through construction specifications, field inspections, material certifications, and pre-operation testing, rather than through a separate, stand-alone CQA Plan.

Because the Project utilizes an existing, historic TDF for tailings placement, CQA activities described in this section apply primarily to new, Project-constructed containment features, including the temporary lined ore stockpile, contact water management facilities, leak detection systems, reclaim infrastructure, and associated conveyance systems.

9.1 Non-Tailings Disposal Facility Liner System Testing and Verification

For new Project-constructed reactive material management facilities where engineered liner systems are required, including the Temporary Ore Stockpile and applicable lined contact water management facilities, liner systems will be installed where practicable and appropriate. Quality assurance for these liner systems will be achieved through a combination of field observations, testing, and documentation, including, as applicable:

- Verification of subgrade preparation, including grading, proof-rolling, and removal of unsuitable materials prior to liner placement.
- Inspection of geomembrane and geocomposite installation, including panel placement, anchorage, and protection against damage.
- Field testing of liner seams, such as air-pressure testing, vacuum box testing, or equivalent methods appropriate to the liner configuration.
- Electronic leak location testing of primary and secondary liner systems.
- Review of manufacturer certifications and quality control documentation for geomembranes and geocomposites.

These activities are intended to confirm that liner systems are installed in a manner that supports effective containment and leak detection, without requiring a separate formal CQA plan. Quality assurance and verification will address installation and integrity of the primary liner, secondary liner, geocomposite leak detection layer, and the GCL or compacted soil component, consistent with the composite liner configuration described in this Plan.

9.2 Leachate Collection and Leak Detection System Testing

Leachate collection and leak detection systems associated with lined facilities will be tested prior to operation to confirm that they are functional and accessible. Pre-operation verification may include, as applicable:

- Inspection of leak detection layers to confirm continuity and proper connection to detection sumps.
- Verification that leak detection sumps, detection piping, and cleanouts are installed in accordance with design specifications and are accessible for monitoring.

- Functional testing of sump pumps or gravity drain components, where applicable, to confirm flow paths and discharge locations.
- Confirmation that liquids collected in leak detection systems can be managed as contact water and routed to treatment, reuse, or permitted discharge systems.

For lined facilities that include liner and leak detection systems, pre-operation testing and verification will be completed before reactive materials are placed in those facilities.

Because NP1 and NP2 do not include composite liners or dedicated leak detection layers, quality assurance and pre-operation testing for the legacy TDF will focus on verification and functional testing of new Project-related infrastructure (e.g., reclaim pumps, conveyance piping, drainage features, and monitoring installations), rather than subsurface liner testing.

9.3 Documentation Prior to Tailings Placement

Prior to placement of Project-generated tailings or operation of lined reactive material management facilities, the Operator will compile and retain documentation demonstrating that construction and pre-operation testing have been completed. Documentation will include, as applicable:

- Records of field inspections and testing conducted during construction.
- As-built drawings for liner systems, leachate collection and leak detection components, reclaim pumping systems, and associated conveyance infrastructure.
- Material certifications for geomembranes, geocomposites, and other critical containment components.
- Documentation of pre-operation inspections and functional testing confirming that systems are ready for service.

Where construction or system installation occurs in phases, documentation will be completed for each phase prior to that phase becoming operational. This documentation will be completed prior to commencement of tailings placement and retained as part of the Project record.

The CQA and pre-operation testing approach described in this section supports the containment performance objectives of this Plan and complements the leak detection and seepage response framework described in Section 8 and the Project Contingency Plan (Appendix H).

10. Cover System and Isolation Measures

Isolation of tailings within NP1 and NP2 will be achieved through direct tailings revegetation, rather than through placement of an engineered soil or geomembrane cover system. This approach is appropriate for the existing TDF configuration and is consistent with the long-term stabilization strategy for legacy tailings facilities where installation of a conventional cover system is not practicable.

Direct revegetation provides isolation by establishing a stable vegetative cover over non-inundated tailings surfaces, thereby reducing exposure pathways associated with wind erosion and surface-water transport. This approach primarily provides surface stabilization and erosion control and is not intended to function as a low-permeability cover for containment purposes. Revegetation is implemented progressively during operations and maintained through closure and post-closure to support long-term surface stability.

10.1 Objectives of Direct Tailings Revegetation

The objectives of the revegetation program for NP1 and NP2 are to:

- Provide sufficient vegetative cover to effectively control fugitive dust generation from exposed tailings surfaces.
- Reduce erosion and transport of suspended solids by surface water runoff across tailings surfaces.

By limiting dust emissions and surface erosion, revegetation reduces the potential for off-site transport of tailings material. Any surface water runoff from revegetated tailings areas remains subject to collection, treatment, and discharge controls implemented for the TDF, providing an additional layer of protection. Treatment of contact-related waters is only expected to be required for a limited time following closure and not result in perpetual care requirements.

10.2 Existing Revegetation Conditions

Revegetation has been implemented historically across substantial portions of both NP1 and NP2. Within NP2, essentially all non-inundated areas within the basin footprint have received some form of revegetation treatment, including applications of seed, fertilizer, mulch, straw crimp, and organic amendments over multiple areas. Field trials and phased applications have been used to inform revegetation techniques and improve long-term performance.

Within NP1, extensive vegetative cover has been established across large portions of the tailings surface and has been maintained for multiple years. Additional areas have been treated through targeted revegetation applications to improve cover where needed. Exterior embankment slopes that include exposed cyclone sands, particularly on NP2, have also received revegetation treatment to improve surface stability and reduce erosion potential. These existing conditions demonstrate that direct revegetation is a proven and effective stabilization method for the TDF and provide a foundation for continued and adaptive revegetation during Project operations and closure.

10.3 Performance Standards for Effective Cover

The effectiveness of direct tailings revegetation is evaluated based on effective cover, which includes live vegetation, ground moss, standing stubble, plant litter, and mulch. Effective cover is assessed in relation to both wind and water erosion protection.

Revegetation performance is considered successful when:

- The average effective cover across a monitored area meets or exceeds 70%.
- Areas of bare or unvegetated tailings are sufficiently limited such that erosion and dust generation potential remain controlled.

These performance criteria reflect commonly applied standards used by state and federal agencies for mine reclamation and tailings stabilization and are intended to ensure that revegetated surfaces provide durable isolation over the long term.

10.4 Long-Term Sustainability of Revegetation

In addition to achieving effective cover, the revegetation program is intended to be self-sustaining over time. Long-term sustainability is demonstrated through:

- Evidence of natural regeneration, including flowering and seed production.
- Species diversity sufficient to promote resilience under site-specific climatic conditions.

Revegetation areas are evaluated over multiple growing seasons to confirm that plant communities can persist without ongoing intensive management, such as repeated fertilization or reseeding.

10.5 Adaptive Management and Completion of Revegetation

Revegetation of NP1 and NP2 is implemented in an adaptive manner, with treatments refined based on observed performance. Areas where effective cover is insufficient may receive additional amendments, reseeding, or other corrective actions to achieve performance objectives.

Revegetation for a given area is considered complete when effective cover and long-term sustainability criteria have been achieved and maintained over the applicable monitoring period. Once completed, revegetated surfaces serve as the primary isolation measure for tailings within the TDF during closure and post-closure conditions.

10.6 Applicability to Other Facilities

The direct tailings revegetation approach described in this section applies only to the NP1 and NP2 TDF. Other Project facilities that manage reactive materials or contact water, including temporary ore stockpiles, crushed ore stockpiles, processing plant, and contact water ponds and water management system, are designed with engineered systems and are not intended to be isolated through revegetation during operations.

Closure and isolation of these facilities will be addressed in detail in the Project Reclamation Plan (Appendix G). At closure, materials managed within these facilities will be removed as

appropriate, and liner systems will be decommissioned and removed or otherwise managed in accordance with reclamation design requirements. Following removal of materials and liners, affected areas will be regraded to promote stable landforms and re-establish natural drainage patterns, and will be covered and revegetated to support long-term stability and post-closure land use.

This approach ensures that each facility type is managed using appropriate and facility specific isolation measures, with direct revegetation serving as the isolation mechanism for legacy tailings surfaces at NP1 and NP2, and engineered liner removal and building demolition, grading, and revegetation serving as the isolation and reclamation approach for other Project-constructed facilities.

11. Professional Certification and Compliance Documentation

This section describes the professional certification, documentation, and recordkeeping requirements associated with construction, testing, and operation of treatment and containment systems addressed in this Plan. Certification and documentation are intended to demonstrate that Project-constructed containment features are designed, constructed, and tested in accordance with design intent and applicable regulatory requirements prior to placement of reactive materials.

11.1 Engineer-of-Record Certifications

For Project-constructed treatment and containment features that require certification under applicable regulations, a Professional Engineer licensed in the State of Michigan will serve as the Engineer of Record. Following completion of construction and pre-operation testing, the Engineer of Record will certify that the applicable systems have been:

- Designed in accordance with approved design criteria
- Constructed in substantial conformance with the design intent
- Tested as required to verify functionality prior to use

Certification will be provided prior to placement of reactive materials and submitted to EGLE for approval. Placement of ore, waste rock, overburden, or tailings in the facility will not begin until approval is received from EGLE.

Because the NP1 and NP2 TDF are existing, historic structures, professional certification under this section applies primarily to new Project-constructed containment features and modifications, rather than to the original TDF embankments. Certifications required for the TDF design and operation will be managed in accordance with Part 315 of NREPA.

11.2 As-Built Documentation

As-built documentation will be prepared for Project-constructed treatment and containment features to document final construction conditions. As-built records may include, as applicable:

- Drawings showing final grades, liner extents, collection and detection system layouts, reclaim infrastructure, and conveyance features.
- Documentation of construction changes or field adjustments made during installation.
- Records demonstrating completion of required testing and inspections.

For lined reactive material management facilities, certification will specifically address proper design, construction, and testing of liner, leak detection, and associated collection features prior to placement of reactive materials. As-built documentation will be completed prior to operation of the applicable facility or system and will be maintained as part of the Project's permanent records.

