

DATE 11 July 2013**PROJECT NO.** 13514250010.513/B.0**TO** Project File**CC** Gareth Digges La Touche**FROM** Eoghan Hayes**EMAIL** ehayes@golder.com**AMULSAR GOLD PROJECT: FLOOD RISK ASSESSMENT**

1.0 INTRODUCTION

This Technical Memorandum documents the risk arising from natural flooding of the Vorotan river and minor streams draining sensitive site areas of Amulsar Mine, Armenia. In order to assess the risk of inundation at the Waste Dump Facility (WDF) and Heap Leach Facility (HLF) sites, hydraulic and hydrologic modelling has been conducted. The modelling approach and assumptions are first set out, followed by some discussion of the results and likely risk.

2.0 MODELLING APPROACH

The WDF and associated infrastructure are located in a hanging valley which is drained by a minor tributary of the Vorotan river. The WDF is located well above the flood plain of the Vorotan river but is sufficiently close to the Vorotan river that it has been deemed appropriate to build a simple HEC-RAS model to identify the flood plain extents for major flood events.

The HLF is drained by two minor ephemeral streams which are tributaries to the Arpa river, eventually draining to the main channel of the Arpa south of Gndevaz village. Simple hydrologic modelling has been conducted to identify the peak flows to be expected through the natural channels for major return period events. The site is approximately 500 metres (m) vertically above the nearest point on the Arpa river and not considered at risk from that source.

A number of different data sources were used in estimating the flood flows for both sites, both through direct interpretation of surface water flow records and analysis of available rainfall records to generate return period rainfall intensities for runoff estimation.

A long term record was obtained for the Borisovka gauge on the Vorotan river (now submerged beneath the Spandaryan reservoir) along with additional information for a newer gauge immediately upstream of the Spandaryan reservoir at Gorayk. The annual instantaneous flow rate maxima for these gauges are compared to the estimates derived from the process described below for validation purposes.

In the absence of definite flow record data, a simple rational method calculation is used to provide a probabilistic measure of peak flows in the local catchments using rainfall intensities derived from a number of sources. Values for 100 year return period and 1000 year return period storm intensities for the critical period were estimated.

2.1 Catchment Parameters

Catchment delineation and physical parameter estimation for the purposes of the modelling was based on-site observations, soviet era mapping and a digital elevation model (DEM) of the catchment areas. Drawing 1 which forms an appendix to this report shows the modelled catchments at the proposed WDF and HLF locations. This drawing also indicates the overall Vorotan valley catchment at the current location of the Gorayk flow gauge.



Values for catchment area, channel length and elevation were measured using ArcGIS and used to calculate the time of concentration for the Site using the Bransby-Williams method. This value represents the response of our study catchment to a rainfall event; essentially how long it takes for storm runoff to propagate from the upper areas of the catchments to the study sites. A uniform runoff coefficient of 0.4 was estimated for the catchment based on the steep slopes and thin cover, further increased by transitional frozen ground in the late spring when peak flows are most likely to occur.

Table 1: Vorotan Catchment Parameters (Rainfall-Runoff)

	Vorotan (Gorayk)	Vorotan (Modelled)	WDF Site
Catchment Area	160 Km ²	124 Km ²	4.95 Km ²
Runoff Coefficient	0.4	0.4	0.4
Channel Length	28 km	19.2 km	3.2 km
Elevation (Upper)	3240 masl	3240 masl	2450 masl
Elevation (Lower)	2100 masl	2250 masl	2250 masl
Linear Profile Slope	24.6	19.4	15
Time of Concentration	2.15 hours	1.59 hours	23.2 minutes

Table 2: HLF Catchment Parameters (Rainfall-Runoff)

	Northern Catchment	Southern Catchment
Catchment Area	2.88 Km ²	4.93 Km ²
Runoff Coefficient	0.4	0.4
Channel Length	3.4 km	4.9 km
Elevation (Upper)	2500 masl	2500 masl
Elevation (Lower)	2000 masl	2000 masl
Linear Profile Slope	6.8	9.8
Time of Concentration	30.3 minutes	38.8 minutes

