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Technical Memorandum

To:	File	**From:**	Terra Michaels & Ronson Chee
Company:		**Date:**	July 24, 2012
Re:	Eagle Gold Heap Leach Facility Water Balance – Revision 2	**Doc #:**	114-320905X-5.3
CC:	Troy Meyer, P.E. (Tetra Tech Inc.)		

1.0 Introduction

This technical memorandum explains the methodologies and assumptions used to create a water balance model that has been developed for the Eagle Gold Heap Leach Facility. The intent of the water balance model is to determine the monthly freshwater make-up requirements for the Heap Leach Facility.

The water balance model accounts for a three phase Heap Leach Pad and a 100-day stockpile, with an ore production rate of 29,500 tons per day for 350 days a year. For 250 days (March-November), ore will be processed through three stages of crushing and stacked on the heap leach pad. For the remaining 100 days (November-February), ore will be sent through the primary crusher only and stored on the 100-day stockpile. When the 250 day stacking period begins the following year, the ore from the stockpile will be sent through remaining crushers (secondary and tertiary) and stacked on the heap leach pad.

The water balance model incorporates all inflows and outflows to this system described above and determines the amount of water available for recycle. The recycle water is assumed to be available for application to the ore in order to reach optimal leaching water content. The remaining water requirements will be fulfilled using freshwater, with monthly and yearly quantities summarized.

2.0 Water Balance Model

The Heap Leach Facility water balance model is shown in Figure 1. Ore is assumed to be mined 350 days year at a constant rate of 29,500 tons per day. For 250 days beginning in March of the first year, 29,500 tons of raw ore each day goes through three (3) stages of crushing, then is stacked on the heap leach pad. Halfway through November of the first year, stacking of the heap will cease and the same rate of ore will go through primary crushing only and will be placed on a stockpile for 100 days. The stockpile exists for the purpose of storing partially crushed ore in the wintertime (November-February), when snowpack on the heap leach pad interferes with stacking and the heap leaching. Beginning in March of the second and every subsequent year, the 29,500 tons of ore mined each day will return to the three stages of crushing, followed by stacking. In addition, the ore stockpiled during the previous winter will also go through the last two stages of crushing and will be stacked during the same 250 days. It is assumed that the stockpiled ore will be moved to the heap at 11,800 tons per day (an even rate over the 250 stacking days.) Thus, a total of 41,300 tons of ore per day is added to the heap from March through part of November beginning in year 2.

All water movement into, out from or within the entire water balance system (characterized by the large green box) is represented by arrows in Figure 1. Red arrows signify ore movement and the associated water content that naturally occurs with it, which is considered an input to the water balance. Crushing is assumed to neither contribute nor take water from the ore, therefore it is assumed that ore water content stays constant during the crushing process. Ore from the stockpile is assumed to have an altered water content due to environmental exposure, however this will also stay constant during crushing after storage. Blue arrows represent water solutions that are added or removed from the system at will of the process operators, such as leaching solution and make-up water. Environmental contributions are highlighted using purple arrows, and symbolize the amount of water that is added or removed from the stockpile and heap due to exposure to the environment.

Environmental contributions change seasonally, therefore a monthly time step was used in the water balance model to reflect the changes. For the heap leach water balance analysis, only average climate conditions (precipitation, evaporation and sublimation) were simulated for each month. The Event Ponds were not modeled as part of the water balance system as it is was designed only to store extreme event conditions. The Event Ponds, however, may be used as temporary storage for excess solution for wet months in which the heap experiences excess rainfall or serve as temporary storage for recycle water.

Make-up water is assumed to be added to the crushed ore after being stacked on the heap, as shown in Figure 1. It is defined as the amount of water required to meet the optimal water content for leaching ore, as specified in the *Process Design Criteria* (Wardrop, 2011b). The make-up water is composed of two parts: recycle water and freshwater. This water balance determines the amount of excess water that accumulates in the heap, which is assumed to be entirely available for reuse. After the amount of available recycle water is found, the freshwater make-up can be determined to complete the water balance.

The stockpile and the heap leach pad are the two areas of primary focus, because they are the areas mainly affected by the inflows and outflows of the water balance. Because stockpiling and stacking take place in different locations, at different rates and during different times of the year, the two areas must be treated as two separate parts that contribute to the entire water balance.

3.0 Water Balance Inflows and Outflows

Figure 1 includes a visual representation of the water balance inputs and outputs that will be considered for this model.

System inflows include:

- Ore Water – Water that naturally occurs in the mined ore.
- Rainfall – Rainfall that is collected within each contributing area (further defined in Section 4.3). The watersheds are bound by the diversion channels and natural topography.
- Snowmelt – Snow accumulates over the winter, but is not released into the system as water until it melts during the warmer months. Snowfall within the contributing areas (further defined in Section 4.3) is considered input into the water balance. This

is distributed each month as a percent of total yearly snowmelt. The snowfall and snowmelt distribution percentages over the year can be found in Attachment A.

- Leaching Application Solution – The solution applied to the heap to leach the ore.

- Make-up Water – The water that will be applied to the ore to raise the water content of the ore to the optimal leaching water content. It consists of a mix of recycle water from the heap and freshwater.

System outflows are:

- Evaporation (from precipitation) – Precipitation that vaporizes on the top surface area of the ore due to high temperatures. Evaporation varies with area and is assumed to only occur on the top area of the heap leach pad and stockpile.