11.3 Recordkeeping and Reporting

Records associated with professional certification, construction verification, testing, and operation of treatment and containment systems will be maintained by the Operator and made available for review upon request. Recordkeeping will include, as applicable:

- Engineer-of-Record certifications
- Inspection and testing records for liner systems, leachate collection systems, leak detection systems, and reclaim infrastructure
- As-built documentation
- Records related to monitoring, response actions, and system performance

Reporting and notification related to treatment and containment systems will be conducted in accordance with applicable permit conditions, monitoring requirements, and contingency procedures. Together, these records demonstrate compliance with the treatment and containment objectives of this Plan and support regulatory oversight throughout operations and closure.

Tables

Table 1-1
Regulatory Basis and Section Alignment
Treatment and Containment Plan
White Pine Copper LLC

Metallic Mineral Mining	Regulation	Requirement Summary	Treatment and Containment Plan Section(s)
Part 2 - Permits	R 425.203(h)	A treatment and containment plan that describes proposed measures to prevent contamination of groundwater and surface water from leaching of acidic water or dissolved metals	-
	R 425.203(h)(i)	The treatment and containment plan required under this subdivision shall apply to earth materials that are determined to be reactive under R 425.203(c)(v).	Section 3
	R 425.203(h)(i)(A)	Design, construction, and operation of stockpiles and storage or disposal facilities for ore, waste rock, overburden, and tailings.	Sections 3, 4, 5, 6, 7
	R 425.203(h)(i)(B)	The management of peripheral rock that has been determined in the environmental impact assessment to have the potential to contaminate groundwater or surface water.	Section 3
	R 425.203(h)	Address rock material generated or exposed during mining where it could generate acidic drainage or leach dissolved metals.	Section 3
	R 425.203(h)(ii)	Account for the generation, volume/rate/movement, and management of leachate and contact water within the containment system.	Section 7
Part 4 - Mining Operations	R 425.409	Manage reactive overburden, ore, waste rock, peripheral rock, and tailings to minimize actual and potential adverse impacts on groundwater and surface water by preventing leaching or runoff of acid-forming and other waste products.	Sections 1, 6, 7, 8, 10
	425.409(a)(i)(A)	A stockpile or storage facility shall have a composite liner system comprised of a flexible synthetic membrane that is not less than 60 mils thick and a layer of at least 3 feet of compacted soil having a maximum hydraulic conductivity of 1.0 X 10-7 cm/sec.	Section 5
	425.409(a)(i)(B)	An alternative liner system may be approved if the operator demonstrates equivalent or better protection.	Section 6
	425.409(a)(i)(C)	A stockpile or storage facility shall have a leachate collection system.The system shall be designed, constructed, and operated to limit the hydraulic head at the lowest point in the system to not more than 1 foot, excluding the collection sump, after construction.	Sections 7, 8
	425.409(a)(i)(D)	Provide a leak detection system for reactive stockpiles / storage facilities.	Section 8
	425.409(a)(i)(E)	The liner, leachate collection system, and leak detection system shall be tested before the placement of overburden, ore, waste rock, or tailings into the facility.	Section 9
	425.409(a)(i)(F)	A cover shall be employed to isolate the reactive overburden, ore, or waste rock from precipitation and air as soon as practicable.	Section 10
	425.409(a)(i)(G)	A registered professional engineer or other qualified individual shall certify the proper design, construction, and testing of all liners, covers, and leachate collection systems required by this paragraph. The permittee shall submit the certification to the department and shall not begin placement of ore, waste rock, overburden, or tailings in the storage facility until approved by the department.	Section 11
	425.409(a)(ii)	Subject to approval of the department, an operator may utilize an alternative plan for a stockpile or storage facility for reactive overburden, ore, waste rock, or tailings. The alternative plan shall incorporate 1 or more of the following: (A) Measures to prevent the generation of leachate by adding a material or materials that counteract or neutralize the acid-forming or toxic characteristics of the ore, waste rock, overburden, or tailings. (B) Measures to treat or neutralize any leachate that may be generated before it migrates outside of the storage facility. (C) Measures to isolate the ore, waste rock, overburden, or tailings from oxygen and other oxidizing substances. (D) Measures to isolate the ore, waste rock, overburden, or tailings from groundwater or surface water.	Section 6
	425.409(b)	An operator may utilize a disposal facility to manage, contain, or isolate reactive waste rock, tailings, overburden, or peripheral rock subject to approval of the department.	Section 4
	425.409(c)	A stockpile or storage or disposal facility under this rule shall be monitored in compliance with R 425.406.	Section 7.2
	425.409(d)	A permittee shall conduct and maintain grading or diking at stockpiles and storage or disposal facilities subject to this rule to assure that surface water drains away from the storage or disposal area.	Section 6.1

Prepared by: MJG4
Checked by: CER2

Table 3-1
Identification of Reactive Materials
Treatment and Containment Plan
White Pine Copper LLC

Identified Reactive Material	Expected Reactivity Classification	Primary Facility/Handling Area	Contact-water Control	Long-term Treatment/Containment Approach
Project-generated tailings	Non-Potential Acid Generation (Non-PAG). Potential for metals leaching.	North Pond 1 (NP1) and North Pond 2 (NP2) TDF	Contact water generated within the (TDF), including reclaim pond water, precipitation on active deposition areas, and bleed water from newly deposited tailings, is collected and routed for reuse, treatment, or discharge under applicable permits; non-contact water is segregated to the extent practicable.	Long-term containment is provided through placement within the existing legacy TDF, engineered water management, seepage interception, monitoring, and progressive/closure revegetation rather than installation of a new composite liner beneath NP1/NP2.
Temporary ore stockpile material	Non-Potential Acid Generation. Potential for metals leaching.	Temporary Ore Stockpile adjacent to the TDF NP1	Runoff and contact water generated during temporary storage are collected and managed as contact water for reuse, treatment, or permitted discharge as applicable.	Temporary operational storage only; long-term treatment/containment is not anticipated because ore is intended for processing rather than permanent disposal. Any engineered containment features for the stockpile are addressed through lined-facility design, as applicable.
Crushed ore	Non-Potential Acid Generation. Potential for metals leaching.	Crushed Ore Stockpile (enclosed under a geodesic dome) prior to processing	Drainage from the crushed ore stockpile area is managed as contact water and routed to the contact water management system.	Temporary operational storage only; crushed ore is processed rather than permanently disposed. Long-term containment is therefore not anticipated beyond routine operational containment and contact-water management during active use.
Mine development rock (mine rock brought to surface)	Small fraction of Nonesuch Formation material has Potential Acid Generation (PAG), remaining materials Non-PAG.	Reuse on TDF NP1 for southern containment berm	Contact water will be captured within the TDF NP1 contact water management system and routed for reuse, treatment, or discharge under applicable permits.	Mine rock brought to surface will be managed as planned construction material for the NP1 southern containment berm; long-term storage as a separate waste rock facility is not proposed.

Notes:

Reactivity classifications shown in this table are based on current Project geochemical characterization and may be refined as additional characterization, design, and operating information becomes available.

Prepared by: MJG4
Checked by: CER2

Table 4-1
Tailings Storage Facility Characteristics
Treatment and Containment Plan
White Pine Copper LLC

Design Criteria	Value	Source
Operating Mine Life	26 years	Stantec Mine Plan received March 16, 2026.
Pre-Production	3 years	From Ausenco Common Values and Reference Sheet 108571-GX-00000-11100-001-02.
Life of Mine Dry Tailings Production	133.6 Mt	From Ausenco's mass balance.
Tailings Specific Gravity	2.768	Table V of Paterson and Cooke's (2025) Test Work Report Rev_B.
Thickener Underflow Solids Concentration	62%	Based on Paterson and Cooke's (2025) Test Report Rev_B.
Fully Sheared Yield Stress	80 Pa	Based on Paterson and Cooke's (2025) Tailings System Hydraulic Report Rev_B.
Beach Slope	2%	Based on Water, Waste and Land's (2026) Report Rev_B.
Zero Free Water Solids Mass Concentration	76.90%	Table VIII of Paterson and Cooke's (2025) Test Work Report.
Final Consolidated Dry Density	1.46 tonnes/ m^3	Based on consolidation testing and review of other thickened tailings performance data.
Final Consolidated Solids Concentration	81%	Based on a specific gravity 2.768 and a final dry basis moisture content of 23%.

