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Design and Analysis of Mine Adit-Plugs within a Tailings Storage Basin

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ABSTRACT: This paper discusses the development of an adit-plug system for sealing historical mine adits located within the Tailings Storage Facility basin at the El Boleo project in Santa Rosalía, Mexico. The historical workings could possibly intersect proposed new mine workings that may allow future tailings to migrate from the TSF into the new mine workings, and eventually to areas outside the TSF creating potential instability and safety concerns. In order to seal the historical workings within the TSF limits, a plug system was designed to incorporate multiple barriers to minimize tailings migration and water inflow into the adits. The plug system includes a structural reinforced concrete plug, shotcrete liners around the plug, and a compacted, structural-fill buttress placed above and in front of the plug. Analytical methods, including an empirical summation-of-forces approach and three-dimensional numerical modeling, have been used to evaluate the global stability of the adit-plug system. Results of the analyses are presented.

1. INTRODUCTION AND BACKGROUND

The Boleo deposit, located near the town of Santa Rosalía, Baja California Sur, Mexico, occurs in the El Boleo Formation which is comprised of at least four minable, bedded, mineralized units called mantos. The deposit was mined using underground methods between 1886 and 1972. The significant copper-cobalt-zinc-manganese resources remaining on the Boleo property will be mined by modern surface and underground mining methods. Over the life of the current mining effort, large volume of tailings will be generated from on-site ore processing, which will be placed in an adjacent Tailings Storage Facility (TSF). The footprint of the tailings will extend over historical mine workings and a portion of the remaining resources that could be exploited. At their ultimate storage height, the tailings will cover all the manto outcroppings within the TSF and exceed the elevation of all nearby historical and planned underground mine workings (Figure 1). The photo of a typical historical mine adit is presented in Figure 2.

The operating mine life is estimated to be 23 years, which will result in tailings deposition to an elevation of approximately 252 meters (m) above Mean Sea Level (MSL). The maximum tailings and supernatant pond height above the lowest mine adit opening is approximately 112 m. Distribution of the tailings will occur along the TSF embankment crests which will accumulate subaqueous depositions of solids against the TSF embankment. The overall average tailings density in the facility at ultimate capacity is estimated to be 0.34 dry tonnes per cubic meter (21.2 pounds per cubic foot) [1].

Site investigation and test mining have indicated that many of the historical, single exploration drifts remain partially open and might serve as a potential conduit for water and tailings. The historical production stopes exposed near manto outcrops appear to have "open" voids that, in some cases, are partially collapsed or illustrate some convergence of the openings. The safety of the operations and the integrity of the TSF itself could be compromised by tailings leakage through the historical or future underground mine workings.

2. ADIT PLUG DESIGN

The usual design for sealing an underground opening is to construct a plug with two parallel rectangular faces in the rock mass that has sufficient shear resistance to withstand the driving force. Such shear resistance can be developed between the edges of the plug and the host rock mass. The design developed by Chekan [2]

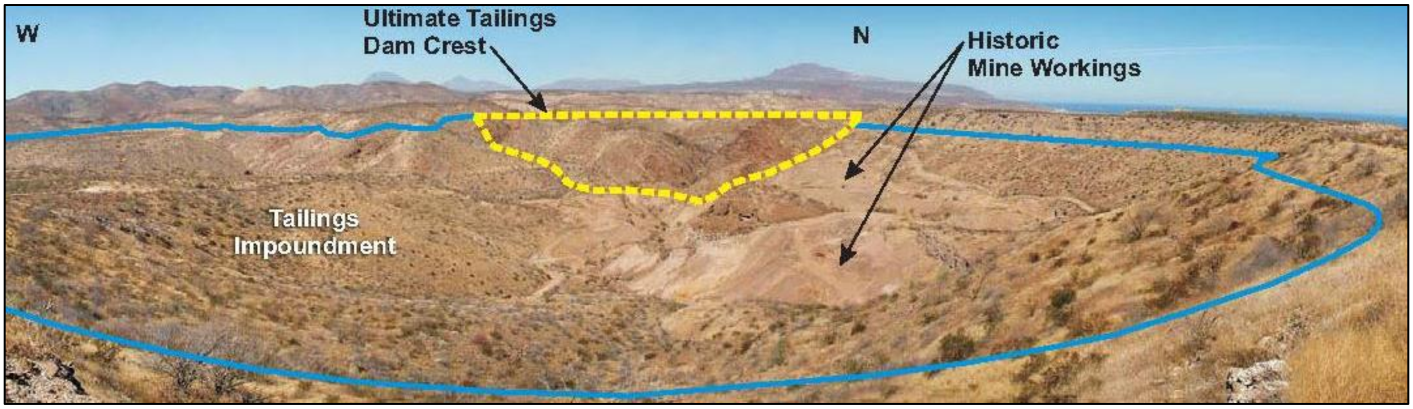

Fig. 1. Panoramic view of TSF and historical mining areas.

Fig. 2. Typical historical mine adit.

has been the standard of the industry for plugs/bulkheads constructed in sound rock formations. This approach was not practical for sealing adits in this case due to the relatively weak nature of rocks encountered at the Boleo site and the need to place the plugs very close to the portals for safety concerns during construction. Because of the weak rock in the ribs and roof of the adits at the Boleo site, gaining access into them would have necessitated temporary adit support.

Therefore, a design utilizing an adit seal method with an extension of approximately 1 to 2 m into the adit was developed. This design relies on the strength characteristics of the rock formation behind the plug and the frictional resistance between the concrete plug and the rock foundation, coupled with the frictional resistance between the concrete plug and the structural fill buttress covering it. A minimum stability factor (SF) of 2.0 was deemed acceptable for this design approach. The plug design is based on the height and width of the plug larger than the portal opening. This design assumes that the plug is placed into compression against the rock face from the hydrostatic load from the overlying tailings and buttress fill. The form inside the adit consists of large rock pushed into the adit along with polyurethane foam to seal voids. A keyway is installed beneath the concrete plug just outside the adit portals. A 200-millimeter (mm)-thick reinforced shotcrete cap layer was designed and installed up to 2 m above and on either side of the plug. The 200-mm-thick shotcrete cap transitions to a 100-mm-thick shotcrete liner and extends over the highwall and up the slope to maintain the structural integrity of the potentially fractured rock slope above the historical workings. Figure 3 illustrates the various components of the plug system. This paper summarizes concepts that were the result of numerous revisions to the design and analysis concepts that were developed over the course of several years. The final version of the design report was issued in early 2012 [3].

