

Design of a Centerline Method Tailings Dam using Mine Waste Rockfill in Perú

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ABSTRACT: The Constancia tailings facility, located in the Peruvian Andes, was designed to store more than 450 million tonnes of copper tailings in an area of relatively high seismicity and seasonally high rainfall. The tailings dam, currently more than 100 meters high, is constructed of compacted rockfill sourced from glacial outwash, quarried sandstone, and open-pit waste rock, and will ultimately be raised to a height greater than 170 meters. The dam comprises an initial downstream-method rockfill embankment with sloping upstream core and chimney drain, with subsequent conversion to a centerline-method, supported by an upstream rockfill platform to provide support to the vertical section of core and filter/drain zones. This paper provides a summary of the design analyses and describes observations made during construction and operation. Site-specific deterministic and probabilistic seismic hazard analyses, seepage, slope stability, and deformation analyses performed to predict the long-term engineering behavior of the dam and impounded tailings are described.

1 INTRODUCTION AND PROJECT SETTING

1.1 *Project Description*

The Constancia open-pit copper mine, owned and operated by Hudbay Perú SAC, is located in southern Perú, approximately 100 kilometers (km) south of Cusco, at elevations ranging from 3900 to 4500 meters above sea level (masl). The Constancia deposit is a porphyry copper-molybdenum system, emplaced in multiple phases of monzonites and monzonite porphyry. Milling rates are in the range of 80,000 to 90,000 tonnes per day (tpd). The original mine plan considered 525 million tonnes (Mt) of ore and 538 Mt of waste rock to be produced over a 17-year life of mine (additional reserves have since been added but are not considered in this paper). The process plant employs a conventional grinding and flotation circuit, from which tailings are pumped approximately 5.3 km through a high-density polyethylene tailings delivery line and spigotted into a partially-lined Tailings Management Facility (TMF). Figure 1 shows the layout of the TMF, processing plant, open pit, and other facilities at the site.

In the TMF, tailings are contained by an arcuately-shaped, cross-valley tailings dam with multiple small saddle dams, all constructed of rockfill, that will have a crest length of 4.1 km at design height. The TMF design was developed by Knight Piésold of Denver, Colorado, USA, to store approximately 290 million cubic meters of tailings at a maximum dam height of 170 meters (m), corresponding to a crest elevation of 4160 masl. Construction of the TMF was initiated in 2013, and the facility was placed into operations in late 2014, with full production achieved in early 2015. Since then, the dam has been incrementally raised to a height of 107 m as of December 2017. Prefeasibility designs have been prepared to raise the ultimate height to 200 m (crest elevation 4190 masl) to accommodate additional reserves, and final design studies for expansion are in progress.

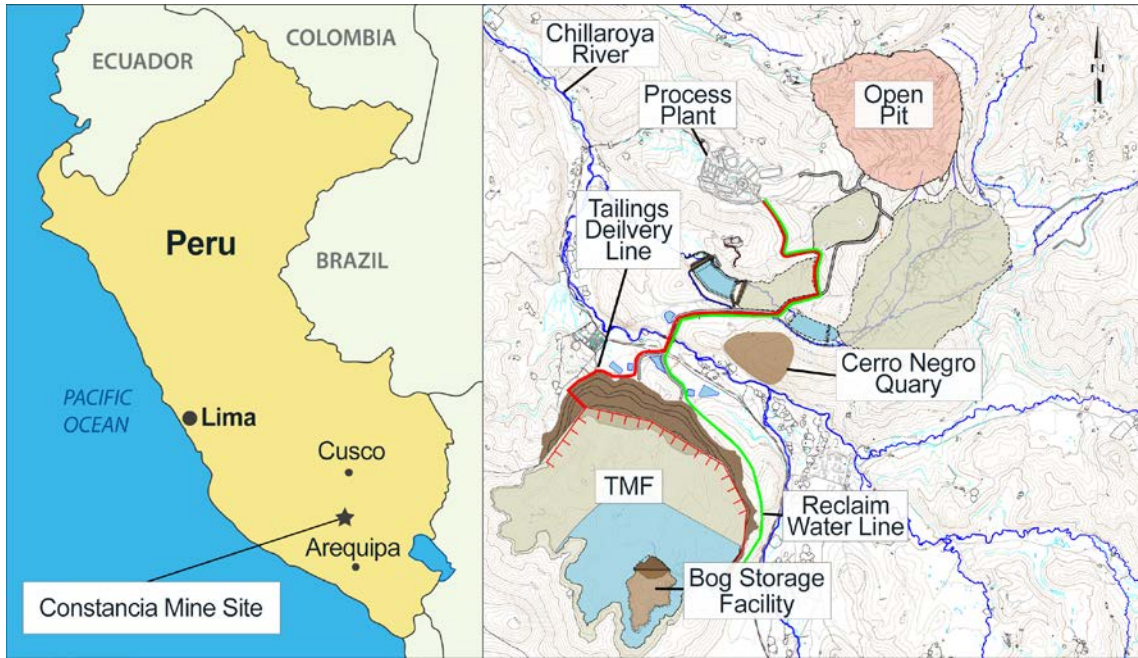


Figure 1. Location of site and general plan view on the Constancia Mine.



Figure 2. Aerial drone photograph of the Constancia TMF in June 2018.

1.2 *Geologic Setting*

The site is located within the eastern cordillera of the Andes mountains. Geomorphology is characterized by moderate relief, influenced by lithologic trends, structural features, and glacial erosion, with low rolling hills approximately 400 to 500 m high. Dissolution surfaces and karst are found where limestone dominates the rock mass but are absent at the TMF site. The Chillaroya river, a tributary of the Apurimac River within the Amazon watershed, flows from southeast to northwest just north of the TMF and captures drainage from the TMF and vicinity.

