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EXTRACTION AND PRIMARY PROCESSING  
OF GOLD-BEARING ORE FROM THE  
ROZINO DEPOSIT

# Report on integrated and sustainable water management.

Issued for:  
Tintyava Exploration AD  
Design order DS 25-01 to the framework agreement of 20 February 2025.

REPORT



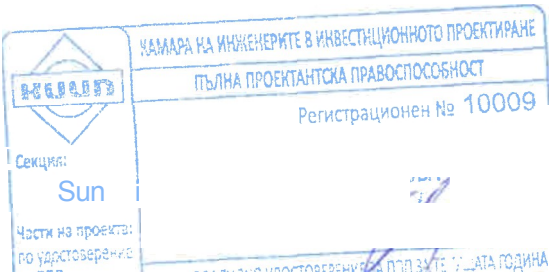
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## 1.0 SUMMARY

Proektiraneto i Analizi EOOD has been commissioned by Tintyava Exploration AD to prepare a report on sustainable water management based on the parameters of the submitted investment proposal for the project: "Extraction and primary processing of gold-bearing ores from the Rozino deposit".

The objectives of the study are to present in detail the adopted water management strategy for the site based on best mining practices and the trend towards closed-loop water management and minimisation of direct emissions (loads) to the environment, using a sustainable "zero discharge" approach to contact water.

Water management for the site has been carefully planned and designed, with a comprehensive strategy based on the principle of sustainability and minimisation of the impact of mining activities on both groundwater and surface water. Protecting the country's water resources is also an important part of the strategy. As far as possible, the project aims to completely close the cycle of water affected by the mine for internal use under normal operating conditions.

At this stage of the project, it is not possible to examine the water quality at the site in detail, which is why an integrated approach to water management within the production site has been formulated. This approach is based on the division of surface water into two streams: water affected by mining activities (water from production cycles, tailings storage facilities, ore pits and open-pit mines) and water that has not been in contact with production (surface water, rainwater, runoff water, water from undisturbed forest areas). This functionality allows for sustainable water management by minimising the amount of contact water in circulation at any given moment during operation. The main challenge for operation will be to find a balance between water surplus and shortage, especially considering that the annual evaporation rate exceeds the annual precipitation rate.

The water management strategy is based on experience from previous projects, the requirements of the BD and the requirements of Tintyava Exploration AD for continuous improvement. The integrated approach adopts the principles of minimising wastewater quantities, for which purpose a strategy has been developed for the minimal use of fresh water and its maximum reuse within the production site.

It is important to understand and assess the risk, as we cannot prevent extreme rainfall events from occurring. Generally speaking, during extreme rainfall (or rather a combination or sequence of rainfall events), there is a possibility that the level of contact water storage facilities will rise. In this regard, the results of the specific scenarios described in detail in the report show that the design maximum water level (forced water level) in the tailings pond's settling pond, corresponding to a rainfall event with the relevant regulatory assurance under Bulgarian legislation of 0.1% (1 in 1000 years), remains below the crest wall level. The free height varies during the period of operation from 0.37 m at the beginning to 0.54 m at the end of operation. This means that **there will be no** overflow of water.

In addition, the report discusses measures for managing the filtration flow under the facilities, so that the infiltrated contact water is captured and returned to circulation, while at the same time an injection curtain is constructed to separate contact from non-contact groundwater. Thanks to this barrier at depth, the mixing of the two flows and the spread of contact water filtering downstream along the Yuren Dere stream is prevented.

The integrated approach to water management in the development of the Water Management Strategy for the Project: Extraction and primary processing of gold-bearing ores from the Rosino deposit is a sustainable approach that uses the best mining practices for water management. This study demonstrates the "**zero** discharge" approach to surface and groundwater, which is necessary for the project to be approved by the Competent Authority.

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Enhanced evaporation facility. Example supplier

### ANNEX C

Data on annual precipitation amounts – primary data for a 46-year period

## 2.0 INTRODUCTION

Tintyava Exploration AD is developing a facility for the extraction and primary processing of gold-bearing ores from the Rozino deposit.

### 2.1 Description of the site

The Rosino deposit is part of the Tintyava area, located in the municipalities of Ivaylovgrad and Krumovgrad, south-eastern Bulgaria, about 350 km (by road) east-southeast of the capital Sofia. To the east and south is the border with Greece, and to the north and west are the municipalities of Lyubimets, Madzharovo and Krumovgrad. The project is located about 2 km south of the village of Rozino and 85 km southeast of the town of Kardzhali.

Mining and processing activities include the operation of an open pit mine, a crushing plant with conventional crushing, grinding and flotation processing to obtain gold concentrate. The location of the tailings storage facility has been chosen to minimise the footprint of the facilities and to be close to the production plant and open pit mine in order to reduce tailings pumping and waste transport costs. The method of tailings placement after thickening was chosen as the most cost-effective solution compared to other options, such as paste or dry cake. The tailings storage facility is of the upstream (upgrading) type. This solution provides greater structural safety compared to other superimposition methods such as upstream or central construction.

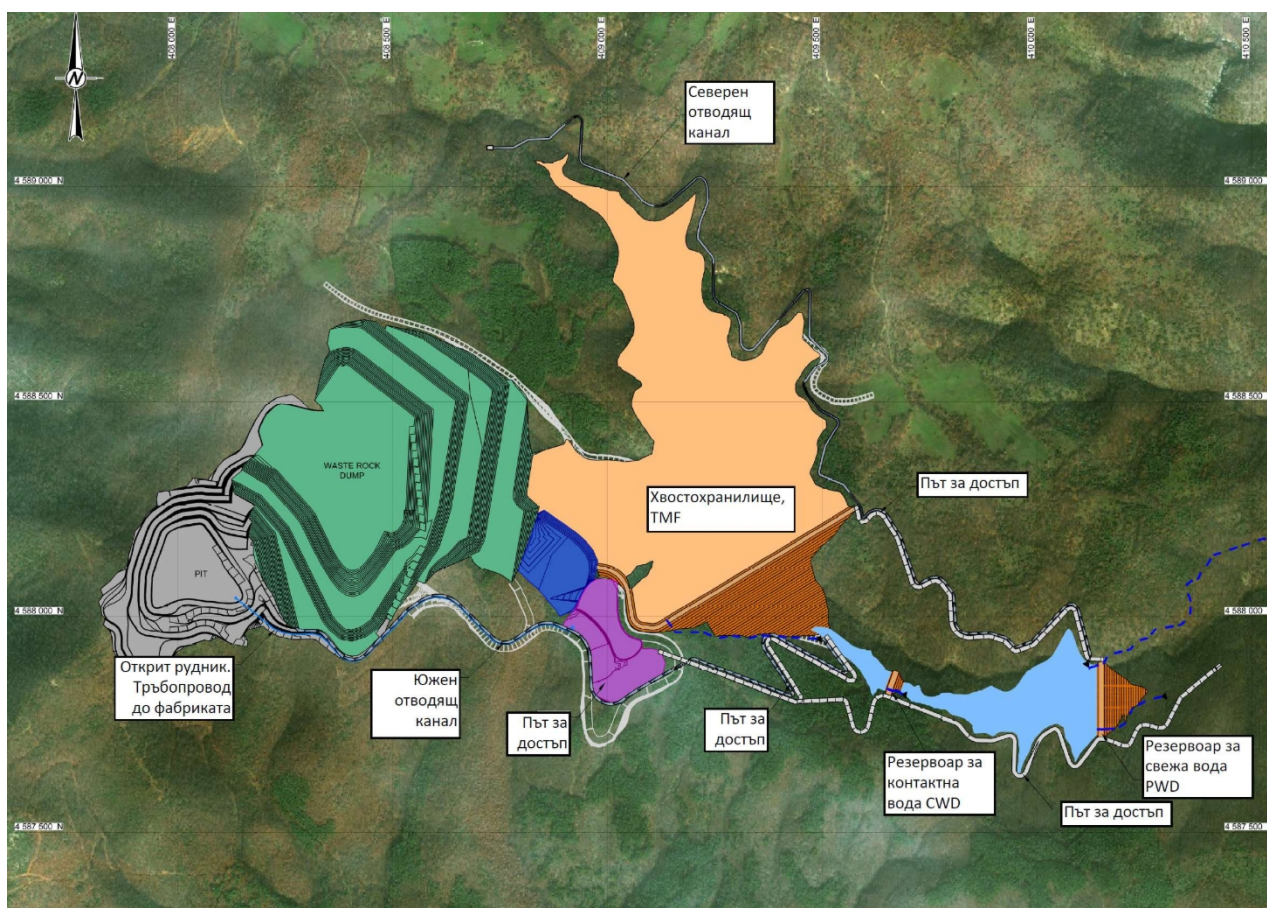


Figure 1: General plan of the Rosino site

The figure above shows the selected layout of the site, showing the selected configuration consisting of a thick tailings facility (TMF), a contact water dam (CWD), a non-contact water dam (RWD), road connections to the open pit mine and other mining infrastructure.

## 2.2 Main objectives and results achieved

This report describes the processes related to water management for the site. Water management facilities, practices, plans and projects are described in the context of the integrated water balance for the entire mining site, as well as for the entire planned operating period, taking into account the climatic characteristics of the area.

The preliminary sizing of key surface water infrastructure was performed based on the results of a hydrological assessment study, as well as reports from past studies, mine plans and topographical data provided by the Client. The preliminary analysis includes a design for a northern and southern diversion channel, emergency spillways for the three water storage facilities (tailings pond, contact water reservoir and fresh water reservoir) and other auxiliary structures (e.g. collection shafts, pipe drains, energy dissipators, etc.) required during the mine's operation. A separate preliminary design for the organisation of surface drainage has been completed for the mine closure period.

Water balance modelling shows that approximately 300,000 m<sup>3</sup> of active volume will be required for the fresh water reservoir, and to achieve this, the reservoir has been designed with a total allowable volume of 365,000 m<sup>3</sup>. This ensures that there will be no water shortage at the plant. Modelling shows that a water import of 50 l/s for 5 months of the year is sufficient to supply the project based on this water volume.

The water balance analysis shows that approximately 14,000 m<sup>3</sup> of active volume is required to implement a contact water reservoir, which is why the reservoir is designed with a total volume of 25,000 m<sup>3</sup>, which is equivalent to approximately 7 days of storage at maximum pump flow in the reservoir.

The projected peak volume of water entering the tailings pond (TMF) is approximately 50,000 m<sup>3</sup> for a dry year scenario, 115,000 m<sup>3</sup> for an average year scenario and 220,000 m<sup>3</sup> for a wet year scenario. The tailings pond is designed to collect this water and ensure its reliable inclusion in the production plant's cycle. The most important and significant result of the water balance modelling, along with strategic water management, is that the design maximum water level in the water storage reservoir remains below its overflow edge throughout the entire life of the deposit. This means that there will be no overflow from the reservoir even in the event of an extreme water event.

## 3.0 CLIMATIC AND GEOGRAPHICAL INDICATORS

The project site is located in the municipality of Ivaylovgrad, Haskovo region, southern Bulgaria. The site is immediately south of the village of Rozino. The Biala River and the Luda River and its tributaries are the main water sources in the area, with the site located between two tributaries of the Biala River (Yuren Dere and Arpa Dere, see Figure 2). It is located on the Biala River, part of the Maritsa River basin, a tributary of the Luda River. The Byala River drains the Măglănik, Irintepe and Sirt ridges of the Eastern Rhodopes and has a total catchment area of 594 km<sup>2</sup>. The river's outflow at the village of Doluno Lukovo is approximately 7.53 m<sup>3</sup>/s and is mainly fed by rainfall. Many of the surface water bodies are ephemeral with no flow during the dry season (dry streams).

### 3.1 Description of the terrain

The site is located in an area of low mountains and flat hills, cut through by steep valleys with an average altitude of 320 m. The project area is bounded to the south by the steep Tashlaka cliffs, which border a river valley fault.

Several small villages and farms are located near the site, with agriculture and farming being the main activities, mainly livestock and tobacco cultivation. The State Forestry Enterprise is also of economic importance. The forests are mainly broadleaf, but there are also coniferous forest areas. The soils are poorly developed and are mainly slightly leached, dark brown forest soils.



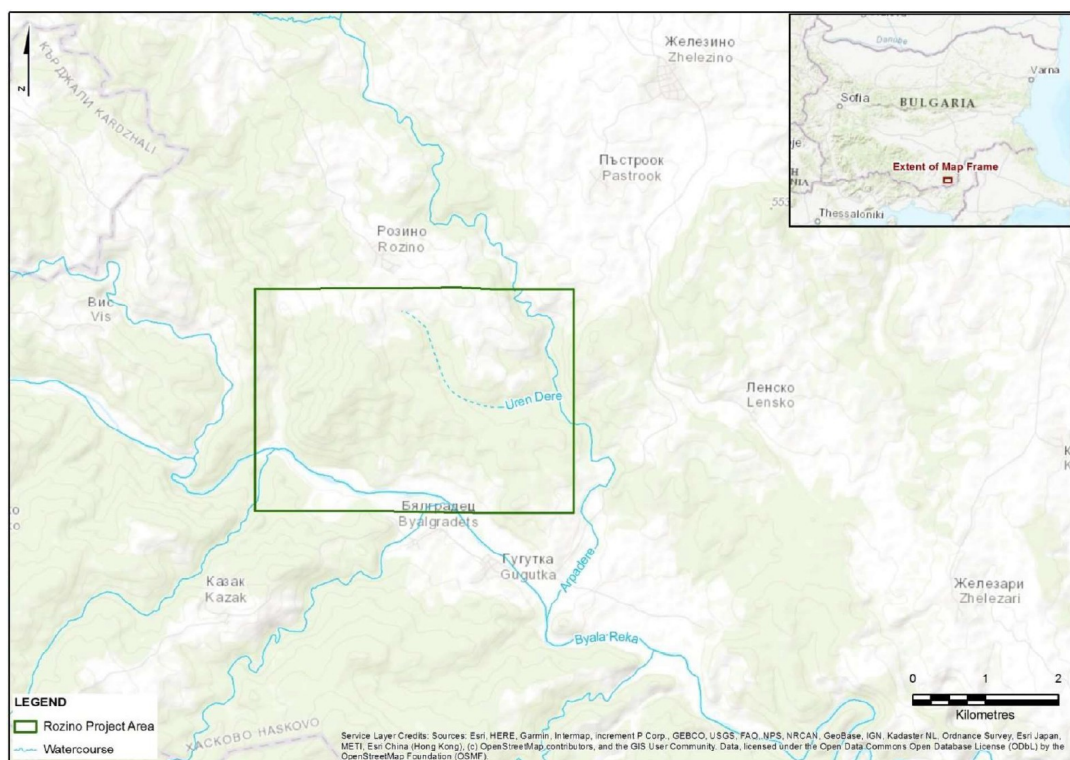


Figure 2: Location of the project site and surface water bodies in the vicinity

### 3.2 Climate assessment

The site is located in an area with a continental Mediterranean climate. It usually has hot, dry summers and warm winters. The climate data was obtained from the Krumovgrad Climate Station, which was determined to be the most suitable source based on a previous assessment of this data against other regional stations.

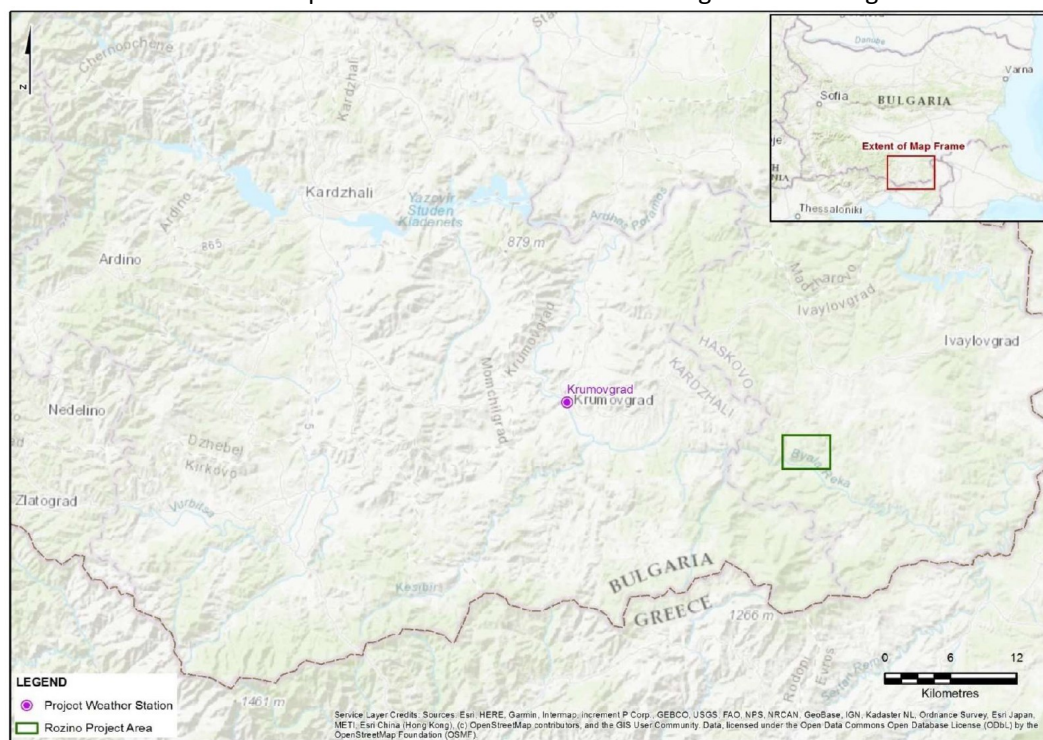


Figure 3: Location of the Krumovgrad Climate Station in relation to the site

### 3.3 Temperature

The temperature data recorded at the Krumovgrad meteorological station between November 2014 and October 2019 are presented in Table 1. The values presented are the minimum, maximum and average values of data for the average daily temperature for the specified period. The average annual temperature is 13.2°C. Temperatures were highest from June to August and lowest from December to February. January is the coldest month with an average daily temperature of 1.9 °C, and August is the hottest month with an average daily temperature of 23.9 °C. The highest recorded daily temperature is 29.5°C (15 July 2016), and the lowest is -10.1°C (1 September 2017).

**Table 1: Temperature in Krumovgrad HMS (November 2014 to October 2019)**

Temperature (°C)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Minimum daily	-10.1	-5.5	-3	4.7	10.9	15.8	16.5	18.2	10.6	3.6	-2.3	-5.4	-10.1
Average daily	1.9	5.4	8.5	12.8	17.1	21.1	23.3	23.9	19.3	12.8	8.4	3.7	13.2
Maximum daily	12.5	14.3	14.8	21	22	29.1	29.5	28.3	27.3	19.8	19	16.5	29.5

### 3.4 Precipitation

The average monthly and annual precipitation amounts for the Krumovgrad station from 1974 to 2019 are given in Table 2. The average annual precipitation is calculated as 745 mm. The wettest month is December, with an average monthly precipitation amount of 92.7 mm. The driest month is August, with an average monthly total precipitation depth of 26.7 mm.