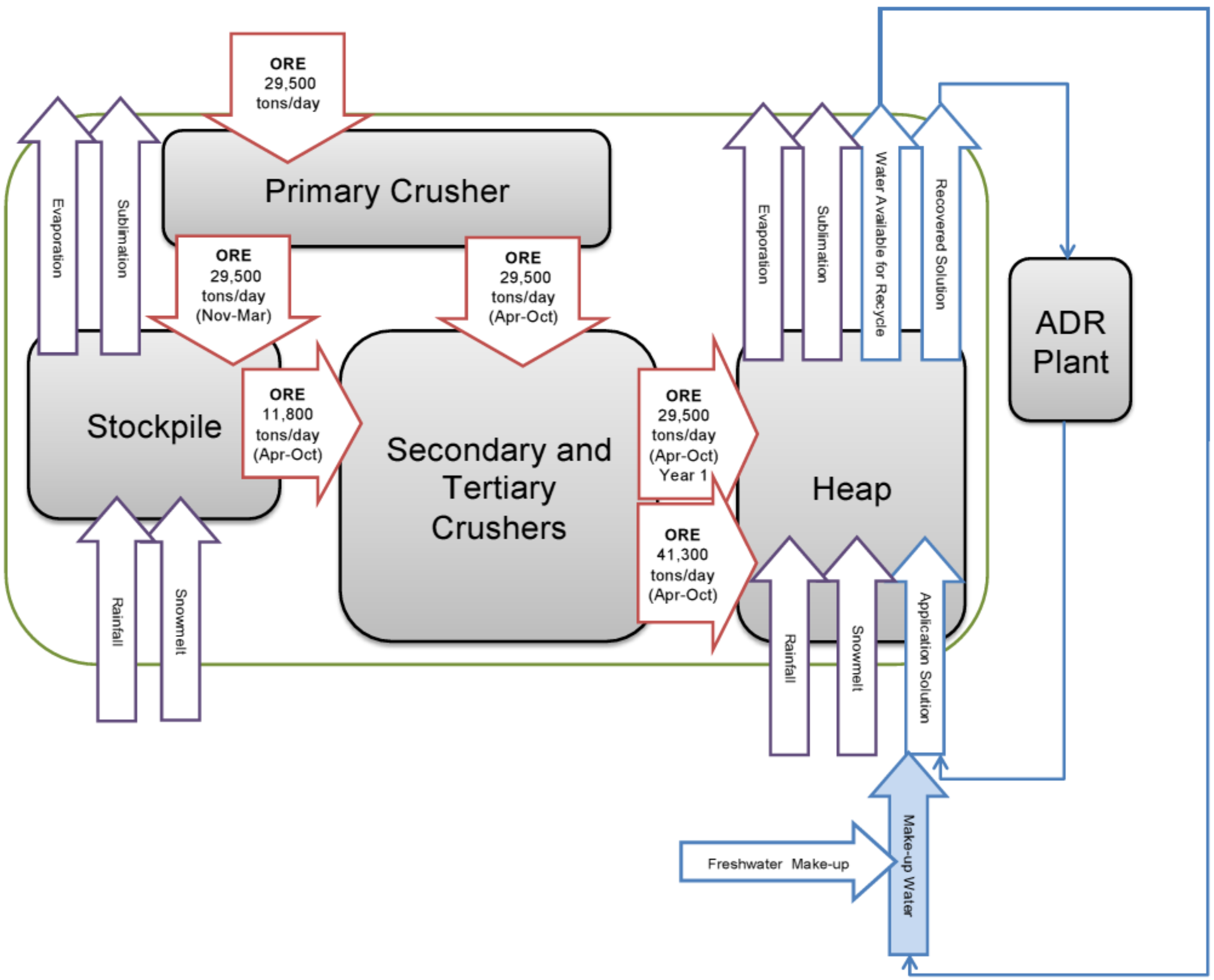
- Evaporation (from leaching solution) – Percent of leaching solution that vaporizes upon application.

- Sublimation – Water that vaporizes from the winter snowpack, reducing snowpack available for snowmelt. This is estimated at 20% of the snowpack (Knight Piesold, 2012).

- Recovered Leachate Solution – Solution that contains minerals extracted from the ore after the leaching process.

The total amount of water available after the leachate has been recovered is assumed to be available for recycle in the make-up water. This also assumes that ore being leached at any given time does not retain water from environmental contributions. Lastly, any water not completely used in a given month is assumed to be available for use in the following month. This excess water may possibly be temporarily stored in the Event Ponds or as determined by Victoria Gold.

Figure 1 Heap Leach Facility Water Balance Model Schematic



4.0 Water Balance Model Assumptions

4.1 Heap Ore Properties

The water content of the raw ore when removed from the pit, before processing is 5.0% (Wardrop, 2011a). All ore processed and sent to the heap leach pad during the 250 stacking days contributes 5.0% water content into the water balance.

All water accumulated on the stockpile is assumed to contribute to the total water content of the ore stockpile. The sum of all contributing water for the entire year is divided evenly per cubic meter of ore, assuming an even distribution of the accumulated water.

The water content for ideal leaching was determined to be 13.3% (Wardrop, 2011a). This consists of accumulated water in the heap that is available for recycle and freshwater. It is assumed that this makeup water will be added to the ore after all crushing and being stacked on the heap leach pad. The ideal leaching water content of 13.3% is assumed to occur only under the active leaching area. Over time, the ore water content of previously leached areas will eventually decrease to about 5% water content.

The "Water Content" worksheet in Attachment B shows the calculations for the water quantities listed above. The ore input values are (Wardrop, 2011a):

- As-delivered ore water content = 5.0%;
- Optimal ore water content = 13.3%;
- Ore bulk density = 1,800 kg/m^3.

4.2 Heap Ore Stacking and Stockpiling

Stacking and stockpiling rates are based on an ore production rate of 29,500 tons of ore per day for 350 days a year, and stockpiling takes place for 100 of those days. Figure 1 reflects the following stacking and stockpiling information and assumptions:

- 29,500 tons/day of ore is processed through three stages of crushing and stacked on the heap leach pad for 250 days (March through part of November every year);
- 29,500 tons/day of ore is processed through the primary crusher and placed on the stockpile for 100 days (the last part of November through February every year beginning in November of the first year);
- 11,800 tons/day of partially crushed ore from the stockpile is sent through the secondary and tertiary crushers and is stacked on the heap leach pad for 250 days (March through part of November after the first year);
- 41,300 tons/day of fully processed (crushed) ore is placed on the heap for 250 stacking days (March through part of November after the first year).