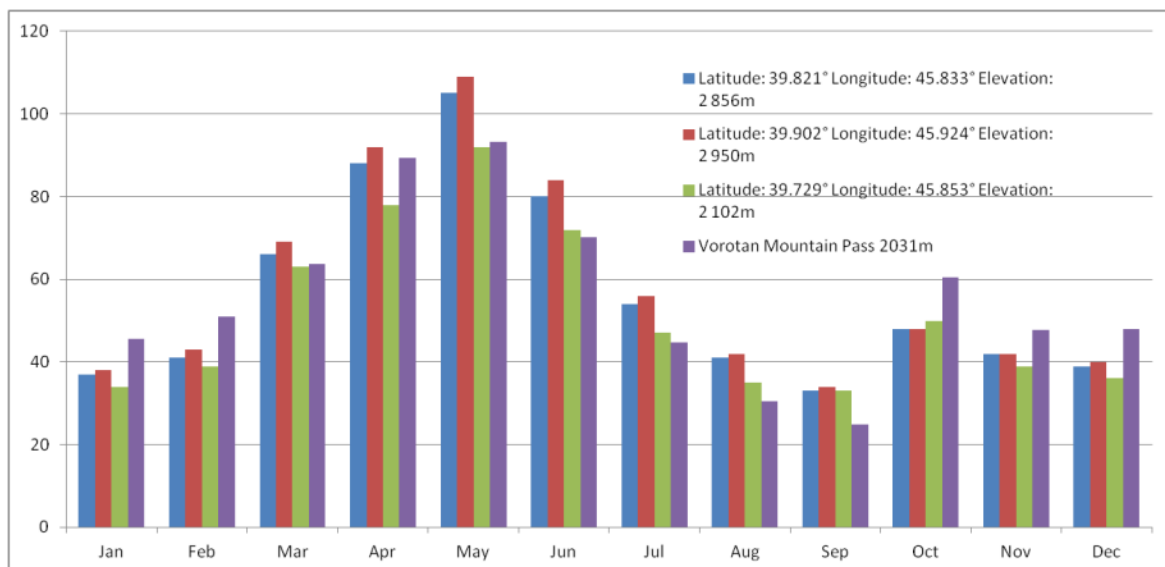
2.2 Rainfall Estimation

The rainfall record data for Vorotan Pass consists of daily rainfall totals only and describes a location at a lower altitude than the majority of the study catchment. This 40 year daily rainfall record is adequate to perform a frequency analysis and estimate the 100 year and 1000 year rainfall intensities for the critical storm at the project area by fitting to a number of distributions, the results of which are tabulated in Table 4.

As numerous sources were available for monthly precipitation averages in the region, this measure was used as a proxy for comparison to catchments for which design intensities have already been derived. The median annual daily maximum rainfall for the Vorotan Pass gauge was used as an additional point of comparison in selecting a donor rainfall site.

Table 3: Design Rainfall Intensities for Vorotan Study Catchment

	Rainfall Depth (mm)	
	100	1000
Return Period:		
Source		
FA of Vorotan Pass 24H Rainfall	77.45	101.49
Maximum Daily Rainfall, Vorotan Pass	74.00	



Source: Climate Research Unit Aquastat tiled data

Figure 1: Long Term Average Monthly Rainfall Totals

As no design intensity data were available for a hydrologically similar catchment in the region, the UK was used as a source for the donor site. While the rainfall patterns in the UK are in general wetter and show less seasonal variation than southern Armenia with most gauges located at a lower elevation, the two regions share some commonality in terms of climate type and the hydrological mechanisms driving rainfall patterns. Mountainous areas of the UK are subject to much higher annual and monthly total rainfall which forced selection of a donor location away from obvious upland sites. On the basis of the chosen points of comparison, a suitable site was found at Hurn, near Bournemouth. A comparison of the long term monthly average rainfall for the UK and Armenian locations is shown in Figure 2 below. The 1D-RMED value for Hurn (35 mm per day) was also in the same range as the long term median annual maximum daily rainfall for Vorotan Pass (33 mm per day).