**Table 2: Average annual precipitation amounts by month for HMS Krumovgrad**

Statistics	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Precipitation amounts (mm)	74.4	76.5	70.1	61.2	59.5	52.6	40.8	<b>26.7</b>	43.8	64.4	82	92.7	745

Table 3 shows the average monthly and annual snowfall for the Krumovgrad station; however, depths are only available for the last 5-year period (November 2014 - December 2019). Snowfall is recorded as accumulated from November to March, with a maximum depth of 69.0 cm occurring in January 2017. January has the highest average snowfall with a depth of 5.1 cm. The average annual snowfall is 7.4 cm.

**Table 3: Average monthly snow cover depth for HMS Krumovgrad**

Statistics	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average (cm)	5.1	0.9	1.3	0	0	0.0	0	0.0	0.0	0.	0.	0.1	7.4
Maximum (cm)	69	31	39.0	0.0	0	0	0	0.0	0.0	0.0	2.0	11	152.0

As snowfall is relatively low in this region, the effect of snowmelt on runoff is not taken into account in water management and water balance.

#### 3.4.1 Forecast precipitation amounts

Data on annual maximum daily precipitation was collected from the 46-year record of Krumovgrad (1974-2019). An analysis of the frequency of the annual maximum time series was undertaken to estimate the depth of precipitation for a number of return periods. A logarithmic normal probability distribution was applied to the data set. The results are presented as follows:

**Table 4: Forecast water quantities for 24-hour rainfall (intensity and depth)**

Confidence period	Depth, (mm)	Intensity, (mm/h)
10,000	195.	8
1,000	160.4	6.7
500	150.0	6.3
200	136.3	5.7
100	125.9	5.2
50	115.4	4.8
25	104.9	4.4
20	101.4	4.2
10	90.3	3.8
5	78.6	3
2	60.1	2.5

### 3.4.2 rain gauge

Due to the lack of data on precipitation within a day, the obtained annual maximum daily precipitation depths (Table 4) are subject to appropriate scaling to estimate precipitation within a day. Several methods were used for this analysis, including those of Bell (1969), Wild (1982) and Herschfield (1961), each of which is described in Adamson and Chong (1992).

The Herschfield method is the most applicable. The method was developed for extreme precipitation events in the United States, but since it reflects a wide range of climates from tropical to arid, it is applicable in many other parts of the world. Due to the high evaporation rates (see next section) compared to available precipitation, the site is classified as semi-arid. This assumption allowed the selection of appropriate scaling factors for calculating daily precipitation. The selected factors are presented in Table 5:

**Table 5: Downscaling factors for 1 to 24 hours**

Storm duration (hours)	Scale factor
1	0.58
2	0.70
3	0.80
6	0.90
12	1.00
24	1.00

For hourly precipitation values, the hourly precipitation values obtained are multiplied by the recommended downscaling factors from Hershfield (1961), which are presented in the following table:

**Table 6: Downscaling coefficients for 1 to 24 hours**

Storm duration (minutes)	Scale factor
5	0.29
1	0.45
15	0.57
30	0.79
60	1.00



Table 7 and Table 8 present the depth-duration-frequency (DDF) and intensity-duration-frequency (IDF) data generated using the above methods, respectively.

**Table 7: Depth-Duration-Frequency**

Return period	Precipitation duration (hours). Depth in [mm]									
	0.083	0.167	0.25	0.5	1	2	3	6	12	24
5	13.2	20.5	26	36	45.6	55.0	62.9	70.7	78.6	78.6
10	15.2	23.6	29.9	41.4	52.4	63.2	72.3	81.3	90.3	90.3
20	17.1	26.5	33.5	46.5	58.8	71.0	81.1	91.3	101.4	101.4
50	19.4	30.1	38.2	52.9	67.0	80.8	92.4	103.9	115.4	115.4
100	21.2	32.9	41.6	57.7	73.0	88.1	100.7	113.3	125.9	125.9
200	22.9	35.6	45.0	62.4	79.0	95.4	109.0	122.6	136.3	136.3
500	25.2	39.1	49.6	68.7	87.0	105.0	120.0	135.0	150.0	150

**Table 8: Intensity-duration-frequency**

Return period	Duration of precipitation (hours). Intensity in [mm/hr]									
	0.083	0.16	0.25	0.5	1	2	3	6	12	24
5	158.6	123.0	103.9	72.0	45.6	27.5	21	11.8	6.5	3.3
10	182.4	141.5	119.5	82.8	52.4	31.6	24.1	13.6	7.5	3.8
20	204.7	158.8	134.1	92.9	58.8	35.5	27.0	15.2	8.4	4.2
50	233.0	180.8	152.7	105.8	67.0	40.4	30.8	17.3	9.6	4.8
100	254.1	197.1	166.5	115.4	73.0	44.1	33.6	18.9	10.5	5.2
200	275.0	213.4	180.2	124.9	79	47.7	36.3	20.4	11.4	5.7
500	302.7	234.9	198.3	137.4	87.0	52.5	40.0	22.5	12.5	6

### 3.5 Evaporation

Potential evaporation at the Krumovgrad station for the period of available temperature data (November 2014 to October 2019) was calculated using the Thornthwaite method (Thornthwaite, 1948). The average monthly and annual evaporation is presented in Table 9 Table 10. The average annual evaporation during this period is approximately 760 mm. Evaporation rates are highest from May to August (with over 100 mm of evaporation each month) and lowest from December to February.

**Table 9: Average monthly potential evaporation**

Statistics	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average (mm)	7.5	14.6	28.8	54.2	89.7	121.7	142.1	136.3	88.7	45.5	22.8	9.3	761.4

Records for four full years are available for assessing potential evaporation, as presented below:

**Table 10: Annual potential evaporation at HMS Krumovgrad (2015–2018)**

Statistics	2015	2016	2017	20
Annual evaporation (mm)	751.7	779.1	745.8	758.9

### 3.6 Humidity

The average monthly and annual humidity for HMS Krumovgrad from November 2014 to December 2019 is presented in Table 11. The average annual humidity is 68.2%, ranging from average monthly values of 74.9% in November to 58.2% in August.

**Table 11: Average monthly humidity**

Statistics	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average (%)	74.4	71.5	69.6	63.7	69	67.7	62.6	58.2	63.4	70.8	74.9	72.8	68.2

### 3.7 Wind

The data for the average monthly and annual wind speed for HMS Krumovgrad (1936-1983) are presented as follows (Table 12):

**Table 12: Average monthly wind speed**

Statistics	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average (m/s)	2.1	2.2	2.1	2	1.7	1.5	1.9	1.7	1.6	1.6	1.8	1.8	1.8

## 4.0 SURFACE WATER MANAGEMENT PLAN

### 4.1 Water Management Strategy

Goldar has developed a preliminary surface water management strategy for the Project based on data provided by the Client and the results of previous project design stages. The Client provided mine development plans for years 1 to 7. The current water management plan covers the period from years 3 to 7, as the surface water facilities at the beginning of year 3 will represent the final configuration of the infrastructure. Surface water and its management during years 1 to 2 (facility construction period) will be carried out by a combination of permanent and temporary open channels, which will be considered at a later stage of design.

The main principles of the surface water management strategy/plan are as follows:

- Development of a system for capturing non-contact water from surrounding slopes and controlled its removal for storage and return to circulation to reservoir for fresh water storage (RWD);
- Collection of contact water runoff from the ore pad and open pit mine and its discharge to the settling pond at the tailings storage facility (TMF) or the pond for storage of contact water (CWD), avoiding discharge of contact water into the environment;
- Protecting unlined channels from erosion through appropriate hydraulic design, limiting flow velocities to maximum permissible levels;
- Provision of spillways to non-contact water facilities, sized for the maximum theoretical rainfall event in order to prevent uncontrolled overflow through the wall body;
- Ensuring the necessary free volume in the contact water reservoirs so that, at the relevant regulatory assurance of the rainfall event, the level of the forced water level is lower than the overflow edge level of the respective contact water facility (TMF and CWD).
- Ensuring a strategy for closing the three water storage facilities (TMF, CWD and RWD) after the mine has been decommissioned.

The proposed water management strategy is presented in the drawings in Appendix A.

To manage runoff from the areas above the facilities, two channels have been developed: a northern diversion channel along the northeast side and a southern diversion channel south of the facility.

the site, which will discharge collected water to the RWD and CWD respectively. Other localised drainage solutions will be explored during the next phase of the project.

In addition, emergency spillways for the operational phase and spillways for the closure phase are dimensioned for all three water storage facilities (TMF, CWD and RWD). It should be noted that the calculated water quantity used for this dimensioning is above standard and is based on the theoretical maximum possible rainfall (PMP) for the site. The 24-hour rainfall rate (PMP) of 506.2 mm is approximately 2.5 times greater than the 24-hour rainfall rate with a probability of 1 in 10,000 years (195.8 mm).

## 4.2 Scope of the project

The surface water management design was developed by GOLDER, and the overall design was prepared in collaboration with other consultants. Specifically, the WRD, the plant site and associated access roads were developed by others. GOLDER received the layout for these areas in early August 2020 and took it into account in developing the water management.

Localised drainage solutions for these areas have not been developed, but the water flowing from them has been taken into account as part of the overall water management strategy/plan.

## 4.3 design criteria

The design criteria related to surface water management elements are summarised in Table 13:

**Table 13: Design criteria for surface water management**

Design elements	Scaling factor
Open channels	Operating period: 1 in 100-year event <sup>(1,3)</sup> or average annual runoff <sup>(2,3)</sup> without overflow; Closure period: 24-hour maximum rainfall (PMF) without overflow;
Erosion control	Provide energy dissipators at selected locations to minimise erosion potential in fresh and contact water intakes, CWD and RWD.
Contact and non-contact water reservoirs	Emergency spillway structures are designed to withstand a sustained 24-hour maximum rainfall (PMF) while maintaining a freeboard between the forced water level and the crest elevation of the dam.  The starting water level is taken as the maximum operating water level in the respective reservoir.

Notes: (1) South Diversion Channel, (2) North Diversion Channel, (3) Additional elements.

## 4.4 Hydrotechnical calculations

### 4.4.1 Characteristics of the individual catchment areas

The project area was analysed and the individual catchment areas for each facility were delineated based on available topographic data. Topographic data with a 2 m contour interval was provided by the Client, covering most of the site. Only 0.2% of the site was not covered. As such, it was necessary to confirm the northern boundary of the project's catchment area against satellite imagery (this is estimated at approximately 0.007 km<sup>2</sup>).

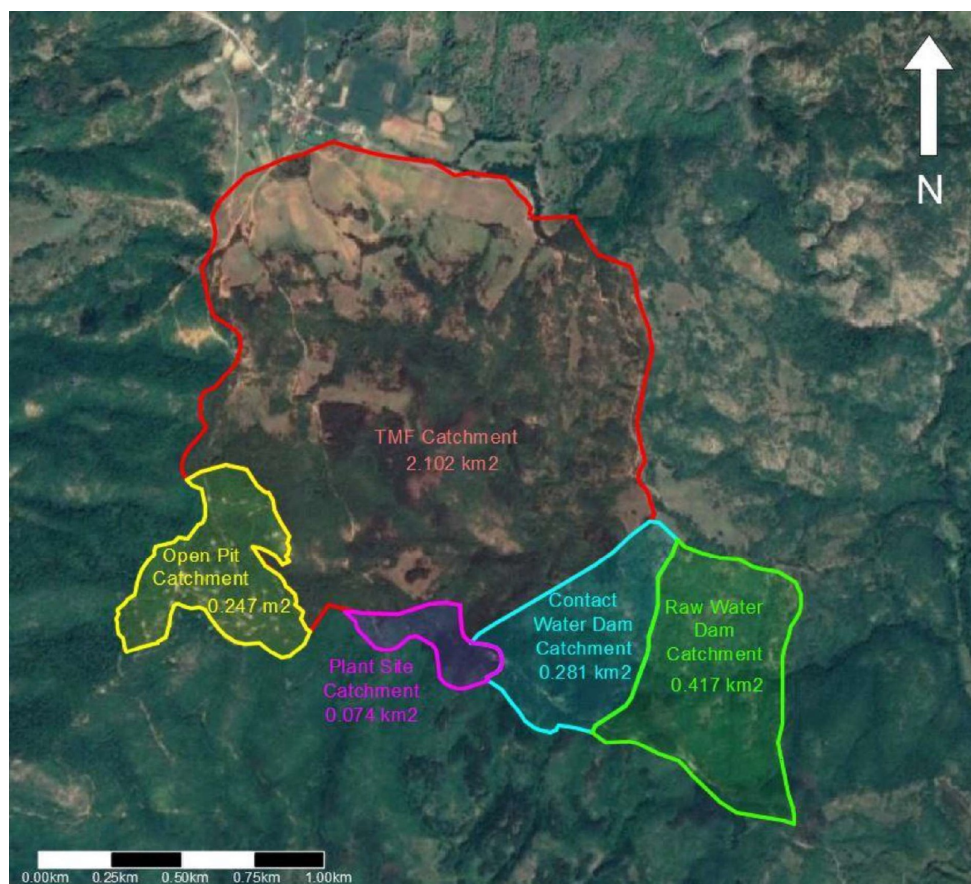


Figure 4: Catchment areas by the end of year 2

From the start of construction work on the mine until the end of year 2, construction work is being carried out to build the walls of the tailings storage facility. There is no production activity and the waters are classified as construction waters. Surface water will be managed by permanent and temporary open channels. The catchment areas are shown in Figure 4.

During the construction period, temporary sedimentation ponds will be constructed along the watercourse to capture solid runoff and discharge clarified water into the Yuren Dere valley.

From the beginning of year 3 onwards, the North Diversion Channel will start operating at the north-eastern end of the tailings storage facility, extending to the RWD area north of the site. This configuration will remain unchanged until the end of operations prior to closure.

The catchment areas formed during the mine's operational period are shown in Figure 5, and the characteristics of the catchments are given in Table 14:

**Table 14: Characteristics of catchment areas during the operational phase**

Catchment	Area (km <sup>2</sup> )	Length of watercourse (m)	Average slope of the terrain, (%)	Runoff time, (mins)
TMF, tailings pond	0.858	91	26.69	12.09
CWD tank for contact water	0.233	374	24.52	7.03
CWD fresh water tank	1.632	3482	22.67	48.27
FP enrichment plant	0.118	899	19.75	14.89
OP, open pit mine	0.239	not applicable	not applicable	not applicable



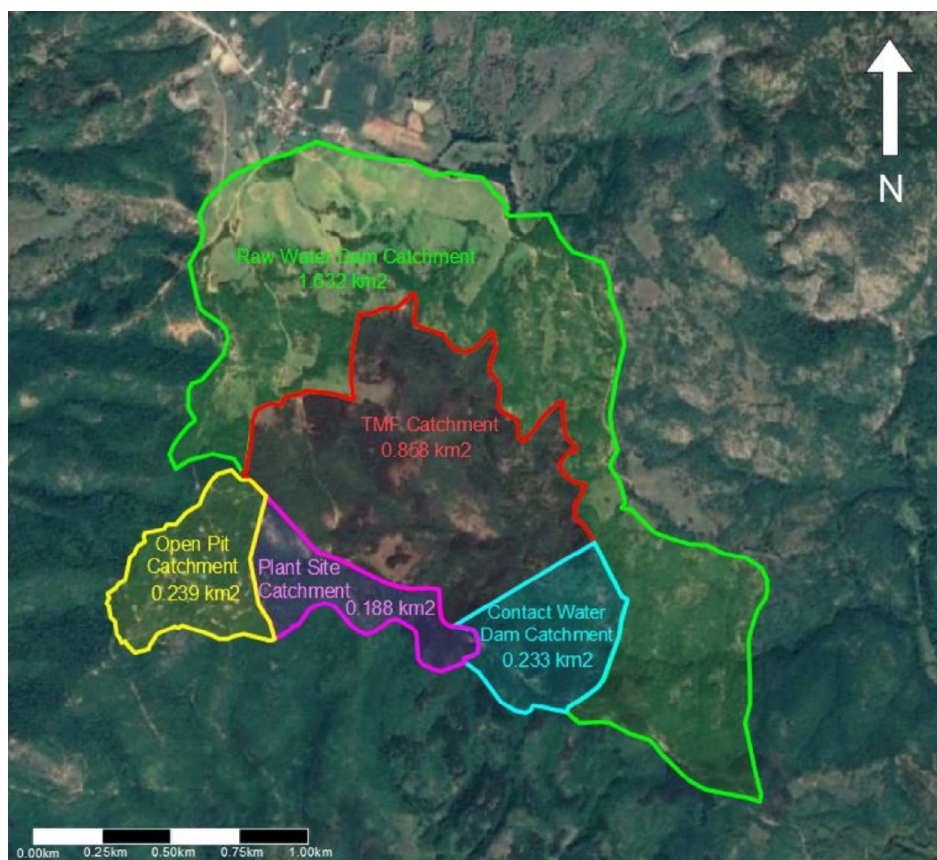


Figure 5: Catchment areas during operation

#### 4.4.2 Maximum water quantities by catchment areas

The software product (HEC-HMS) of the Corps of Military Engineers was used to develop design hydrographs with a 1 in 100 year return period. The following methods were used in the calculation model: "SCS Curve Number loss method" and "SCS Unit Hydrograph transform method".

The SCS Curve Number loss method uses standardised curves to determine water losses in the soil (infiltration, based on soil conditions and catchment cover). The curve numbers are determined for each catchment. It has been calculated that the soil within the site has a hydraulic conductivity in the range of  $1.52 \times 10^{-6}$  to  $1.38 \times 10^{-7}$  m/s. Therefore, the soil is classified as hydrologic soil group D (USDA 2007a), soils with very low infiltration rates. The characteristics for each area are determined depending on its use, and then a weighted average coefficient is obtained for each individual catchment. The weighted average values obtained range from 79.0 to 87.7. The SCS Unit Hydrograph transform method uses an empirical model to transform precipitation events into runoff for the respective section.

In addition to curve numbers, hydrological models require other input parameters. The complete list of model input data required for each catchment is presented below:

- Characteristics of the catchment area:
  - Catchment area;
  - Curve number;
  - Percentage of impermeability;

- Delay time (the time between the peak rainfall and the peak runoff for a given section. For the present analysis, a coefficient of 0.60 of the runoff time has been adopted);
- Meteorological data:
  - Precipitation data assumed for a storm duration of 1 hour (values entered using the "Frequency Storm" method in HEC-HMS);

Table 15 below presents the results of the HEC-HMS model calculations for the catchment areas during the operational period:

**Table 15: Design water quantities for a 100-year event**

Catchment	Design water quantity, (m <sup>3</sup> /s)
TMF tailings pond	24.0
Contact water tank, CWD	6.0
Fresh water tank, RWD	15.6
Enrichment plant, FP	4.3

#### 4.4.3

The Probable Maximum Flood (PMF) has been assessed to contribute to the design of surface water infrastructure including emergency spillways for the tailings storage facility, both CWD and RWD reservoirs during operation and during site closure. The emergency spillways (i.e. HDPE pipe spillways) for the tailings pond, and CWD and RWD reservoirs during operation, and the spillways for the tailings storage facility and RWD during closure are designed to maintain a free height of approximately 1.0 m above the forced water level for each facility in the event of a PMF (Probable Maximum Flood) event.