Notes:
Pa = Pascals
m^3 = cubic meters
Mt = Million tonnes

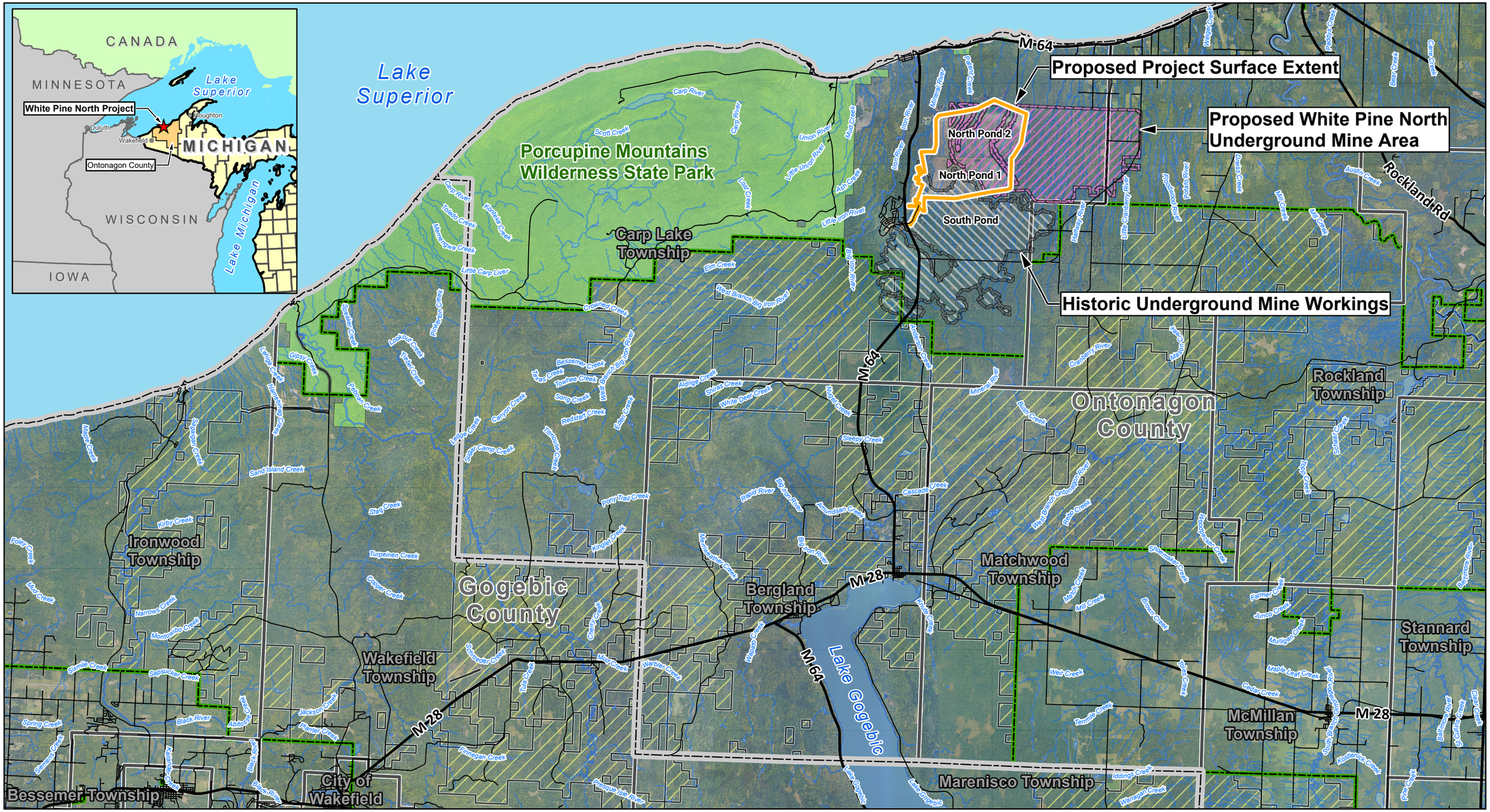
Prepared by: MJG4
Checked by: CER2

Table 7-1
Summary of Contact and Non-contact Waters
Treatment and Containment Plan
White Pine Copper LLC

Water Source	Water Type
Historic Mine Dewatering	Non-Contact
South Pond Runoff	
Runoff from Legacy Tailings	
Contact Water Pond Discharge	Contact
New Mine Dewatering	
Future Tailings Discharge (Bleed Water)	
Runoff from Future Tailings Deposition Area	

Prepared by: MJG4
Checked by: CER2

Figures



NOTES:

1. 2024 - 18in resolution air photo from the USDA NAIP.
2. Horizontal coordinate system is NAD1983 UTM Zone, 16N, units in meters.
3. Road, boundary and hydrographic data from Michigan GIS Open Data.

LEGEND

- Major Road
- Minor Road
- River/Stream
- Lake/Pond
- Proposed Project Surface Extent
- Proposed White Pine North Underground Mine Area
- Historic Underground Mine Workings
- Municipal Boundary
- County Boundary
- Ottawa National Forest Boundary
- Porcupine Mountains Wilderness State Park
- USDA Forest Service Owned Property

WHITE PINE COPPER

Foth

N

Foth Infrastructure & Environment, LLC			
REVISED	DATE	BY	DESCRIPTION
PREPARED BY:		DATE:	
REVIEWED BY:		DATE:	
APPROVED BY:		DATE:	

WHITE PINE COPPER LLC

FIGURE 2-1
PROJECT LOCATION
TREATMENT AND CONTAINMENT PLAN
WHITE PINE NORTH, ONTONAGON COUNTY, MI

Scale: 0 1.5 3 Miles

Date: JUNE 2026

Drafted By: DAT

Project: 25W006

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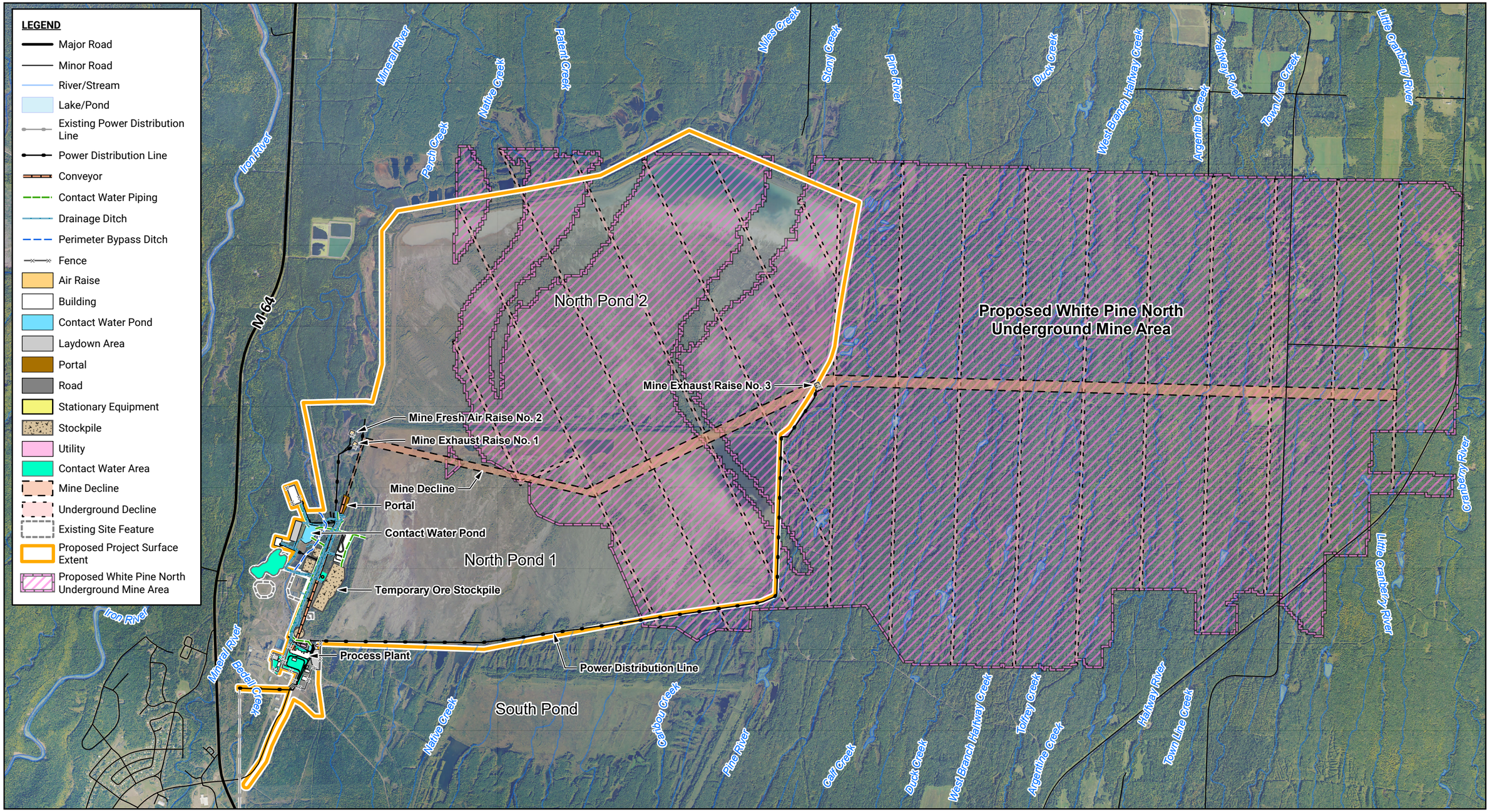

NOTES:

1. 2024 - 18in resolution air photo from the USDA NAIP.
2. Horizontal coordinate system is NAD1983 UTM Zone, 16N, units in meters.
3. Road, boundary and hydrographic data from Michigan GIS Open Data.
4. Site layout and infrastructure data from Ausenco, March 2026.

WHITE PINE COPPER

Foth

N

Foth Infrastructure & Environment, LLC			
REVISED	DATE	BY	DESCRIPTION
PREPARED BY:			DATE:
REVIEWED BY:			DATE:
APPROVED BY:			DATE:

WHITE PINE COPPER LLC

FIGURE 2-2
MINE SITE LAYOUT
TREATMENT AND CONTAINMENT PLAN
WHITE PINE NORTH, ONTONAGON COUNTY, MI

Scale: 0 1,500 3,000 Feet

Date: JUNE 2026

Drafted By: DAT

Project: 25W006

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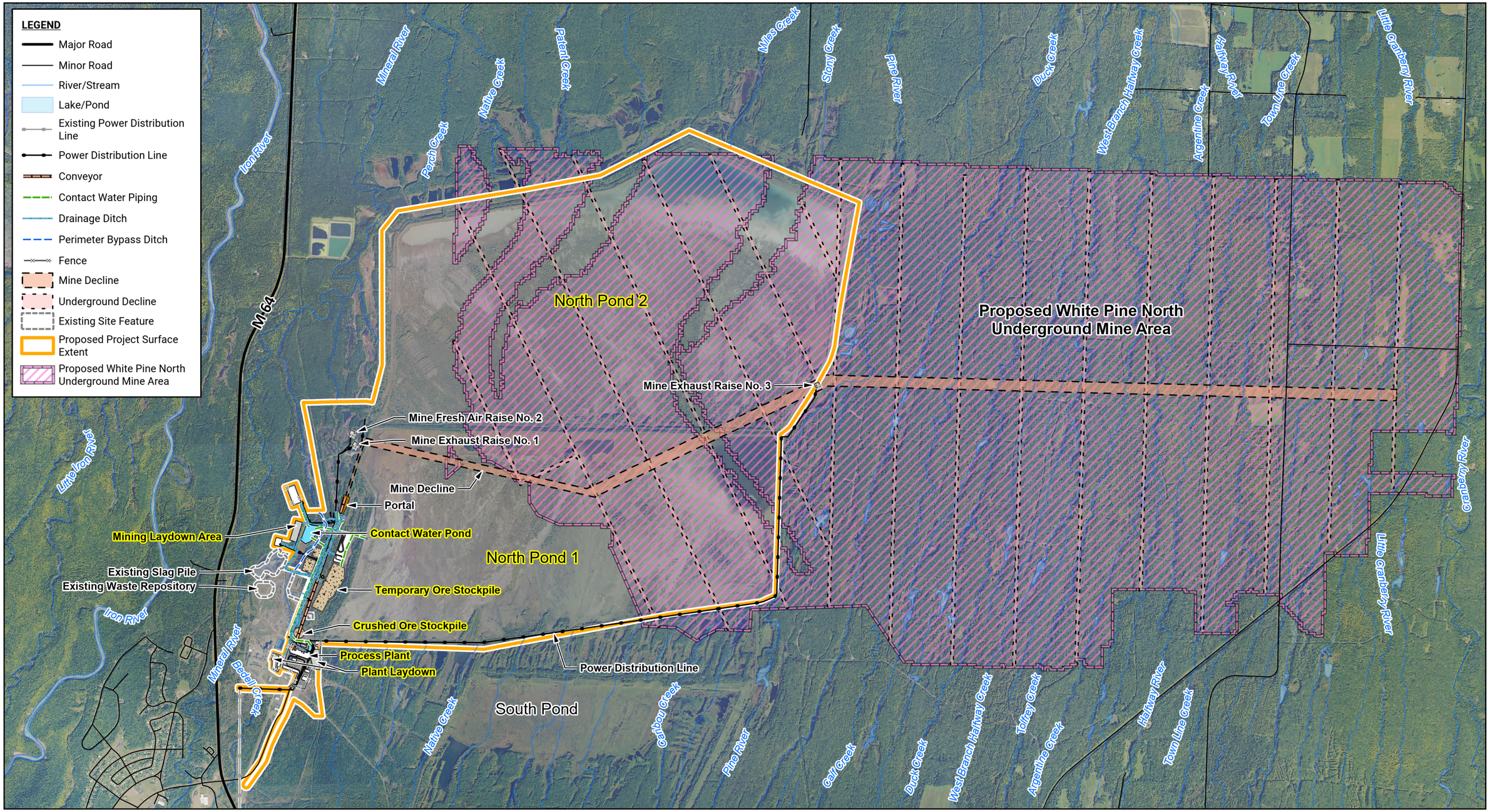

NOTES:

1. 2024 - 18in resolution air photo from the USDA NAIP.
2. Horizontal coordinate system is NAD1983 UTM Zone, 16N, units in meters.
3. Road, boundary and hydrographic data from Michigan GIS Open Data.
4. Site layout and infrastructure data from Ausenco, March 2026.

Highlighted labels identify reactive materials.

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N

Foth Infrastructure & Environment, LLC			
REVISED	DATE	BY	DESCRIPTION

PREPARED BY:	DATE:
REVIEWED BY:	DATE:
APPROVED BY:	DATE:

WHITE PINE COPPER LLC

FIGURE 3-1
IDENTIFICATION OF REACTIVE MATERIALS
TREATMENT AND CONTAINMENT PLAN
WHITE PINE NORTH, ONTONAGON COUNTY, MI

Scale: 0 1,500 3,000 Feet

Date: JUNE 2026

Drafted By: DAT

Project: 25W006

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NORTH POND 2
(REFER TO DRAWING
108571-EB-03100-31345-007-05)

NORTH POND 1

NP1 CONTACT WATER POND
(NOTE 5)

NP1 RECLAIM PUMP STATION
(6m x 6m) (NOTE 4)

NEW SPILLWAY

CONTROL LINE - NP1 WEST AND
NORTH BERM WIDENING

BEDELL POND

EXISTING BEDELL POND
DISCHARGE TO NP1

APPROXIMATE LIMITS OF WASTE
AND TEMPORARY STOCKPILES
(BY OTHERS)

DISCHARGE LOCATION OF
EMERGENCY TAILINGS UPSET
PIPELINE

NP1 AND NP2 TAILINGS AND
RECLAIM WATER PIPELINES
JUNCTION

TAILINGS PIPELINE
(NOTE 3)

APPROXIMATE LOCATION
OF PROCESS WATER TANK

PROPERTY BOUNDARY

FINAL DEPOSITION EL:281.00

FINAL DEPOSITION TOWER EL: 281.00

EG EL:260.99

EG EL:260.77

EG EL:261.57

ACCESS CAUSEWAY (TYP)

RECLAIM WATER PIPELINE
(NOTE3)

SOUTH POND DIVERSION

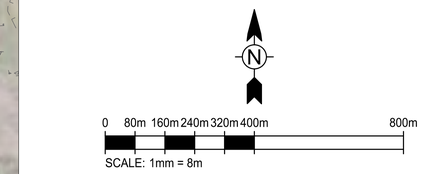
SOUTH DIVERSION DITCH

CONTROL LINE - NP1 SOUTH BERM

750.00m

2.0%

A 31345-005-05
B 31345-005-05
C 31345-005-05
D 31345-005-05
1 31345-010-05
3 31345-010-05
6 31345-011-05
11 31345-013-05
12 31345-011-05
13 31345-013-05



NOTES

1. PROJECT COORDINATE SYSTEM IS UTM-WGS 1984 DATUM, ZONE 16 NORTH, METER; CENT. MERIDIAN 87D W.
2. DRAWING UNITS ARE IN METERS UNLESS OTHERWISE SPECIFIED.
3. PIPELINES SHOWN ARE INDICATIVE. PIPELINE SHALL BE INSTALLED IN BERMS UNLESS NOTED OTHERWISE. REFER TO DRAWING 108571-EB-03100-31345-009-05 FOR PIPING SCHEDULE AND ROUTING.
4. RECLAIM PUMP STATION SHALL BE LOCATED ON NEW CREST PAD IN NORTHEAST CORNER OF NP1.
5. NP1 CONTACT WATER POND INCLUDES NORTHERN AREA OF NP1 ABSENT OF DEPOSITION OF TAILINGS.

LEGEND

- EXISTING CONTOURS (2m MINOR, 10m MAJOR)
- DESIGN CONTOURS (2m MINOR, 10m MAJOR)
- BYPA — SOUTH POND DIVERSION PIPELINE
- REW — RECLAIM WATER PIPELINE
- PRW — TAILINGS PIPELINE
- PRW — FORMER MINE DEWATERING PIPELINE
- EMERGENCY TAILINGS UPSET DISCHARGE PIPELINE
- PROPERTY BOUNDARY
- CONTACT WATER POND
- APPROXIMATE LIMITS OF DISTURBANCE
- DEPOSITION TOWER

NP1 TAILINGS DEPOSITION (2.0% SLOPE)

CUT/FILL	VOLUME* (m^3)	DRY MASS** (M TONS)
FILL	42,584,581	62,173,488

*DEPOSITION VOLUME IS SUBTRACTIVE OF CAUSEWAY FILL VOLUME.
**DRY MASS BASED ON A CONSOLIDATED DRY DENSITY OF 1.46 T/m^3.

NP1 TAILINGS DEPOSITION (2.0% SLOPE)

3d SURFACE AREA (m^2)	SPIGOTS (EA)
4,984,445	3

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MARK	DATE	DESCRIPTION	BY
A	02/27/26	ISSUED FOR REVIEW	RNW
B	03/27/26	ISSUED FOR REVIEW	RNW
C	04/06/26	ISSUED FOR STUDY	RNW
D	08/17/26	ISSUED FOR PERMITTING	

WHITE PINE NORTH
TAILINGS MANAGEMENT
PRE-FEASIBILITY STUDY

TAILINGS DISPOSAL FACILITY
NP1 TAILINGS DEPOSITION
PLAN VIEW (YR1 - YR11)

PROJ:	117-372636-25001
DESN:	C. TOMLINSON
DRWN:	R. WOOLSEY
CHKD:	C. TOMLINSON

108571-EB-03100-31345-004-05

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Bar Measures 1 inch, otherwise drawing not to scale

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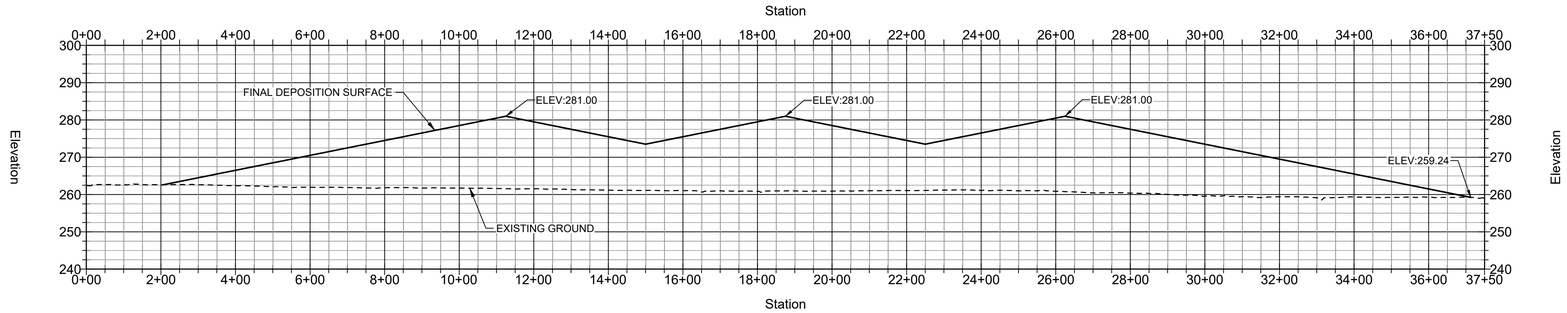