The oldest rocks in the area are sandstones and mudstones of the Lower Cretaceous-age Chillaroya Formation, overlain discordantly by limestones, calc-arenites, and conglomerates of the Arcurquina and Ferrobamba formations of Upper Cretaceous age. These formations were intruded by Oligocene-age dioritic/granodioritic plutons of the Andahuaylas-Yauri Batholith, and several monzonitic stocks and dikes intrude and cross-cut the sedimentary lithologies. The intrusives have been exposed to extensive weathering, leading to a deep oxidation profile.

Bedrock beneath the TMF comprises diorite/granodiorite of the Apurimac Batholith and sandstone of the Chillaroya formation. The degree of weathering is highly variable with compe-

tent rock exposed in some areas, but also with frequent exposure of moderate-to-soft rock overlain by low-to-moderately plastic soils. Diorite is exposed over most of the TMF basin and below the western portion of the TMF embankment, while sandstone is exposed in the northeastern portion of the basin and the east half of the dam. The diorite exhibits significant weathering to several meters depth beneath the west abutment of the dam, requiring deep stripping in some areas to reach competent rock for the dam foundation. Structural features in the TMF area correspond to stratification, faulting and fracturing systems of the sedimentary rock strata formed by the magmatic intrusions. Three fault systems were identified near the TMF, which were investigated by trenching and considered to be inactive in recent geologic time.

Quaternary deposits of alluvium and bog partially cover bedrock in the valley bottoms of the TMF area. The alluvium, which is typically non-plastic, was eroded from the weathered diorite, carried downslope and deposited in the valley bottoms. Bog deposits, consisting of organic-rich soils or peat, are several meters deep in many valleys. The bog deposits are highly compressible and have low strength, so the bog and loose alluvium deposits were removed from under the embankment and geomembrane-lined areas and placed into designated bog disposal areas.

1.3 Climate and Hydrologic Setting

Climate at the site is humid and seasonably cool with well-defined rainy and dry seasons. Average daily maximum temperatures range from 13 to 16 degrees Celsius and average daily minimum temperatures range from -11 to 0 degrees Celsius. Virtually all precipitation occurs during the wet season, which is defined from October to April of each year. Climatological data for use in the designs were based on data from multiple climatological stations located in the region, supplemented by data from a single on-site station, and a regression analysis was used to correlate on-site precipitation data with outlying stations. Estimates for the design 24-hour storm events were calculated using a theoretical Extreme Type I Gumbel probability distribution on precipitation data from approximately 48 years of historical records. The probable maximum precipitation (PMP) was estimated according to the World Meteorological Organization (WMO, 1973) methodology for all stations with 24-hour maximum precipitation records greater than 10 years. The resulting design data and parameters are provided in Table 1.

Table 1. Summary of Climatological and Hydrological Design Data

Description	Design Value	Description	Design Value
Annual average precipitation	1,000 mm	Annual maximum precipitation	1,353 mm
Annual minimum precipitation	590 mm	Average wet season precipitation	932 mm
Average dry season precipitation	72 mm	100-year/24-hour storm precipitation	92 mm
PMP/24-hour storm precipitation	263 mm	Potential annual evaporation	961 mm
Annual evaporation from natural ground	480 mm	Evaporation from dry tailing	455 mm
Evaporation from wet tailing	865 mm	Evaporation from waste area	296 mm
Evaporation from water surfaces	673 mm		

1.4 Seismicity and Seismic Hazard

The site is located within a seismically-active region of Perú, where the Nazca Plate subducts under the South American Plate at a rate of about 80 millimeters/year. The region experiences earthquakes associated with the subduction zone and crustal faulting within the continental intraplate zone. Deterministic seismic hazard analyses and probabilistic seismic hazard analyses (PSHA) were performed originally by Alva (2013) and updated by GeoPentech (2018). Maximum credible earthquake (MCE) scenarios were developed for each of three categories of seismic sources: subduction interface (SDI), deep intraslab (DIS), and shallow crustal (SCS). MCE scenarios and resulting peak horizontal ground accelerations for 84th-percentile ground motions are summarized in Table 2. The PSHA results were used to select operating basis earthquake ground motions for the TMF and maximum design earthquakes for non-critical structures, and to inform the deterministic analyses. The MCE response spectrum for the DIS and SCS scenarios fell mostly between the uniform hazard spectra for 2475-year and 5000-year probabilistic ground motions. The 10,000 peak ground acceleration (PGA) is about 0.8g, and the return period for PGA of 0.6g (corresponding to the MCE) is approximately 2475 years.

Table 2. MCE Scenarios

Source Category	Moment Magnitude	Distance (km)	PGA	Ground Motion Models used
Subduction Interface	9.0	250	0.09g	Abrahamson et al. (2016), Zhao et al. (2006), Atkinson & Boore (2003, 2008)
Deep Intraslab	7.75	100	0.59g	Abrahamson et al. (2016), Zhao et al. (2006)
Shallow Crustal	6.75	1.5	0.62g	NGA-West2 (Boore et al. 2014)

2 EMBANKMENT DESIGN

2.1 *Design Criteria*

Design criteria, developed jointly by Hudbay and Knight Piésold, generally followed the Canadian Dam Association (CDA) dam safety guidelines, with consideration to other international guidelines. Dam consequence classification of Very High was considered realistic for this facility, considering the relatively low population at risk and the lack of critical infrastructure in the downstream area, but the more conservative Extreme consequence category was used for design, given the height of the dam, the perceived cultural and environmental value of the surrounding area, and the potential for reputational impact on the owner. The design earthquake and design storm deserve special discussions provided in this section, as follows.