PMF is the flood associated with theoretical maximum precipitation (PMP), which is defined as the theoretically largest precipitation for a given duration that is physically possible over a given area at a given location at a given time of year, according to current meteorological conditions (WMO 2009). PMP is calculated from the daily rainfall record in Krumovgrad using a statistical procedure described by WMO (2009), which gives a 24-hour rainfall depth of 506.2 mm.

This theoretical storm (PMP) was entered into HEC-HMS to model the outflow from the tailings pond (the facility furthest upstream) using a Type II SCS storm distribution applied to the 24-hour PMP (Chow 1988). The Type II distribution has four cumulative precipitation curves with the steepest distribution gradient, allowing for initial wetting of the catchment basins prior to the period of very intense precipitation. The use of this method is expected to result in "worst case" estimates of rainfall intensity.

The outflow from the tailings pond was directed to the emergency spillway (catchment area 2.03 km<sup>2</sup>), and the outlet structure (i.e. HDPE pipe drain) was dimensioned to achieve the required freeboard between the forced water level and the crest wall elevation. A maximum peak flow rate of 0.46 m<sup>3</sup>/s was obtained from the tailings pond during the PMF event for Year 7. The same size and slope of the HDPE pipe was adopted for the CWD and RWD reservoirs, which are assumed to have to transmit a similar flow rate downstream. A more detailed analysis of these facilities will be carried out at the next design stage.

A similar approach has been adopted for determining the size of the tailings pond spillway during closure and reclamation. The theoretical maximum precipitation (PMP) forecast is entered into HEC-HMS and the runoff is directed to the spillway, which for the closure period is an open trapezoidal channel designed for a flow capacity of 7.4 m<sup>3</sup>/s. At present, the dimensions of the spillways of the fresh and contact water reservoirs have not been specified in detail.

However, it is assumed that the capacity of these facilities is large enough to mitigate the inflow from floods during PMF, while maintaining the desired water level.

The inflow from the tailings pond at closure will be collected from an open channel and discharged downstream to Yuren Dere. A peak flow rate of 8.4 m<sup>3</sup>/s has been accepted, taking into account the additional water inflow from the catchment area downstream of the TMF (additional catchment area of 0.15 km<sup>2</sup>). Therefore, the channel will carry the peak flow of 7.4 m<sup>3</sup>/s from the tailings pond plus an additional peak inflow of approximately 1.0 m<sup>3</sup>/s.

It is assumed that the peak overflow water volumes from the CWD and RWD reservoirs during the PMF in the closure phase will be considered non-contact water (water from reclaimed land) and will be discharged directly into the Yuren Dere without being taken up by the open channel.

## 4.5

### 4.5.1 General layout of the

The northern discharge channel starts at the northern end of the tailings pond. It runs along the eastern edge of the tailings pond and along the CWD to the RWD, where the water carried into the channel is discharged through an overflow into the fresh water lake.

The upper northern part of the channel will consist of an unlined trapezoidal channel. Three reinforced concrete flow control structures will be located along the length of this initial section. They will be designed to divert flows into the tailings storage facility so that the northern diversion channel does not overflow during floods that exceed its capacity (i.e. water will be discharged in a controlled manner to the TMF). The downstream section of the channel will consist of a stepped trapezoidal channel lined with riprap. The stepping and lining are intended to reduce the high velocities and erosion potential of the flow that would result from the slope of the topography.

The preliminary dimensioning of the channels was performed using Manning's equation, taking into account the design slopes between 0.4% and 2%. Roughness coefficients of 0.040 (m<sup>1/3</sup>/s)<sup>-1</sup> and 0.033 (m<sup>1/3</sup>/s)<sup>-1</sup> were taken into account for the unlined and lined channel sections, respectively (Chow 1959).

The cross-section of the channel will remain constant along its entire length, as summarised in the following table:

**Table 16: Northern drainage channel. Geometry**

Facility	Bottom width of channel, (m)	Depth of channel, (m)	Slope of slopes, (H:1V)	N=Longitudinal slope, (m/m)
Northern drainage channel	1.0	1.0	2	0.004~0.020

As designed, the channel will convey the average annual flow without overflow, with average flow velocities of 0.68 m/s and 1.39 m/s for the unlined and lined sections, respectively.

The water carried by the channel will be discharged into the fresh water reservoir (RWD) through a 500 mm diameter HDPE pipe with an approximate slope of 25% and flow velocities not exceeding 10 m/s. To prevent erosion at the outlet of the drain, an energy dissipator lined with riprap will be constructed.

### 4.5.2 Alternatives

A second diversion channel could be considered, starting from the northern end of the tailings storage facility (see point A1, Appendix A) and running along the western edge of the tailings storage facility. This surface water channel would collect runoff from the natural catchment areas located above and separate non-contact water from mixing with contact water. This solution would increase

the volume of fresh water available to replenish the system, reducing the volume of water needed to be pumped from Arda Dere to the RBD. The applicability of this channel will be clarified in the next design stage.

## 4.6 South drainage channel

### 4.6.1 General layout

The southern diversion channel starts at the south-eastern corner of the open pit, passes south of the flotation plant site and discharges into the contact water reservoir (CWD) via a steeply sloping section along the access road to the tailings storage facility.

The channel consists of four sections, two sections will be unlined and two sections will be stepped and lined with riprap. The steps and riprap lining are necessary to reduce the high velocity and erosive power that would be generated on the steep slope of the access road.

The preliminary dimensioning of the southern drainage channel was performed using Manning's equation, taking into account that the channel has a slope between 1.3% and 1.6%. Roughness values of  $0.040 \text{ (m}^{1/3}/\text{s)}^{-1}$  and  $0.033 \text{ (m}^{1/3}/\text{s)}^{-1}$  were used for the unlined and lined sections, respectively (Chow, 1959).

The cross-section of the channel will remain constant along its entire length, as summarised in the following table:

**Table 17: Northern drainage channel. Geometry**

Facility	Bottom width of channel, (m)	Depth of channel, (m)	Slope of slopes, (H:1V)	N=Longitudinal slope, (m/m)
South drainage channel	2.0	1.0	2	0.013

As designed, the channel will carry the peak discharge from the respective catchments for a 1 in 100 year event without overflow and with average flow velocities of 1.73 m/s and 2.10 m/s for the unlined and lined sections, respectively.

Where the channel crosses the access road from the flotation plant, a culvert (HDPE pipe) with a diameter of 1200 mm is installed. The culvert is dimensioned for a peak flow with a 1 in 100 year probability 4.3 m<sup>3</sup>/s.

### 4.6.2 Alternatives

A partial diversion of the water flow to the tailings storage facility reservoir may be considered, if the elevation allows, in order to reduce the peak flow in the southern diversion channel during floods with a significant degree of certainty. The diversion could be constructed at the transition from an unlined to a lined profile at the eastern end of the flotation plant site (see Appendix A, point B3), in a south-east to north-west direction. This possible solution will be studied in the next phase of the project.

Another possible alternative to reduce the load on the contact tanks in case of heavy rainfall is to form two sections in the channel, one for contact water and one for non-contact water. This will allow a reduction in the volume of water discharged from the southern channel into the contact water basins.

## 4.7 Emergency spillways

Emergency outfall structures will be provided for the tailings storage facility (TMF) and the contact (CWD) and non-contact/fresh water (RWD) reservoirs during the mine's operational phase. As discussed



discussed in the previous sections, the facilities are designed to maintain a free height above the maximum (forced) water level in the event of water inflow resulting from a PMF event.

Preliminary calculations have been made for the sizing of an emergency spillway for the tailings storage facility, assuming a 500 mm diameter HDPE pipe with a 1% slope. The spillway is located at the southern corner of the facility.

Upon exiting the tailings pond wall, the pipeline will continue down the slope around 350 m before it enters the CWD contact water reservoir. The slope of this section of the pipeline will be steep and will be finalised in the next design phase.

The same design for emergency spillways has been considered in the cases of CWD and RWD reservoirs. The pipes will be selected with the same diameter and material as for TFM. The outlets will have a 1% slope in the section passing through their respective embankments and a steep slope following the sloping surface of the embankment. The CWD outlet pipe will carry water approximately 125 m downstream before discharging into the RWD. The RWD outlet pipe will transport water approximately 170 m downstream and discharge into the Yuren Dere.

In order to meet the design criterion for erosion control, energy dissipation facilities will be installed at all three outlets.

It should be noted that emergency release valves will be equipped with appropriate locking devices sealed by the authorising authorities, i.e. they will be normally closed and can only be used after explicit notification to the authorising authorities.

#### 4.8 closure stage

A preliminary water management strategy/plan has been prepared for the mine closure phase. The following assumptions were used in developing the strategy:

- It is assumed that the tailings storage facility and the CWD and RWD reservoirs will be retained;
- The emergency spillways (pipes) will be replaced with permanent spillways in the form of open channels;
- It is assumed that the water quality of the entire runoff collected in the tailings pond and CWD meets the discharge criteria and the runoff will be discharged directly into Yuren Dere;
- The access road between the tailings storage facility and the RWD on the eastern side of the site will be replaced by an open channel.

The preliminary design considers a surface water channel that starts at the tailings pond sedimentation pond (TMF), runs along the eastern side of the facilities through the access road between the tailings storage facility and the fresh water reservoir, and discharges its waters downstream of the reservoir wall into the Yuren Dere stream. A maximum design flow of 8.4 m<sup>3</sup>/s has been obtained based on a PMF event.

The channel is stepped and lined with riprap to manage the potentially high flow velocities and erosion forces that would be generated by the steep topography. Preliminary sizing was performed using Manning's equation, considering a slope of 1% and a roughness value of 0.033 (m<sup>1/3</sup>/s)<sup>-1</sup>, appropriate for the lined section of the channel (Chow, 1959). The geometry of the channel is summarised in the following table:

**Table 18: Drainage channel in the closure stage**

Facility	Channel bottom width (m)	Depth of channel, (m)	Slope of slopes, (H:1V)	N=Longitudinal slope, (m/m)
Drainage channel after closure	2.0	1.	2	0.0

The next stage of design will confirm the applicability of a stepped channel lined with riprap, after comparison with other alternatives, such as a stepped channel lined with concrete, etc.

## **4.9 Water outside the storage facilities at**

### **4.9.1 Groundwater from the mine:**

The bottom of the mine is expected to reach an elevation of 435 m. According to the drilling work carried out, no groundwater has been reached at this elevation and therefore no additional water inflow to the pit is expected other than that from rainfall and snowfall.

### **4.9.2 Surface water in the pit, dumps and industrial site:**

The water that has entered the mine and the dumps as a result of rain and snowfall will be directed to the lowest parts of the respective facility. In the mine pit, a sump is planned to be formed at each working level to collect surface water, which will be used to irrigate the mine roads. If necessary, excess mine water will be pumped out and redirected to the contact water facility to replenish the circulating water. All dumps will be constructed and developed with slopes at each stage to ensure gravity drainage of surface water back to the outer edges. A surface water drainage system will be constructed and directed to the ore processing plant or to the contact water reservoir. The upper horizons of the pit, which have an open contour, will help to redirect rainwater by gravity and carry it outside the perimeter of the pit, which would reduce the need for drainage during the extraction of the mine, as well as the associated capital and operating costs.

### **4.9.3**

Bottled water will be supplied for drinking purposes. Industrial water will be used for domestic purposes.

### **4.9.4 Sewage**

There are no plans to build a central sewerage system due to the lack of a treatment plant near the site. As an alternative, the possibility of using mobile sanitary facilities maintained by a specialised company is being considered. This company would periodically collect the generated water and transport it to a licensed treatment plant. Another option is to design and build a mobile treatment plant tailored to the site's consumption, which will be removed once the site is no longer in operation and is closed down. At present, domestic wastewater is considered insignificant and is not taken into account in the water balance.

### **4.9.5 Industrial water – fresh water to replenish the turnover**

According to the preliminary conceptual design, to compensate for the expected annual water deficit of approximately 125,000 to 310,000 m<sup>3</sup> in the plant's water supply, water will be supplied from a pumping station at Arpa Dere near the village of Gugutka (approximately 1.7 km east of the Rozino site).

The study envisages pumping water directly from Arpa Dere during the wet months of the year (from January to May inclusive). The water intake from Arpa Dere will be close to the confluence with Yuren Dere and adjacent to the existing pumping station in the village of Rozino. This pumping station is designed to pump water from a spring that flows into the whirlpool. The flow rate of this spring varies between 6 and 11 l/s throughout the year, depending on the season. It has been calculated that the flow rate required to supply the village of Rozino is approximately 0.34 l/s. The excess water from the spring, after the relevant justification, could be used for the industrial needs of the site throughout the year.

It is planned that the water intake will be carried out from a naturally formed pool without the need to build a dam or other construction works blocking the river. During operation and upon closure of the site, the suction pipe of the pumping station will be dismantled with minimal impact on the riverbed.

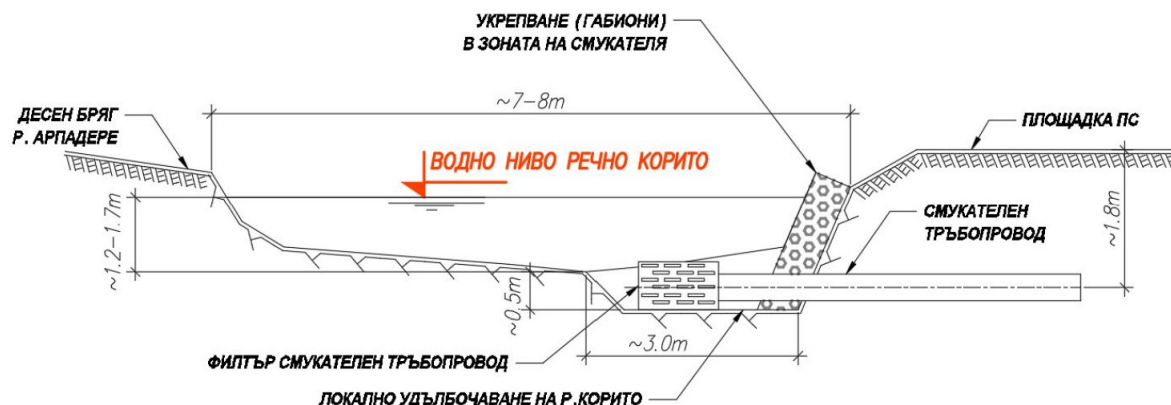


Figure 6: Water intake from Arpa Dere

The intention is to build a pumping station adjacent to the existing pumping station and to use the existing 20 kV power supply. The pumping station will have three pumps (two working, one standby) designed to supply water to the fresh water facility at a flow rate of up to 50 l/s. The distance to the site is 1.15 km. The hydraulic head between the pumping station and the point of inflow into the fresh water tank is 61 m. The project envisages that all equipment for the pumping station will be modularly housed in separate containers, which will be dismantled during the closure phase with minimal impact on the environment. The pipeline will be underground, at a depth of approximately one metre, and will run along the length of the Yuren Dere. The necessary air vents and mud traps are provided along the length of the route.

#### 4.9.6

The possibilities for accelerated evaporation technology have been investigated with the aim of removing water from the contact water tank (CWD) and thus from the system as a whole.

The concept of accelerated evaporation is to disperse approximately 5 l/s of water from the tank (for each device) 18 m into the air with an average droplet size of less than 100  $\mu\text{m}$ . This is achieved by means of mobile mechanical evaporation devices, enabling the achievement of the "zero discharge" target at the site during the mine closure phase.

Expected climatic conditions, based on historical records, have been used to assess the potential effectiveness of the enhanced evaporation device in the model. Table 19 presents statistics on climatic conditions in the area of the site.

Table 19: Climate conditions for accelerated evaporation

Item	Value
Average annual wind speed	1.8 m/s
Average humidity – April to September	64
Average daily temperature – April to September	19.4 °C
Predominant wind direction	North

The data in the table above show that the climate has favourable conditions for enhanced evaporation. The high average temperatures and relatively low humidity during the months of April to September indicate that these could be the optimal months for enhanced evaporation.

Annex B provides an example with specific manufacturer data for reference and information.

The devices are mounted either on the ground or on floating platforms. This allows for operational flexibility and repositioning depending on wind direction. With this system, all water that has not evaporated is captured within the reservoir's catchment area.

The facilities will be actively used in the mine closure process (as provided for in the investment intention 5-year period of active conservation) when the collected drainage water in the contact reservoir will evaporate during the summer months to provide sufficient free volume for drainage water during the winter period when evaporation is ineffective.

The enhanced evaporation device can accommodate dissolved solids content within the range of 30,000 to 40,000 parts per million (ppm). The forecast for accelerated evaporation, even at a minimum efficiency of 30%, is up to 3028 m<sup>3</sup> per month per device.

## 5.0

### 5.1 Description and objectives of the model

To facilitate planning and design in surface water management at the current mine, a water balance for the entire site has been developed using GoldSim software. GoldSim is one of the best software programs using the Monte Carlo method to perform simulations and predict future processes and events, while providing a quantitative and qualitative assessment of changes in the systems under study.

The model simulates the life of the mine (LOM). The mine closure scenario has not been modelled. The individual components of the system after closure are designed for maximum possible flow (PMF) cases as described in section 4.8 above.

The model simulates the catchment areas and the outflow formed by the following facilities and infrastructure:

- Open pit mine;
- Tailings storage facility, TMF;
- Flotation Plant, PP;
- Contact Water Tank, CWD
- Fresh water tank, RWD

The results support the designed water management system. The main objectives of the water balance model are described as follows:

- Simulation and planning of future water management at the site;
- Simulation of the volume and maximum water level stored in the tailings pond sedimentation tank and other water tanks to support the sizing of facilities and the design of emergency spillways;
- Confirmation of pumping rates between different water storage facilities;
- Predicting whether there will be excess water at the site; and
- Predicting whether the water brought to the site and rainfall are sufficient to meet the needs of the flotation plant.

## 5.2 Conceptual model

A conceptual block diagram of the water balance processes (PFD) for the entire site is shown in and represents the structure used in the GoldSim model. The principles and details of water management at the site are described in the previous section.

### Incoming flows:

- Incidental total rainfall directly on the open pit areas, the tailings pond, the contact and fresh water reservoir, RWD and CWD;
- Runoff from the catchment basin from internal and external areas (runoff from natural catchment basins located upstream is diverted to the RWD);
- Fresh water with a flow rate of 50 l/s from Arpa Dere at the Rozino pumping station for the months of January to May (i.e. 5 months of the year); and
- Subsurface water inflows from the open pit mine and other filtration;

At this stage, the model assumes that the outflow is from precipitation. Snowmelt inflows represent less than 10% of annual precipitation and are not expected to significantly change the dynamics of the water balance.

### Outflows:

- Evaporation from water surfaces in the open pit mine, the tailings pond and the RWD and CWD ponds;
- Water entrained in the tailings and/or filtration flows from the tailings pond;
- Water for domestic use, consisting of 50 litres per person per day for a staff of 300 (75% of which, after treatment, is returned via the tailings storage facility for use in the production facility); and
- Water for dust suppression, which varies per month from 140 m<sup>3</sup> (February) to 10510 m<sup>3</sup> (August), and which is drawn from the TMF.
- Forced evaporation through an evaporator installation to control the level in the contact water pond;

At this stage, forced evaporation is not being considered, as it will not be effective during the operational phase of the facilities. Forced evaporation is effective during the hot months of the year, but it is precisely during this period that the demand for fresh water is highest. During the rainy season, the installation is not effective, and experience with similar facilities shows that it is used extremely rarely, which is why the water balance has been calculated without considering this additional option for water control.