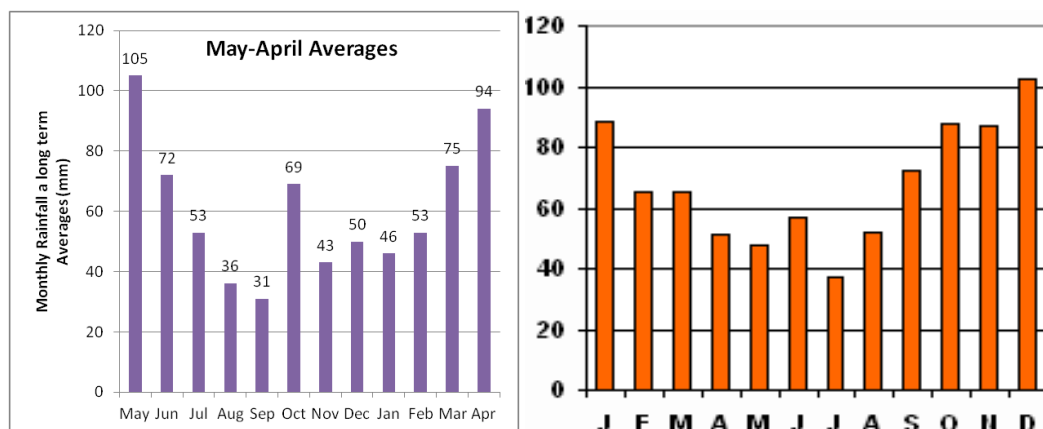


Figure 2: Vorotan Pass average monthly rainfall (left) compared to UK Donor catchment average monthly rainfall. The Vorotan pass monthly data have been time shifted to match UK wet/dry Periods)

100 year and 1000 year rainfall depths were then generated using the FEH software DDF rainfall function. The appropriate design depths were then found by interpolating for the design storm length and scaling this depth based on the ratio of total annual rainfall at the locations (see Table 4). The maximum daily rainfall was also identified from the Vorotan Pass rain-gauge record (active from 1962 to 1994 and 2000 to 2008) as an additional validation check.

Table 4: Design Rainfall Intensities (FEH Scaled)

	WDF Catchment	HLF Southern Catchment	HLF Northern Catchment
1 in 100 year	34.9 mm/hr	29.7 mm/hr	25.3 mm/hr
1 in 1000 year	74.0 mm/hr	61.1 mm/hr	51.2 mm/hr

2.3 Flow Estimation

The figures for design rainfall and the catchment parameters described in the preceding sections were used to calculate a number of rational method flows for the smaller catchments considered in this study. The estimates for the WDF site are tabulated below.

Table 5: WDF Model input Flows based on Rainfall–Runoff Estimates

	Q₁₀₀	Q₁₀₀₀
FA of Vorotan Pass 24H Rainfall	30.2 m³/s	39.6 m³/s
FEH Scaled Proxy	13.6 m ³ /s	28.5 m ³ /s
Maximum recorded daily Rainfall VP	28.9 m ³ /s	

Table 6: HLF Northern Catchment Flows based on Rainfall–Runoff Estimates

	Q₁₀₀	Q₁₀₀₀
FA of Vorotan Pass 24H Rainfall	31.8 m³/s	41.7 m³/s
FEH Scaled Proxy	10.4 m ³ /s	21.0 m ³ /s
Maximum recorded daily Rainfall VP	30.4 m ³ /s	

Table 7: HLF Southern Catchment Flows based on Rainfall–Runoff Estimates

	Q₁₀₀	Q₁₀₀₀
FA of Vorotan Pass 24H Rainfall	18.6 m³/s	24.4 m³/s
FEH Scaled Proxy	7.1 m ³ /s	14.7 m ³ /s
Maximum recorded daily Rainfall VP	17.79 m ³ /s	

These numbers represent the instantaneous peak flow associated with the design storm intensities. The Vorotan Pass rainfall depths are converted to intensities by assuming the daily depth falls over the course of one hour for this validation check.

While these estimates based on the rational method are likely over estimating the magnitude of the peak flows at the site, given the data available, these provide a reasonable upper bound to define the flood extents and potential impacts at the sites.

In estimating the Vorotan main channel flood flows, rainfall runoff methods were not deemed appropriate; instead, reference was paid to available flow gauge data. A study related to the design of a small hydro power plant (SHPP) located north (upstream) of the mine site in the Vorotan valley includes a record of maximum flows recorded at the Borisovka / Gorayk gauge. These annual maxima are plotted in the HEP design report and allow estimates to be made for the Q₁₀₀ and Q₁₀₀₀ events, presented in the table 8.