For the purposes of this water balance, March 1, 2013 is assumed to be the beginning of the 250 day stacking season in the first year.

4.3 Heap Leach Pad and Stockpile Design/Contributing Area

The heap leach pad will be constructed and stacked in three phases, determined from preliminary stacking plans developed by Tetra Tech. It is assumed that temporary diversion channels will be placed upstream of each lined pad phase area. These diversion channels serve as a boundary to identify the contributing watershed areas. Since each area is lined, all precipitation that falls within that area is assumed to add to the water balance.

The stockpile area also contributes to the water balance, and will remain constant during the life of the mine. All precipitation that falls within the stockpile area is assumed to add to the water balance. It is also assumed that all precipitation runoff from higher elevations will be diverted from the stockpile area.

The contributing areas that collect precipitation for each phase of the Heap Leach Pad and 100 day stockpile are as follows:

- Phase 1: 410,007 square meters;
- Phase 2: 940,920 square meters;
- Phase 3: 1,134,970 square meters;
- 100-day Stockpile: 185,425 square meters.

Evaporation from the heap leach pad is calculated using only the top area of the current lift, which changes incrementally. These top areas of the heap were estimated using the preliminary stacking plans. The stockpile areas were estimated using an approximate footprint and contributing area, and are assumed to increase evenly during the 100 days of stockpiling. Similarly, the stockpile area is assumed to decrease incrementally during 250 stacking days. As more information becomes available on the physical geometry and layout of the stockpile, the water balance may be adjusted.

4.4 Climate Data Input

Monthly climate data was provided by Knight Peisold (2012) for a site near the proposed heap leach facility. Averages were calculated from the entire dataset, collected between October 2006 and June 2011. This data includes monthly values for:

- Precipitation (rainfall and snowfall);
- Snowmelt; and
- Evaporation.

Only evaporation values were included in the provided data, not pan evaporation values, therefore it is assumed that any pan evaporation coefficients are not necessary for the calculations. The climate data input can be found in Attachment A.

4.5 Solution Application

Solution application and solution recovery are represented using two blue line arrows as shown in Figure 1. Evaporation calculations from the leaching solution drip emitters is included on the

"Environmental Contribution Water" sheet in Attachment C. Except for this evaporation, it is assumed that the amount of leaching solution and recovered leachate solution are equal, therefore are not included in the calculations.

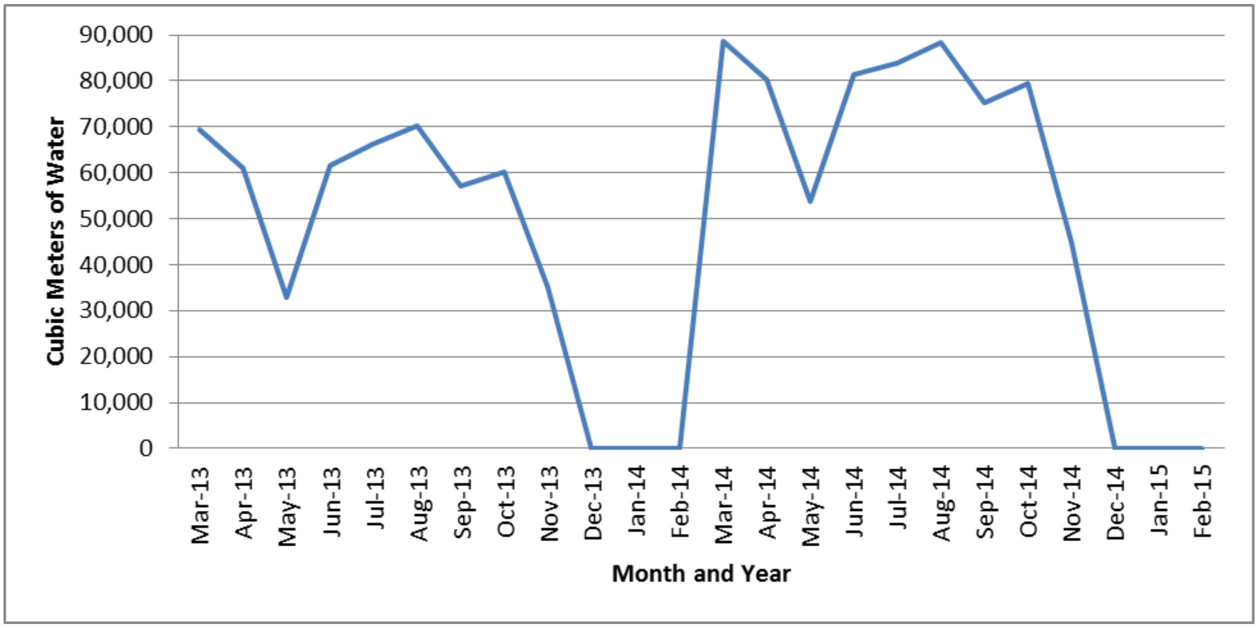
5.0 Results

Calculated monthly freshwater make-up requirements are displayed graphically in Figures 2, 3, and 4, representing Phases 1, 2, and 3, respectively. The numeric data can be found in Attachment C in the "Monthly Makeup Water Requirements" table. Each section discusses the results for each phase.

5.1 Phase 1 Results

Phase 1 (Figure 2) exhibits roughly the same pattern in freshwater demand each year. In the first year, the water demand appears smaller than the subsequent years due to not having a contributing stockpile from the previous winter. The make-up water peaks in August only during the first year, with another peak in March. The primary peak in year two occurs during March, corresponding with the month that stacking begins on the heap leach pad. Make-up water decreases in April of both years, when precipitation begins to contribute larger amounts to the water balance. By May, most of the winter snowpack has melted, resulting in a large quantity of water available for recycle. From June through October, water demand rises again and remains high, as ore stacking continues but water available for recycle decreases. November always experiences a smaller demand for freshwater due to stacking only occurring for half of the month. December through February does not exhibit any demand for water, because there is no stacking on the heap.