On the other hand, during the active conservation period (the first 5 years after closure), the installation will have an irreplaceable effect on controlling drainage water during the summer.

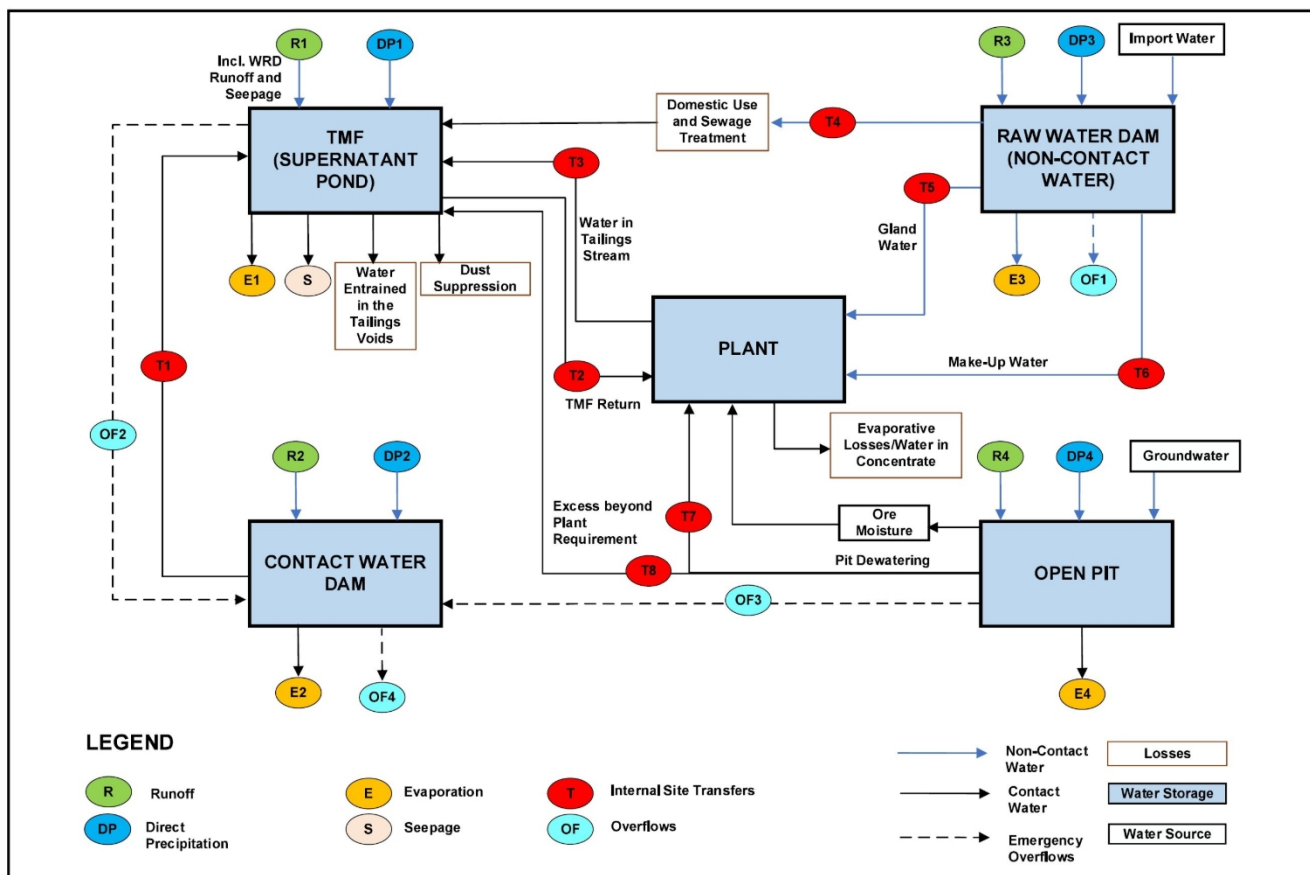


Figure 7: Block diagram of the main flows in the water balance

The conceptual model of the water balance for the site is shown in the following figure:

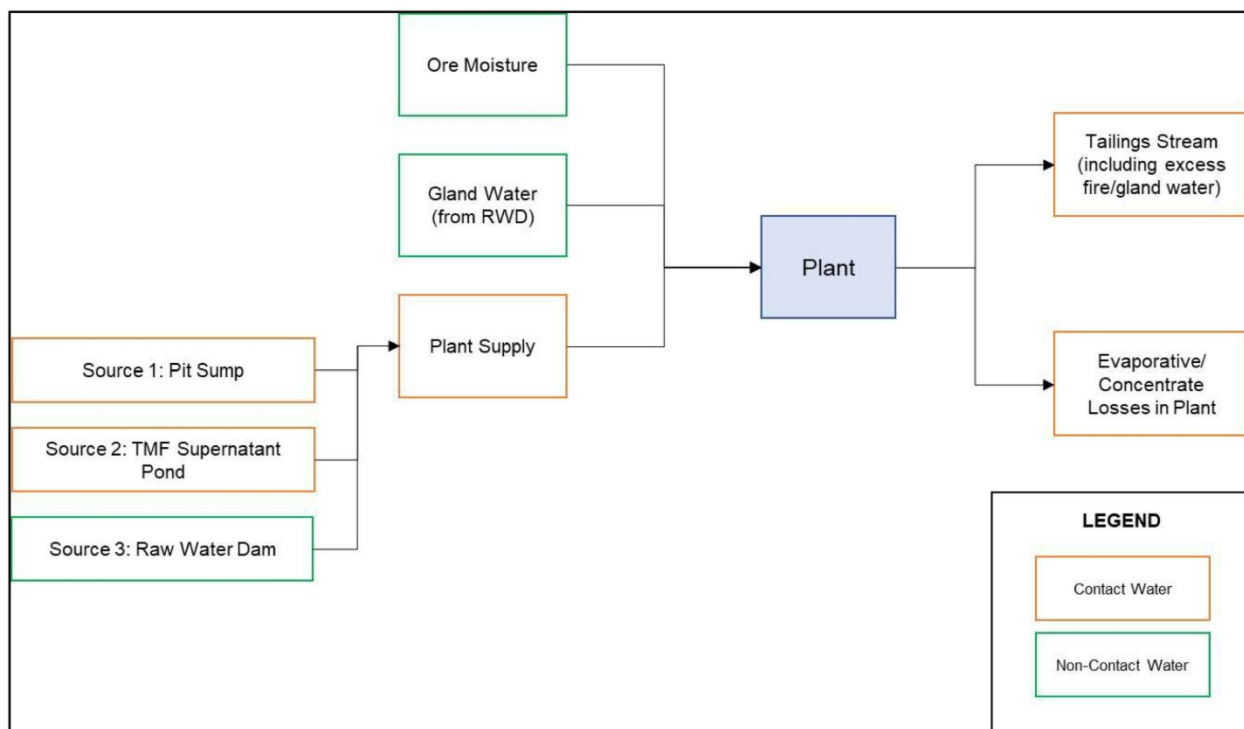


Figure 8: Conceptual model of the water balance

### 5.3 Input data and basic assumptions for the model

#### 5.3.1 model simulation settings

The model simulation period (7 years) is consistent with the duration of the LOM. The model is run on a daily time step. The model is run deterministically (i.e. a single, predefined daily time series of precipitation is simulated at a given moment, with no stochasticity built into the model).

#### 5.3.2 Climate input data

##### Precipitation

The model simulates one of three deterministic precipitation series at a time, allowing for the analysis of dry, average and wet years. The daily precipitation from Krumovgrad (described in the report above) is statistically analysed to develop the precipitation sequences.

Initially, 1996 was selected as a year with average precipitation, with annual precipitation of 736 mm – the closest recorded value to the Krumovgrad HMS (745.1 mm). The total monthly precipitation for 1996 also corresponds to the long-term monthly averages for Krumovgrad. The daily precipitation for the selected average year (1996) is repeated seven times to create a suitable average precipitation sequence to drive the 7-year LOM model.

Daily precipitation for the average year (1996) was used as a basis for creating dry and wet annual sequences. The generated precipitation depths (for dry and wet years) with a 2% probability (1 in 50 years) were determined by analysing the moving 84-month cumulative depths of the precipitation records of the Krumovgrad HMS using a logarithmic distribution. A scaling factor was applied to the daily precipitation for 1996 so that the total amounts of the 7-year cumulative precipitation correspond to the determined 1 in 50 years for wet or dry 7-year precipitation amounts.

**Table 20: Summary of input precipitation for the water balance model**

Scenario	7-year total precipitation, (mm)	7-year return period (%)	1-year total precipitation, (mm)	Annual return period
Dry year	3755	1 in 50 years (98%)	536	1 in 10 years dry (90%)
Average year	5152	1 in 2 years (50%)	736	1 in 2 years (50%)
Wet year	6683	1 in 50 years (2%)	955	1 in 10 years wet (90%)

Therefore, the scenarios simulated within the model are as follows:

- Average year: Closed 7-year cycle of cumulative daily records for years close to the average annual precipitation for HMSA Krumovgrad 1974-2019. The selected average year is 1996 with MAP 736.5 mm;
- Dry year: Using a logarithmic normal distribution, a dry series of precipitation (1 in 50 years) is determined based on the annual precipitation from HMS Krumovgrad (3755 mm, i.e. 536 mm per year). Daily precipitation for the selected average year (1996) is scaled to create a 7-year dry series for simulation;
- Wet year: Also using a logarithmic normal distribution, a wet series of precipitation (1 in 50 years) was determined based on the annual precipitation from HMS Krumovgrad (6683 mm, i.e. 955 mm per year). The daily precipitation for the selected average year (1996) is scaled to create a 7-year wet series for simulation;

##### Evaporation

Based on a review of available potential record years for evaporation, 2018 was determined to have a total evaporation depth closest to the 4-year average for the Krumovgrad station. The 2018 record was used as a representative daily input for time



rows to the model for the average, dry and wet precipitation scenarios. The table below presents the monthly and annual total potential evaporation for 2018, which is consistent with the average values presented in Table 9 above.

**Table 21: Average monthly potential evaporation**

Statistics	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average (mm)	8.2	10.7	28.1	69.2	101.1	116.8	133.1	132.3	84.4	47.9	20.8	6.1	758.9

### 5.3.3 Hydrological input data

Rainwater runoff was calculated using the Soil Conservation Service Curve Number Method (Kent, 1973), where each curve number depends on soil type, land cover and previous moisture conditions. The curve numbers were selected for three types of land cover, as shown in the following table:

**Table 22: Curve numbers for runoff calculations**

Terrain description	Curve Number
Undisturbed natural terrain	70
Urbanised areas (roads, concrete, etc.)	88
Mine walls and lined areas	98

The catchment areas for the various mining facilities for each year of the LOM are listed in the following table:

**Table 23: Catchment areas. Development by year in m<sup>2</sup>**

Description	1	2	3	4	5	6	7
<b>Fresh water lake, RWD</b>							
Built-up	10635	10635	27177	27177	27177	27177	10635
Unbroken	406757	406757	1604912	1604912	1604912	1604912	1621454
<b>Total</b>	<b>417,392</b>	<b>417,392</b>	<b>1632089</b>	<b>1632089</b>	<b>1,632,089</b>	<b>1632089</b>	<b>1632089</b>
<b>Contact water reservoir, CWD</b>							
Built-up	92318	92318	99276	99276	99276	99276	99276
Unbroken	189118	189118	133637	133637	133637	133637	133637
<b>Total</b>	<b>281,437</b>	<b>281,437</b>	<b>232,913</b>	<b>232,913</b>	<b>232,913</b>	<b>232,913</b>	<b>232,913</b>
<b>Tailings storage facility, TMF</b>							
Built-up	210880	30595	294,476	294,476	294,476	294,476	300899
Unbroken	1771213	1670213	226,490	226,490	226,490	226,490	220066
Tiled	125490	125490	337385	337385	337385	337385	337385
<b>Total</b>	<b>2107583</b>	<b>2101662</b>	<b>858,351</b>	<b>858,351</b>	<b>858,351</b>	<b>858,351</b>	<b>858,351</b>
<b>Mine pit, OP</b>							
Built-up	16542	16542	16542	130426	141854	145041	91937
Unbroken	14362	14512	1922	2366	2366	2366	0
Mine board	182708	215841	250883	159516	148088	144,901	146,791
<b>Total</b>	<b>213,612</b>	<b>246,896</b>	<b>269,347</b>	<b>292,308</b>	<b>292,308</b>	<b>292,308</b>	<b>238,729</b>

### 5.3.4

The inflow of groundwater from the open pit mine is estimated as follows:



**Table 24: Expected inflow of groundwater from the mine pit**

Year	2	3	4	5	6	7
m <sup>3</sup> /year*10 <sup>3</sup>	50	300	150	200	225	100

### 5.3.5 Water storage facilities

The key geometric properties for the different water storage facilities are given in the following table. The model tracks water volume, elevation, and surface area in each facility throughout the simulation period. The sizing of RWDs is discussed further in the next section.

**Table 25: Geometric characteristics of the main facilities**

Scenario	Total operating volume, m <sup>3</sup> 2m free height	Area, m <sup>2</sup> 2m free height	Crown elevation, m
RWD	365000	4305	302
CWD	2500	6823	311
TMF-1g.	1056503	105573	358
TMF-2g.	2333301	179844	367
TMF-3g.	3356801	230013	372
TMF-4g.	4639261	282012	377
TMF-5~7	6235877	352199	382

### 5.3.6 Basic operating rules

#### Open pit mine sumps

The sumps (which collect the outflow and inflow of groundwater) are kept as empty as possible, with water pumped to the plant as a priority in order to meet the requirements of the technological plant. If there is excess water, it is pumped to the tailings pond for temporary storage.

#### Contact water pond

The CWD is designed to capture contact water flows (outflow from disturbed areas) that cannot be conveyed to or stored in the tailings storage facility by gravity. The CWD is kept as empty as possible. Water collected in the reservoir is pumped to the tailings storage facility if the tailings storage facility is less than 90% full.

#### Freshwater lake

In the model, water is imported from Arpa Creek for five months of the year (January to May) at an average flow rate of 50 l/s and stored in the RWD, unless the RWD is more than 95% full. This fresh water is used for domestic purposes or as fresh water to supplement the factory's water cycle when needed.

#### Tailings storage facility

The thickened tailings are deposited in the tailings storage facility until year 6, after which they are deposited in the open pit. The tailings storage facility is modelled to have a conical shape with a slope of 1V: 100H. The settling pond is used as a source of water for dust suppression.

#### Dust suppression

The quantities accepted for analysis as necessary for dust control are as follows:

**Table 26: Monthly water costs for dust control**

Consumption	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
m <sup>3</sup> /month	155	140	155	1831	2610	4065	8648	10510	6117	2872	150	155

## Water supply plan for the factory

Groundwater and surface runoff from the open pit mine are used as the primary source of water for the production facility. This water is pumped from the open pit mine sumps to the plant reservoirs. The water stored in the tailings pond is the second source of water for the plant, with RWD being used as a supplementary water source (third source) if the water from the open pit mine and the tailings pond is not sufficient or of the required quality to meet the factory's water needs. The factory's water requirement is calculated by subtracting the % of solids from the tailings flow and dividing the resulting mass flow by the density of water.

A summary of the values used in the model is provided in the following table:

**Table 27: Main calculation parameters**

Parameter	Value	Unit of measurement	Note
TOTAL			
Mine life	7	years	Tintyava
Annual ore production	1.5~1.75	10E+6 tonnes/year	Tintyava
RWD			
Fresh water inflow from Arpa Dere	50	l/s	Tintyava
Months for water abstraction (January-May)	5	months	Tintyava
TMF			
Annual tailings production	1.4~1.7	10E+6 tonnes/year	Tintyava
LOM plan for disposal in the tailings storage facility	8.575	10E+6 tonnes/year	Tintyava
Solid particle content (by weight)	55	%	Slurry
Specific gravity of solids in tailings	2.7	tonne/m <sup>3</sup>	Slurry
Bulk density of consolidated tailings	1.4	tonne/m <sup>3</sup>	Slurry
Coefficient of pores in consolidated tailings	0.9285		Calculated
Slope of the beach in the tailings pond	1	%	Accepted
Volume of water entrained in the tail	3650	m <sup>3</sup> /day	Calculated
Water saturation of the consolidated tailings	10	%	Accepted
Beach area (wet surface)	90	%	Accepted
Tailings filtration coefficient	8E-11	m/s	
CWD			
Pump flow rate to TMF	10	l/s	Optimised
Open pit mine			
Pump flow rate to TMF (excess water)	12	l/s	Calculated
Pump flow rate to the factory	~40	l/s	Calculated

## 5.4 analysis results

The results obtained from the water balance model are presented in the following sections.

### 5.4.1 Fresh water tank capacity, RWD

The water supply to the plant is vital to the project. Drainage from the open pit mine will provide the primary source of water, the settling pond in the tailings storage facility will be the secondary source, and the remainder will be obtained from the fresh water reservoir. The capacity of the RWD must also be sufficient to supply clean water for domestic use. An external supply of fresh water (Arpa Dere pumping station) with a flow rate of 50 l/s is available to the project for 5 months of the year. It will be

stored with the RWD and used during the dry part of the year when a negative water balance is expected. The client provided three indicative options for the volume of this reservoir for study in the model, i.e. option 1 (87,000 m<sup>3</sup>), option 2 (365,000 m<sup>3</sup>) and option 3 (536,000 m<sup>3</sup>); these capacities were used as a starting point for assessing the water storage required in the model.

For the 7-year dry climate scenario, the capacity of the RWD has been varied according to the options described in 1 to 3. For Option 1, the storage of fresh water is insufficient and will lead to a shortage of water supply to the plant. Option 2 provides sufficient water storage to supply the plant during the dry season for the 7-year dry climate scenario. Approximately 300,000 m<sup>3</sup> of active volume will be required to be provided in the RWD. To achieve this, the reservoir is designed with a total volume of 365,000 m<sup>3</sup>, which takes into account unforeseen water costs/revenues. With this capacity and the current rates of fresh water import, there is no water shortage for the factory.