Selection of the design earthquake was guided by CDA (2007, 2013), ANCOLD (2012), ICOLD (2010, 2016), and the experience of the designers and Independent Professional Review Board (IPRB). For extreme hazard dams, CDA (2013) recommends the MCE or 10,000-year earthquake. ANCOLD (2012) recommends the 10,000-year earthquake during operations and the MCE for post-closure. ICOLD (2016) recommends either the MCE or an earthquake with a “long return period, for example 10,000 years” and that “deterministically-evaluated earthquakes may be more appropriate in locations with relatively frequent earthquakes that occur on well-identified sources, for example near plate boundaries.” Because the Constancia site is in an area of frequent seismic activity with relatively short recurrence intervals between large magnitude events, the PGA estimated for a 10,000-year earthquake exceeds the PGA estimated for the 84th-percentile MCE, although the MCE ground motions at longer periods for crustal sources are higher than the 10,000-year uniform hazard spectrum. The 84th percentile MCE was ultimately selected as the appropriate design earthquake.

Once the dam reaches final height, a permanent spillway designed for the PMP will be constructed into natural ground near the left abutment. However, during operations, the TMF will have no spillway. As a result, the design storm criteria during operations were developed using a probabilistic water balance to maintain sufficient volume above the tailings surface to contain the volume of water corresponding to an annual exceedance probability (AEP) of 0.1% (i.e., 1:1000 years), which is consistent with recommendations for extreme-consequence, non-release dams described in ANCOLD (2012). The raise sequence of the dam was planned to maintain a minimum of 2 m of freeboard above the 0.1% water surface elevation at all times. Further discussion of the water balance method and application are provided in Section 2.6.

2.2 *Geometric Design and Embankment Zonation*

Plan view of the TMF is shown in Figure 1. The main dam crosses two valleys separated by a central ridge, creating separate basins in the initial years of operation. A repository for bog and other unsuitable material removed from the valleys within the TMF footprint during construction is in the southern end of the east basin and will eventually be inundated by water and tailings. The reclaim pond is maintained at the southern end of the east basin, which is fully geomembrane-lined. The geomembrane liner is a 2-mm linear low-density polyethylene (LLDPE) with double-sided texturing. Only a portion of the west basin is lined.

The maximum section, depicted in Figure 3, is in the west valley. The embankment is comprised of the zones and materials summarized in Table 3. The width of the core (Zone A) ranges from a maximum of about 24 m at the base of the maximum section to 4 m wide in the vertical section (i.e., centerline-raised portion). Zone B (filter/drain) is 2 m wide for the entire dam height was designed to meet both piping criteria (to filter Zone A) and permeability criteria.

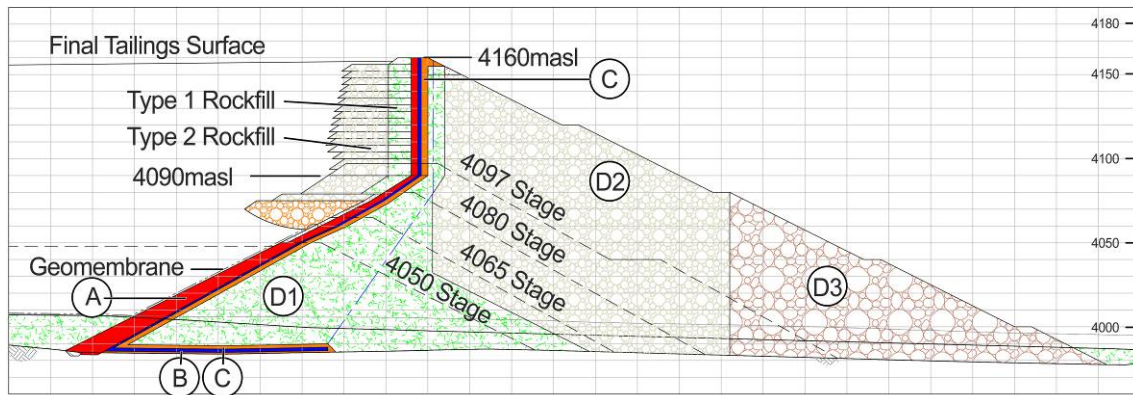


Figure 3. Typical cross-section of TMF embankment at maximum section.

Zone C (transition/coarse filter) is 4 m wide for the entire dam height, and its gradation was designed to meet only piping criteria (to filter Zone B material). A drainage blanket of Zone B/Zone C material was placed beneath the upstream portion of the dam to collect upward seepage from the foundation and seepage through the core. This drainage blanket directs seepage toward a central collection drain of coarse gravel in the bottom of the two main valleys, which empty into two sump, from which water is pumped and returned to the process circuit.

The downstream shell comprises three zones: D1, D2, and D3. Zone D1 was used to construct the starter dam and provides transition between Zone C and Zone D2 in the vertical core section. The maximum allowable fines content of 20% for Zone D1 is higher than normal for good-quality rockfill but was allowed since Zone D1 is well-compacted and located deep in the dam's interior. Most of the material placed in this zone had fines content of less than 15% based on quality control testing. Zone D2 comprises the greatest percentage of dam volume and is described in the following paragraphs. To date, no Zone D3 material has been placed.

Initially, Zone D2 material was sourced from a sandstone quarry (Cerro Negro), hauled in 40-tonne trucks and compacted with 19-tonne rollers. Then, once open-pit mining started, Zone D2 material was sourced from non-acid-generating waste rock—mostly unaltered limestone/sandstone—and delivered and compacted with 240-tonne mine trucks. Extensive testing has been performed on the Zone D2 material during construction and operation, including 1-m diameter triaxial tests and companion six-inch diameter triaxial tests.

A key feature of the design is the upstream rockfill zone (Type 1 and Type 2 Rockfill), which provides support for the vertical core section. A crest detail showing the upstream rockfill zone is shown in Figure 4, and a photograph of construction is provided in Figure 5. The initial lifts of this zone—referred to as the “Initial Working Platform”—were placed over tailings, but once the initial working platform was built, subsequent lifts are placed on compacted rockfill, using 0.5-m to 1.0-m lift thicknesses, with compaction provided either by a 190-tonne roller or loaded haul trucks. Thus, the design concept and construction sequence produce a compacted rockfill zone that is fully supported on the upstream slope of the downstream-method starter dam. By starting the Initial Working Platform construction when the tailings surface is about 20 m below the conversion from downstream construction to centerline construction, the initial rockfill lifts displace into the soft tailings and form a stable working surface for subsequent lifts.