The Vorotan adjacent to the WDF has a shorter time of concentration, is more steeply sloped and has a lower average runoff coefficient in comparison to the much larger catchment upstream of the Borisovka gauge. Hence areally scaling reported flows at the Borisovka / Gorayk gauge will tend to underestimate the

magnitude of the flow in the faster responding, upstream Vorotan. In order to account for these variations, the "GOST 2.01.14-83" method was applied to estimate maximum return period inflows at the SHPP. These values have been scaled up for use as the input steady-state flows for the Vorotan main channel in the hydraulic model.

Table 8: Analysis of Flow Gauge Data¹

Description	Borisovka/Gorayk Gauge	SHPP Location	Vorotan adjacent to WDF
Catchment Area	507 km ²	85.2 km ²	124 km ²
Q₁₀₀	188 m ³ /s	92 m ³ /s	<i>133.8 m³/s</i>
Q₁₀₀₀	309 m ³ /s	151 m ³ /s	<i>219.8 m³/s</i>
Maximum recorded flow (1968)	211 m ³ /s	-	-

Modelled flows shown in italics

While the reliability of the return period flow estimates is limited by the available data, the maximum recorded flow at Borisovka is within this range.

2.4 HEC-RAS Model

A hydraulic model of the study reach was developed in HEC-RAS with cross-section and profile data extracted from a 30 m resolution DEM of the Vorotan valley and surrounds using the Geo-RAS ArcGIS extension. Sections were taken at multiple locations encompassing the channel and the valley areas in which the WDF is to be located. Figure 3 below illustrates the location of the relevant sections with respect to the WDF.