The simulated volume in the RWD for the life of the mine under the three precipitation scenarios (dry, average and wet year) are illustrated in the following graph:

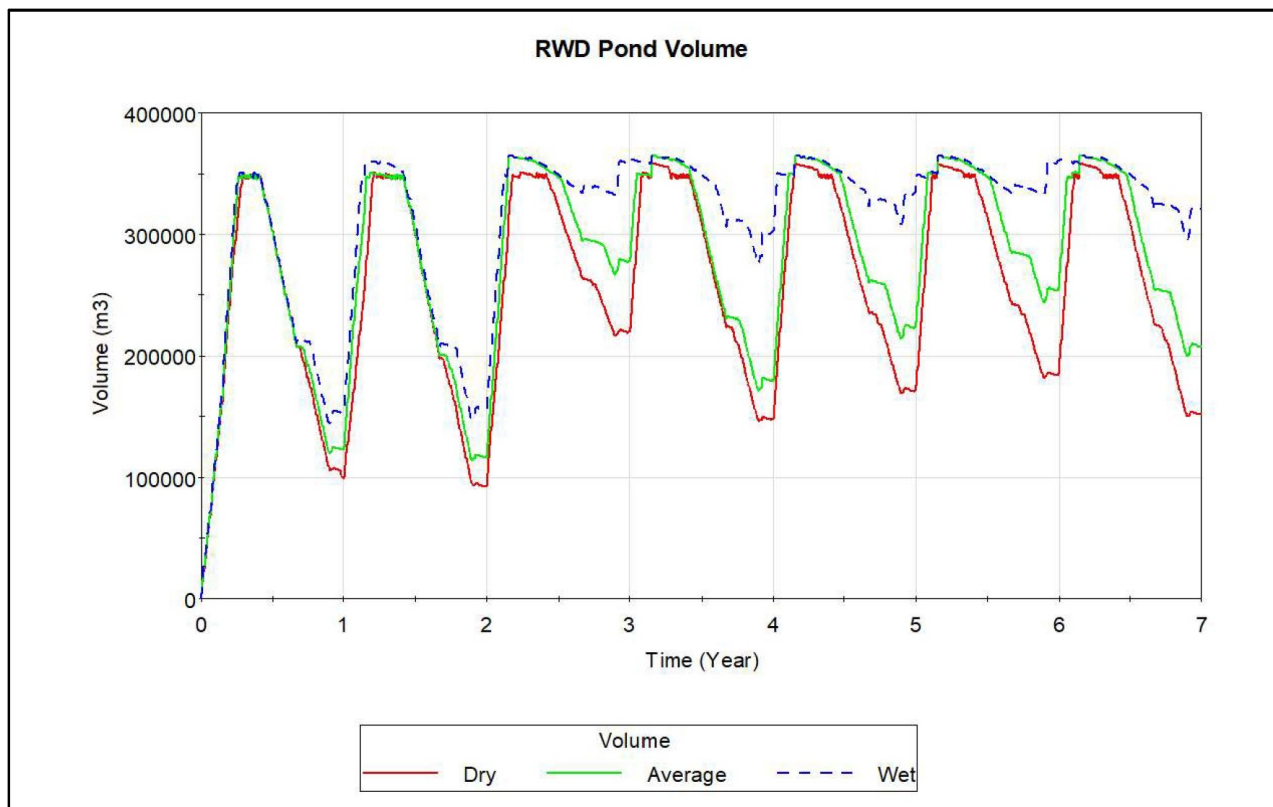


Figure 9: RWD volume for dry, average and wet year scenarios

As illustrated in the following figure, the cumulative volume of imported water is lower in the wet year scenario than in the average and dry year scenarios, as imports are stopped when the RWD reaches 95% of its capacity. Modelling shows that water imports of 50 l/s for 5 months of the year are sufficient to supply the project.

### Alternatives for supplying fresh water throughout the year

At the next stage of design, attention should be paid to the possibility of importing fresh water from alternative sources, such as the existing irrigation pumping station near the village of Gugutka on the Biala River or the springs at the pumping station in the village of Rozino on the Arpa Dere. The possibility of year-round supply of fresh water from an alternative source in the order of 6-10 l/s would lead to a significant optimisation of the size of the fresh water lake, and thus to a reduction in capital costs and environmental impact.

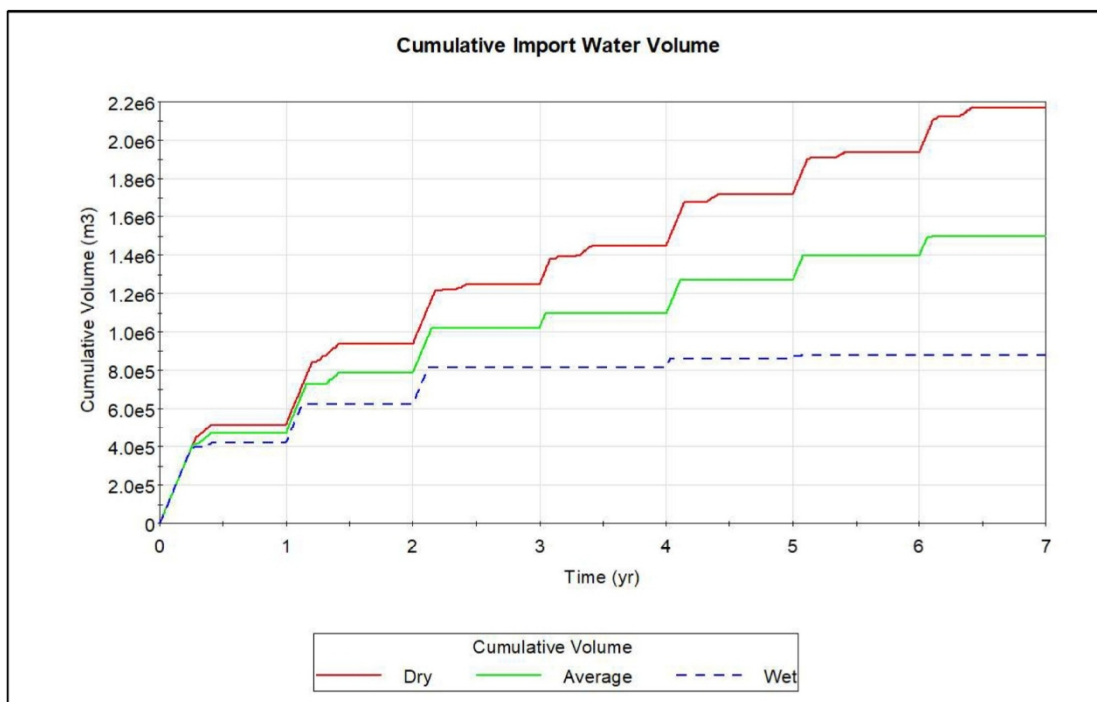


Figure 10: Cumulative volume of water from an external water source for RWD

#### 5.4.2

As shown in the following figure, 14,000 m³ of active volume is required for the wet year scenario (in this case, the most excess contact water is generated). Based on this, a total volume of 25,000 m³ has been accepted to account for unforeseen circumstances. The maximum capacity of 25,000 m³ is equivalent to approximately 7 days of water supply to the plant at maximum pump flow.

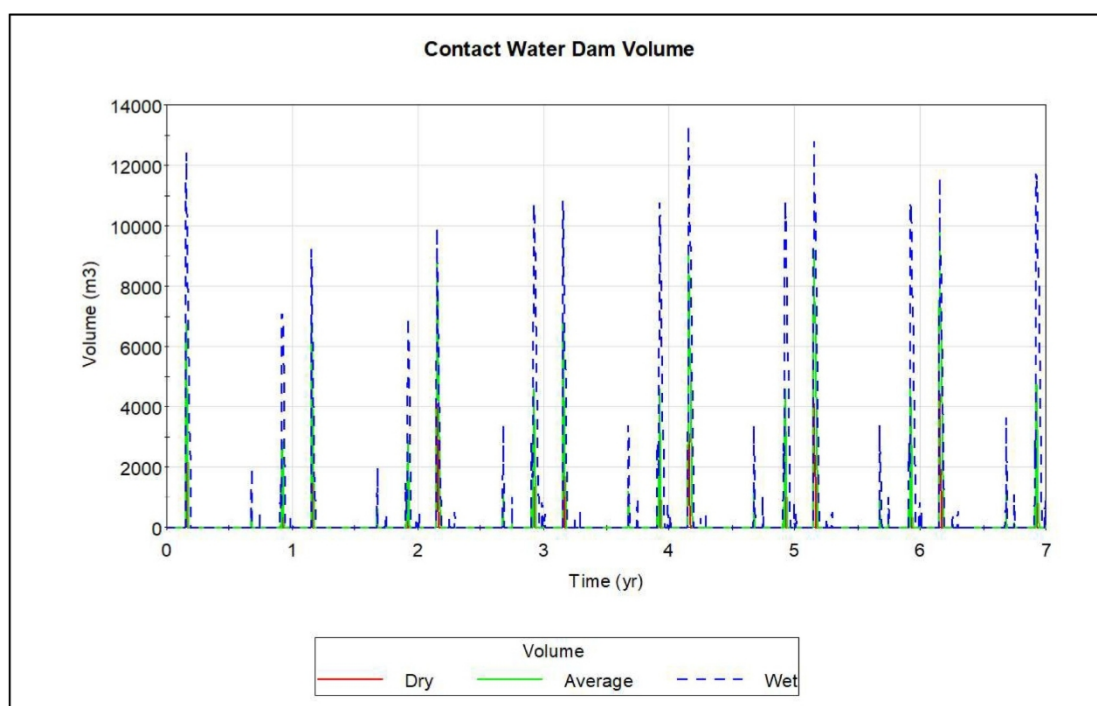


Figure 11: CWD volume for dry, average and wet year scenarios

### 5.4.3 Capacity of sumps in the open- mine

Water management in the open pit mine will be examined in detail at the next stage of design. In the present analysis, the outflow and inflow of groundwater to the open pit mine are captured in a series of temporary unlined reservoirs built at the base of the open pit mine for each active phase. To estimate the pumping rates and the necessary water storage volumes, dimensioning was performed for the three scenarios adopted in the model for a dry, average and wet year.

Inflow rates were estimated for the outflow resulting from the maximum extent of the open pit mine catchment area (year 3 of operations). Outflow was estimated for a 1 in 100-year event using the same methodology described in the transitional sections. The resulting total inflow rate was compared to the range of pumping rates to determine the appropriate balance between pump size and mine shaft volume. The total volume required in the event of an emergency was determined to be 12,500 m<sup>3</sup> if a pumping rate of 120 l/s to the tailings storage facility was used. It should be noted that this peak volume will not appear in the results of the daily step-in-time model, as this is a five-day inflow.

The simulated volume of water stored in the open pit mine shaft for the three deterministic rainfall scenarios is shown in the following figure. Since the water is first pumped to the plant and then the excess water is pumped to the TSF, the water accumulates only for short periods during days of heavy rainfall in the wet year scenario.

The pumping rate and total volume of the sumps are selected so that the sumps will be kept empty for most of the year.

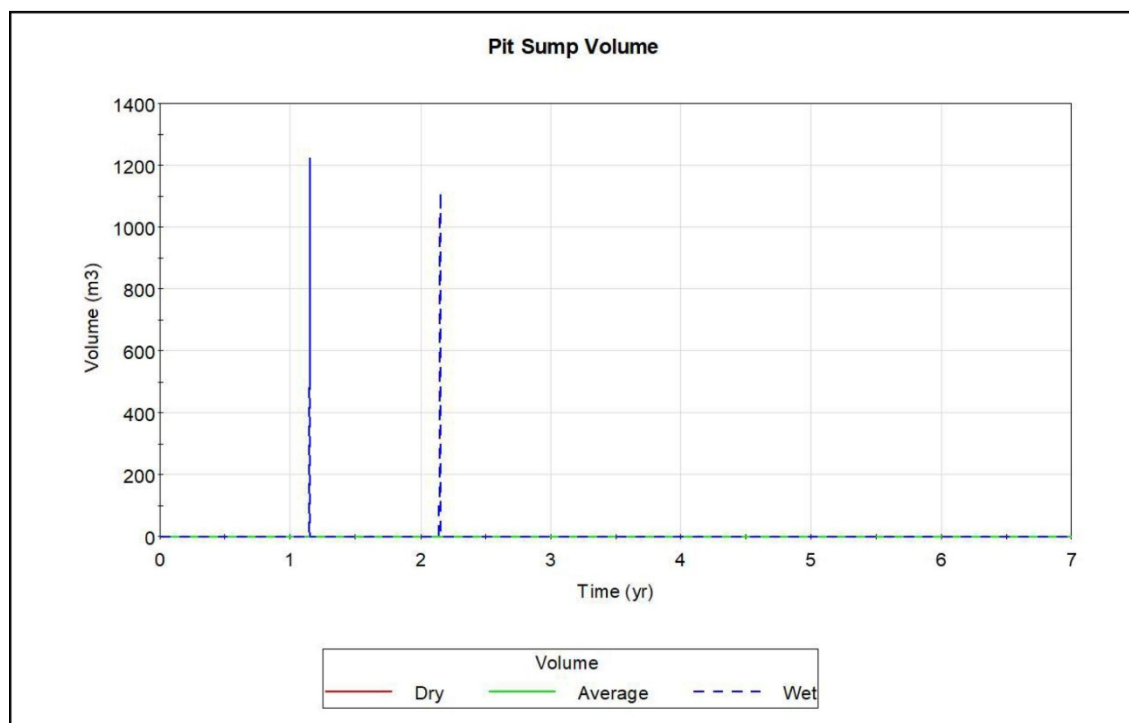


Figure 12: Cumulative volume of sumps in the open pit mine

### 5.4.4 Capacity of the tailings pond settling lake, TMF

The capacity of the tailings storage facility increases each year until year 5, at which point the crest (including a 2 m freeboard) is at 384 m above sea level. The modelled peak water volume in the settling pond is approximately 50,000 m<sup>3</sup> for a dry precipitation scenario, 115,000 m<sup>3</sup> for an average scenario and 220,000 m<sup>3</sup> for a wet year. The tailings storage facility is designed to accommodate the volume of the wet year scenario and to have a free height up to the crest of the wall.

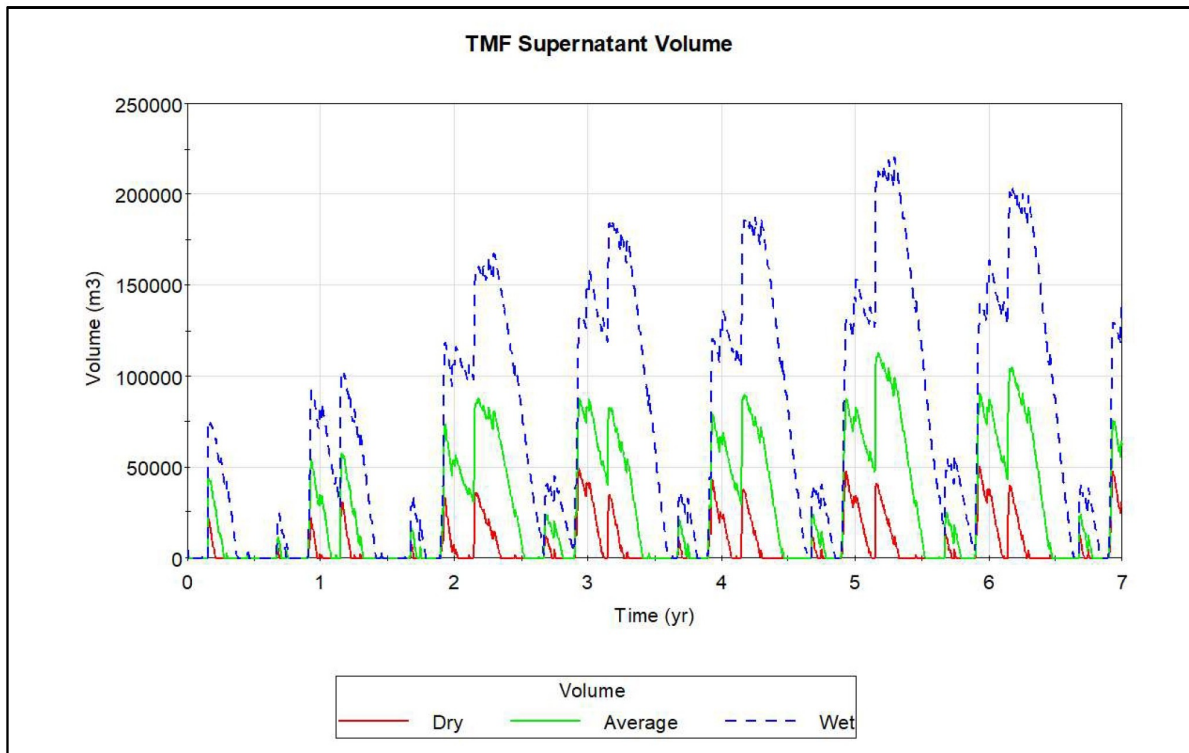


Figure 13: Simulated volume of the tailings pond sedimentation lake

For the wet year scenario, the levels reached by the settling pond and the wall crest are close, but there is always a free board above the forced water level, as shown in the following graph. This figure illustrates that there is sufficient freeboard to retain water and tailings in the tailings pond.