3. PLUG DESIGN PARAMETERS

The plug design necessitates the determination of applicable rock strength characteristics, the destabilizing pressure that needs to be resisted, and a method by which the design criteria can be implemented in order to provide suitable resistance against the destabilizing pressure. These items will be discussed in the following sections.

3.1. Rock Strength Determination

A preliminary underground mining geotechnical performance study was performed at the El Boleo Mine by Agapito Associates, Inc. (AAI) in 2006 [4]. One of the tasks completed as part of the study was the collection and testing of geotechnical core samples of the ground surrounding the test mine to characterize pertinent physical properties. Laboratory testing included unconfined compression strength tests (UCS), Brazilian tensile strength tests, and point load tests on samples of sandstone (roof stratum), manto (minable ore body), and conglomerate (floor). The physical properties reported represent the intact rock strength values. The average UCS values of the various rock materials were determined to be: Manto = 543 tonnes

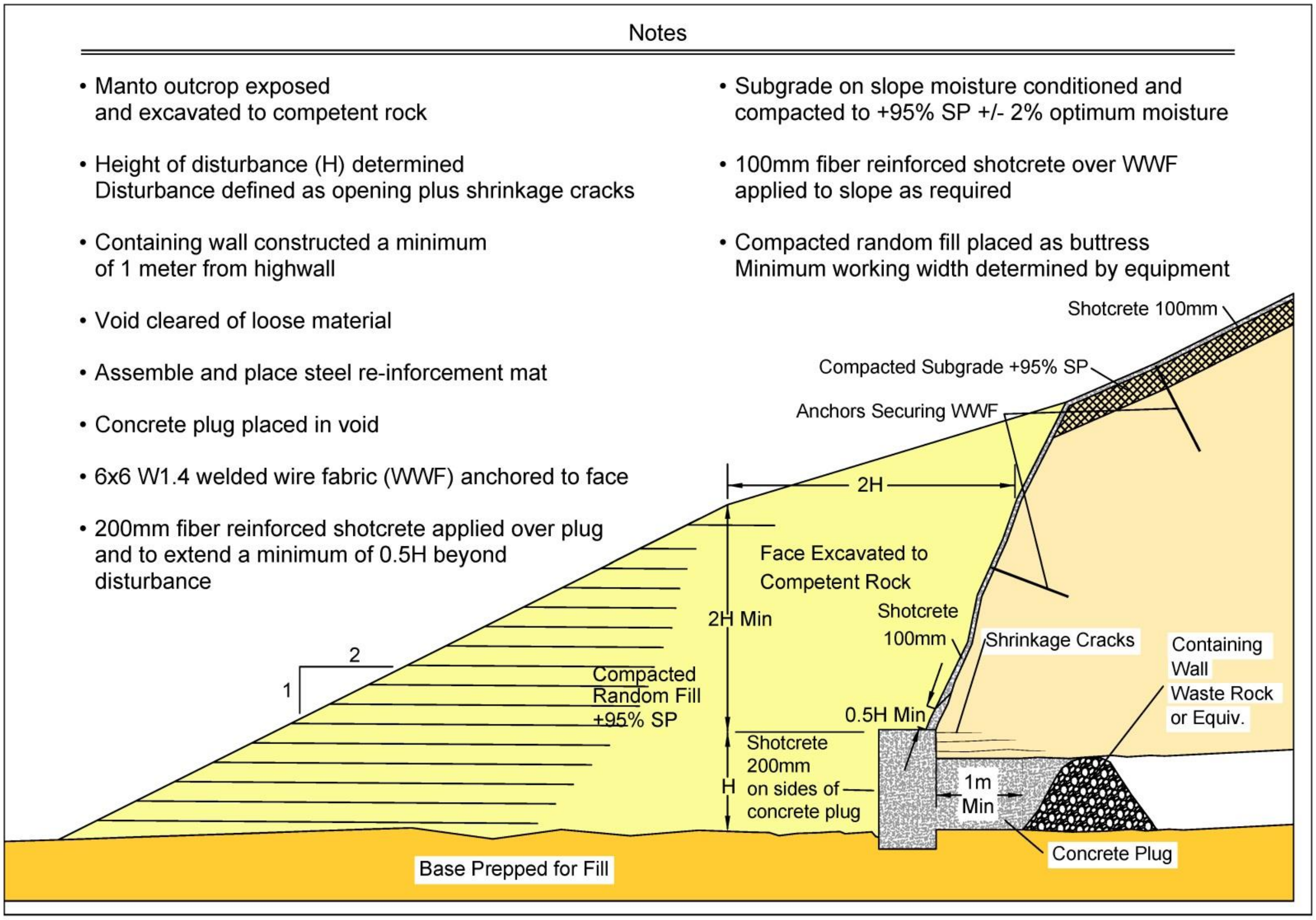

Fig. 3. Typical arrangement of adit plug components.

per square meter (tonne/m^2); Sandstone = 246 tonne/m^2; and Conglomerate = 721 tonne/m^2.

Following the preliminary study, a geotechnical evaluation was conducted as part of a definitive feasibility study for MMB for underground mine development 2007 [5]. The purpose of the geotechnical evaluation was to characterize the strength, structure, and loading/failure characteristics of the proposed underground excavations and surrounding rock strata in order to optimize the underground mine design. As part of this study, additional samples were obtained from the three rock types listed above and tested to determine specific geotechnical characteristics to supplement the findings of the previous study.

The two-dimensional (2D) numerical modeling program FLAC was used to calibrate the rock mass strength characteristics to reflect the ground closure values obtained from field measurements during preliminary test mining. The calibrated rock mass strength and deformability properties used for the underground mine design are summarized in Table 1.

3.2. *Anticipated Tailing Pressure*

The greatest pressure anticipated from the tailings placement in the TSF basin will occur at the maximum tailings depth. The maximum crest elevation of the TSF embankment, considering normal freeboard, will produce a tailings and supernatant pond elevation of approximately 252 m. The elevation of the starter dam is designed at 232 m, which corresponds to a tailings and supernatant pond elevation of 230 m. The lowest elevation within the TSF basin is approximately 140 m. The density of saturated tailings is reported to be 1,249.4 kg/m^3.

Considering the height of tailings that will be contained by the starter embankment, and the lowest elevation in the basin, the resultant pressure that will develop is calculated as follows:

$$\text{Total Pressure } (P_T) = \text{tailing wet density } (\gamma_t) \times \text{height } (H) \quad (1)$$

$$P_T = 1{,}102.7 \text{ kPa} \sim 1.1 \text{ MPa}$$

The wet tailings density of 1,249.4 kg/m^3 was utilized because that density will be typical during the time when it is most important to limit seepage into the

Table 1. Calibrated rock mass strength and deformability properties.