2.3 Material Characterization

Material properties were developed during design based on field and laboratory investigations of materials deemed representative at the time. However, borrow-source investigations were limited due to access restrictions related to land ownership during project development. As a result, the Observational Method (Peck, 1969) was applied, and supplemental investigations and laboratory testing were performed during construction and operation to confirm or update the design parameters, based on actual materials. Table 4 presents selected geotechnical parameters for the primary materials of interest. The following sections provide discussion of the embankment materials and tailings material characterizations.

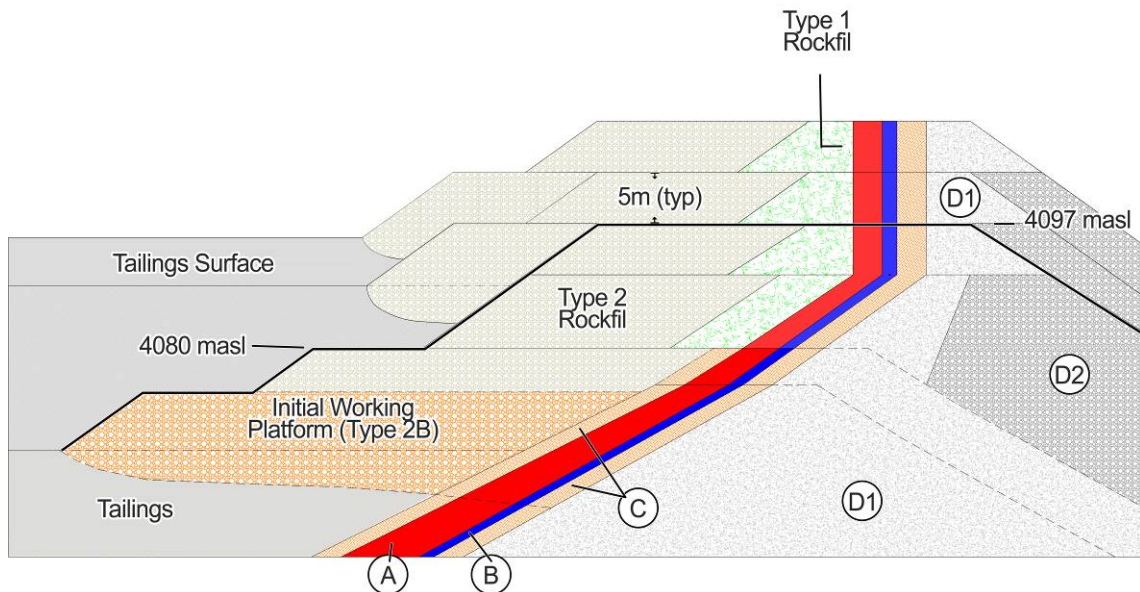


Figure 4: Crest detail showing upstream rockfill platform.

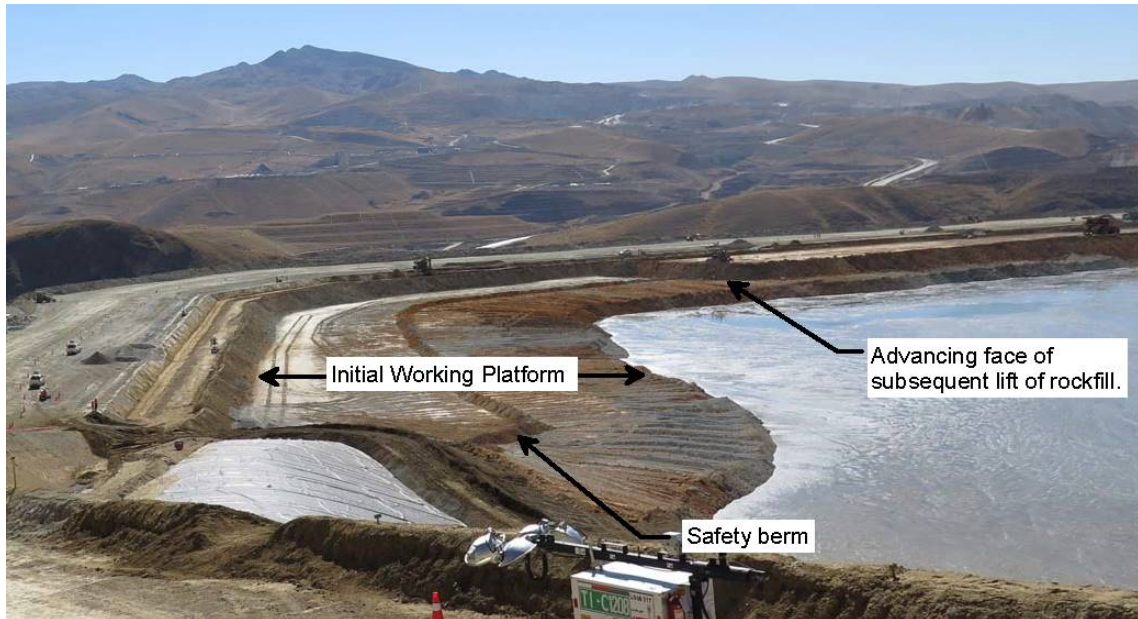


Figure 5: Construction of initial working platform.