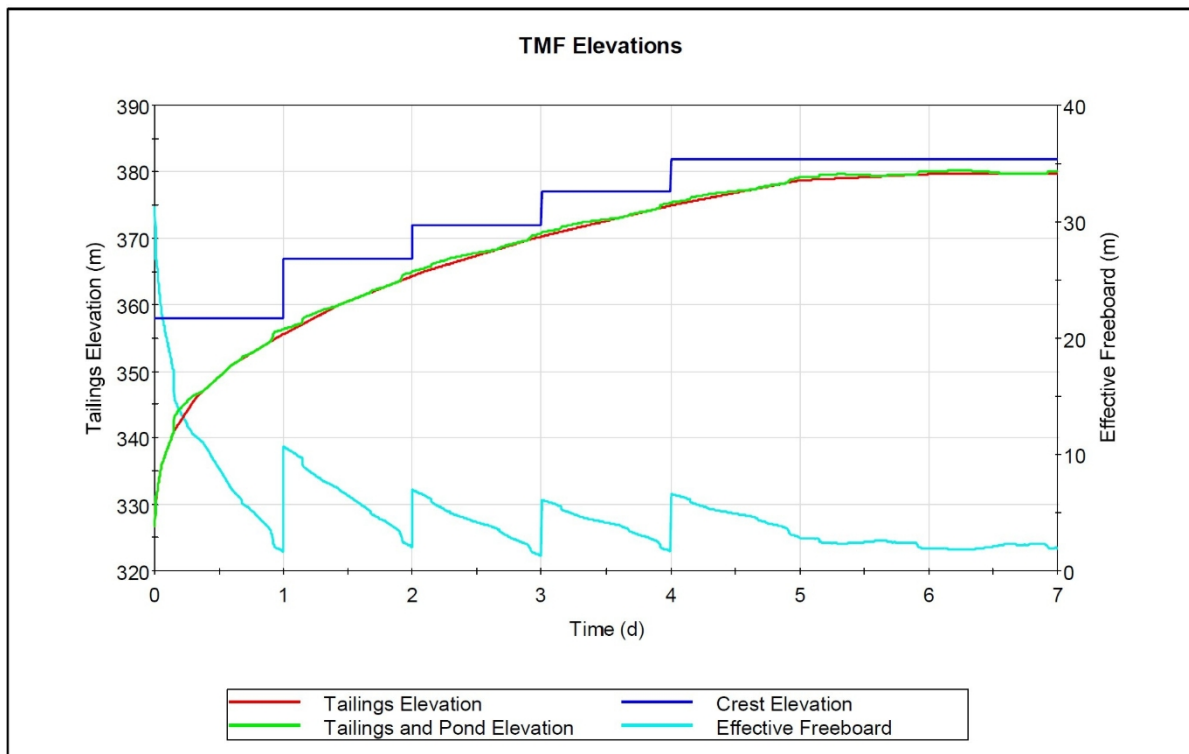


Figure 14: Level in the sedimentation pond in the case of a wet year



### 5.4.5 Summary of water balance scenarios

The following three diagrams illustrate the average flows (m<sup>3</sup>/d) and storage volumes (m<sup>3</sup>) for a dry, average and wet year:

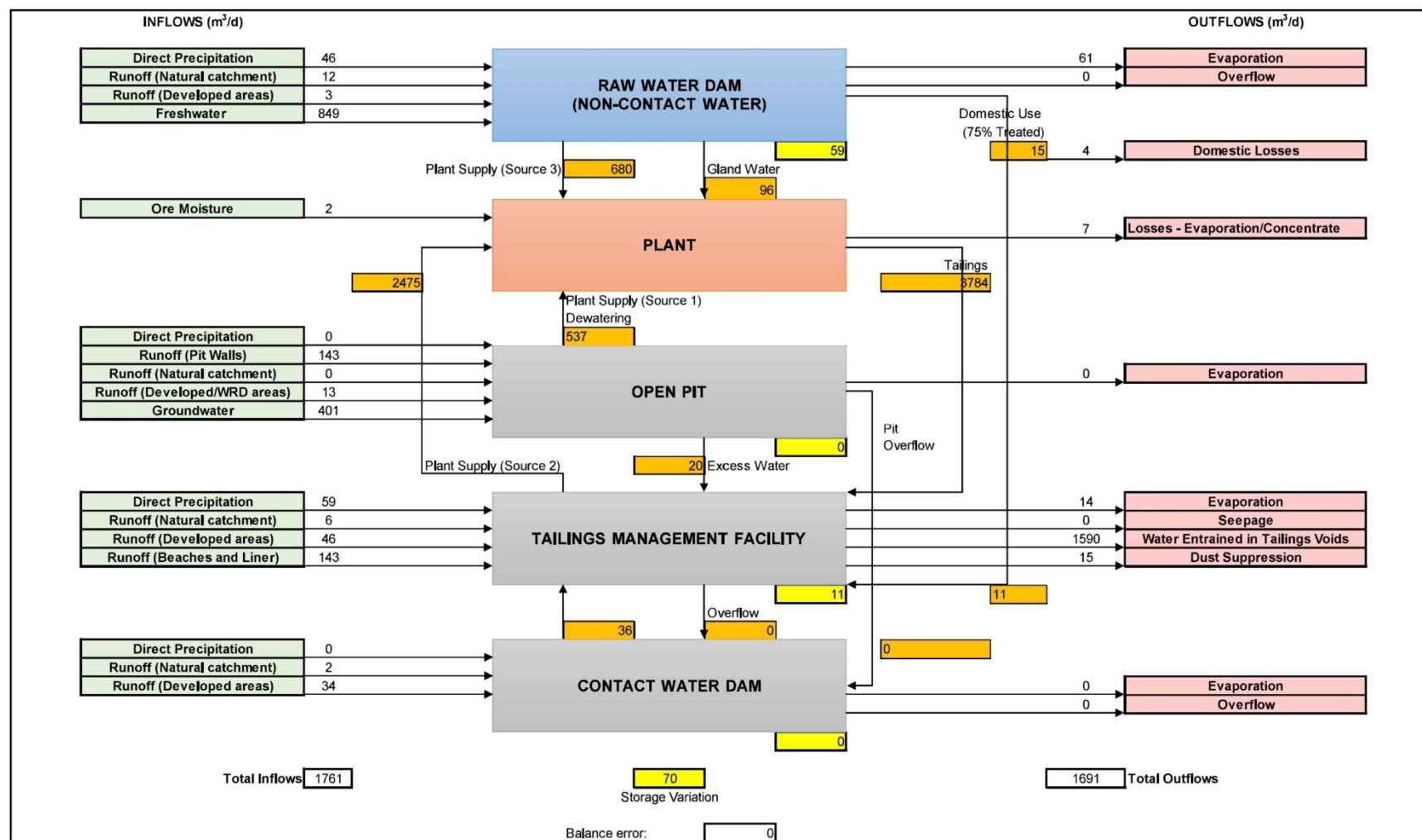


Figure 15: Water balance in the event of a dry year

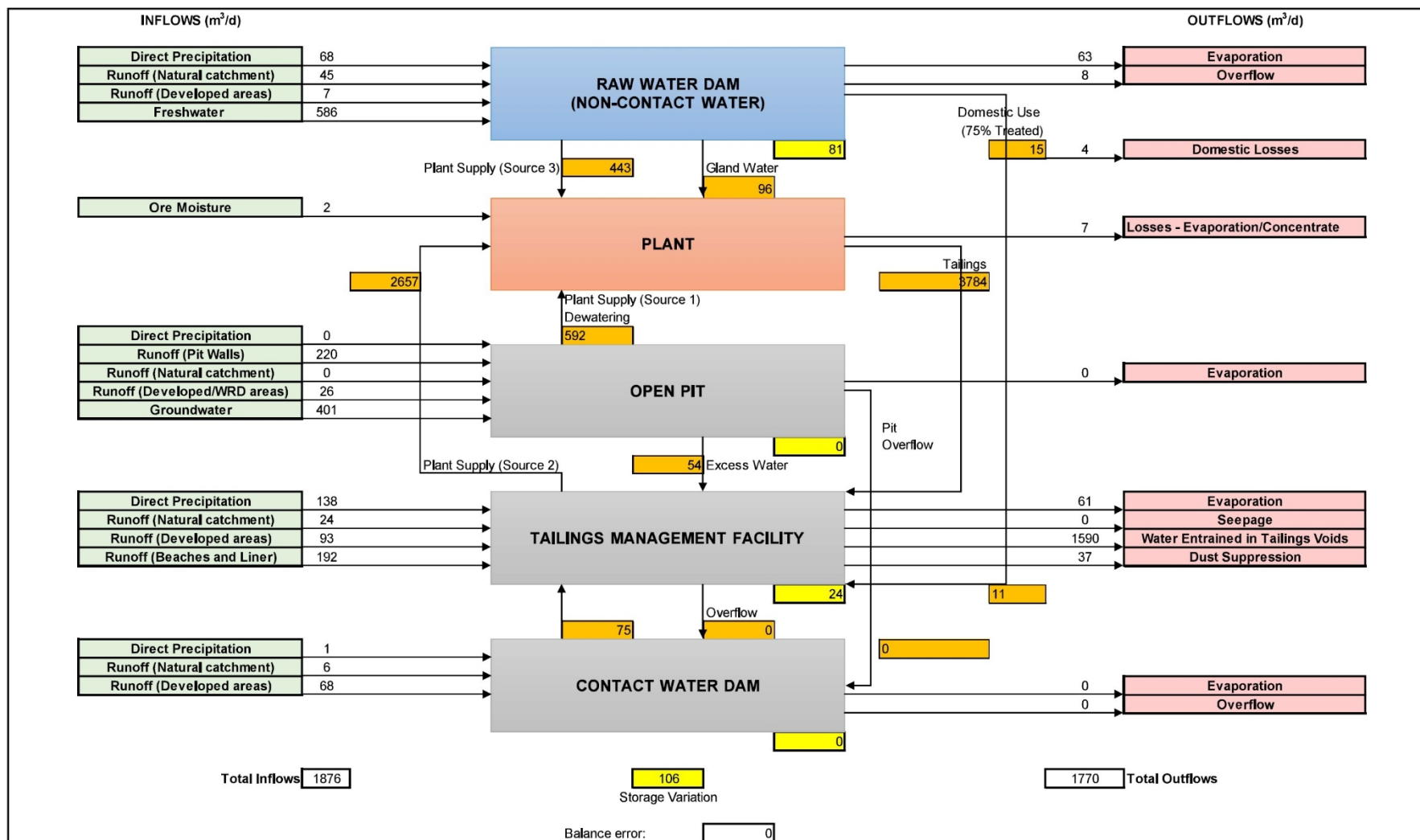


Figure 16: Water balance in the case of an average wet year



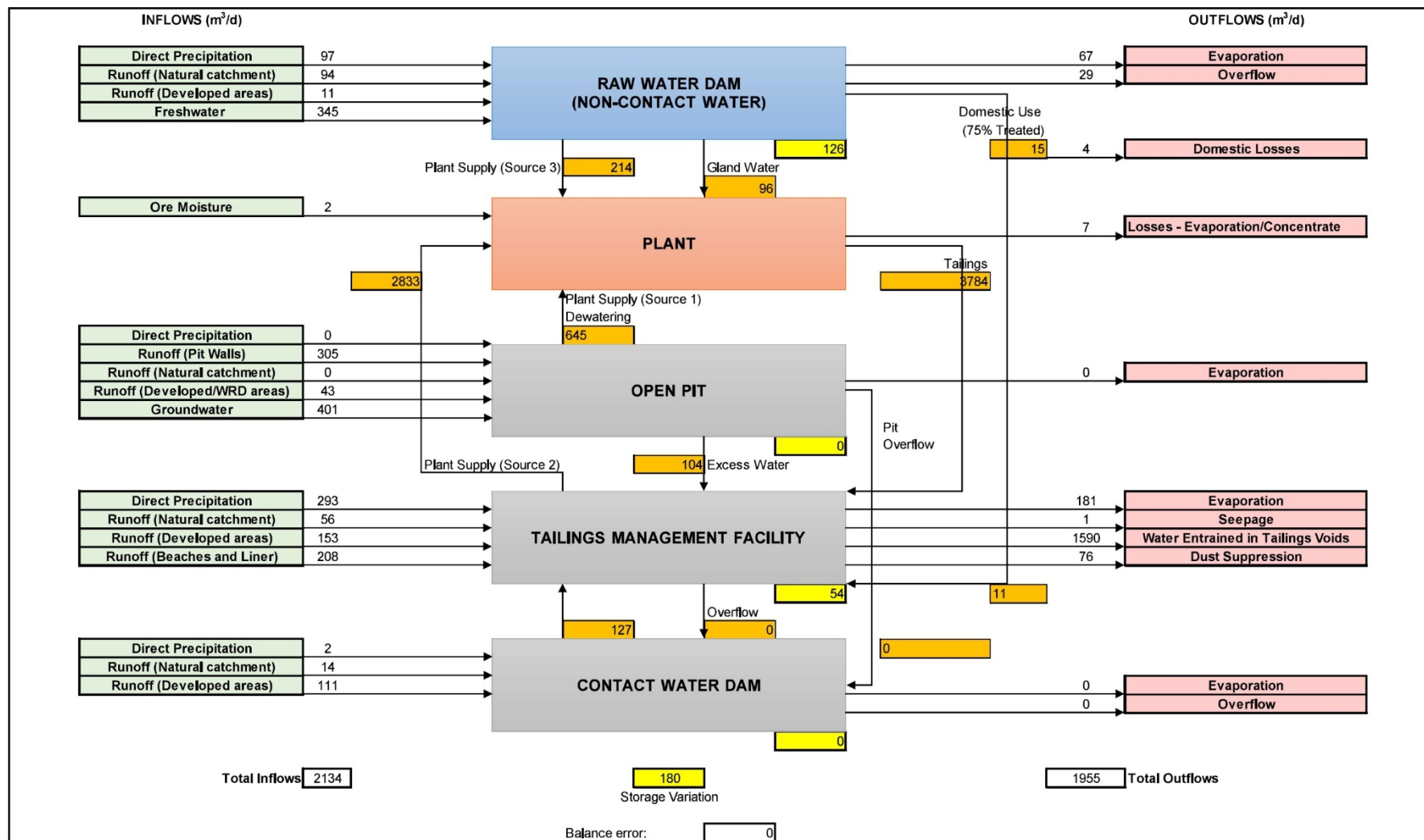


Figure 17: Water balance in the case of a wet year

The graphs show that integrated water flow management is sufficiently well balanced and even in the case of a wet year, "zero discharge" to the environment is achieved. For all three graphs, the outflows from the contact water lake are zero.

## 6.0 CLASSIFICATION OF FACILITIES

### 6.1 Classification of facilities according to national legislation

According to the national legislation in force, the water storage reservoirs and the tailings pond are HTS facilities and, according to document "Standards for the design of hydraulic engineering facilities. Basic provisions" of 13 December 2012, their class and the corresponding design safety factor of their spillway facilities are specified.

**Table 28: Classification of main HTS facilities**

Facility	Tailings storage facility TMF	Contact water reservoir water, CWD	Fresh water tank water, RWD
Rock geological base:	Weathered/fresh gneisses	Weathered gneisses	Weathered gneisses
Filtration coefficient:	1E-6 m/s	1E-6 m/s	1E-6 m/s
Height of the facility:	67 m	12 m	36 m
Volume of stored tailings	6.125*1E+6 m <sup>3</sup>		
<b>Class of facility</b>	<b>Second class, II</b>	<b>Fourth class, IV</b>	<b>Third class, III</b>
Minimum security for spillway dimensioning	<b>0.1%</b> , (1 per 1000 g)	<b>1%</b> , (1 in 100 g)	<b>0.5%</b> (1 in 200 g)

At the next point, the contact water tank is checked in addition to the regulatory minimum of 1% and 0.1%, which ensures that the entire contact water storage system has the necessary retention capacity for a valid event with a probability of 1 in 1000 years.

### 6.2 Checking the retention capacity and free height

As shown in the previous section of the report, each facility is designed with a free height between the forced water level and the crest elevation. The following table provides an assessment of the retention capacity of each facility, assuming that the emergency spillways are sealed and not operational.

**Table 29: Estimated retention capacity**

Parameter	Tailings storage facility, TMF			Contact water reservoir, CRD	
	Stage	Stage 2	Stage 3	Stage 1	
Security	0.1% (1 in 1000)			1	0.1
Crown elevation	360	375.0	384	311	311
Elevation/water level, m	358	373.5	383	308	306.2
Starting height, m	2	1.5	1	3	4.8
Free height, m	<b>0.37</b>	0.77	<b>0.5</b>	0.3	1.9

Where: "Initial height" is the height between the water level at the tail end when the emergency event starts, "free height" is the difference between the forced water level and the crest elevation.

As can be seen from the table above, each facility has the capacity to accommodate the additional water masses that would be directed to its volume as a result of a significant event with the corresponding regulatory security. It should be noted that the calculations are made based on the following assumptions:

- The starting level in each reservoir is assumed to be the maximum working water level.
- The discharge coefficient is assumed to be 0.90;

The assumptions are in favour of safety and ensure a conservative assessment of the retention capacity.

It should be noted that the contact water tank has a minimum free height capacity of 0.30 m, provided that the water level starts to rise from elevation 308, which corresponds to a theoretical volume of 25,000 m<sup>3</sup>. If we take the calculated water volume according to the water balance of 14,000 m<sup>3</sup>, which corresponds to elevation 306.2 m, it turns out that the reservoir is able to handle a significant event of 1 in 1000 years and still have a significant reserve in height up to the crest elevation.

### 6.2.1 s to CWD

The following graph shows the curve of the stored volumes for the contact water tank (CWD):



Figure 18: Curve of the stored volumes in the CWD head, [m<sup>3</sup>]

The curve shows that the potential of the sump is large and with only a one-metre increase in the crest level, we gain an additional 13,000 m<sup>3</sup> of free volume. We recommend that the next design phase assess the possibility of increasing the crest elevation to 312 m, which will ensure reserve capacity for a 0.1% event even for water levels of k.308 (25,000 m<sup>3</sup>).

## 7.0 WATER FILTRATION MANAGEMENT

### 7.1 Introduction

The filtration water management strategy is based on experience from previous projects, the requirements of the BD and the requirements of Tintyava Exploration AD for continuous improvement of the design process. An integrated approach is also applied here, with filtration water being divided into contact and non-contact water according to the following principle:

- Contact filtration water: this is all water that has infiltrated from the contact water storage facilities, including all natural groundwater in these areas;
- Non-contact filtration water: this is all water that has infiltrated deep into the fresh water storage facility;

The management of non-contact filtration flows is only hinted at in this study, as they are not defined as an environmental load and, although an important factor, will be considered in the next phases of the project.

The subject of this study is the basic configuration and main design criteria for the main facilities for controlling contact filtration water. The following diagram shows a basic diagram for the management of filtration water:

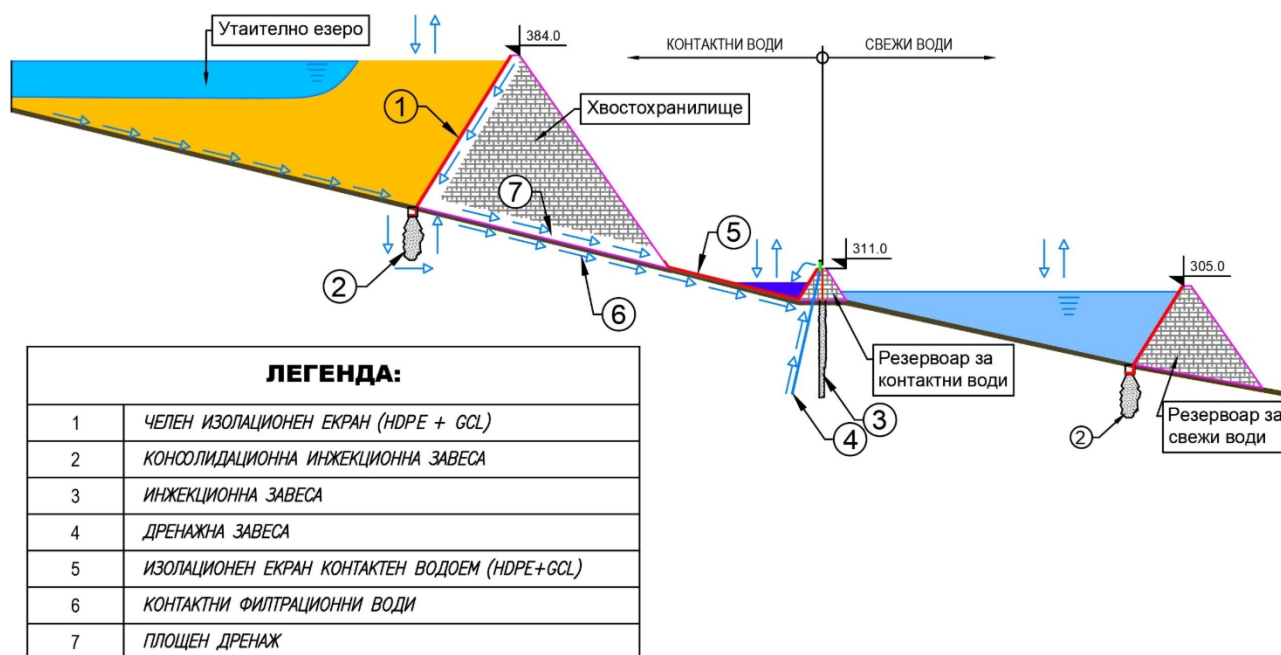


Figure 19: Basic diagram for the management of filtration water

It should be noted that, in order to reduce the amount of infiltrated contact water, a decision has been made to line the bottom and walls of the contact water tank with an insulating screen (Position 5). In principle, no filtration water should seep from the tank, except in the event of a membrane defect. Any water from possible leaks will join the flow of filtration water coming from the tailings storage facility.