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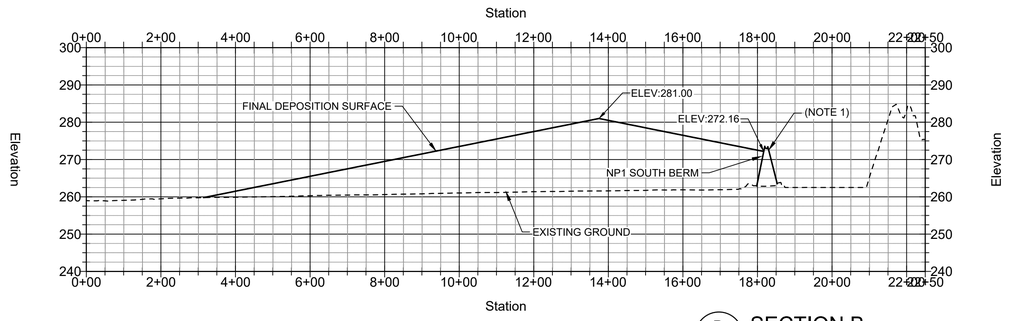

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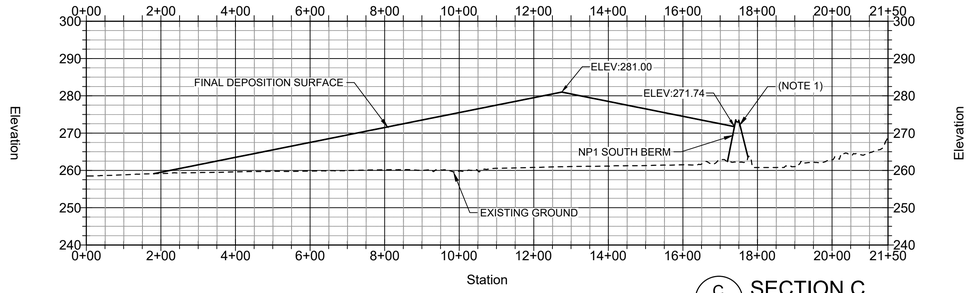

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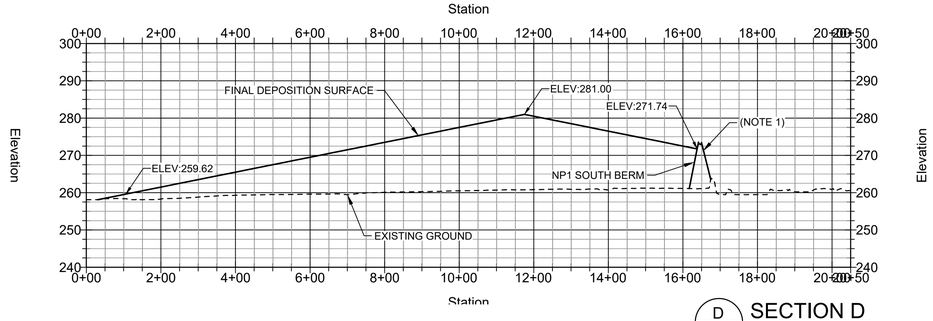

D SECTION D
31345-004-05 SCALE: 1:8

NOTES

1. WATER ACCUMULATED AGAINST SOUTH BERM SHALL BE MANAGED WITH TEMPORARY PUMPING SYSTEMS AND CONSTRUCTION OF TEMPORARY CONVEYANCE CHANNELS.

0 80m 160 240 320 400m 800
SCALE: 1mm = 8m

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C	04/06/26	ISSUED FOR STUDY	RNW
D	08/17/26	ISSUED FOR PERMITTING	RNW

WHITE PINE NORTH
TAILINGS MANAGEMENT
PRE-FEASIBILITY STUDY

TAILINGS DISPOSAL FACILITY
NP1 TAILINGS DEPOSITION
SECTIONS

PROJ:	117-372636-25001
DESN:	C. TOMLINSON
DRWN:	R. WOOLSEY
CHKD:	C. TOMLINSON

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APPROXIMATE LOCATION OF EMERGENCY SPILLWAY

PROPOSED NP2 RECLAIM SPILLWAY

NP2 DIVIDER BERM (NOTE 5)

NON-CONTACT WATER POND

RECLAIMED WATER POND FOR NORTH POND 2

11 31345-013-05 RECLAIM PUMP STATION (6m x 6m) (NOTE 4)

2 31345-010-05

RECLAIM FLUSH WATER, BEDELL PIPELINE, AND MINE DEWATERING PIPELINE (NOTE 3)

4 31345-010-05

NP2 CONTACT/NON-CONTACT WATER CONVEYANCE BERM (REFER TO DETAIL)

NORTH POND 2

1000.00m 1000.00m

6 31345-011-05 INITIAL DEPOSITION TOWER EL: 248.18

EG EL:243.18

INITIAL DEPOSITION TOWER EL: 248.85

EG EL:243.85

INITIAL DEPOSITION TOWER EL: 248.30

EG EL:243.30

CAUSEWAY (TYP)

3 31345-010-05

BEGIN WIDENING BERM DOWNSTREAM BY 3m TO RECLAIM PUMP STATION LOCATION

TAILINGS PIPELINE (NOTE 3)

RECLAIM FLUSH WATER PIPELINE (NOTE 3)

NORTH POND 1 EMERGENCY SPILLWAY

10 31345-013-05 CREST DEPOSTION POINTS (~STA 28+00 - ~STA 54+00)

FORMER MINE DEWATERING PIPELINE

SOUTH POND DIVERSION (3) 60" DIA PIPELINES

NORTH POND 1
(REFER TO DRAWING 108571-EB-03100-31345-004-05)

N

0 80m 160m 240m 320m 400m 800m

SCALE: 1mm = 8m

NOTES

1. PROJECT COORDINATE SYSTEM IS UTM-WGS 1984 DATUM, ZONE 16 NORTH, METER; CENT. MERIDIAN 87D W.
2. DRAWING UNITS ARE IN METERS UNLESS OTHERWISE SPECIFIED.
3. PIPELINES SHOWN ARE INDICATIVE. PIPELINE SHALL BE INSTALLED ON EMBANKMENTS UNLESS NOTED OTHERWISE. REFER TO DRAWING 108571-EB-03100-31345-009-05 FOR PIPING SCHEDULE AND ROUTING.
4. RECLAIM PUMP SYSTEM SHALL BE LOCATED ON NEW CREST PAD LOCATED IN NORTHWEST CORNER OF NP2.
5. NON-CONTACT AND CONTACT WATER PONDS FORMED BY EXISTING PONDED AREAS. THE TWO PONDS SHALL BE SEPARATED BY CONSTRUCTION OF A SEPARATION BERM.

LEGEND

- EXISTING CONTOURS (2m MINOR, 10m MAJOR)
- DESIGN CONTOURS (2m MINOR, 10m MAJOR)
- BYPA — SOUTH POND DIVERSION PIPELINE
- REW — RECLAIM WATER PIPELINE
- PRW — TAILINGS PIPELINE
- PRW — FORMER MINE DEWATERING PIPELINE
- PROPERTY BOUNDARY
- CONTACT WATER POND
- NON-CONTACT WATER POND
- APPROXIMATE LIMITS OF DISTURBANCE
- WATER CONVEYANCE BERM
- DEPOSITION TOWER
- PERIMETER DISCHARGE SPIGOT

NP2 WEST EMBANKMENT

CUT/FILL	VOLUME (m³)	CREST ELEV (m)
FILL	25,235	244.13

NP2 CONTACT/NON-CONTACT CONVEYANCE BERM

CUT/FILL	VOLUME (m³)	CREST ELEV (m)
FILL	78,350	VARIES
CUT	16,680	VARIES

NP2 INITIAL CAUSEWAYS

CUT/FILL	VOLUME (m³)	CREST ELEV (m)
FILL	858,805	VARIES

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C	04/06/26	ISSUED FOR STUDY	RNW
D	08/17/26	ISSUED FOR PERMITTING	RNW

WHITE PINE NORTH

TAILINGS MANAGEMENT
PRE-FEASIBILITY STUDY

TAILINGS DISPOSAL FACILITY MODIFICATIONS PRIOR TO DEPOSITION ON NP2 (YR11)