2.3.1 Embankment Materials

The stability and performance of the TMF dam is largely dependent on the Zone D2 and Zone D3 materials, which comprise about 70 percent of the embankment volume, and to a lesser degree on the Zone D1 materials. By nature of the geologic development of the ore body, the waste rock is highly variable, based on the lithology and type of thermal alteration. The primary rock types to be used in dam construction are unaltered sedimentary formations (limestone/sandstone) and monzonite. The monzonite materials are altered as typical to porphyry development—argillic, phyllic, propylitic, potassic, and silicic—although not all alteration types are suitable for dam construction, depending on their potential for acid generation (the design considered that only non-acid generating materials would be placed downstream of the core).

Most of the Zone D2 material to date was sourced from fresh limestone waste rock, sandstone from the Cerro Negro quarry, and a small amount of argillically-altered monzonite. A laboratory testing program on samples of the Zone D2 materials selected during construction was performed, comprising large-scale (1000-mm diameter) triaxial testing performed at the IDIEM laboratory at the University of Chile in Santiago and medium-scale (150-mm diameter) triaxial testing at the Knight Piésold laboratory in Denver, Colorado. Samples were prepared using the

parallel gradation technique described by Lowe (1964) and by Frossard et al. (2012) to match representative gradations measured on in-place materials (associated with large-diameter water replacement density tests). Extensive characterization of index and engineering properties were performed on each sample.

Table 3. Embankment Zone Materials and Specifications.

Zone	Description	Borrow Source	Max. Particle Size	Fines* Content	Loose Lift Thickness	Compaction
A	Clay Core	Glacial Till/Outwash	6 inch (152 mm)	25-85%	200 mm (8 inch)	95% of ASTM D 1557
B	Fine Filter/Drain	Wash plant	3 inch (76 mm)	0-5%**	500 mm (19 inch)	4 passes of roller or tamped
C	Coarse Filter/Transition	Screened quarried rock	4 inch (102 mm)	0-10%†	500 mm (19 inch)	95% of ASTM D 1557
D1	Structural Fill	Glacial Outwash, Quarry	12 inch (300 mm)	0-20%	500 mm (19 inch)	95% of ASTM D 1557
D2	Structural Rockfill	Quarry or Waste Rock	24 inch (600 mm)	0-8%††	1000 mm (39 inch)	Method specification/test fills
D3	Thick-Lift Structural Rockfill	Waste Rock	1200 mm	0-5%	2000 mm (68 inch)	Method specification/test fills
Type 1, 1A	Upstream Rockfill Platform	Waste Rock	24 inch (600 mm)	0-20%	500 mm (19 inch)	Method specification
Type 2, 2A, 2B	Upstream Rockfill Platform	Waste Rock	24 inch (600 mm)	0-10%†	1000 mm (39 inch)	Method specification/test fills

*Fines content defined as percent passing 0.075 mm (No. 200 sieve)

**Zone B: <10% of tests within 30-test running average allowed to have fines content between 5%-8%.

†Zone C: <10% of tests within a 30-test running average allowed to have fines content between 10-15%. Within the downstream-raised section, Zone C was allowed to have up to 20% fines content.

††Zone D2: <10% of tests within 30-test running average allowed with fines content between 8-12%.

Laboratory testing results on the limestone waste rock material are shown in Figure 6 in the stress-dependent framework of Leps (1970). The triangular symbols represent peak shear strengths for specimens prepared to relatively high density (relative density, D_R , of approximately 75%) with a relatively coarse gradation (less than 10% percent passing No. 200 sieve in field-scale samples). The blue best-fit line in Figure 6 passing through the triangle symbols is parallel to the Leps (1970) lines and slightly higher than the “Low Leps” relationship, supporting the use of the Low Leps relationship in design. The diamond symbols represent “lower-quality” rockfill specimens, which were prepared either to a relatively low density ($D_R \approx 33\%$) and with a coarse gradation, or to a relatively high density ($D_R \approx 75\%$) with a finer gradation (approximately 20% passing No. 200 sieve). The resulting best-fit line is approximately 3 degrees lower than the “higher quality” material, but higher than the “lower limit” proposed by Linero et al. (2007).

The lower-strength relationship was used in optimization studies to evaluate the potential for increasing lift thickness, which lowers the relative density, or to consider uses for other lower-quality rockfill scenarios. All medium-scale results presented in Figure 6 were adjusted for maximum particle size based on the Frossard et al. (2012) method. The comparison between the large-scale results from IDIEM was favorable to the corresponding size-adjusted medium-scale results, which validated the use of the Frossard method.

Conventional laboratory testing was also performed on the core, filter/drain, and transition materials. Table 4 provides a summary of the most relevant material properties for each embankment materials used in the various geotechnical analyses described in Section 2.4.