The amount of filtered water from the tailings pond will be the main load on our facilities. The flow rate will be controlled by performing consolidation grouting at the foot of the tailings pond wall (Position 2). The depth and number of grouting rows will be determined at the next design stage, taking into account the specific geological conditions at the wall section. Consolidation grouting must be carried out along the entire length of the anchor groove of the screen so that the necessary primary barrier against infiltration is formed. The minimum depth of the <sup>first</sup> row of injection boreholes is 15.0 m.

The contact filtration water passing through any breaches in the screen (Position 1), as well as the filtration water under the base of the facility, will enter the contact reservoir either directly through the area drainage (Position 7) or after being captured by the drainage curtain (Position 4) and pumped back into the contact water reservoir.

The filtration water that has passed through the curtain will be stopped by the injection barrier (Position 3). The injection curtain is not only a barrier to the contact filtration water, but also a barrier to the non-contact water. It prevents the two flows from mixing, which is why we can say with certainty that the contact filtration flows are limited along the axis of the CWD tank.

In the plan, the two curtains (drainage and injection) run along the axis of the crown wall and continue into the slopes at least up to elevation 340 (see Appendix A) in order to create a reliable barrier for bypass filtration from the tailings storage facility. Given its remoteness and the low permeability of the consolidated tailings ( $1.0\text{E-}6 \sim 1.0\text{E-}7$  m/s), no significant filtration pressure from bypass filtration is expected.

In the next stage of the project, the dimensions and depth of the curtain will be determined based on a specific geological picture and a comprehensive analysis of the two facilities – the tailings pond and the contact water reservoir.

The following lines set out the basic principles and guidelines for the design and implementation of the planned injection and drainage curtains.

## 7.2 e injection curtain

The main tasks of the anti-filtration injection curtain are:

- to reduce the water permeability of the rock mass to values below 1 Lyujon (filtration coefficient  $K_f \leq 1 \times 10^{-7} \text{ m/s}$ ) in this rock mass, where the water permeability is greater than 1 Lyujon;
- to stop the filtration of water through the rock mass towards the Yuren Dere valley, which will prevent the mixing of filtered contact water with non-contact water downstream the stream and
- prevent the possibility of concentrated filtration currents (suffusion) forming in the rock base along the axis of the injection curtain.

The main elements of the anti-filtration curtain are: width, depth and length.

The required minimum width of the curtain will be ensured by two rows of vertical boreholes, with a distance of 0.5 m between the rows. With the expected (accepted) radius of propagation of the injection solution for this rock – 1.25 m, on either side of the axis of the two rows, the injected rock mass will be compacted to a minimum width of 0.5 m. In this case, the width of the injection curtain will be:  $0.5 \text{ m} + 2 \times 0.5 \text{ m} = 1.5 \text{ m}$ .

The depth of the injection curtain must be in line with the requirements for it and the water permeability of the rock mass. The anti-filtration curtain will penetrate practically impermeable rocks with a water permeability of 1 Lyuzhon. Since the pressure gradient decreases with increasing depth, the design width of the curtain does not need to maintain its width throughout its entire depth. Only the first series of boreholes from the first row will reach the assumed design depth of 30 m. The second and third series of boreholes from the first row are shorter by 1 injection interval (5 m), and the boreholes from the second row are shorter by two injection intervals (10.0 m). Therefore, it can be recommended that the design width of the curtain of 1.5 m is ensured at a depth of 20 m.

At this stage of the project, the length of the injection curtain is planned to be 340 m. It covers the Yuren Dere valley from the left to the right slope at an elevation of 340 m. The approximate total length of the drilling for the two rows is 6,340 m. The maximum number of boreholes in each row is 137. The water permeability requirement for the completed injection curtain is 1 Lyuzhon and will be verified by control boreholes. These represent an additional volume of approximately 10% for drilling. A schematic profile along the length of the curtain (including drainage) is shown in the following graph, and a plan of the curtain is presented in Appendix A.

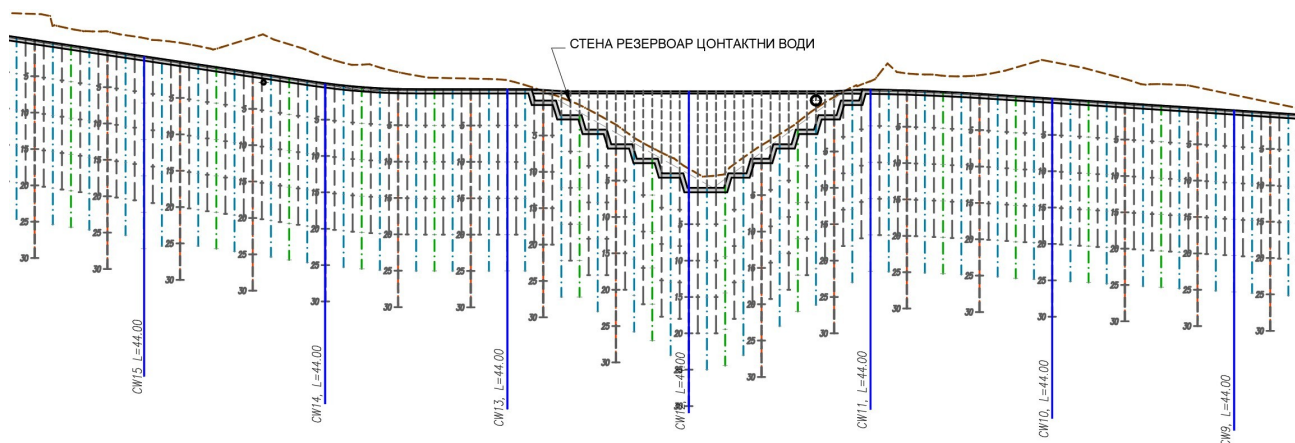


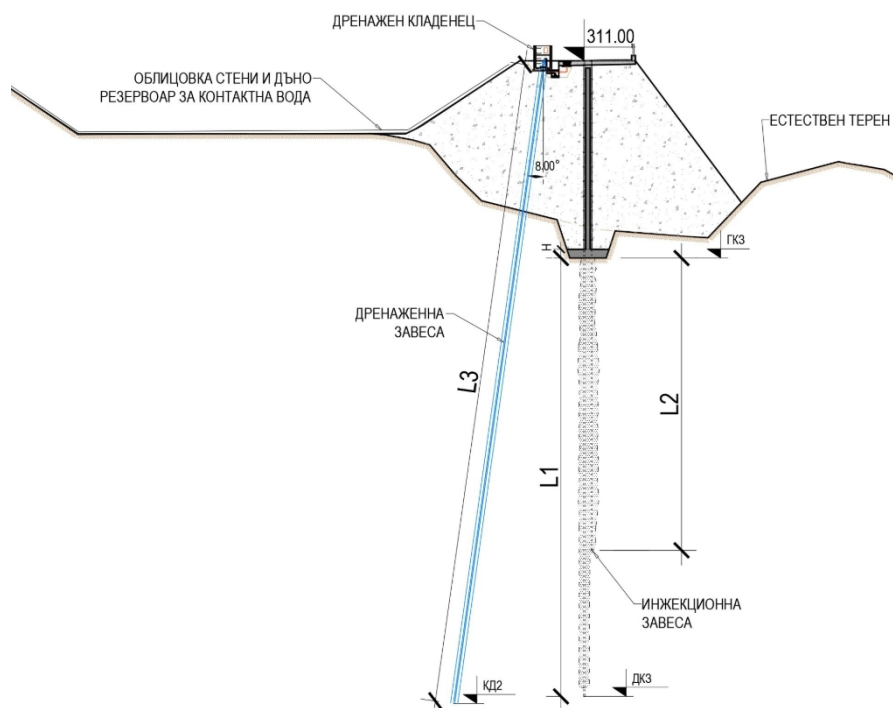
Figure 20: Drainage and injection curtain. Typical longitudinal section

Each row and each borehole of the injection curtain shall be marked on site with the numbering corresponding to the Project. The sequence of execution of the rows requires that the first row of the curtain be executed first, followed by the second row. The boreholes in the rows will be executed using the "sequential compaction" method, in three sequences - first, the first sequence of boreholes (every 10.0 m) will be executed, then those from the second sequence, which are located between the boreholes from the first sequence - at a distance of 5 m. Finally, the boreholes from the third sequence will be drilled, which halve the distance between the first and second sequence boreholes to 2.5 m. The following table presents an approximate specification of the planned injection curtain:

**Table 30: Specification of injection curtain boreholes**

Injection borehole		Depth, m	Distance in plan	Total number	Total length, m
I <sup>vi</sup> row	I <sup>va</sup> sequence	30	10	35	1050
I <sup>vi</sup> row	II <sup>ra</sup> sequence	25	10.	35	875
I <sup>vi</sup> order	III <sup>rd</sup> sequence	25	5	69	1725
II <sup>nd</sup> place	I <sup>va</sup> sequence	20	10	35	700
II <sup>nd</sup> order	II <sup>ra</sup> sequence	20	10	35	700
II <sup>nd</sup> row	III <sup>rd</sup> sequence	20	5	69	1380

It is important to note that the transition from one sequence to the next will take place after the need for its implementation has been established. Based on the results of the water test and the injection of the boreholes from a given sequence, the depth of the boreholes from the next sequence will be further specified. No drilling of boreholes from one sequence will be performed until all boreholes from the previous sequence have been injected in a specific section of the curtain.



*Figure 21: Drainage and injection curtain. Typical cross-section*

The number and lengths of the boreholes indicated in the table above are the maximum envisaged if the water samples show that the criterion of 1 Lyujon has been achieved, the injection in a given sequence shall be discontinued.



### 7.3

The drainage curtain consists of 16 boreholes (spaced 20-25 m apart) that operate as a single system pumping the filtered water into a common collector that carries the water to the CWD contact water lake. At this stage of the project, the drainage wells are planned to be constructed to a depth of up to 45 m.

The drainage curtain is constructed parallel to the injection curtain in plan but at an angle of 8-10 degrees to the vertical (see Figure 21). In the direction of flow, the drainage curtain is constructed before the injection curtain and is located between it and the contact water pond.

Each well in the drainage curtain must be equipped with a drainage pumping station. Each of the pumping stations is designed to pump the incoming water quantities into the rock mass in front of the anti-filtration curtain. For preliminary analysis, the flow rate of the pumps can be determined as 0.35 l/s at a head of 45 m. They are expected to operate for several hours a day, or the water flow from the drainage curtain will be 40-50 m<sup>3</sup>/day.

In structural terms, the borehole is encased in steel pipe for the first 1.5-2 m, after which the space between the pipe and the borehole walls is filled with cement mortar to a depth of 4 m. In total, the first 5~10m are filled with non-perforated pipe. The water intake section is the next 35m, in which a perforated PVC pipe and a washed sand filter are laid. A schematic diagram of the drainage well is shown below:

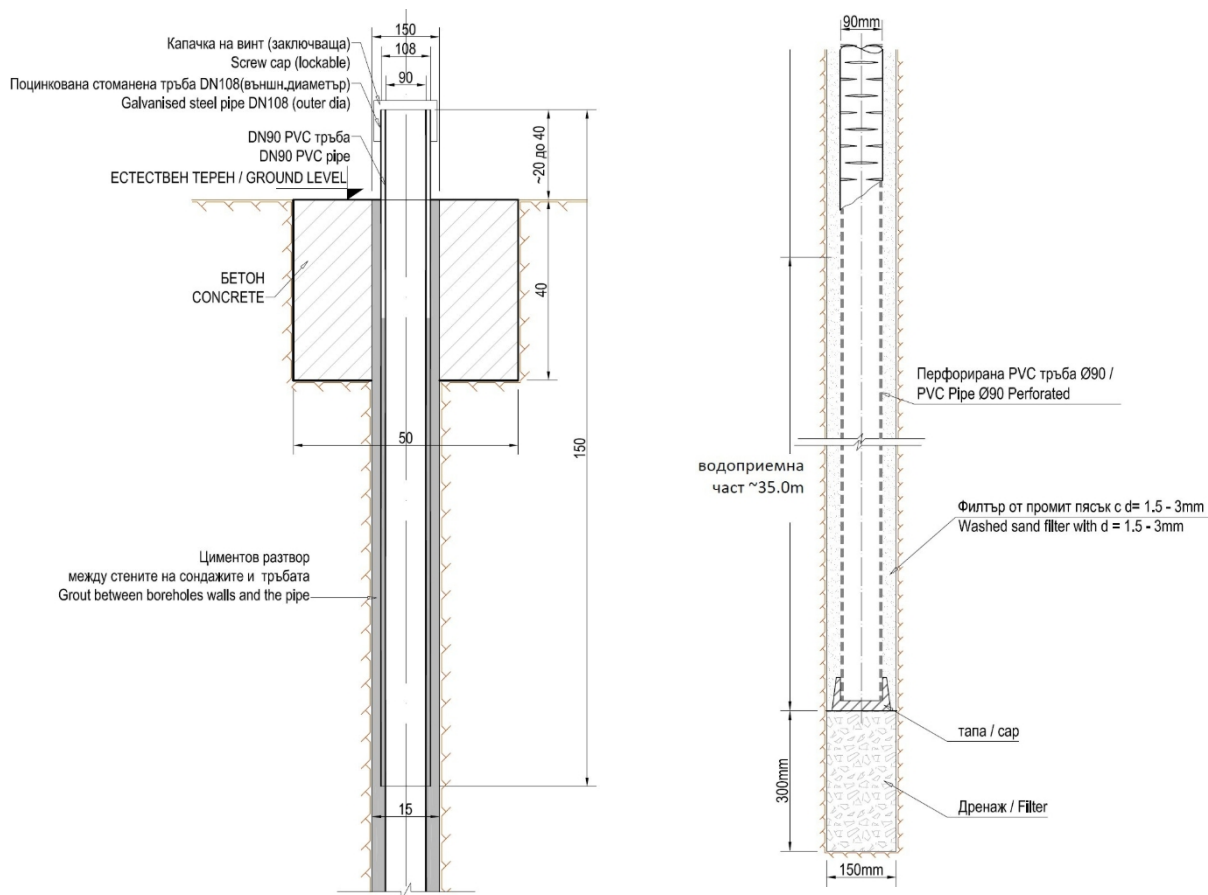


Figure 22: Drainage well, details for the upper and lower ends

The layout of the curtain is shown in Appendix A.

## **8.0 REFERENCE DOCUMENTS**

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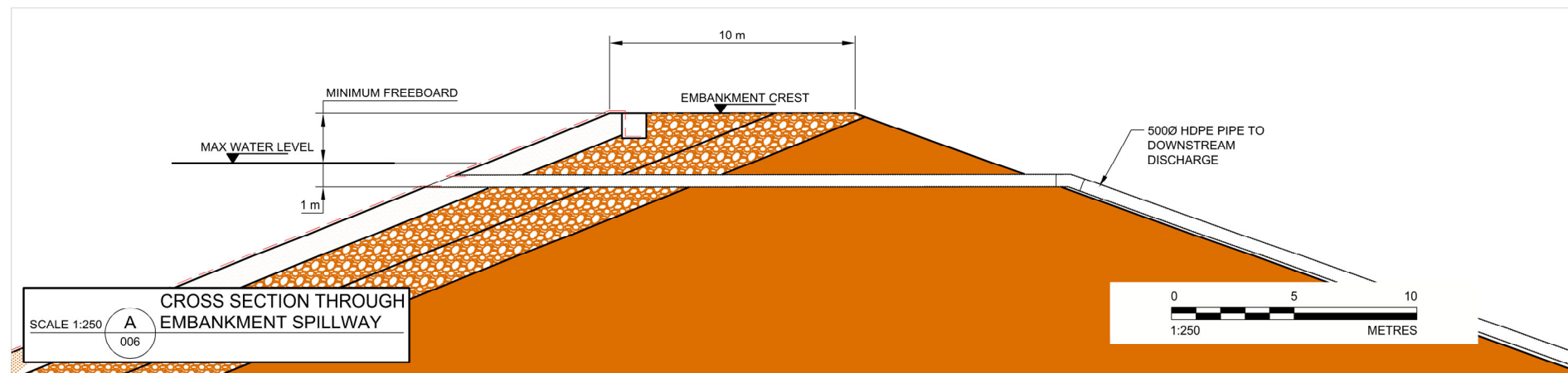
# APPENDIX A