Rock Type	Young's Modulus (GPa)	Peak UCS (MPa)	Peak Cohesion (MPa)	Residual Cohesion (MPa)	Peak Friction (°)	Residual Friction (°)
Sandstone roof	0.85	0.70	0.41	0.30	29.5	23.6
Manto 3	0.40	0.16	0.16	0.12	14.1	11.3
Conglomerate floor	3.98	4.36	0.88	0.72	45.7	41.8

GPa = gigapascal; MPa = megapascal

underground workings. When the tailings consolidate over time, the tailings density will increase to approximately 1,600 kg/m^3. This long-term condition will, however, not produce the higher potential seepage conditions that the earlier wet tailings will produce. Therefore, the wet tailings density is the typical design parameter used for analyses for both the summation of forces and the numerical modeling analyses.

4. PLUG STABILITY ANALYSIS

4.1 Summation-of-Forces Method

A novel approach utilizing engineering mechanics was developed to evaluate the stability of the plugging system, which accounts for the shear resistance in the rock mass behind the plug that needs to be overcome by the destabilizing force due to the tailings load. Considering shear resistance of the rock mass, a conceptual diagram of the likely failure surfaces is presented in Figure 4, along with typical dimensions of a plug, assuming a typical adit height and width of 2 m. The manto, roof, and floor strata thicknesses presented in the figure are representative of the field conditions. The likely failure surface has a truncated pyramid shape and is generated by projecting the inner edges of the plug onto the inner edges of the plug extension inside the adit.

In order for the shear failure to initiate, the shear strength of the rock layers must be exceeded. In addition, the frictional resistance at the bottom interface of the concrete plug and conglomerate base needs to be exceeded to induce lateral movement in the plug. The destabilizing force is the lateral force active on the plug due to the weight of the tailings. An estimation of the destabilizing force is presented below.

Maximum pressure on the plug (P_T) = 1.1 (MPa)

Surface area of plug face (A_f) = 24.94 m^2

$$\text{Total destabilizing force } (F) = P \times A_f = 1.1 \times 24.94 = 27.43 \text{ (MN)} \qquad (2)$$

The shear resistance of the rock mass in the rock slope behind the concrete plug is a function of the shear strength of the rock mass. Since the potential cone of failure passes through three formations (sandstone roof, Manto 3, and conglomerate floor), the shear strength values for the respective rock type was used. Since most of the adits are located in Manto 3, this layer and its associated roof/floor strata were used in the analysis. For each rock type, two sets of shear strengths were considered, "calibrated" rock mass properties and "intact" rock properties. The rationale behind using two sets of properties stemmed from the need to understand the effects "calibrated" and "intact" values may have on the reaction and stability of the plug. Because the size of the drifts and the plugs are much smaller than the underground openings, the authors are of the opinion that when subjected to load from the concrete plug, rock behavior will be governed by the intact rock properties more than by the calibrated rock mass properties. This is logical since the rock mass parameters were defined by calibration of convergence observed over much larger excavations immediately following test mining. To illustrate the impact of strength parameters on stability of the plugs, both calibrated and intact rock properties were used, as presented in Table 2.

Friction along the bottom of the concrete plug is anticipated to resist lateral movement of the block. This resistive force was estimated by multiplying the normal load on the surface area of the plug embedded in ground with a suitable coefficient of friction (the assumed value is 0.7). The frictional resistance calculations for a plug with the dimensions illustrated in Figure 4 are presented below:

- Weight of concrete plug (W_c) = 0.1 MPa
- Normal pressure on plug's bottom,
  (P_b) = 1.2 MPa

Frictional resistance along plug concrete and ground interface:

- Ground embedded surface area of plug,
  (a_b) = 20.46 m^2
- Concrete-rock interface friction coefficient,
  (μ_r) = 0.7
- Frictional resistance along plug concrete and ground interface = 17.19 MN

Frictional resistance along plug concrete and buttress fill interface:

- Area on top of concrete plug (a_t) = 17.98 m^2

Table 2. Shear resistance of rock mass.

Formation	Surface Area (MPa)	Rock Mass Cohesion (MPa)	Rock Mass Angle of Internal Friction (deg)	Rock Mass Shear Strength (MPa)	Intact Rock Cohesion (MPa)	Intact Rock Angle of Internal Friction (degrees)	Intact Rock Shear Strength (MPa)	Shear Resistance (MN) Rock Mass Properties	Shear Resistance (MN) Intact Rock Properties
Sandstone Roof	16.00	0.41	29.5	1.02	0.89	29.5	1.50	16.3	23.98
Manto 3	9.81	0.16	14.1	0.43	0.61	14.1	0.88	4.22	8.59
Conglomerate Floor	5.06	0.88	45.7	1.98	2.57	45.7	3.67	10.03	18.58
					Total Shear Resistance (MN)			30.56	51.15