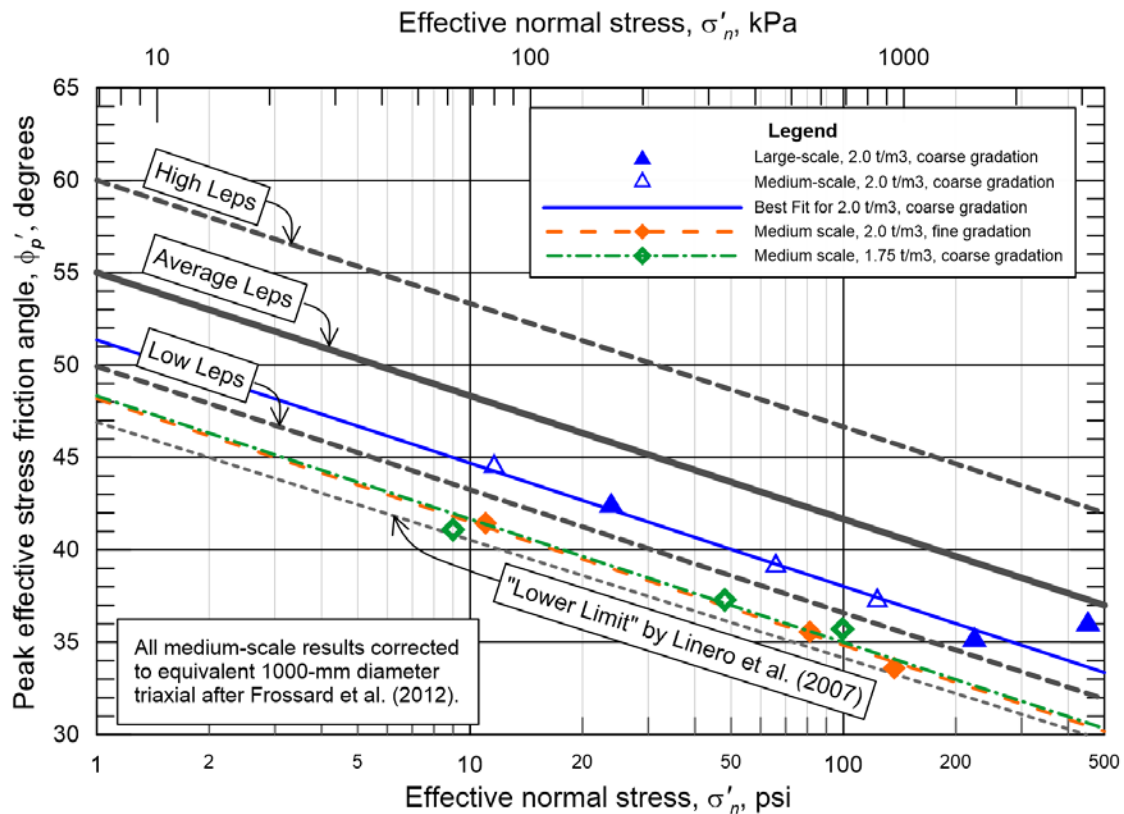


Figure 6. Shear strength characterization of the Zone D2 rockfill material.

2.3.2 Tailings

Two prototype tailings samples were selected during design from bench-scale metallurgical testing of core samples and were subjected to index property tests, triaxial strength and permeability tests, settling and sedimentation tests, and seepage-induced consolidation tests (SICT). These idealized materials exhibited 100% by weight passing the No. 50 sieve and 70% passing the No. 200 sieve. During operations, index property testing has been regularly performed on whole tailings samples from the tailings thickener underflow, which demonstrate a range of 88% to 100% passing No. 50 sieve and 50% to 80% passing No. 200 sieve, indicating the prototype materials were generally on the fine side of the range of tailings produced by the milling operation.

The key parameters for tailings characterization are the drained shear strength, undrained yield shear strength, liquefaction susceptibility, residual (post-liquefaction) undrained shear strength, permeability, and in situ density. Strength and permeability characteristics used in design are summarized in Table 4. The tailings were assumed to completely liquefy during the design earthquake. The relationship between in situ density and effective stress was developed using CONDES software (Gjerapic and Znidarcic, 2007), which was used to forecast the average tailings density over time, ranging from 1.2 tonnes/cubic meter (t/m³) after one year to 1.55 t/m³ after 16.5 years. Bathymetric surveying results over the first 3 years of operations indicate the actual average density is approximately 1.39 t/m³, just slightly lower than the value of 1.42 t/m³ indicated by the consolidation model, but within the usual range of uncertainty.

A confirmatory laboratory and field investigation was performed to characterize the tailings after about 3 years of operations. The field investigation relied mostly on cone penetration testing (CPT), supplemented with vane shear and limited drilling and sampling. Interpretation of the CPT data supported the assumed yield undrained shear strength ratio of 0.22 and supported raising the residual (post-liquefaction) undrained shear strength ratio from 0.04 to 0.05.

2.4 Geotechnical Analyses

Geotechnical analyses to support the design included two-dimensional seepage analyses using SEEP/W, limit-equilibrium slope-stability analyses using SLOPE/W, simplified dynamic-

deformation analyses, and advanced dynamic analyses using the FLAC software. Key material properties used in the analyses are summarized in Table 4.

Table 4. Selected Material Properties for Geotechnical Analyses

Material	Unit Weight (kN/m ³)	Permeability (cm/s)	Porosity	Shear Strength Parameters	Constitutive Model in FLAC	G _{max} @ 1atm (MPa)
Tailings	1240	3x10 ⁻⁶	0.549	$\phi' = 29^\circ, c = 0$ $s_u / \sigma'_v(\text{yield}) = 0.22$ $s_u / \sigma'_v(\text{liq}) = 0.04$	UBCSand*	49
Zone A	2160	4x10 ⁻⁷	0.212	$\phi' = 25^\circ, c = 0$	UBCHyst**	120
Zone B	2060	5x10 ⁻²	0.228	$\phi' = 38^\circ, c = 0$	UBCHyst	242
Zone C	2160		0.193	Leps High	UBCHyst	346
Zone D1	2160	1x10 ⁻⁴	0.193	Leps High	UBCHyst	346
Zone D2	2160	1x10 ⁻¹	0.193	Leps Average	UBCHyst	242
Zone D3	2160	1x10 ⁻¹	0.193	Leps Low	UBCHyst	242
Type 1/2 Rockfill	2160	1x10 ⁻¹	0.193	Leps Average	UBCHyst	242
Alluvium	1340	1x10 ⁻⁴	0.504	$\phi' = 29^\circ, c = 0$	Mohr-Coulomb	77
Foundation (weathered rock)	2630	1x10 ⁻⁴	0.100	$\phi' = 36^\circ, c = 0$	Mohr-Coulomb	650
Foundation (sound rock)	2630	1x10 ⁻⁴	0.100	Not Applicable	Elastic	650

*Beatty & Byrne (1998), **Naasgaard (2011)

2.4.1 Seepage and Stability Analyses

In the seepage analyses, boundary conditions for intermediate height cases were conservatively applied to represent a fully-submerged condition for the entire tailings impoundment (i.e., no exposed beach), with total head values set to the water elevations predicted by the water balance for flood conditions. For final height (end of operations), the edge of the saturated zone was set 250 m upstream from the crest, representing a minimum pond setback requirement to be incorporated at closure. The geomembrane liner on the upstream face of the dam and in the bottom of the tailings basin was conservatively omitted to allow flow through the foundation and the low-permeability core. The outlet of the drain layer into the central drain (i.e., flow perpendicular to the plane of the mesh) was ignored, another conservative assumption. A drainage sump located about 20 m beyond the ultimate downstream toe was modeled as a zero-pressure head. Review nodes were applied along the crest, downstream slope, and downstream toe area of the TMF embankment, allowing the model to iterate and estimate a conservative phreatic surface within the downstream shell, which is shown on Figure 7.