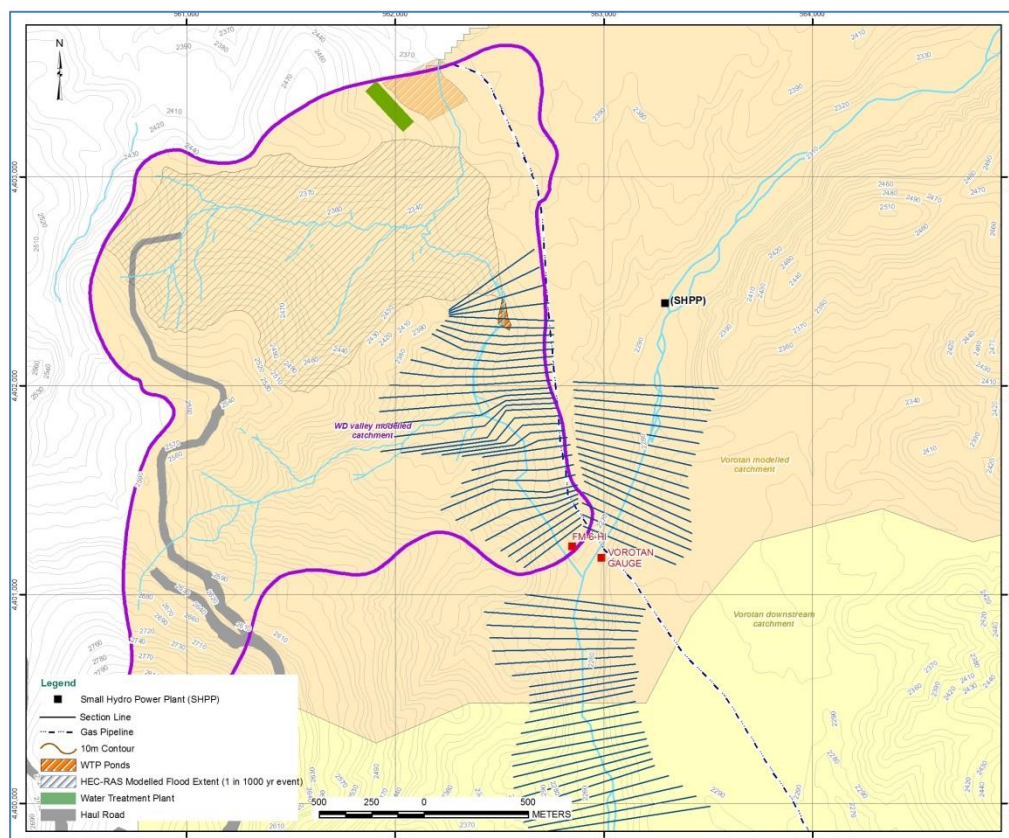


Figure 3: HEC-RAS Section Locations

¹ Source "Mane" SHPP Working Draft General Description, Yerevan 2011

The modelled Vorotan River extends from a point approximately 1 km upstream to a point 1 km downstream of the confluence with a watercourse draining the WDF valley. Approximately 1.5 km of the WDF valley watercourse upstream of the confluence has been modelled. The steeply sloped character of the watercourses dictated that a steady-state model could adequately represent the required level of detail. A Manning's value of 0.05 was used for overbank and channel areas on both watercourses.

Model runs were conducted with steady state flow inputs, with 2 input flow profiles to represent the highest return period flow estimates for the 1 in 100 year and the 1 in 1000 year events. Flows were input at the upstream end of the modelled Vorotan and WDF valley stream reaches. This is reasonable in the case of the Vorotan but overestimates the flow at the upper end of the modelled WD reach, adding a measure of conservatism. Furthermore, given the Vorotan has a much longer time to peak than the WDF valley watercourse, peak flows are highly unlikely to occur concurrently. However, as a conservative measure, a lag has not been introduced between the events defining the flood plains.

Figures 4 and 5 below show the modelled flood extent for the 1 in 100 and 1 in 1000 year floods respectively.