PROJ:	117-372636-25001
DESN:	C. TOMLINSON
DRWN:	R. WOOLSEY
CHKD:	C. TOMLINSON

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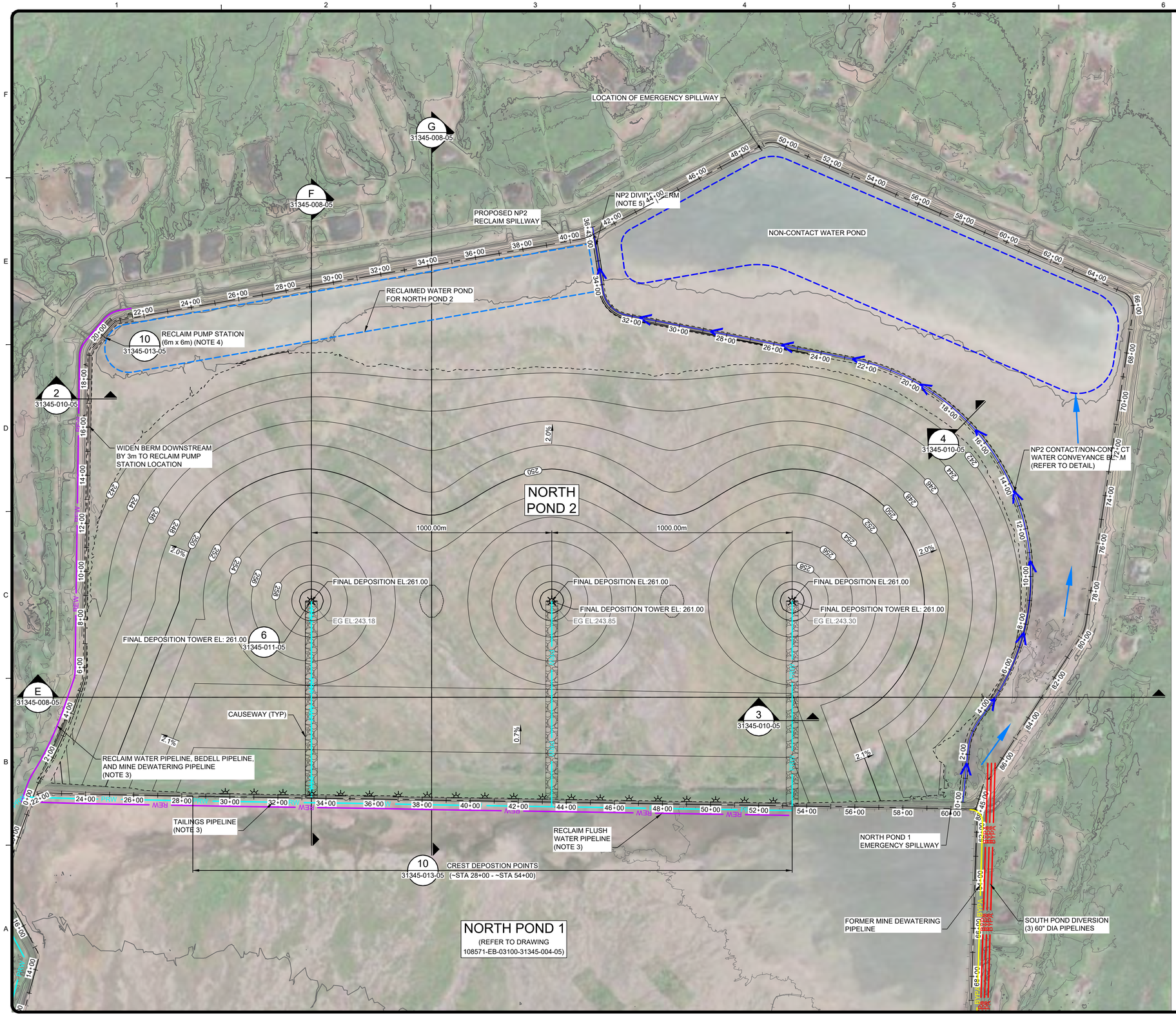

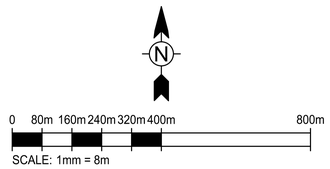

NOTES

1. PROJECT COORDINATE SYSTEM IS UTM-WGS 1984 DATUM, ZONE 16 NORTH, METER; CENT. MERIDIAN 87D W.
2. DRAWING UNITS ARE IN METERS UNLESS OTHERWISE SPECIFIED.
3. PIPELINES SHOWN ARE INDICATIVE. PIPELINE SHALL BE INSTALLED ON EMBANKMENTS UNLESS NOTED OTHERWISE. REFER TO DRAWING 108571-EB-03100-31345-009-05 FOR PIPING SCHEDULE AND ROUTING.
4. RECLAIM PUMP SYSTEM SHALL BE LOCATED ON NEW CREST PAD LOCATED IN NORTHWEST CORNER OF NP2.
5. NON-CONTACT AND CONTACT WATER PONDS FORMED BY EXISTING PONDED AREAS. THE TWO PONDS SHALL BE SEPARATED BY CONSTRUCTION OF A SEPARATION BERM.

LEGEND

- EXISTING CONTOURS (2m MINOR, 10m MAJOR)
- DESIGN CONTOURS (2m MINOR, 10m MAJOR)
- BYPA — SOUTH POND DIVERSION PIPELINE
- REW — RECLAIM WATER PIPELINE
- PRW — TAILINGS PIPELINE
- PRW — FORMER MINE DEWATERING PIPELINE
- PROPERTY BOUNDARY
- CONTACT WATER POND
- NON-CONTACT WATER POND
- APPROXIMATE LIMITS OF DISTURBANCE
- WATER CONVEYANCE BERM
- DEPOSITION TOWER
- PERIMETER DISCHARGE SPIGOT

POND 2 THICKENED TAILINGS DEPOSITION (2.0% SLOPE)(CENTRAL)

CUT/FILL	VOLUME* (m³)	DRY MASS** (M TONS)
FILL	38,223,221	55,805,903

*DEPOSITION VOLUME IS SUBTRACTIVE OF CAUSEWAY FILL VOLUME.
**DRY MASS BASED ON A CONSOLIDATED DRY DENSITY OF 1.46 T/m³.

POND 2 THICKENED TAILINGS DEPOSITION (2.0% SLOPE)(EMBANKMENT)

CUT/FILL	VOLUME (m³)	VOLUME (M TONS)
FILL	10,816,385	15,791,922

**DRY MASS BASED ON A CONSOLIDATED DRY DENSITY OF 1.46 T/m³.

TETRA TECH
www.tetratech.com
4750 West 2100 South, Suite 400
Salt Lake City, UT 84120
Phone: (801) 736-7451 Fax (801) 364-2021

FOR PERMITTING
NOT FOR CONSTRUCTION

MARK	DATE	DESCRIPTION	BY
A	02/27/26	ISSUED FOR REVIEW	RNW
B	03/27/26	ISSUED FOR REVIEW	RNW
C	04/06/26	ISSUED FOR STUDY	RNW
D	08/17/26	ISSUED FOR PERMITTING	

WHITE PINE NORTH
TAILINGS MANAGEMENT
PRE-FEASIBILITY STUDY

TAILINGS DISPOSAL FACILITY NP2 TAILINGS DEPOSITION PLAN VIEW (YR12 - YR26)

PROJ:	117-372636-25001
DESN:	C. TOMLINSON
DRWN:	R. WOOLSEY
CHKD:	C. TOMLINSON

108571-EB-03100-31345-007-05

Copyright: Tetra Tech

Bar Measures 1 inch, otherwise drawing not to scale

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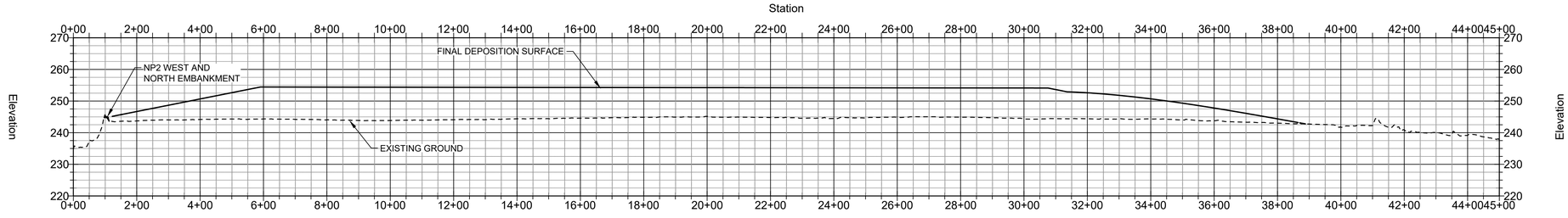

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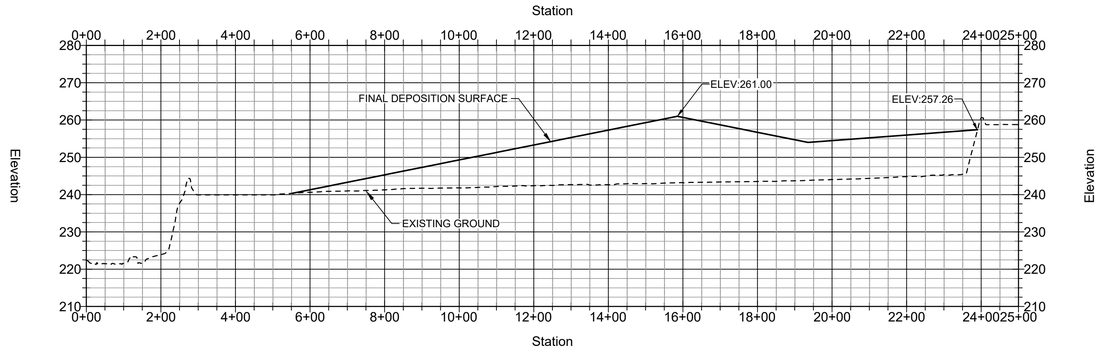

F SECTION F
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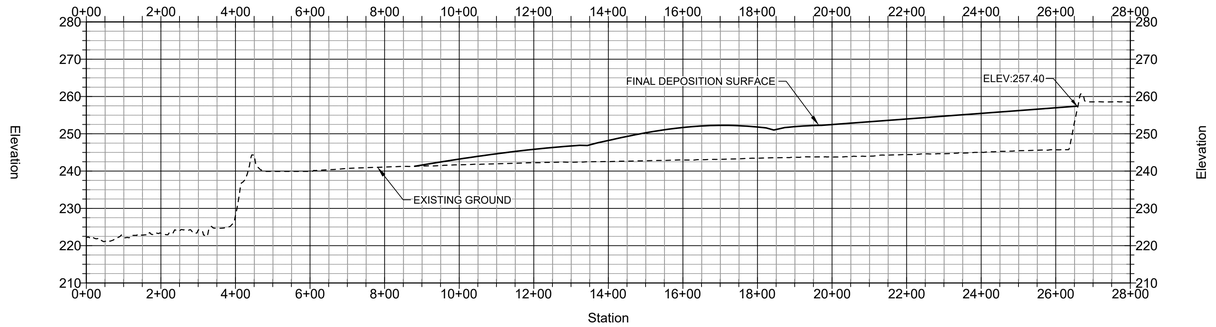