## Drawings





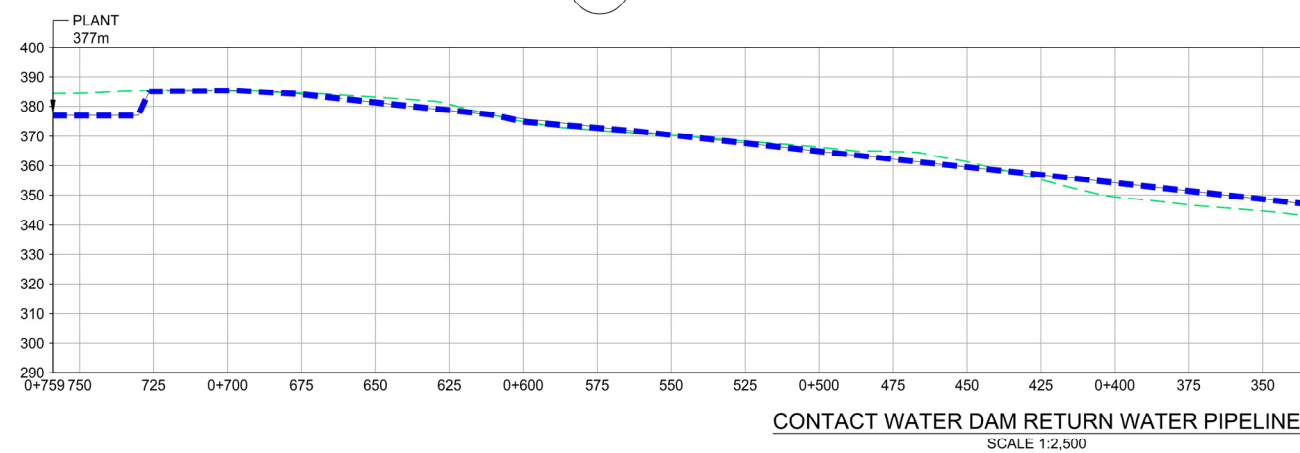
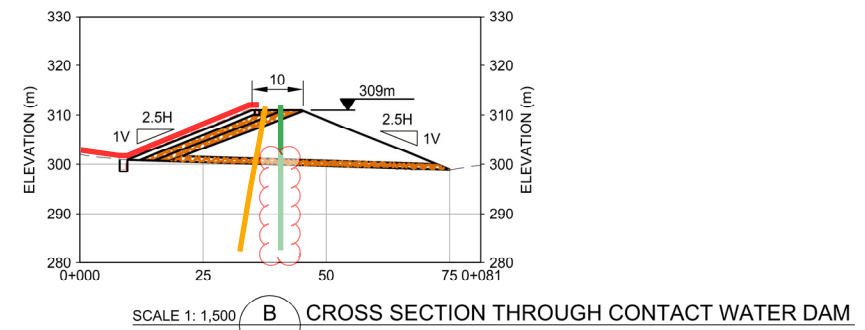
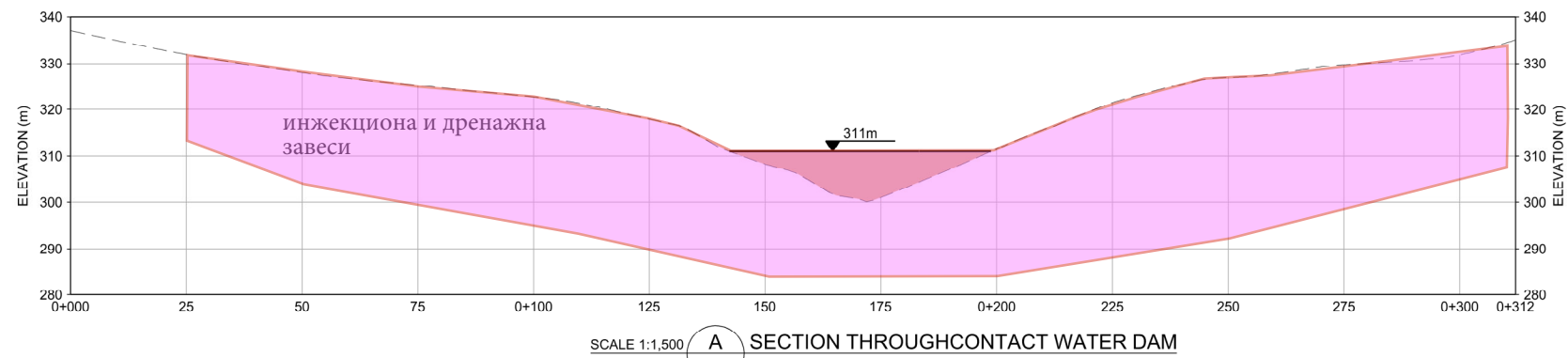
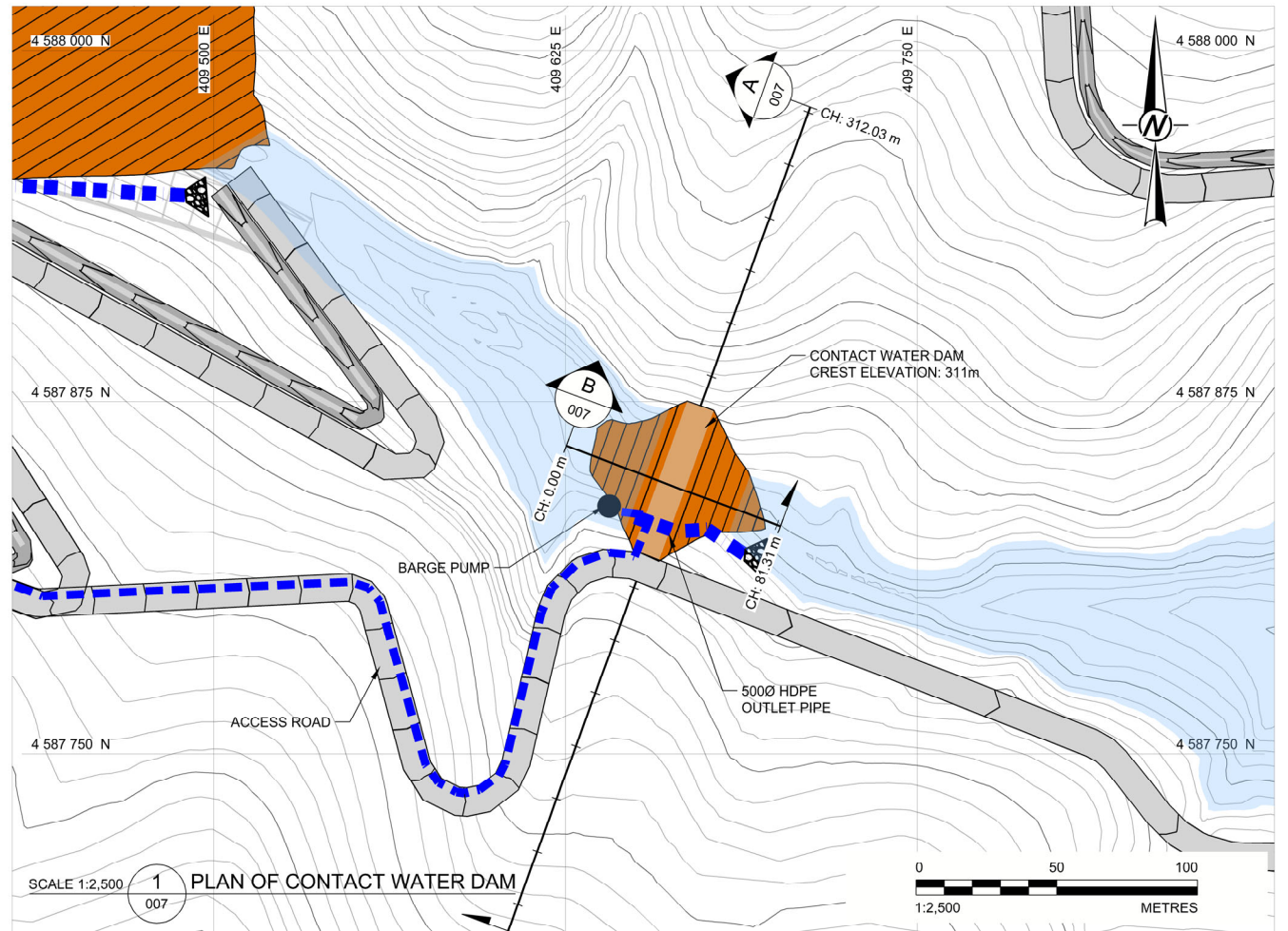
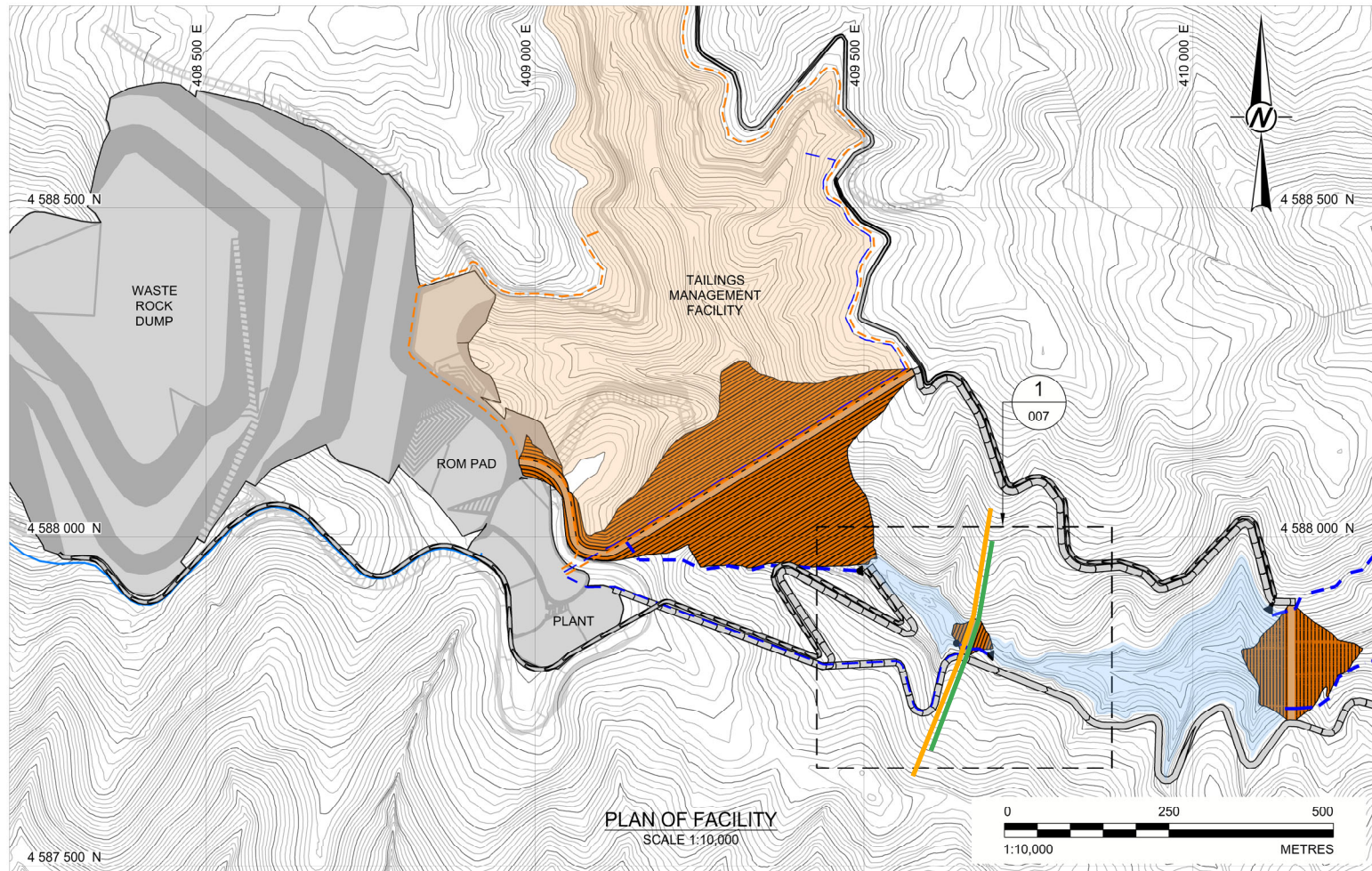




PROJECT NO.	CONTROL	REV.	DRAWING
19127003	3005-PFS	A	006



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- LEGEND**
- EMBANKMENTS AND FACILITIES
  - POND
  - TAILINGS
  - RETURN WATER PIPELINE
  - EMERGENCY OUTFALL

- NOTES**
- DO NOT SCALE FROM THIS DRAWING
  - ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE INDICATED
  - ALL CONTOURS SHOWN AT 2 m AND 10 m INTERVALS.
- дренажна завеса
  - инжекционна завеса
  - изолационне екран

CLIENT  
VELOCITY MINERALS  
TINTYAVA EXPLORATION AD.

CONSULTANT



YYYY-MM-DD 2020-09-25  
DESIGNED HO  
PREPARED MR  
REVIEWED DB  
APPROVED XXX

PROJECT  
ROZINO PRE-FEASIBILITY STUDY

TITLE  
**CONTACT WATER DAM  
GENERAL ARRANGEMENT & SECTIONS**

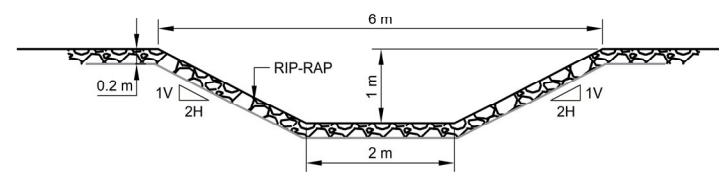
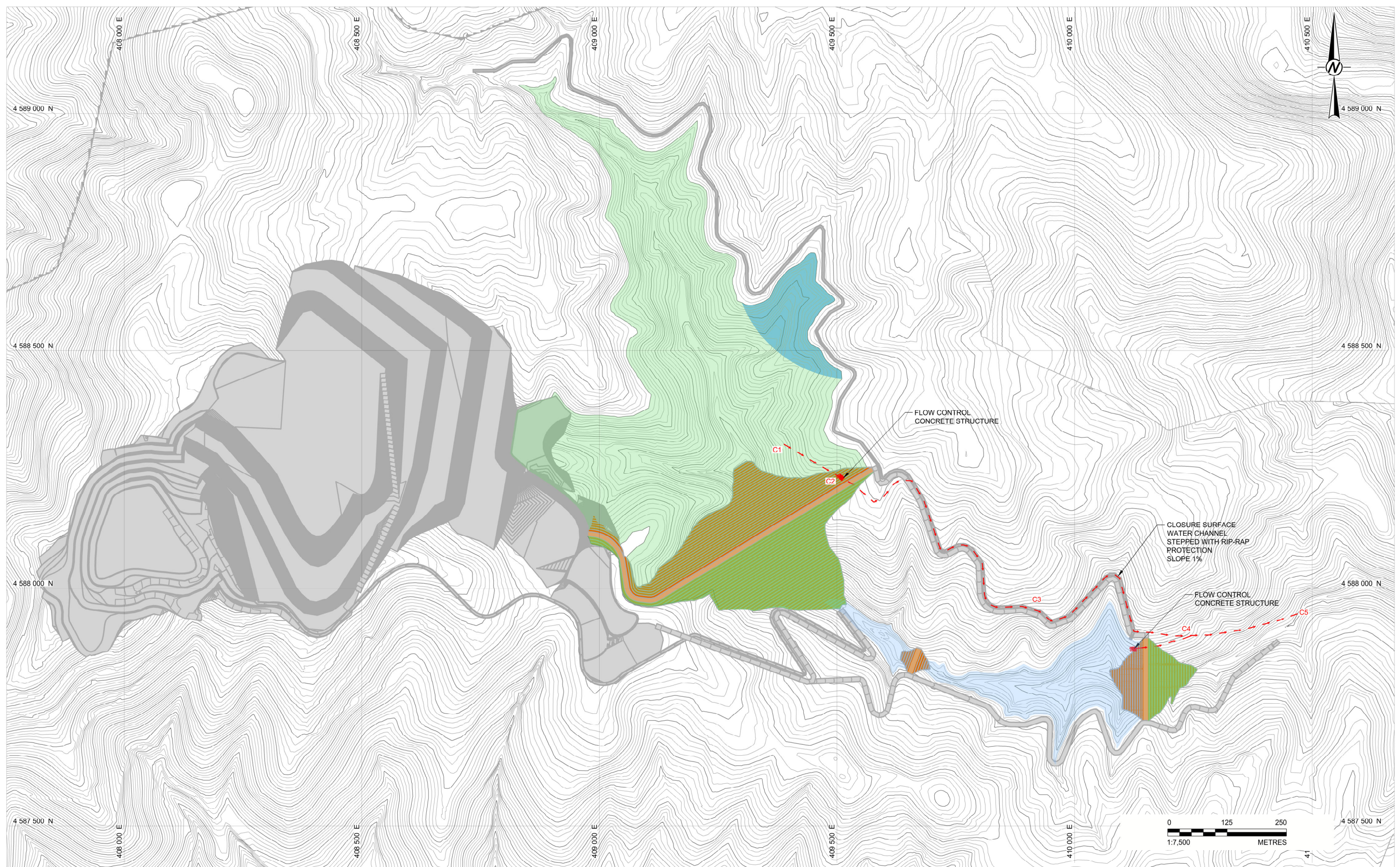
PROJECT NO. 19127003  
CONTROL 3005\_PFS  
REV. A  
DRAWING 007

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A3









TYPICAL SECTION THROUGH CLOSURE CHANNEL  
SCALE 1:100

## NOTES

1. DO NOT SCALE FROM THIS DRAWING
2. ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE INDICATED
3. ALL CONTOURS SHOWN AT 2 m AND 10 m INTERVALS.

### LEGEND



CLOSURE CHANNEL ALIGNMENT

CLIENT  
VELOCITY MINERALS  
TINTYAVA EXPLORATION AD.

CONSULTANT



YYYY-MM-DD	2020-09-25
DESIGNED	HO
PREPARED	MR
REVIEWED	DB
APPROVED	XXX

PROJECT  
ROZINO PRE-FEASIBILITY STUDY

TITLE  
TAILINGS MANAGEMENT FACILITY  
CLOSURE PLAN

PROJECT NO.  
19127003

CONTROL  
3005\_PFSREV.  
ADRAWING  
010



# APPENDIX B

## Enhanced evaporation facility. Sample supplier



## TURBOMISTER MINI Model EV24



Paint over stainless steel standard



Scotchkote 134 finish optional



- 25 Horsepower 460volt 60hz electric motor
  - Available in 380,415,440,525 and 575 Volts & 50hz
  - Up 30 Spiral jet telfon nozzles
  - Turbomist Quick connect stainless steel nozzle ring
  - Starter & control panel with off/on evap & optional pump
  - Lockable wheels and multi position wind tunnel
  - Up to 62 gpm volume at 90 psi
  - Throws waste to 40 feet high out 60-70 feet. Ideal for small ponds and lower volumes than the EV30 model
  - Unit must be crated to ship internationally.
  - Optional Scotchkote 134 powder coat finish is good for ph of 2 to 11, a must option in caustic waste. (\$2400.00 upgrade)
- 25 Мощност 460 volt 60hz ел.двигател
  - Предлага се в 340,415,440,525 и 575 Волта и 50Hz
  - До 30 Спираловидни струйни тefлонови накрайници
  - Връзката с Turbomist е посредством бърз неръждаем стоманен пръстен
  - Стартер и Пулт за управление с изключване / включване и опция помпа
  - Заклучващи се колела и няколко позиции на аеродинамичната тръба
  - До 62 gpm при 90 psi
  - Изхвърляне на отпадъка до 12.192m във височина, дължина 18.288m до 21.336m подходящ за малки резервоари и по-малки обеми в сравнение с модела на EV30
  - Да се създаде контакт за международно доставяне
  - С опция за прахово покритие е добро за Ph от 2 до 11 възможна е опция за отпадък замърсен със сода каустик. ( доплащане 2400\$)



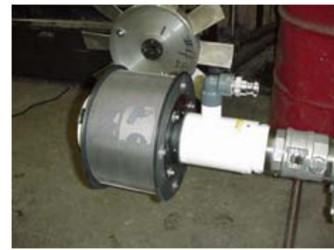
- Price is \$36,500 including pump and intake system outlined below
- Stainless submersible 4" J Class sandhandler
- 5 Hp pump includes self cleaning filter intake system, 50 feet of intake hose
- Includes stainless steel skid to sit submersed in the pond.
- Volumes to 62 gpm at 90psi
- With this option the starter is wired into Evap control panel
- Цената е 36 500 щатски долара включително помпа и смукателна система, описана по-долу
- Неръждаема потопяема 4" J Class sandhandler
- 5 Hp помпа включваща самопочистващ се филтър, 15.24 m водоземна част
  - Включва приспособление за спускане от неръждаема стомана за потапяне в резервоара
  - Обем от 62 gpm до 90psi
  - Стартера е изтеглен в контролния панел на Evap.



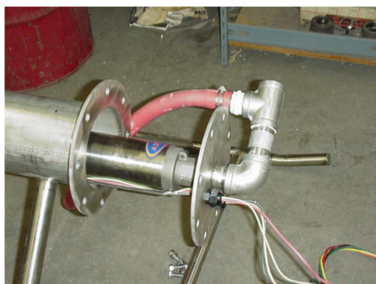
Overview / Общ изглед



Quick connect / Бърза връзка



Self cleaning filter / Самопочистващ се филтър



Pump 89,280 US gallons/day  
 This amounts 2,678,400 per month  
 At 30% efficiency that is over 800,000 gallons evaporated per month. Efficiency varies by climate but 30% is normally minimum obtained by 98% of clientele



Помпа 89,280 US gallons/day или 338 m<sup>3</sup> / ден

При 30% ефективност това е над 800 000 галона изпарени на месец (3028 m<sup>3</sup> / месец). Ефективността варира в зависимост от климата, но 30% обикновено е минимум, получен от 98% от клиентите.