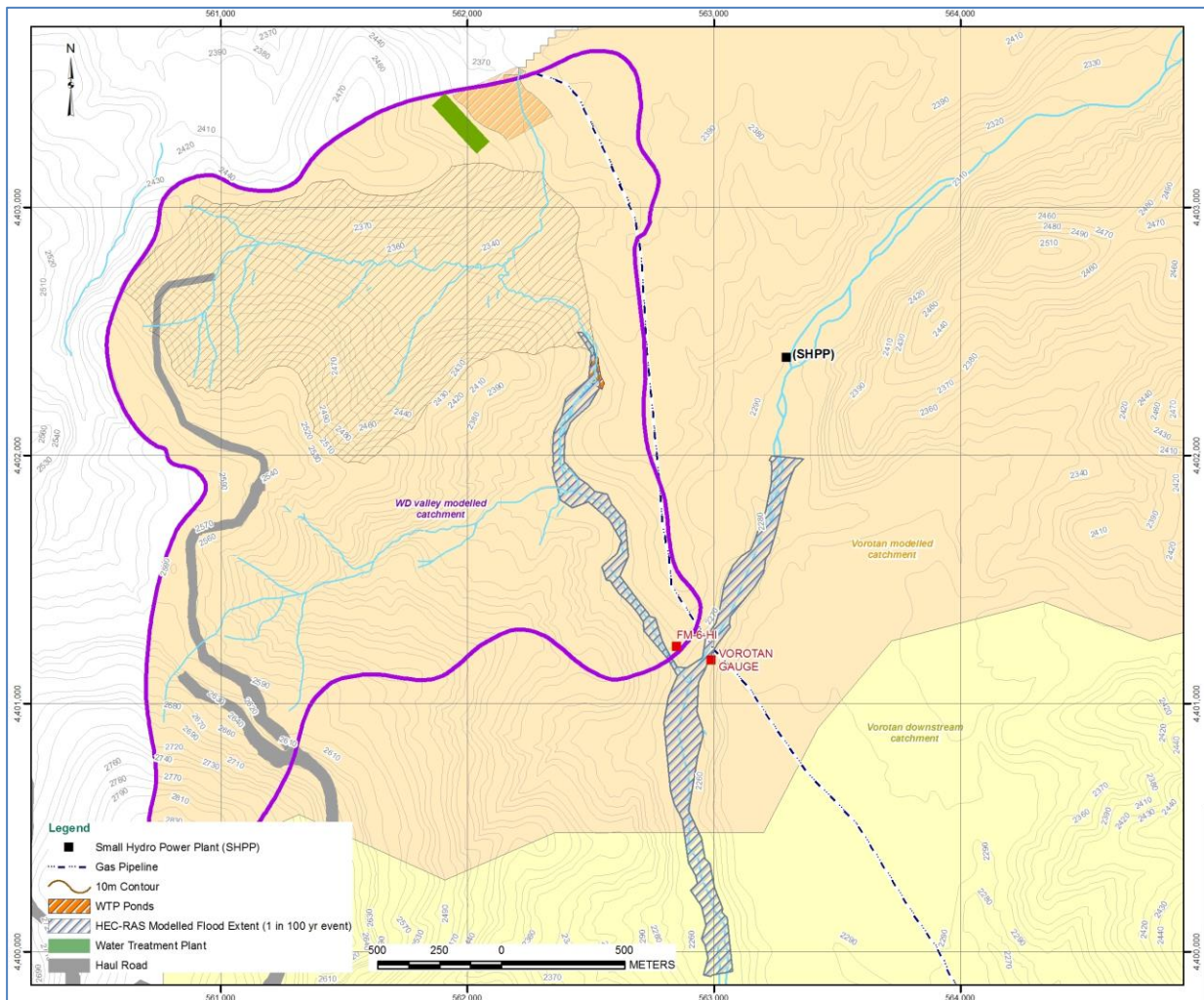


Figure 4: 1 in 100 year modelled pre-development flood extents

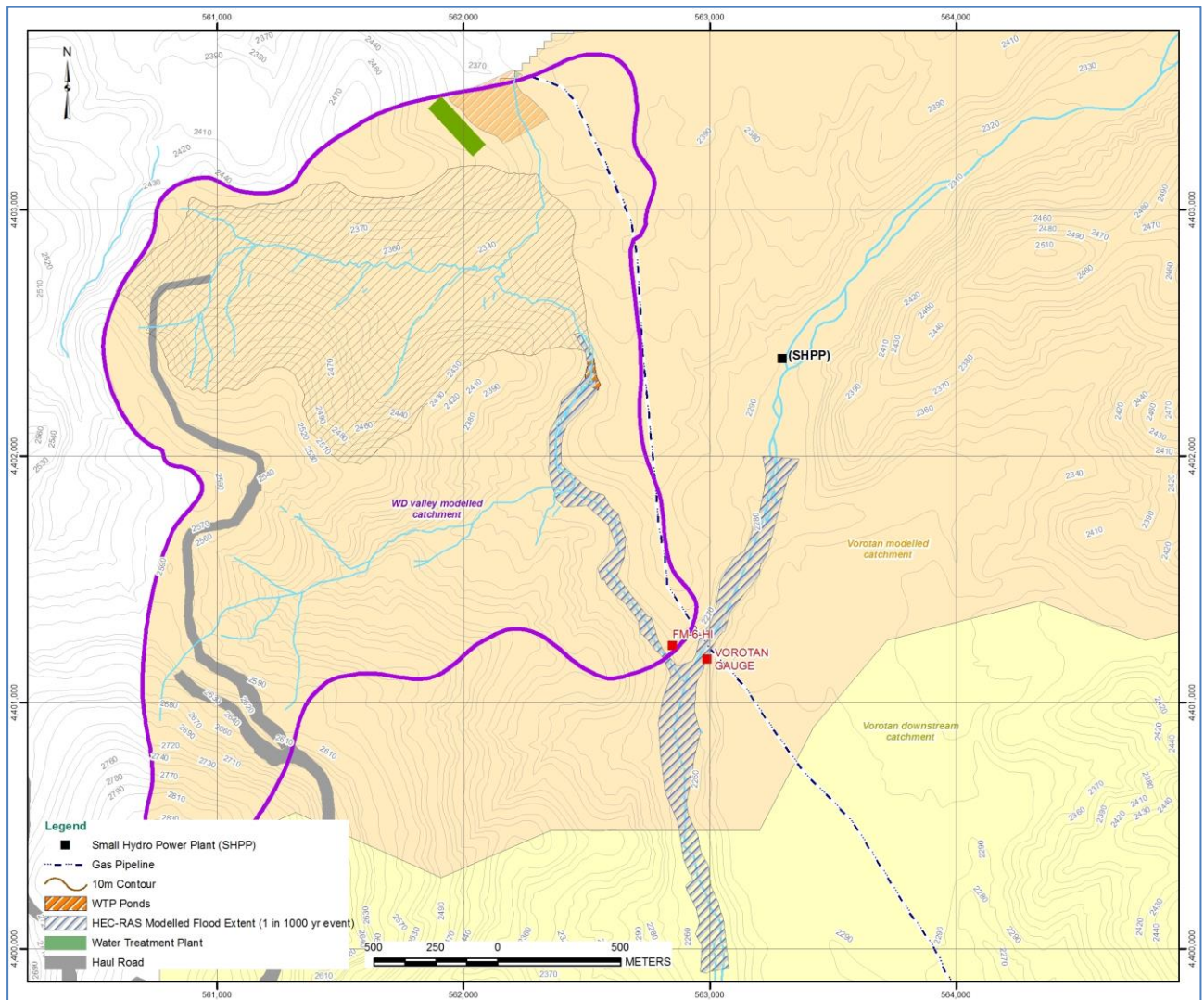


Figure 5: 1 in 1000 year modelled pre-development flood extents

3.0 RESULTS AND DISCUSSION

Neither of the scenarios modelled in this study (including flows higher than the highest recorded flows for the Vorotan at a downstream gauge) result in water levels which place the critical infrastructure (main ponds, WTP, Dump) of the WDF at risk. The maximal flood 1 in 100 year flood extents for the Vorotan do not approach the footprint of the proposed infrastructure.

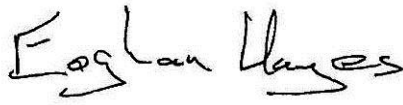
The maximum water surface elevation for the 1 in 1000 year storm of 2293.0 masl at the upstream end of the modelled reach; the spur between the Vorotan and the WD valley is at an elevation of over 2300 masl adjacent to this point.

On the side channel, the modelled 1 in 1000 year flow results in a maximum water surface elevation of 2325.8 masl; this corresponds to a water depth of 1.37m. As stated earlier in the report, this over-estimates the likely flows as includes the entire valley catchment; nevertheless, this flood level is below the WDF toe level of 2326.0 masl. The proposed design does not encroach on the floodplain and will not result in any loss of flood storage in the WDF valley. Runoff from the WDF valley will either be diverted to the WDF ponds for treatment or diverted around the facility; this will result in a lower flood flow entering the modelled reach and hence the modelled predevelopment flood level predictions are conservative. At the HLF site, the ephemeral streams are to be diverted around the facilities with flows being returned to their natural

catchments downstream of the proposed development. Flood flows are to be contained in appropriately sized diversion channels and will pose no risk to the proposed HLF development.

Internal management of flood risk within the HLF and WD facilities is described in the site wide water balance memo (Golder Associates, 2013). Ponds are to be sized to contain both operational volume and the cumulative volume of the wettest month on record.

It is concluded that there is no significant risk of flooding to the critical infrastructure at the proposed WDF or HLF sites.



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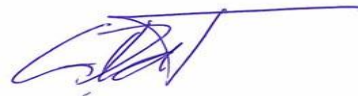
EH/GDLT/nk

References

Golder Associates (UK) Ltd, 2013. Amulsar Gold Project: Site Wide Water Balance. Technical Memorandum 13514250010.503.B.0, July 2013