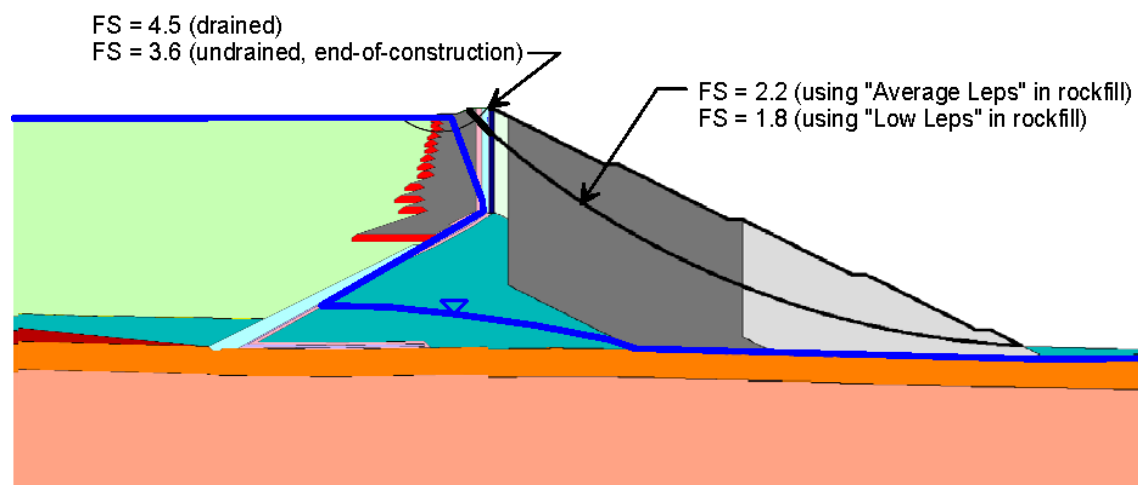


Figure 7. Critical shear surfaces and factors of safety from limit-equilibrium analyses.

Limit-equilibrium slope stability analyses were performed according to the Spencer (1967) method of slices. Model geometry and pore pressure conditions were imported from the SEEP/W output into SLOPE/W for the slope stability analyses. Minimum acceptable factor of safety (FS) values, based on CDA (2013), were 1.5 for long-term loading conditions, 1.3 for the end-of-construction conditions, and 1.2 for post-earthquake loading conditions. For long-term (steady-state) conditions, the minimum FS of 1.5 was applied to a conservative strength characterization in the rockfill (Low Leps relationship) and a minimum FS of 1.8 was used for "average" strength characterization. Figure 7 illustrates the calculated factors of safety for the downstream slope along with the phreatic surface predicted by seepage analyses. The resulting calculated factors of safety are well above the minima, allowing for future optimization of the design.

For design of the Initial Working Platform, an end-of-construction analysis was used to estimate the penetration of rockfill into soft tailings by varying the thickness of the platform to obtain a computed FS = 1.0 at the upstream edge while using undrained strengths in the tailings. The width of setback for additional lifts was then varied to achieve a minimum FS = 1.3 for a "global" failure mode on the upstream slope of the platform. A "safe operating zone" for the loaded haul trucks was established by applying a concentrated load to the platform to represent the truck weight and varying the distance from the upstream edge to achieve the minimum FS.

Simplified deformation analyses were performed using Makdisi and Seed (1978), Bray and Travasarou (2007), and Swaisgood (2014) to confirm the design would perform satisfactorily during the design earthquake. The results were compared to the numerical analyses described in the following section.

2.4.2 Dynamic Deformation Analyses

Seismic deformation analyses of the dam were performed using the explicit finite-difference software FLAC 8.0. The model mesh was set up to consider eight construction stages. These stages were subjected to gravity (static) loading incrementally, which allowed stresses and deformations to come to equilibrium after each stage was added. A seepage analysis was performed for each stage with the objective of obtaining a pore pressure distribution similar to the pore pressure distribution estimated by the SEEP/W model. Material properties used in the analyses are summarized in Table 4.

Seven spectrally-matched acceleration time histories were developed for use in the seismic deformation analysis and applied to the base of the model. The largest amount of deformation was obtained with SCS time history based on the 2009 L'Aquila earthquake in Italy. Displacements induced by this controlling MCE ground motion are presented in Figure 8. The FLAC model indicated a maximum total displacement of 1 m and maximum vertical displacement of 0.9m, which is caused primarily by an upstream rotation of the top portion of the upstream rockfill platform into the liquefied tailings. The downstream shell exhibits a slight downstream horizontal movement with very little to no vertical deformation. The model results indicate no significant damage to the core and filter, and no significant loss of freeboard during the MCE.