G SECTION G
31345-007-05 SCALE: 1:8

0 80m 160m 240m 320m 400m 800m
SCALE: 1mm = 8m

Bar Measures 1 inch, otherwise drawing not to scale

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Salt Lake City, UT 84120
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MARK	DATE	DESCRIPTION	BY
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B	03/27/26	ISSUED FOR REVIEW	RNW
C	04/06/26	ISSUED FOR STUDY	RNW
D	08/17/26	ISSUED FOR PERMITTING	

WHITE PINE NORTH
TAILINGS MANAGEMENT
PRE-FEASIBILITY STUDY

TAILINGS DISPOSAL FACILITY NP2 TAILINGS DEPOSITION SECTIONS

PROJ:	117-372636-25001
DESN:	C. TOMLINSON
DRWN:	R. WOOLSEY
CHKD:	C. TOMLINSON

108571-EB-03100-31345-008-05

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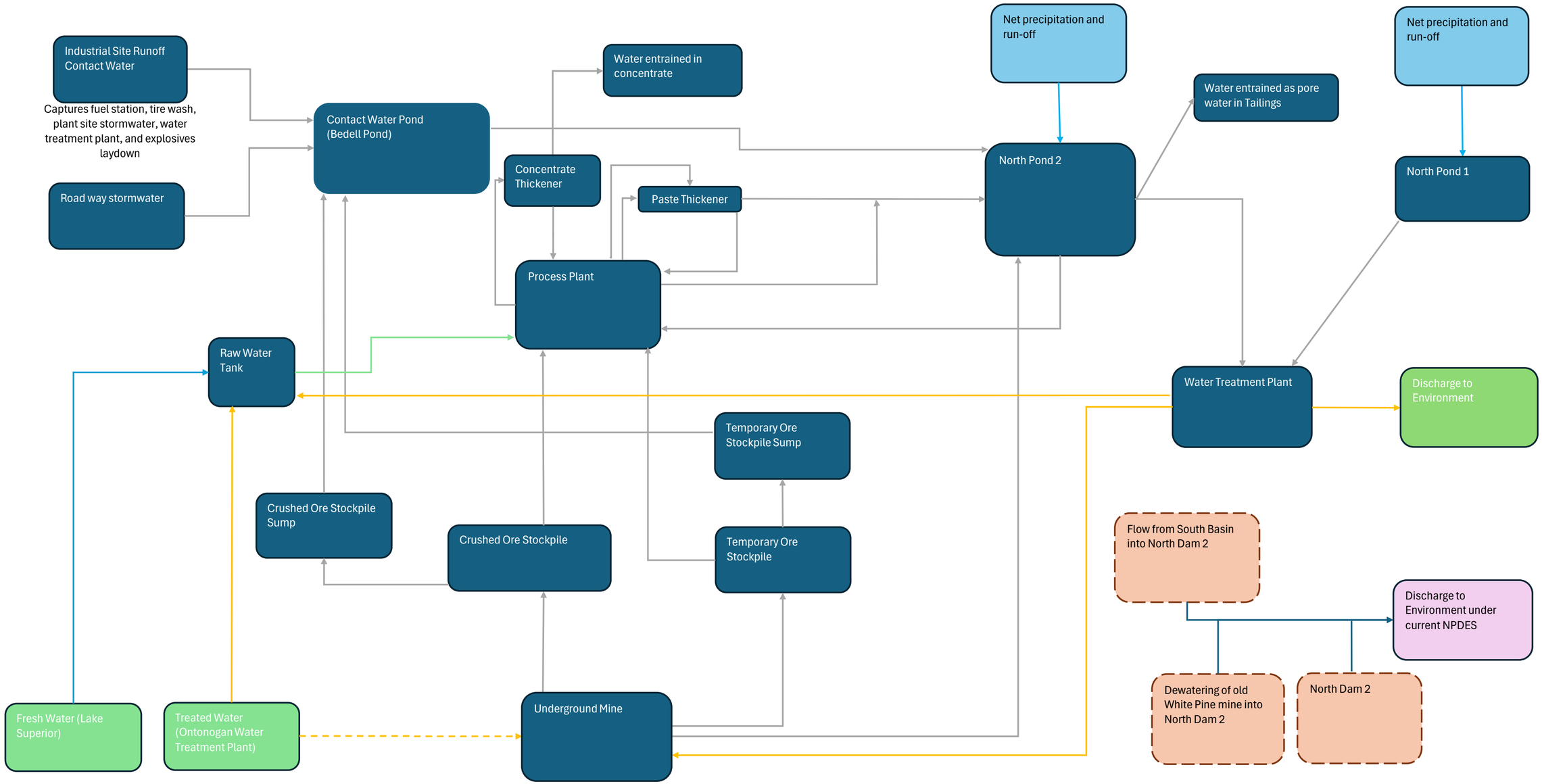


NOTES:

- FRESH WATER = water from Lake Superior
- TREATED WATER = water from water treatment plant
- RAW WATER = FRESH WATER + TREATED WATER
- PROCESS WATER = untreated water, made up of recycled water from process streams (such as conc and thickener o/f, decant from TDF) and water from surface contact water sources

WHITE PINE COPPER LLC

FIGURE 7-1

Conceptual Water Balance

WHITE PINE NORTH

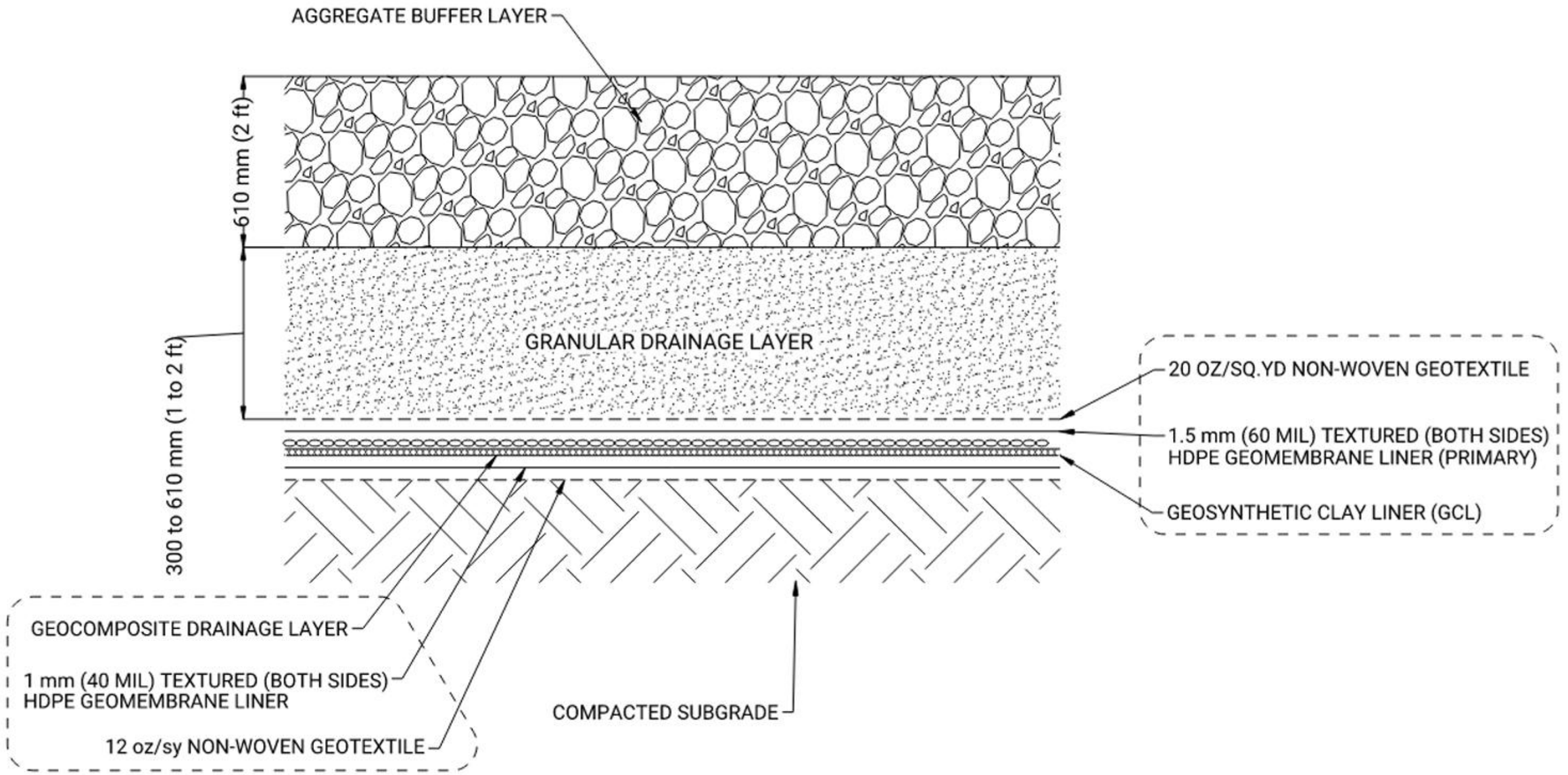
ONTONAGON COUNTY, MICHIGAN

Date: JUNE 2026	Revision Date:	
Drawn By: MJG4	Checked By: CER2	Project: 25W200

Foth

Path: pw:\\Clients\White Pine Copper\23W200\12000 Design Data and Calculations\GoldSim Modeling\Documentation Memo\Att 1 - WPC Operational Water Balance - Paste Tailings - 6.8.26_GPM.xlsx

Date: 2026-06-23



NOTES:

Composite liner system is required for the ore stockpile as detailed in MI Administrative Code R 425.409 and will be comprised of the following from bottom up:

- Compacted subgrade
- 12 ounce per square yard (oz/sy) non-woven cushioning geotextile
- 1 millimeter (mm) (40 mil) High Density Polyethylene (HDPE) geomembrane (secondary liner)
- 200 mil geocomposite
- GCL or 0.91 meters (m) (3 foot) compacted soil layer
- 1.5 mm (60 mil) HDPE geomembrane (primary liner)
- 20 oz/sy non-woven cushioning geotextile

This drawing is neither a legally recorded map nor a survey and is not intended to be used as one. This drawing is a compilation of records, information and data used for reference purposes only.