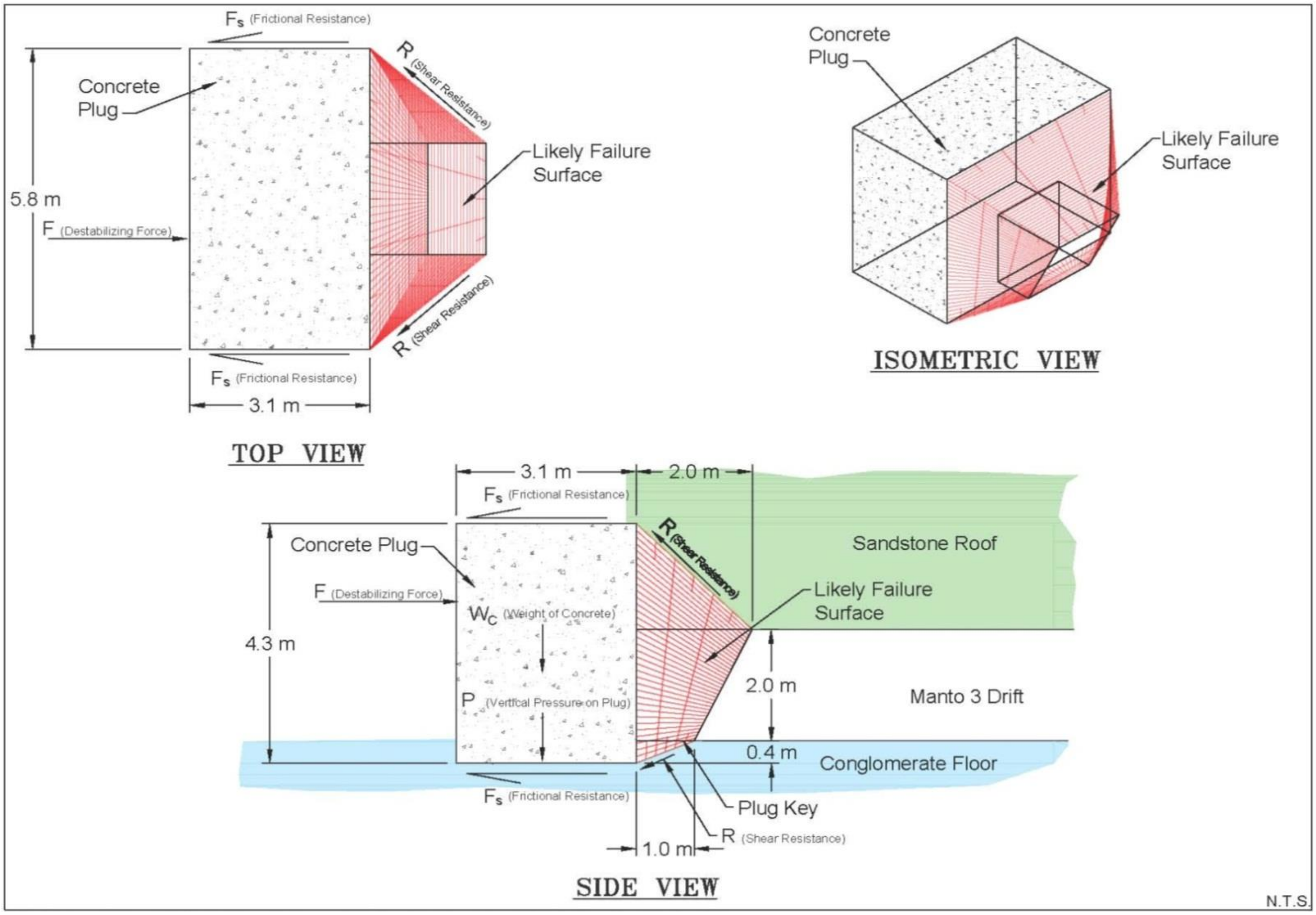

Fig. 4. Likely failure surfaces and forces acting on the concrete plug.

- Area of concrete plug sidewalls (a_{sw}) = 24.18 m^2
- Buttress fill-concrete plug interface friction coefficient (μ_f) = 0.25
- Frictional resistance along concrete plug top and buttress fill interface = 4.94 MN
- Frictional resistance along concrete plug sidewalls and buttress fill interface = 6.65 MN

The total resistive force to prevent movement of the concrete plug is a sum of the shear resistance of the rock mass in the rock slope face and the frictional resistance between the concrete plug bottom and the conglomerate floor. The SF against plug failure was estimated by dividing the total resistive force with the destabilizing force. The calculations are presented below:

- Total Resistive Force (*R*)
  = 59.34 MN (based on rock mass properties)
  = 79.93 MN (based on intact rock properties)
- The Destabilizing Force (from Eq. 2)
  F = 27.43 MN
- Stability Factor (SF) against plug failure = *R/F*
  = 2.16 (based on rock mass properties) (3)
  = 2.91 (based on intact rock properties)

Using "calibrated" rock mass and "intact" rock properties, the calculated SF values against plug failure were 2.16 and 2.91, respectively. The authors are of the opinion that the actual SF value will tend to be closer to 2.91 than 2.16 and is likely to be stable in the long term. It is intuitive that plug dimensions can be reduced for higher adit elevations, with decreasing tailings pressure.

4.2 Numerical Modeling Analysis of Global Stability

A 3D continuum analysis, using FLAC3D [6], was also performed to evaluate the stability of the adit-plug system. The analysis accounted for all the constituents of the plug design (i.e., buttress fill, structural concrete plug, adit opening, highwall slope, and shotcrete cap and liner). Two 3D models were developed to simulate the tailings load on the adit-plug system; the first model representing a tailings depth of 70 m, and the second model representing a tailings depth of 110 m, as measured from the adit invert elevation.

The adit and highwall slope geometry utilized in the models was idealized (i.e., input as smooth, planer surfaces). The representative model geometry in section is presented in Figure 5. Utilizing the symmetry of the plug, shotcrete, and fill about the center vertical plane of the adit, only half of the adit-plug system geometry was modeled, to limit the computing effort. Both the models were developed using an adit dimension of 2 m × 2 m. The manto was modeled as 2 m thick, and the roof and floor were modeled as sandstone and conglomerate, respectively.

The appropriate plug dimensions for this size of adit were determined from an independent design and incorporated in the models with a plug thickness of 3.1 m for the first model (70-m tailings depth) and a plug thickness of 3.5 m for the second model (110-m tailings depth). The slope face was modeled to be inclined at an angle of 75° from the top of the structural concrete plug. Shotcrete was applied around the plug and on the rock face as designed. The rock-fill inside the adit to support the form for placement of the plug insert was also included in the models. Once the plug configuration was developed in each model, the buttress fill was included prior to adding the respective tailings loads. The tailings load applied on top of the first model, corresponding to a 70 m depth, was 0.73 MPa, and the tailing load on top of the second model, corresponding to a 110 m depth, was 1.22 MPa (assuming a tailings density of 1,249 kg/m^3). *In situ* stresses were not considered within the rock slope. The buttress fill was modeled to be an elastic material, since the yielding of the buttress fill due to the tailing load is inconsequential to the overall stability of the adit-plug system and the elastic material assumption precludes the buttress fill from yielding, decreasing computational effort in the process. All other constituents in the models were assigned Mohr-Coulomb elastic-perfectly-plastic behavior. In the absence of actual characterization data, the properties of the rock-fill were assumed to be the same as manto. The rock strength properties used for the sandstone, manto, and conglomerate are those that have been summarized in Table 1. A summary of all input material parameters used in the FLAC3D numerical models are presented in Table 3.

The results indicate that the long-term stability of the analyzed adit-plug configuration is unlikely to be impacted by the load from tailings deposition up to 110 m in height. The distribution of stability factors (i.e., strength-to-stress ratio) around the adit-plug interface in the vertical section indicates SF values in excess of 2.0 (Figure 6). Very small pockets above and below the edges of the plug insert exhibit SF values close to unity, which are simply localized stress concentrations and are unlikely to adversely impact the global stability of the adit-plug system. The plot of yielded zones (Figure 7) does not indicate any large-scale yielding in the adit-plug interface due to the plug. However, very small pockets of yielding are present in the zones discussed above that are anticipated to concentrate stresses. The yielded zones surrounding the adit may be attributed to deformation under gravitational loading in the immediate aftermath of mining.