Figure 2 Phase 1 Freshwater Make-up Requirements



Year 1 monthly water demand extremes are:

- Maximum freshwater make-up requirement of 70,174 m^3 occurring in August 2013;
- Minimum freshwater make-up requirement of 32,860 m^3 occurring in May 2013;
- Yearly minimum freshwater make-up requirement of 0 m^3 occurring in December 2013.

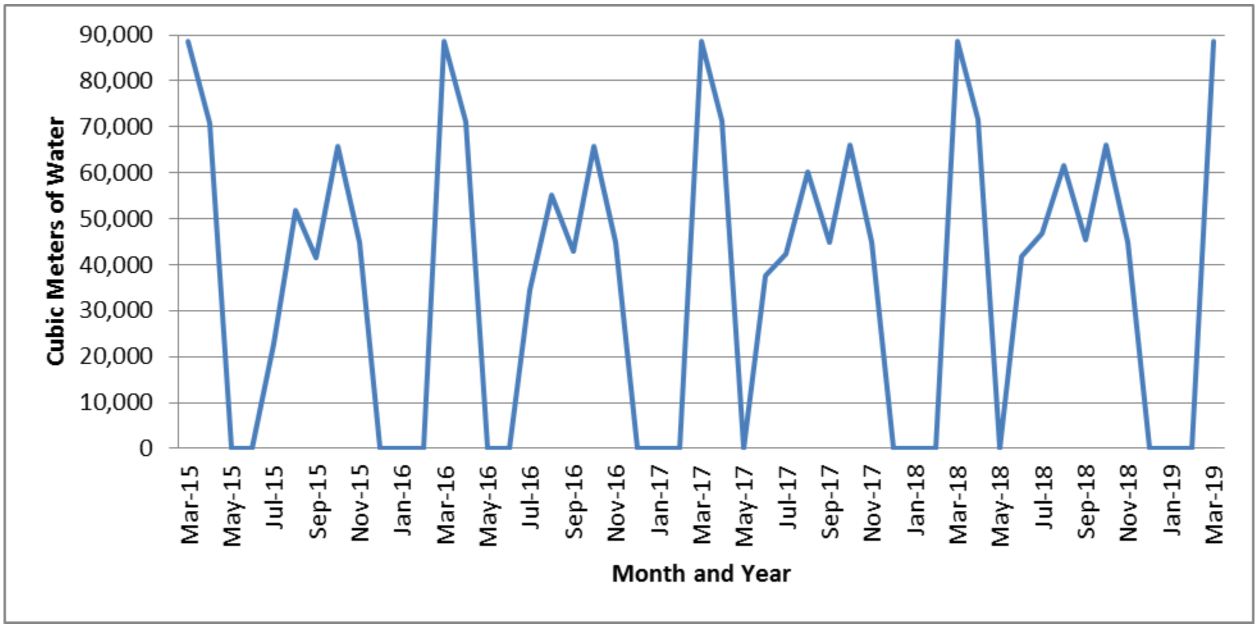
Phase 1 monthly water demand extremes:

- Maximum freshwater make-up requirement of 88,748 m^3 occurring in March 2014;
- Minimum freshwater make-up requirement of 32,860 m^3 in May 2013;
- Yearly minimum freshwater make-up requirement of 0 m^3 in December 2013 and 2014, and January and February 2014 and 2015.

5.2 Phase 2 Results

Freshwater demand shows the same pattern year to year during Phase 2 (Figure 3). The water demand peaks each year in March and decreases slightly in April, similar to Phase 1. In May, however, freshwater demand drops to zero, due to the notably larger Phase 2 area collecting more precipitation in the warmer months. In most years, the water available for recycle collected in May and often June is greater than the water demand, resulting in extra recycle water available for subsequent months. From July through November, water demand rises again as snowmelt reduces, revealing a second yearly peak in October. December through February again do not exhibit any demand for water, because there is no stacking on the heap.

Figure 3 Phase 2 Freshwater Make-up Requirements



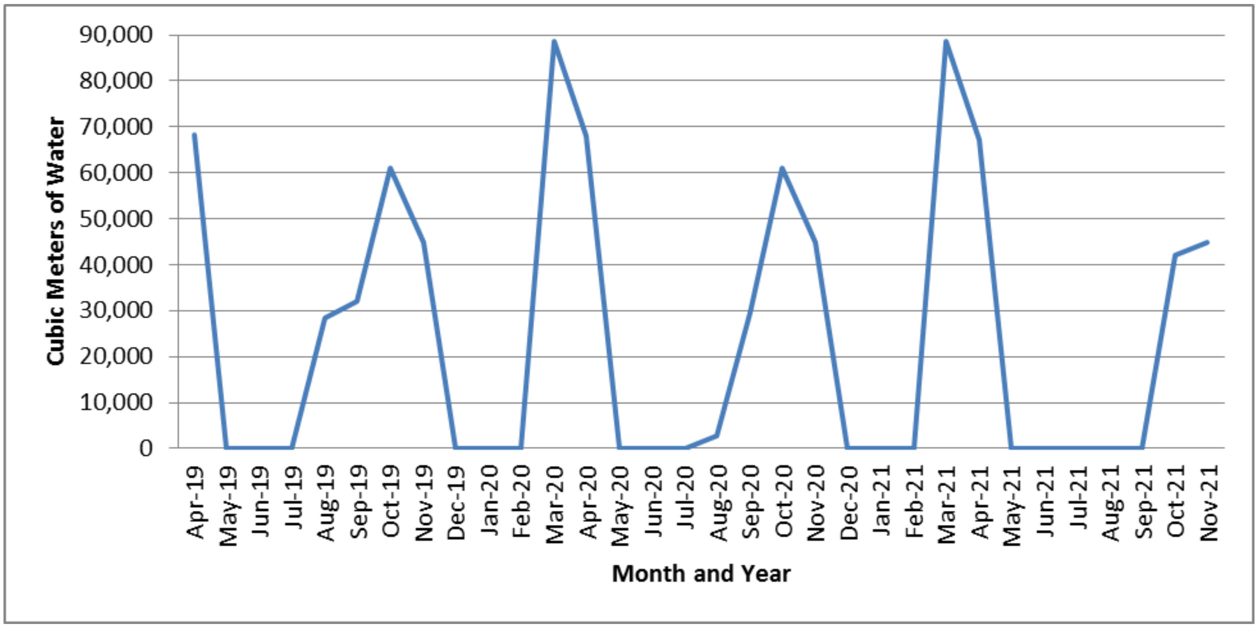
Phase 2 monthly water demand extremes:

- Maximum freshwater make-up requirement of 88,748 m^3 occurring in March 2015 through 2019;
- Secondary peak maximum freshwater make-up requirement of 66,028 m^3 occurring in October 2018;
- Minimum freshwater make-up requirement of 0 m^3 occurring in May 2015 through 2018, and June 2015 through 2016;
- Yearly minimum freshwater make-up requirement of 0 m^3 occurring in December 2015 through 2018, and January and February 2016 through 2019.

5.3 Phase 3 Results

Freshwater demand exhibits the same pattern again year to year during Phase 3 (Figure 4), with water demand peaks each year in March, followed by decreases in April. In May, freshwater demand drops to zero similar to Phase 2, but remains at zero through July, August or September. This is due to the increase of the Phase 3 area, which in turn increases precipitation and recycle water available. The second yearly peak occurs in October of each year, as it did during Phase 2. December through February again do not exhibit any demand for water, because there is no stacking taking place.

Figure 4 Phase 3 Freshwater Make-up Requirements



Phase 3 monthly water demand extremes:

- Maximum freshwater make-up requirement of 88,748 m^3 occurring in March 2020 and 2021;
- Secondary peak maximum freshwater make-up requirement of 61,157 m^3 occurring in October 2019;
- Minimum freshwater make-up requirement of 0 m^3 occurring in May through July 2019 through 2021, and August and September 2021;
- Yearly minimum freshwater make-up requirement of 0 m^3 occurring in December 2019 through 2020, and January and February 2020 and 2021.

5.4 Total Freshwater Required

Overall, water available for reuse increases every year, resulting in a decreasing need for freshwater make-up. Minus the initial freshwater requirements, year 2014 requires the greatest amount of water, at over 676,000 cubic meters for the year. This decreases to only 243,000 cubic meters in year 2021. Twenty-four (24) months of rinsing following operations are also included.

The one month initial water requirement assumes that an initial amount of water is required to bring the ore from its as delivered water content (5.0%) to the optimal leaching content (13.3%) and that it takes approximately one month for this solution to travel through one month's worth of stacked ore (29,000 tons/day for 31 days) and be available for recycle.

Rinsing estimates assume that the same application rate (2,770 m^3/hr) comprised of clean water from the treatment plant is applied over the heap. The rinsing makeup water estimate assumes that no rewetting of ore is required to bring the rinsing area up to water content of 13.3%. Thus, no additional fresh makeup water is required as the clean water used for rinsing will come from recycle water that has gone through the treatment plant.

Table 1 Total Freshwater Volume Requirements

Mine Phase	Year	Makeup Moisture (m^3)	Yearly Makeup Moisture (m^3)	Mine Phase Water Demand Summary (m^3)
1 Month Initial Water Requirement	2013	72,000	585,727	3,835,118
10 Months Operation		513,727		
12 Months Operation	2014	676,139	676,139	
12 Months Operation	2015	386,302	386,302	
12 Months Operation	2016	403,404	403,404	
12 Months Operation	2017	456,127	456,127	
12 Months Operation	2018	466,864	466,864	
12 Months Operation	2019	323,226	323,226	
12 Months Operation	2020	294,647	294,647	
11 Months Operation	2021	242,682	242,682	
1 Month Closure (Rinsing)		0		0
12 Months Closure (Rinsing)	2022	0	0	
11 Months Closure (Rinsing)	2023	0	0	

6.0 Conclusion

Each phase exhibits similar trends in the demand for freshwater make-up requirements. All phases require the most make-up water in March of every year, when stacking on the heap leach pad begins and before most of the winter precipitation melts and is available for recycle. The maximum in March is about the same for each phase, around 89,000 cubic meters. Phase 2 and Phase 3 both have zero minimum water requirements during the stacking season due to the larger areas that catch greater quantities precipitation, which in turn increase the amount of water available for recycle.

Makeup water requirements are greatest in 2014, or year two of operations, calling for over 676,000 cubic meters for the entire year. After year two, however, makeup water requirements generally decrease every year as water available for recycle increases. This is due to the excess of recycle water that accumulates in the Phase 2 and Phase 3 increasingly larger watersheds from rainfall and snowmelt during stacking months. By the end of Phase 3, water requirements are under 243,000 cubic meters for the last year, with the need for makeup water only spanning five to six months a year. At closure there is no additional freshwater required for rinsing.

REFERENCES

Budhu, M. (2007). *Soil Mechanics and Foundations, 2nd Edition.* Hoboken, NJ: John Wiley & Sons, Inc.

Knight Piesold (2012). *Eagle Gold Project – Water Balance Model Transitional Summary*. April 30, 2012.

Tetra Tech (2011). *In-Heap Pond, Spillway and Event Ponds Sizing.* December 23, 2011.

Wardrop (2011a). *Heap Leach Facility Design Criteria.* April 19, 2011.

Wardrop (2011b). *Process Design Criteria.* February 1, 2011.