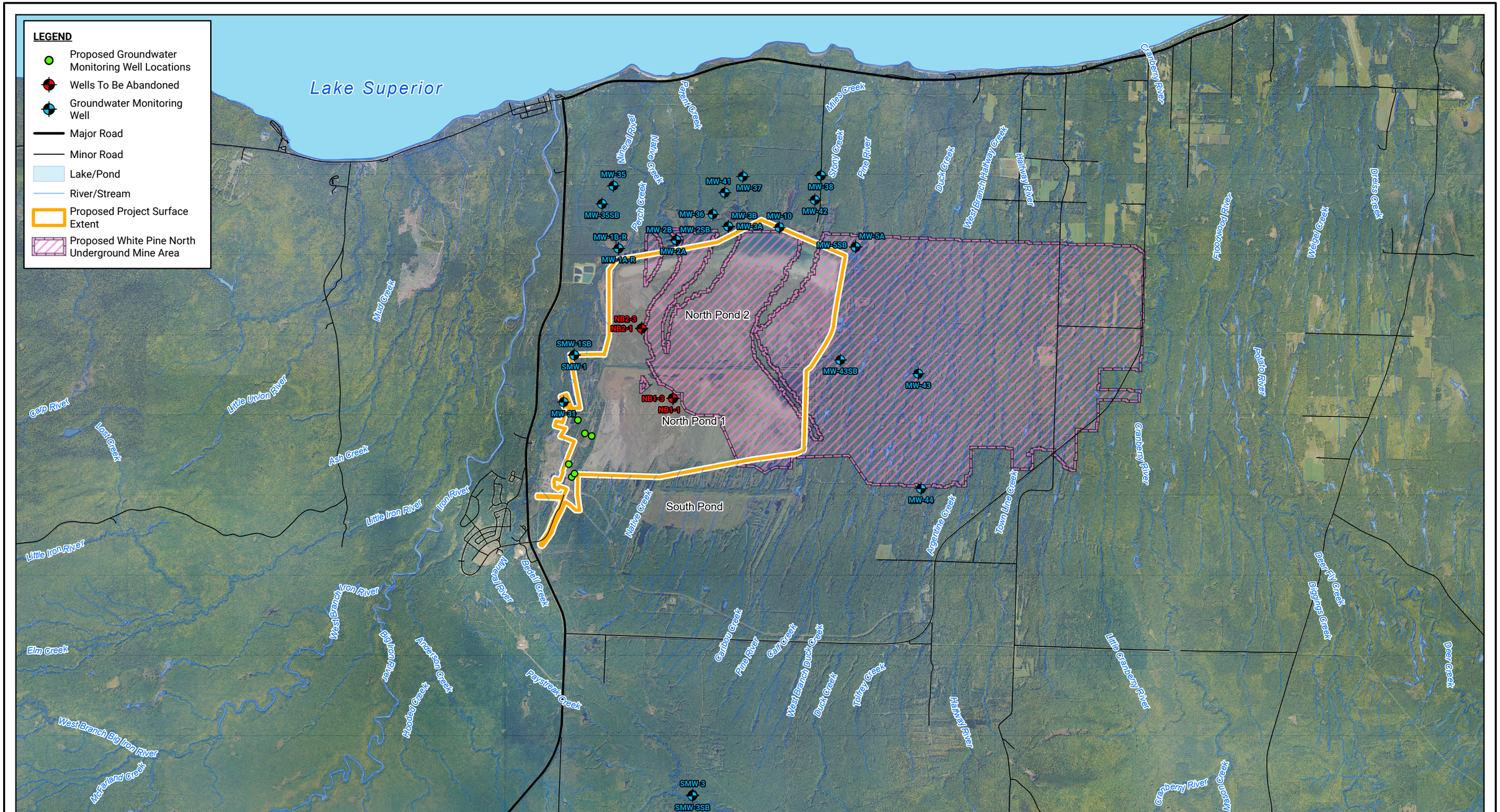
Foth

WHITE PINE COPPER LLC

FIGURE 8-1

TEMPORARY ORE STOCKPILE LINER DESIGN
WHITE PINE NORTH
TREATMENT AND CONTAINMENT PLAN

Date: JUNE 2026	Revision Date:	
DRAWN BY: MJG4	CHECKED BY: CER2	PROJECT: 25W006



NOTES:

1. 2024 - 18in resolution air photo from the USDA NAIP.
2. Horizontal coordinate system is NAD1983 UTM Zone, 16N, units in meters.
3. Road, boundary and hydrographic data from Michigan GIS Open Data.

WHITE PINE COPPER

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Foth Infrastructure & Environment, LLC			
REVISED	DATE	BY	DESCRIPTION
PREPARED BY:			DATE:
REVIEWED BY:			DATE:
APPROVED BY:			DATE:

WHITE PINE COPPER LLC

FIGURE 8-2
GROUNDWATER MONITORING LOCATIONS
TREATMENT AND CONTAINMENT PLAN
WHITE PINE NORTH, ONTONAGON COUNTY, MI

Scale: 0 3,000 6,000 Feet

Date: JUNE 2026

Drafted By: SSG1

Project: 25W006

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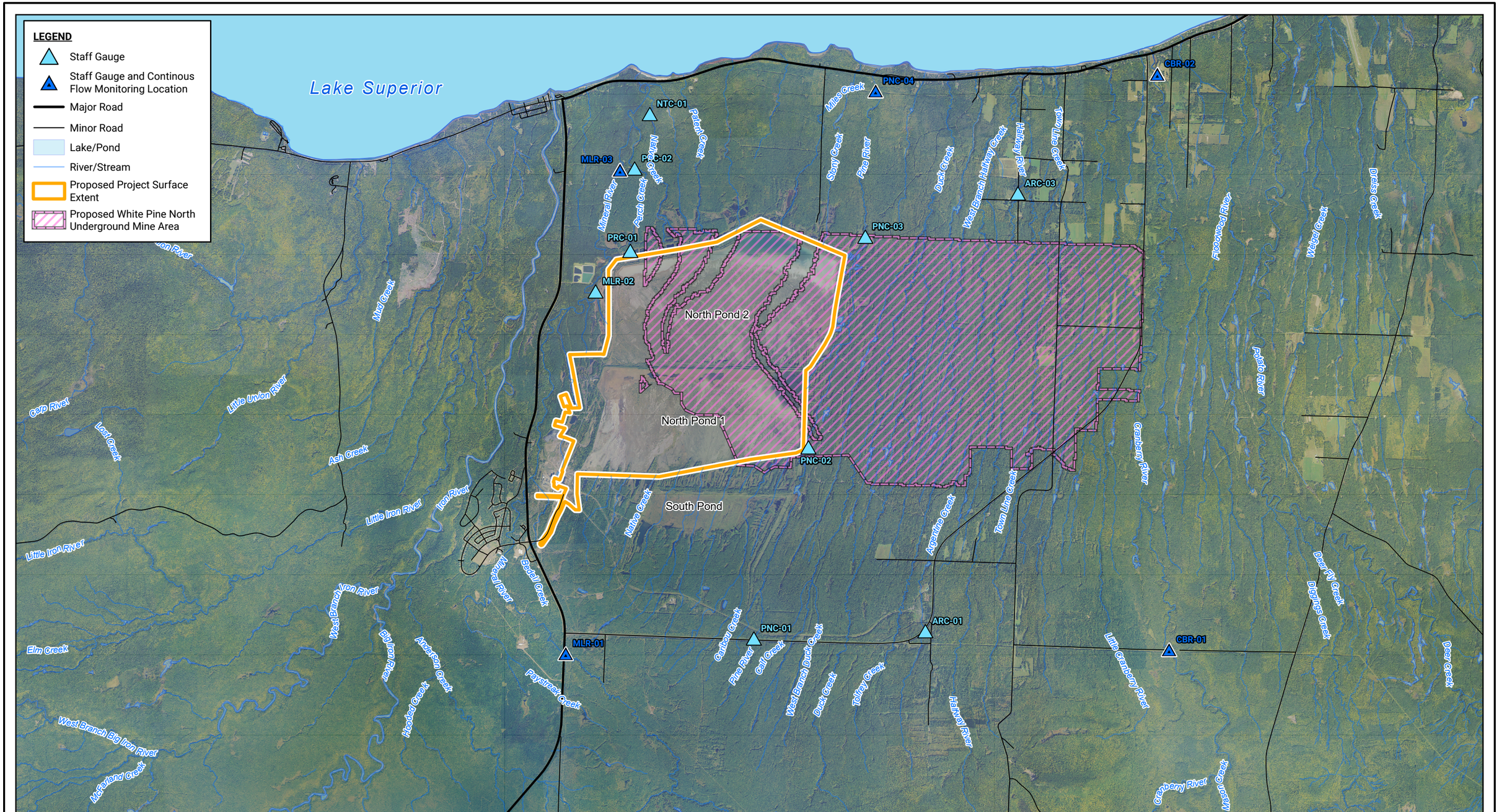

NOTES:

1. 2024 - 18in resolution air photo from the USDA NAIP.
2. Horizontal coordinate system is NAD1983 UTM Zone, 16N, units in meters.
3. Road, boundary and hydrographic data from Michigan GIS Open Data.

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PREPARED BY:		DATE:	
REVIEWED BY:		DATE:	
APPROVED BY:		DATE:	

WHITE PINE COPPER LLC

FIGURE 8-3
SURFACE WATER MONITORING LOCATIONS
TREATMENT AND CONTAINMENT PLAN
WHITE PINE NORTH, ONTONAGON COUNTY, MI

Scale: 0 3,000 6,000 Feet

Date: JUNE 2026

Drafted By: SSG1

Project: 25W006

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