The SF contours for the first model (70-m tailing depth) along a horizontal section through the adit-plug system (1 m above the adit floor) resulted in a SF value greater than 2.0. Figures representing analysis of the horizontal section under load from 70-m tailing height are not included in this paper. The manto rock mass surrounding the adit has SF values close to unity, which may again be attributed to deformation of the relatively weak manto by gravitational loading. These zones are behind the rock-fill and could result in minor surface spalling of the roof and rib of the adit, but are unlikely to affect global stability of the adit-plug system. It can be inferred from this case that the horizontal confinement provided by the plug along with the shotcrete lining leads to stabilization of rock slope at the interface. The plot of yielding in the same horizontal section indicates that exertion of tailings load on the concrete plug of the existing design will not create massive yielding or failure of the rock slope.

The analysis results for the second model (110-m tailings height) are presented in Figures 8 and 9. The distribution of SF values around the adit-plug interface in the vertical section indicates values in the 1.25–1.5 range (Figure 8). Small pockets above and below the edges of the plug insert exhibit SF values close to unity, which are localized stress concentrations and are unlikely to adversely impact the global stability of the adit-plug system. The plot of yielded zones (Figure 9) in the same vertical plane does not indicate any large-scale yielding due to shear in the adit-plug interface

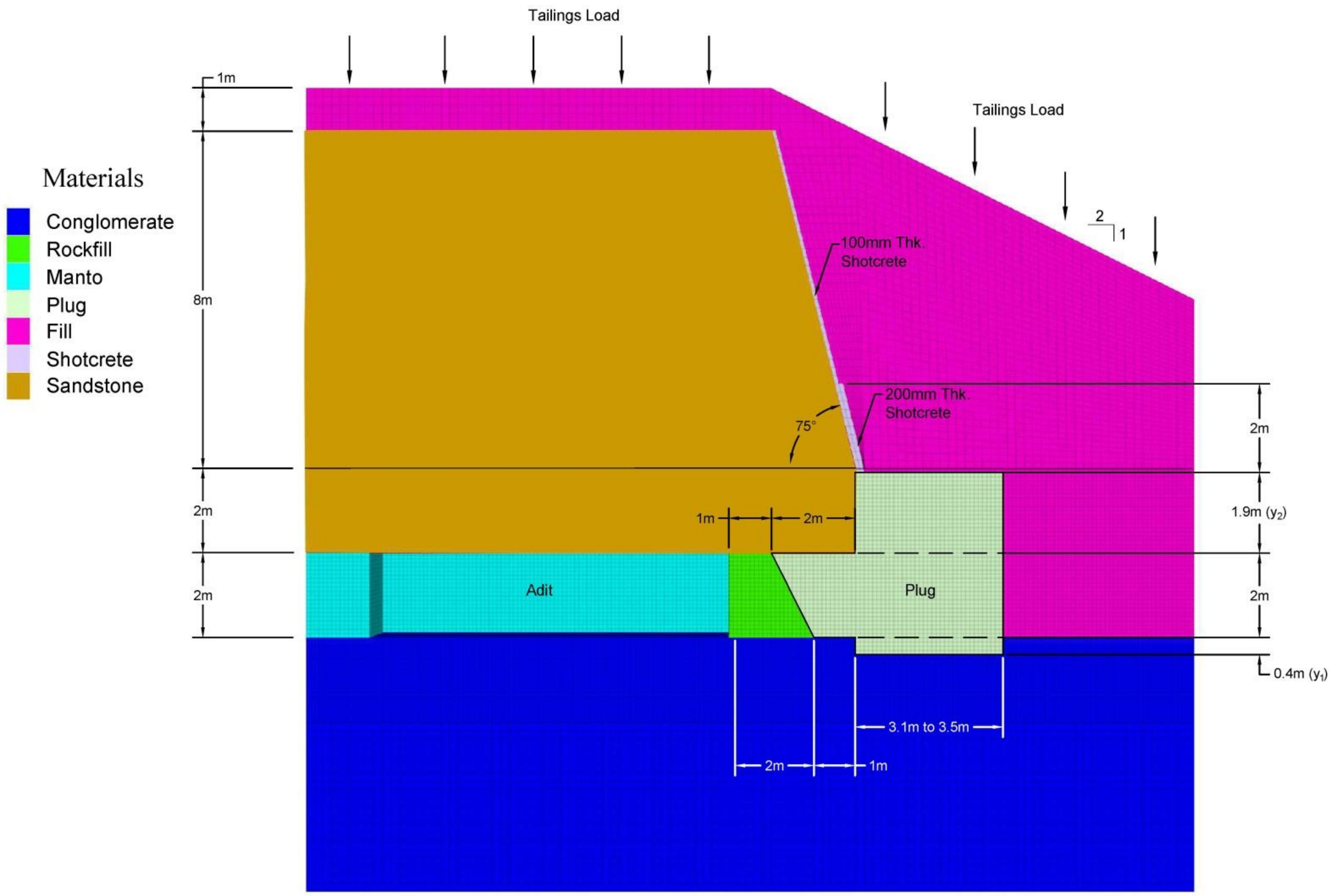

Fig. 5. Model geometry for vertical section.