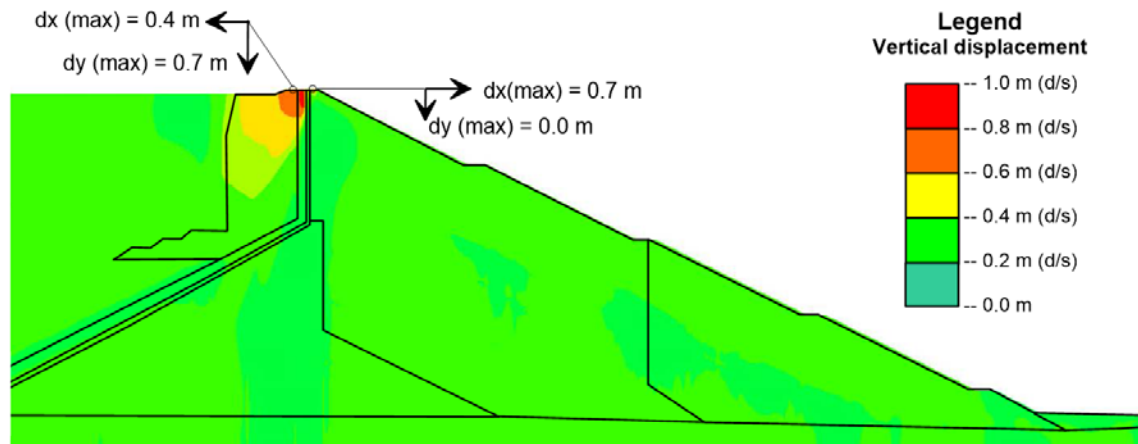


Figure 8. Results of FLAC dynamic deformation analyses for the controlling MCE

3 WATER BALANCE ANALYSES

Probabilistic water balance analyses, generally following the approach described in Truby et al. (2010), were developed cooperatively between Knight Piésold and Hudbay. The site-wide water balance model applies the 48-year precipitation record using to the Indexed Sequential Method (House & Ungvari, 1983) through a customized GoldSim® model that considers the complex interactions among the natural hydrologic conditions and the various systems in operation at the mine. Statistical analysis is performed on the multiple outcomes to estimate the water volumes corresponding to various AEPs. The construction sequence for raising the dam is planned to maintain at all times an empty volume that could safely store the 0.1 percent AEP water volume, plus an additional 2 m of freeboard. The water balance is updated by Hudbay technical services staff on at least a monthly basis to provide ongoing guidance for operation of various pumping systems, when to discharge and when to collect (harvest) water from runoff or pit dewatering efforts. Figure 9 shows historical and predicted volumes corresponding to the crest elevation, tailings surface elevation, and water surface elevations for average, 1 percent AEP and 0.1 percent AEP conditions.

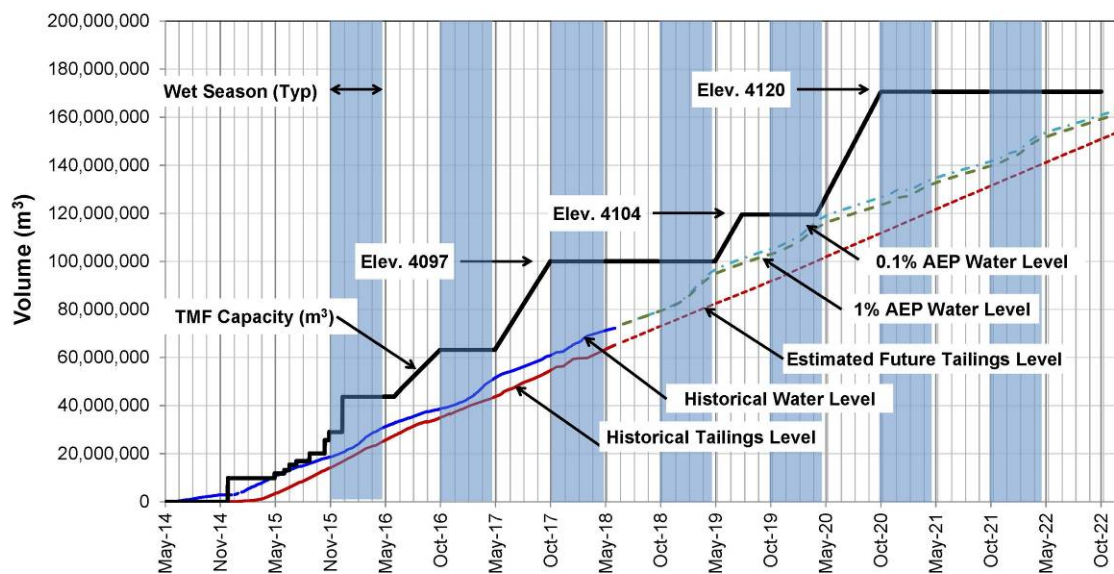


Figure 9. Probabilistic water balance projections.

4 SUMMARY

The Constancia tailings dam is a successful application of the Observational Method resulting in a robust, cost-effective design that has overcome several technical challenges. The use of compacted mine waste rock to construct to significant height a dam inherently resistant to liquefaction and stable under the expected high earthquake load is a best practice that is perhaps underappreciated with the modern emphasis on reduction of risk by new technologies such as filtering. The design philosophy of making appropriately conservative assumptions using relatively limited data, combined with reasonably conservative design criteria (such as factors of safety higher than the minimum required) provides flexibility in the design that allows optimization based on actual conditions realized during construction and operation. Regular involvement of an experienced Engineer of Record in construction and operations, combined with the oversight of an independent review board, reduces risk associated with dam stability and safe operation and maximizes the investment made by mine owners. Integration of multiple disciplines—mine planning, construction services, tailings operation, water balance, and geotechnical engineering, and environmental/permitting services—has made the Constancia project an exceptional case history.

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ACKNOWLEDGEMENTS

The authors recognize the contributions of the IPRB: Dr. Leslie Smith, Dr. Iain Bruce, Mr. James Swaisgood, and (previously) Dr. Gabriel Fernandez. The authors also thank Highbay’s management, including Mr. Alan Hair, Mr. Cashel Meagher, and Sr. Javier del Rio, who have supported the technical excellence making this world-class facility possible.