ATTACHMENT A
HEAP LEACH WATER BALANCE CALCULATIONS
MODEL INPUTS

Input Data

	Year Round	Mar - Oct from stockpile	April - Oct Total	
Ore Production Rate =	29,500	11,800	41,300	tons/day
	16,389	6,556	22,944	m^3/day

	Heap Leach Pad	Ore Stockpile		
Heap Contributing Area (phase 1)=	410,007	185,425	m^2	assumes diversion channels in place upstrea
Heap Contributing Area (phase 2)=	940,920		m^2	
Heap Contributing Area (phase 3)=	1,134,970		m^2	
Ore Bulk Density	1.8		t/m^3	
Density of Water	1,000		kg/m^3	
Solution Application	66,480		m^3/day	

As-Delivered Ore Water Content	5.0%
Sublimation losses	20.0%
Optimal Water Content	13.3%

	# Stockpiling days	# heap days
Jan	29.75	
Feb	26.75	
Mar		29.75
Apr		28.75
May		29.75
Jun		28.75
Jul		29.75
Aug		29.75
Sep		28.75
Oct		29.75
Nov	13.75	15
Dec	29.75	
	100	250

Average Monthly Climate Data

Month	Rainfall	Snowfall SWE	Snowmelt Distribution per month	Evaporation	Est. Emitter Evaporation
	(mm)	(mm)	(%)	(mm)	(%)
Jan	0.00	26.04	0.00%	0.00	0.0%
Feb	0.00	20.50	0.00%	0.00	0.0%
Mar	0.00	26.79	0.00%	0.00	0.0%
Apr	0.17	23.28	8.02%	5.65	0.0%
May	23.26	7.69	67.88%	46.88	1.0%
Jun	84.16	2.11	8.42%	77.14	1.3%
Jul	99.96	0.00	0.00%	81.34	1.2%
Aug	72.49	0.56	0.23%	61.84	1.0%
Sep	52.47	13.65	5.67%	24.71	0.6%
Oct	4.28	47.00	9.77%	1.60	0.0%
Nov	0.00	36.55	0.00%	0.00	0.0%
Dec	0.00	36.32	0.00%	0.00	0.0%
Total	336.79	240.49		299.15	

ATTACHMENT B
HEAP LEACH WATER BALANCE CALCULATIONS
STOCKPILE WATER AND
ORE WATER CONTENT

Stockpile Precipitation and Sublimation Calculations

	Heap Leach Pad		
Stockpile Contributing Area (all phases)=	185,425	m^2	assumes diversion channels in place upstream of Pad
Sublimation losses	20.0%		

	Climate Data									
Date	**Rainfall**	**Snowfall SWE**	**Snowmelt Distribution per month**	**Top Area**	**Rain on Heap Area**	**Snowfall SWE on Heap Area**	**Sublimation from Snowpack**	**Snowfall SWE Minus Sublimation**	**Snowmelt**	**Stockpile Precipitation and Sublimaiton**
	(mm)	(mm)	(%)	(m^2)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)
Jan	0.0	26.0	0.0%	108,242	0	4,828	966	3,863	0	0
Feb	0.0	20.5	0.0%	160,624	0	3,801	760	3,041	0	0
Mar	0.0	26.8	0.0%	174,392	0	4,968	994	3,974	0	0
Apr	0.2	23.3	8.0%	152,697	32	4,317	863	3,453	2,861	2,893
May	23.3	7.7	67.9%	131,003	4,313	1,426	285	1,141	24,216	28,529
Jun	84.2	2.1	8.4%	109,308	15,605	391	78	313	3,004	18,609
Jul	100.0	0.0	0.0%	87,613	18,535	0	0	0	0	18,535
Aug	72.5	0.6	0.2%	65,548	13,441	104	21	83	80	13,522
Sep	52.5	13.7	5.7%	43,853	9,729	2,531	506	2,025	2,023	11,752
Oct	4.3	47.0	9.8%	22,158	794	8,715	1,743	6,972	3,485	4,279
Nov	0.0	36.6	0.0%	18,311	0	6,777	1,355	5,422	0	0
Dec	0.0	36.3	0.0%	53,078	0	6,735	1,347	5,388	0	0
Totals		240.5				44,593		35,674		98,118

Stockpile Water

Input Data

	Mar-Oct	Mar	
Ore Production Rate =	29,500	11,800	tons/day
	16,389	6,556	m^3/day

Total Stockpile Area	185,425	m^2	assumes diversion channels in place upstream of Pad
As-Delivered Ore Water	0.09	m^3/ m3 ore	from Water Content Sheet
Total Stockpiling Days	100		

Ore Destination	Date	# days per month	# Stockpiling days	# Heap days	Evaporation Data	Average Stockpile Area	Ore Water	Precipitation and Sublimation	Evaporation	Monthly Stockpile Water
					(mm)	(m^2)	(m^3)	(m^3)	(m^3)	(m^3)
Stockpile	Jan	31	30	0	0.0	108,242	41,792	0	0	41,792
Stockpile	Feb	28	27	0	0.0	160,624	37,577	0	0	37,577
Heap	Mar	31	0	30	0.0	174,392	0	0	0	-
Heap	Apr	30	0	29	5.6	152,697	0	2,893	862	2,031
Heap	May	31	0	30	46.9	131,003	0	28,529	6,141	22,388
Heap	Jun	30	0	29	77.1	109,308	0	18,609	8,431	10,178
Heap	Jul	31	0	30	81.3	87,613	0	18,535	7,127	11,408
Heap	Aug	31	0	30	61.8	65,548	0	13,522	4,053	9,468
Heap	Sep	30	0	29	24.7	43,853	0	11,752	1,083	10,669
Heap	Oct	31	0	30	1.6	22,158	0	4,279	36	4,243
Stockpile/Heap	Nov	30	14	15	0.0	18,311	19,315	0	0	19,315
Stockpile	Dec	31	30	0	0.0	53,078	41,792	0	0	41,792
Yearly Totals			100	250						210,861

Leaching (250 days) 36400

TETRA TECH

Water Content

Ore Water Content, as delivered	w	5.00%
Optimal Water Content	w	13.30%

Densities

Bulk, initial	γb	1800	kg/m3
Water	γw	1000	kg/m3
Dry	γd	1714.29	kg/m3
Specific Gravity**	Gs	2.7	