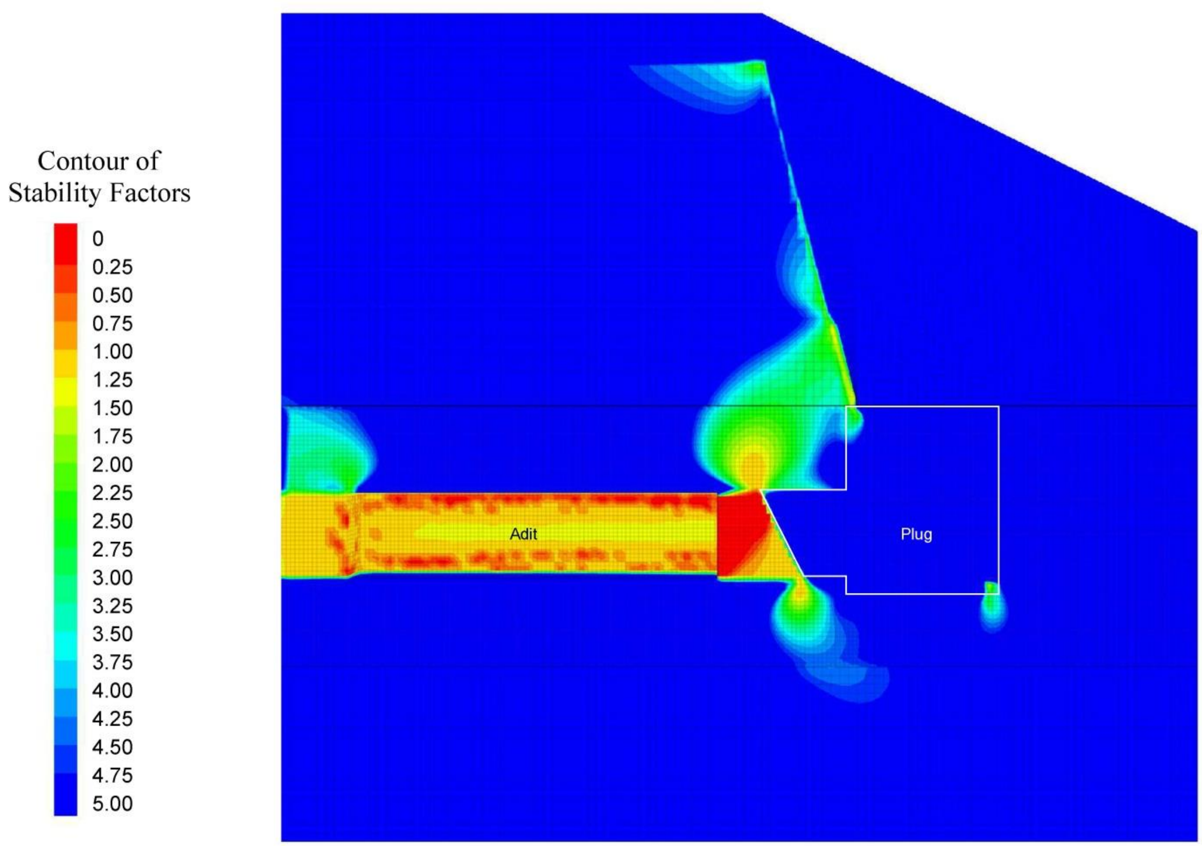

Fig. 6. Stability factor contours (70m tailings depth).

Table 3. Summary of material input parameters used in FLAC3D analysis.

Material	Bulk Modulus (GPa)	Shear Modulus (GPa)	Tensile Strength (MPa)	Peak Cohesion (MPa)	Peak Friction Angle (°)	Density (kg/m^3)
Conglomerate—floor	4.42	1.47	0.50	0.88	45.7	2,200
Manto 3—adit	0.44	0.15	0.02	0.16	14.1	1,600
Sandstone—roof	0.94	0.31	0.50	0.41	29.5	1,800
Concrete plug	14.29	13.04	3.5	10	35.0	2,400
Buttress fill	0.01	0.0019	0.00	0.00	30.0	2,000
Shotcrete—cap/liner	14.30	12.00	3.0	7.5	35.0	2,400
Rock-fill concrete backstop	0.44	0.15	0.02	0.16	14.1	2,000
Tailings						1,249

due to tailing load on the plug. Small pockets of yielding are seen in the zones of stress concentration discussed above, which are slightly larger in size than the yielded zones in the first model, due to the increased loads from the tailings.

The SF contours for the first model along a horizontal section through the adit-plug system (1 m above the adit floor) resulted in SF value at the adit-plug interface in this horizon is also in the 1.25–1.5 range. Figures representing analysis of the horizontal section under load from 110-m tailing height are not included in this paper. The manto rock mass surrounding the adit has SF values close to unity, which may again be attributed to deformation of the relatively weak manto by gravitational loading. These zones are behind the rock-fill and could result in minor surface spalling of the roof and rib of the adit, but are unlikely to affect global stability of the adit-plug system. The horizontal confinement provided by the plug along with the shotcrete lining on the rock slope at the interface continues to have a stabilizing effect even at the tailings depth of 110 m. The plot of yielding in the same horizontal plane indicates that exertion of tailings load on the concrete plug of the proposed design is unlikely to create massive yielding and failure of the rock slope.

Overall, the results of the numerical modeling have provided confirmation of the analytical approach (summation of forces) discussed in the previous section, with the predicted SF values from either method being similar in magnitude. Even though the SF values at the plug-adit interface decrease as the tailing depth is increased from 70 m to 110 m, the SF values are still acceptable for long-term stability. The simulated scenarios are very conservative in the sense that they do not account for the high tensile strength of the steel reinforcement grid, which is proposed to be embedded in the shotcrete lining. Such reinforcement is anticipated to increase the flexural strength of the shotcrete lining, which will in turn enhance the stability of the plug-adit interface. The authors have also noted that the horizontal displacement of the plug into the rock slope due to loading is predicted to be less than 5 centimeters (cm) with the analyzed adit-plug configuration. In addition to the two models discussed above, the authors also ran a model without the application of any shotcrete on the slope face and found that shotcrete application provides a stabilizing influence at the adit-plug interface. In conclusion, the proposed plug systems have demonstrated suitable stability factors in the long term with tailing deposition of up to 110 m in height.

The upper portion of the portal cap, which bears upon the rock above the portal during loading from the TSF, will develop tensile stresses on the inside face. The design of the portal cap, therefore, includes design of reinforcing steel near the inside face to prevent cracking of the concrete as the structure is loaded. This is based on a structural concrete having a 28-day compressive strength of 27.6 MPa (4,000 pounds per square inch [psi]). Steel reinforcement is assumed to be American Standard of Testing Materials (ASTM), grade 60 with yield strength of 413.7 MPa (60,000 psi). The requirements of the American Concrete Institute (ACI 318) were followed in developing the minimum concrete thickness of the portal cap and steel reinforcement requirements. Table 4 presents the dimensional requirements of the adit portal cap/plug system for adits subjected to 90 m of tailing pressure. For adits of differing sizes, construction drawings included various plug dimensions for adits located at greater and lesser tailings depth.

In all cases, the length of plug beyond the cap into the adit will be a minimum of 1.0 m. Thus, the total thickness of the concrete cap/plug will vary from 1.75 m for a 0.5-m-high opening to 5.7 m for a 3.0-m-high opening. Steel reinforcement required along the inside face of the portal cap will vary from 19- to 25-mm-diameter bars at 100 mm to 300 mm on center each way for various cap thicknesses.