Attachments

Drawing 1 Surface Water Impacts - Flood Risk Assessment

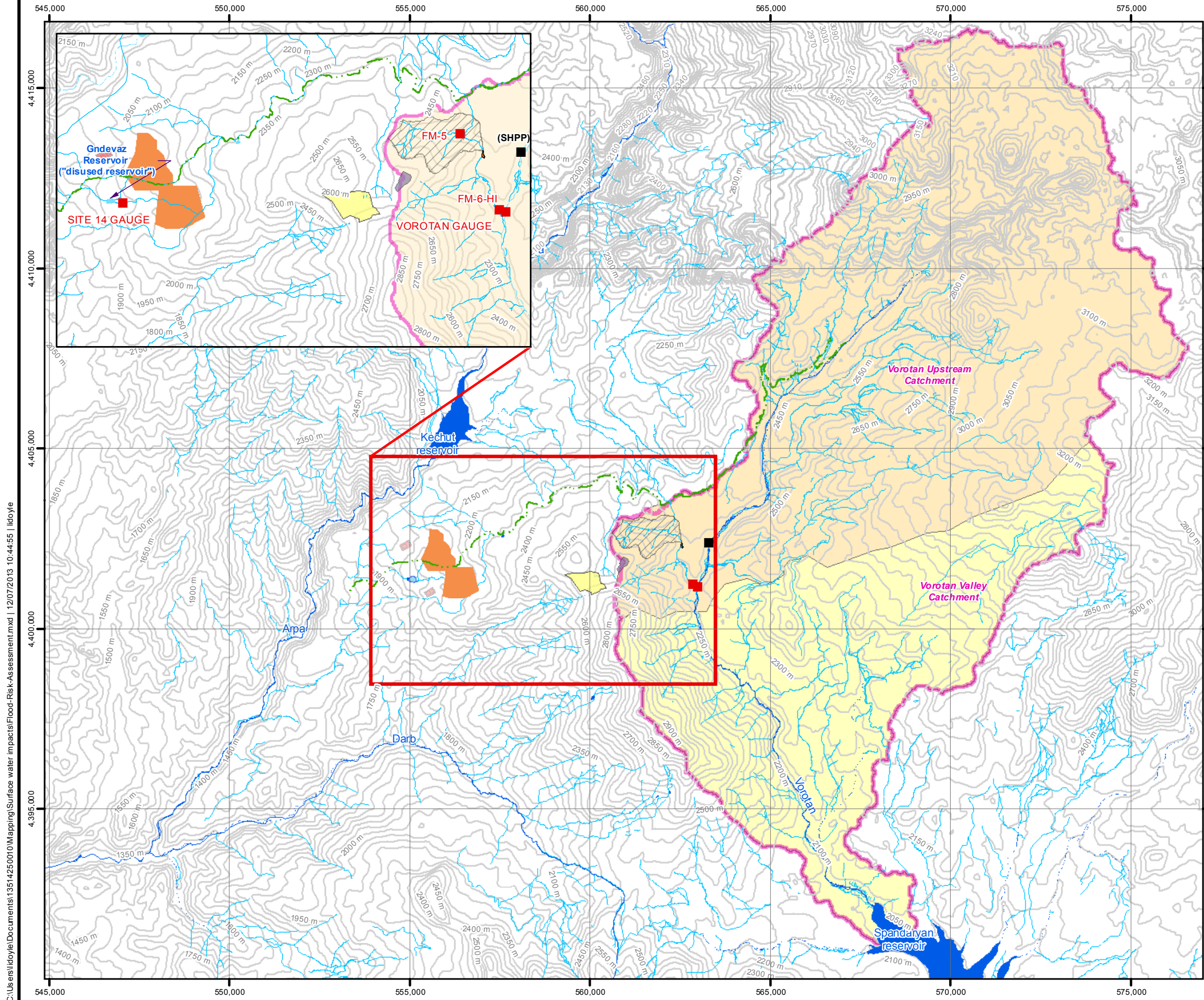


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DRAWING 1

Surface Water Impacts - Flood Risk Assessment





Legend


- Small Hydro Power Plant
- Surface Water Monitoring Location
- Gas Pipeline
- Tunnel Arpa Sevan
- River
- 50m Contour
- Haul Road
- Mine Pit
- Heap Leach Facility
- Crusher Area
- Truck Shop Area
- HLF Pond
- Catchment
- Reservoir

REFERENCES

Coordinate System: WGS 1984 UTM, Zone 38 North, Meter.
Data provided by client



SCALE 1:110,000 1 CENTIMETER = 1.1 KILOMETERS
WHEN PRODUCED AT SIZE A3

PROJECT				
LYDIAN INTERNATIONAL LTD. AMULSAR GOLD PROJECT				
TITLE				
SURFACE WATER IMPACTS- FLOOD RISK ASSESSMENT				
 Golder Associates Bourne End, UK	PROJECT No. 13514250010		FILE No.	
	DESIGN	EH	13/06/2013	SCALE AS ABOVE
	GIS	LD	12/07/2013	REV 0
	CHECK	EH	12/07/2013	DRAWING 1
	REVIEW	JB	12/07/2013	