Stockpile Inputs

Ore added to stockpile	16,389	m3/day
Total Ore	1,638,889	m3
Total Water in Stockpile	210,861	m3
Water per m3 ore		

INITIAL CONDITIONS, VOLUME OF WATER IN ORE

Name	Variable	Amount	Units
Weights*			
Solid, per 1 m3 ore	Ws	1714.29	kg
Water, per 1 m3 ore	Ww	85.7143	kg
Volumes			
Water, per 1 m3 ore	Vw	0.09	m3
Solid, per 1 m3 ore***	Vs	0.63	m3
Air, per 1 m3 ore	Va	0.28	m3
Void (Water + Air), per 1 m3 ore	Vv	0.37	m3
Porosity	n	0.37	
Void Ratio	e	0.575	
Saturation	S	23.48%	

WATER VOLUME FOR OPTIMAL LEACHING

Name	Variable	Amount	Units
Weights			
Solid, per 1 m3 ore	Ws	1714.29	kg
Water, per 1 m3 ore	Ww	228	kg
Volumes			
Water, per 1 m3 ore	Vw	0.23	m3
Solid, per 1 m3 ore	Vs	0.63	m3
Air, per 1 m3 ore	Va	0.14	m3
Void (Water + Air), per 1 m3 ore	Vv	0.37	m3
Porosity	n	0.37	
Void Ratio	e	0.575	
Saturation	S	62.45%	

STOCKPILE WATER VOLUME

Name	Variable	Amount	Units
Weights			
Solid, per 1 m3 ore	Ws	1714.29	kg
Water, per 1 m3 ore	Ww	128.661	kg
Volumes			
Water, per 1 m3 ore	Vw	0.13	m3
Solid, per 1 m3 ore	Vs	0.63	m3
Air, per 1 m3 ore	Va	0.24	m3
Void (Water + Air), per 1 m3 ore	Vv	0.37	m3
Porosity	n	0.37	
Void Ratio	e	0.575	
Saturation	S	35.24%	

Stockpile Water Content w 7.51%

*weight of air is negligible
**using assumed value for Gs (Budhu, 2007)
***Solid volume calculated using specific gravity

Water Content Equations:
γd = γb / (1 + w)
Ww = w * Ws
Vw = Ws / γw
Vs = Ws / (Gs * γw)
Va = 1 - Vs - Vw
V = Va + Vw + Vs = 1
Vv = Va + Vw
n = Vv / V

Ore properties provided by Wardrop (Wardrop, 2011).
The same calculations were used in Tetra Tech's *In-Heap Pond, Spillway and Event Ponds Sizing* Memo, December 23, 2011.

ATTACHMENT C
HEAP LEACH WATER BALANCE CALCULATIONS
HEAP LEACH PAD MAKE-UP WATER REQUIREMENTS

TETRA TECH

Phase 1 Precipitation and Sublimation Calculations

	Heap Leach Pad		
Heap Contributing Area (phase 1)=	410,007	m^2	assumes diversion channels in place upstream of Pad
Sublimation losses	20.0%		

Phase 1
Phase 2
Phase 3

	Climate Data								
Date	**Rainfall**	**Snowfall SWE**	**Snowmelt Distribution per month**	**Rain on Heap Area**	**Snowfall SWE on Heap Area**	**Sublimation**	**Snowfall SWE Minus Sublimation**	**Snowmelt**	**Precipitation and Sublimation, Phase 1**
	(mm)	(mm)	(%)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)
Jan	0.0	26.0	0.0%	0	10,677	2,135	8,541	0	0
Feb	0.0	20.5	0.0%	0	8,405	1,681	6,724	0	0
Mar	0.0	26.8	0.0%	0	10,984	2,197	8,787	0	0
Apr	0.2	23.3	8.0%	70	9,545	1,909	7,636	6,326	6,396
May	23.3	7.7	67.9%	9,537	3,153	631	2,522	53,545	63,082
Jun	84.2	2.1	8.4%	34,506	865	173	692	6,642	41,148
Jul	100.0	0.0	0.0%	40,984	0	0	0	0	40,984
Aug	72.5	0.6	0.2%	29,721	230	46	184	178	29,899
Sep	52.5	13.7	5.7%	21,513	5,597	1,119	4,477	4,473	25,986
Oct	4.3	47.0	9.8%	1,755	19,270	3,854	15,416	7,707	9,462
Nov	0.0	36.6	0.0%	0	14,986	2,997	11,989	0	0
Dec	0.0	36.3	0.0%	0	14,891	2,978	11,913	0	0

Yearly Totals	78,882	216,957

Phase 2 Precipitation and Sublimation Calculations

	Heap Leach Pad		
Heap Contributing Area (phase 2)=	940,920	m^2	assumes diversion channels in place upstream of Pad
Sublimation losses	13.0%		

Phase 1
Phase 2
Phase 3

	Climate Data								
Date	**Rainfall**	**Snowfall SWE**	**Snowmelt Distribution per month**	**Rain on Heap Area**	**Snowfall SWE on Heap Area**	**Sublimation**	**Snowfall SWE Minus Sublimation**	**Snowmelt**	**Precipitation and Sublimation, Phase 2**
	(mm)	(mm)	(%)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)
Jan	0.0	26.0	0.0%	0	24,502	3,185	21,316	0	0
Feb	0.0	20.5	0.0%	0	19,289	2,508	16,781	0	0
Mar	0.0	26.8	0.0%	0	25,207	3,277	21,930	0	0
Apr	0.2	23.3	8.0%	160	21,905	2,848	19,057	15,789	15,949
May	23.3	7.7	67.9%	21,886	7,236	941	6,295	133,632	155,518
Jun	84.2	2.1	8.4%	79,188	1,985	258	1,727	16,576	95,764
Jul	100.0	0.0	0.0%	94,054	0	0	0	0	94,054
Aug	72.5	0.6	0.2%	68,207	527	68	458	443	68,650
Sep	52.5	13.7	5.7%	49,370	12,844	1,670	11,174	11,162	60,532
Oct	4.3	47.0	9.8%	4,027	44,223	5,749	38,474	19,234	23,261
Nov	0.0	36.6	0.0%	0	34,391	4,471	29,920	0	0
Dec	0.0	36.3	0.0%	0	34,174	4,443	29,732	0	0