5. CONCLUSIONS AND RECOMMENDATIONS

With the risk of tailings migrating from the TSF into the Boleo mine workings, both historical and future

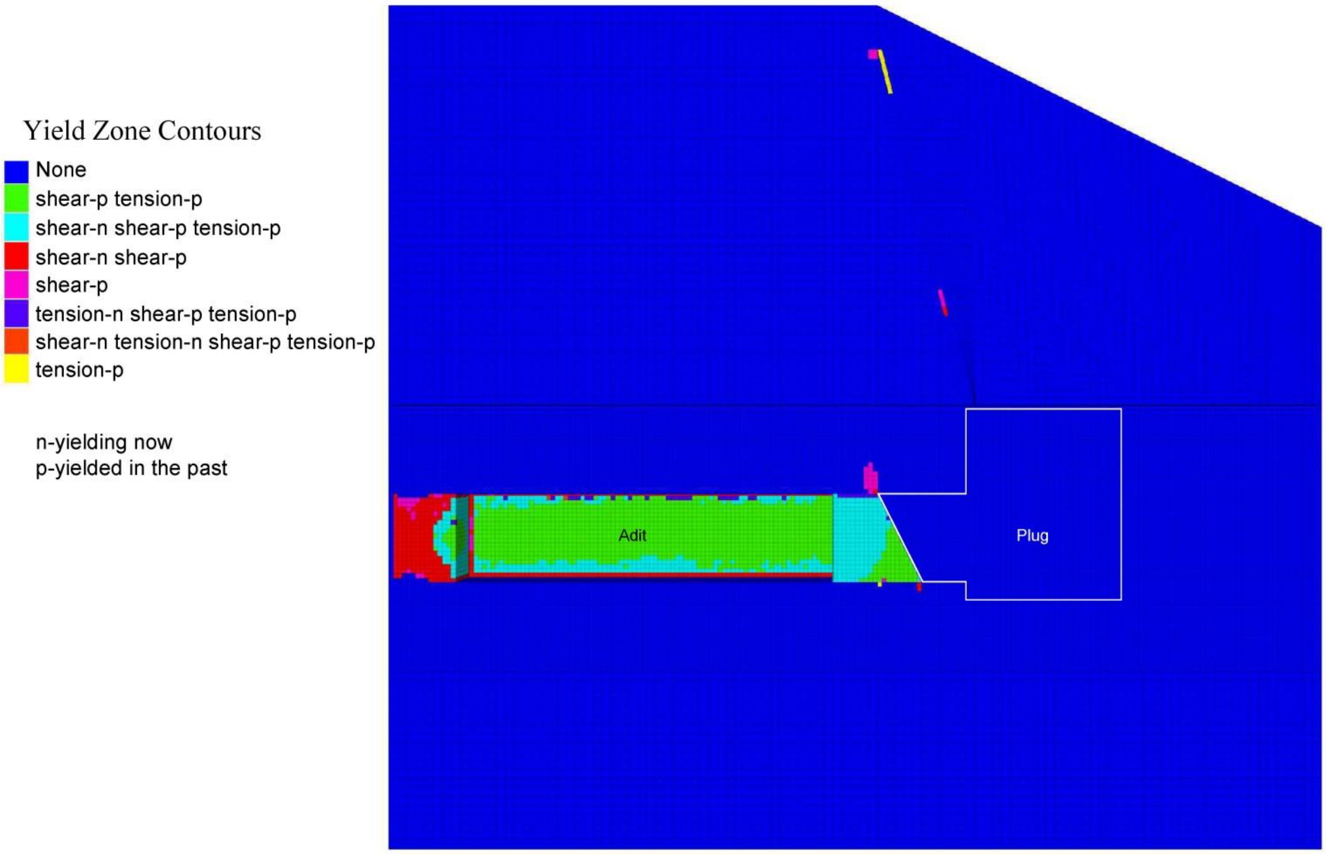

Fig. 7. Yield zone contours (70m tailings depth).

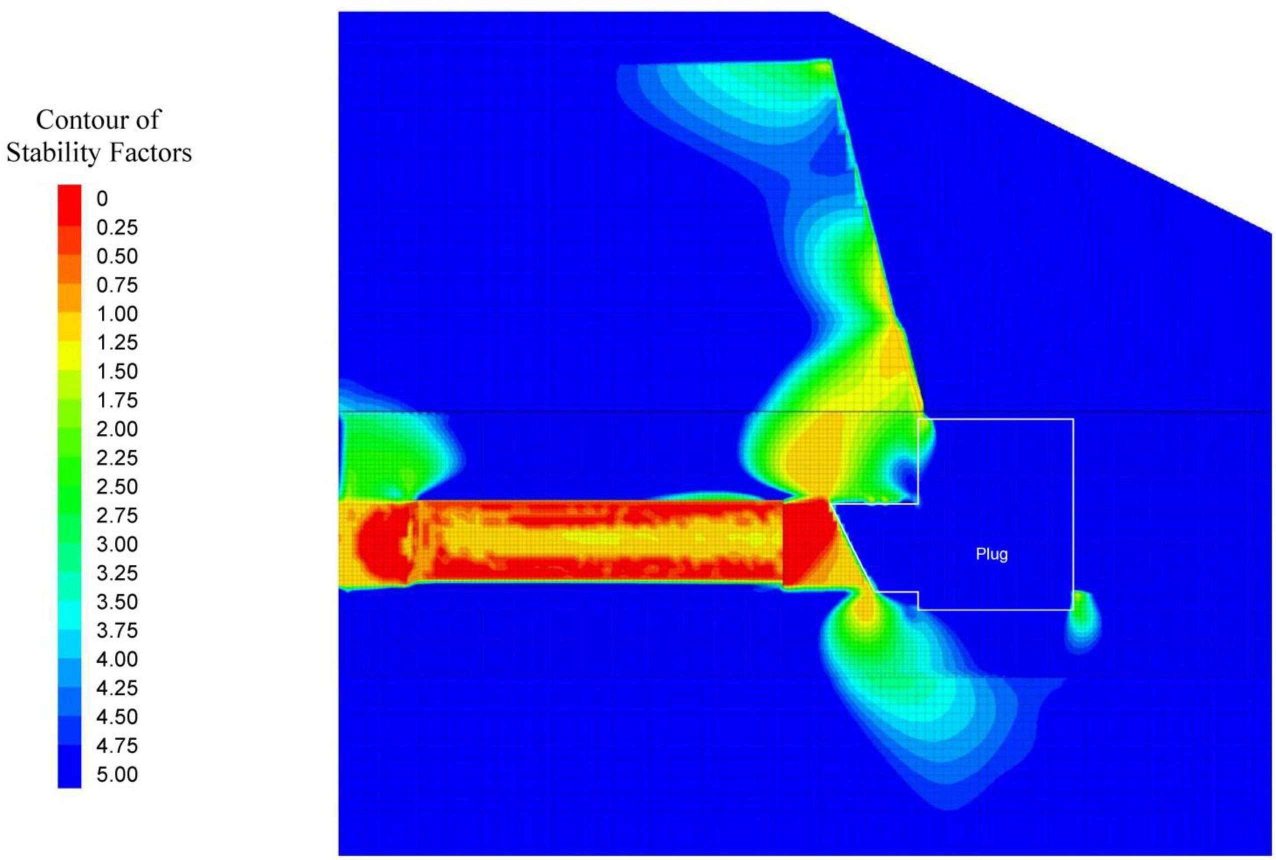

Fig. 8. Stability factor contours (110m tailings depth).

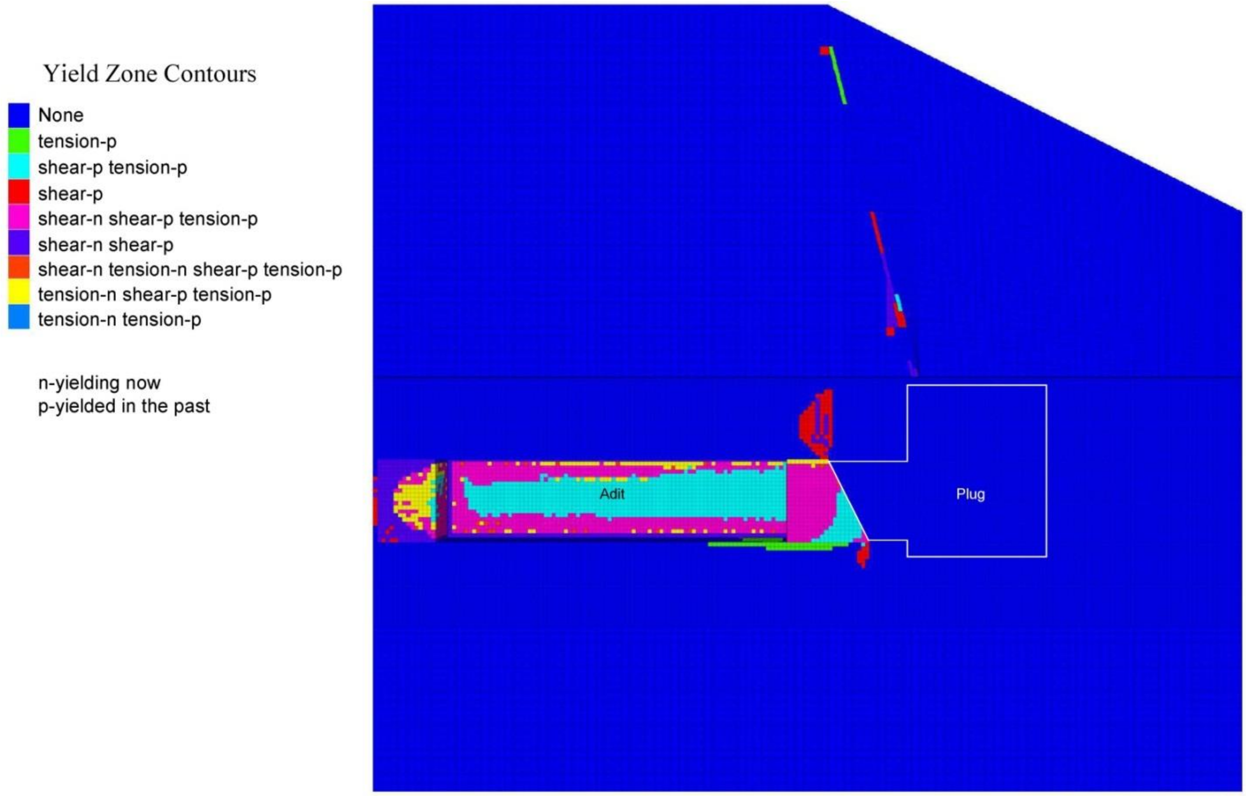

Fig. 9. Yield zone contours (110m tailings depth).

Table 4. Dimensional and reinforcement requirements for the adit structural concrete plug.

Height of Adit, *H* (m)	Dimension below Adit Invert, y_1 (m)	Dimension from Edge of Adit to Outside Edge of Cap, y_2 (m)	Cap Thickness, *t* (m)	Steel Reinforcement Bar Diameter and Spacing Each Way
0.5	0.3	0.4	0.75	19 mm @ 300 mm
1.0	0.3	0.9	1.5	22 mm @ 300 mm
1.5	0.4	1.4	2.4	22 mm @ 225 mm
2.0	0.4	1.9	3.1	22 mm @ 150 mm
2.5	0.5	2.3	3.9	25 mm @ 150 mm
3.0	0.6	2.6	4.7	25 mm @ 100 mm

Note: Dimensions and reinforcement requirements provided are based on 90 m of tailing pressure.

developments, multiple lines of impediment through the adit-seal system presented in this paper is likely to protect miners, mining equipment, and the environment. The mine adit plugging system at the Boleo project has been designed with a multi-component system consisting of the structural reinforced concrete plug, adjacent reinforced shotcrete, and an engineered structural fill buttress. It is expected that the combination of this multi-component system will provide an adequate barrier against significant tailings release into underground workings and reduce seepage to acceptable levels.

The structural concrete adit plugs have been analyzed using a summation-of-forces approach on the concrete plug, based upon the loads created by the placement of 90 to 110 m of tailings over the plugs. The intact rock strengths and the calibrated rock strengths developed during historic underground test mining were used. Stability factors against rock failure vary from approximately 2.16 to 2.91. Potential concrete failure modes in shear and bending are expected to have SFs higher than 2.0 because the concrete and shotcrete strengths are higher than the strongest rock strengths against which the concrete and shotcrete components are constructed. The results obtained from numerical modeling confirmed the findings of the summation-of-forces approach. The SF values observed in the rock face from both 70 m and 110 m of tailing deposition indicate long-term global stability of the plug system. Based on the result of the analyses, actual plug designs for adits located at depths greater than 110 m and less than 70 m were developed.

The plug has been designed as part of a system that consists of the concrete plug, adjoining shotcrete, and structural fill buttress. These three components can be considered multiple defense mechanisms that will prevent tailings migration and minimize seepage into underground workings. The concrete plug is sufficient to seal the mine adits; however, the shotcrete will provide added strength to the surrounding rock and provide a hydraulic barrier for fluid migration. The structural fill buttress will serve as a long-term protective component for the concrete and shotcrete, and will act as a filter in preventing the migration of tailings from the tailings basin into a mine opening behind the plug and shotcrete. Post-construction field monitoring will need to be implemented in the aftermath of tailings deposition in order to measure the overall efficacy of the plug system.

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