Yearly Totals		
	196,865	513,728

TETRA TECH

Phase 3 Precipitation and Sublimation Calculations

	Heap Leach Pad		
Heap Contributing Area (phase 3)=	1,134,970	m^2	assumes diversion channels in place upstream of Pad
Sublimation losses	13.0%		

Phase 1
Phase 2
Phase 3

	Climate Data								
Date	**Rainfall**	**Snowfall SWE**	**Snowmelt Distribution per month**	**Rain on Heap Area**	**Snowfall SWE on Heap Area**	**Sublimation**	**Snowfall SWE Minus Sublimation**	**Snowmelt**	**Precipitation and Sublimation, Phase 2**
	(mm)	(mm)	(%)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)
Jan	0.0	26.0	0.0%	0	29,555	3,842	25,713	0	0
Feb	0.0	20.5	0.0%	0	23,267	3,025	20,242	0	0
Mar	0.0	26.8	0.0%	0	30,406	3,953	26,453	0	0
Apr	0.2	23.3	8.0%	193	26,422	3,435	22,987	19,045	19,238
May	23.3	7.7	67.9%	26,399	8,728	1,135	7,593	161,192	187,591
Jun	84.2	2.1	8.4%	95,519	2,395	311	2,083	19,995	115,514
Jul	100.0	0.0	0.0%	113,452	0	0	0	0	113,452
Aug	72.5	0.6	0.2%	82,274	636	83	553	535	82,809
Sep	52.5	13.7	5.7%	59,552	15,492	2,014	13,478	13,464	73,016
Oct	4.3	47.0	9.8%	4,858	53,344	6,935	46,409	23,200	28,058
Nov	0.0	36.6	0.0%	0	41,483	5,393	36,090	0	0
Dec	0.0	36.3	0.0%	0	41,222	5,359	35,863	0	0

Yearly Totals		
	237,466	619,677

Environmental Contribution Water

Phase 1
Phase 2
Phase 3

Solution Application 66,480 m^3/day

Date	# days per month	Top Area	Evaporation Data	Est. Emitter Evaporation	Precipitation and Sublimation	Evaporation from Precipitation	Solution Application Drip Emitter Evaporation	Monthly Environmental Contribution Water
		(m^2)	(mm)	(%)	(m^3)	(m^3)	(m^3)	(m^3)
Mar-13	31	53,597	0.0	0.0%	0	0	0	0
Apr-13	30	87,616	5.6	0.0%	6,396	495	0	5,901
May-13	31	127,130	46.9	1.0%	63,082	5,959	20,609	36,514
Jun-13	30	127,130	77.1	1.3%	41,148	9,806	25,927	5,415
Jul-13	31	163,164	81.3	1.2%	40,984	13,272	24,731	2,981
Aug-13	31	163,164	61.8	1.0%	29,899	10,090	20,609	-800
Sep-13	30	161,291	24.7	0.6%	25,986	3,985	11,966	10,034
Oct-13	31	161,291	1.6	0.0%	9,462	258	0	9,203
Nov-13	30	160,047	0.0	0.0%	0	0	0	0
Dec-13	31	160,047	0.0	0.0%	0	0	0	0
Jan-14	31	160,047	0.0	0.0%	0	0	0	0
Feb-14	28	160,047	0.0	0.0%	0	0	0	0
Mar-14	31	160,047	0.0	0.0%	0	0	0	0
Apr-14	30	157,319	5.6	0.0%	6,396	888	0	5,508
May-14	31	157,319	46.9	1.0%	63,082	7,374	20,609	35,099
Jun-14	30	140,759	77.1	1.3%	41,148	10,857	25,927	4,363
Jul-14	31	140,759	81.3	1.2%	40,984	11,450	24,731	4,804
Aug-14	31	146,193	61.8	1.0%	29,899	9,041	20,609	250
Sep-14	30	146,193	24.7	0.6%	25,986	3,612	11,966	10,407
Oct-14	31	155,955	1.6	0.0%	9,462	250	0	9,212
Nov-14	30	155,955	0.0	0.0%	0	0	0	0
Dec-14	31	155,955	0.0	0.0%	0	0	0	0
Jan-15	31	155,955	0.0	0.0%	0	0	0	0
Feb-15	28	155,955	0.0	0.0%	0	0	0	0
Mar-15	31	161,644	0.0	0.0%	0	0	0	0
Apr-15	30	161,644	5.6	0.0%	15,949	912	0	15,036
May-15	31	163,991	46.9	1.0%	155,518	7,687	20,609	127,222
Jun-15	30	163,991	77.1	1.3%	95,764	12,649	25,927	57,187
Jul-15	31	163,991	81.3	1.2%	94,054	13,340	24,731	55,984
Aug-15	31	179,184	61.8	1.0%	68,650	11,081	20,609	36,961
Sep-15	30	179,184	24.7	0.6%	60,532	4,427	11,966	44,139
Oct-15	31	193,864	1.6	0.0%	23,261	311	0	22,950
Nov-15	30	193,864	0.0	0.0%	0	0	0	0
Dec-15	31	193,864	0.0	0.0%	0	0	0	0
Jan-16	31	193,864	0.0	0.0%	0	0	0	0
Feb-16	29	193,864	0.0	0.0%	0	0	0	0
Mar-16	31	193,864	0.0	0.0%	0	0	0	0
Apr-16	30	211,800	5.6	0.0%	15,949	1,196	0	14,753
May-16	31	211,800	46.9	1.0%	155,518	9,928	20,609	124,981
Jun-16	30	211,800	77.1	1.3%	95,764	16,337	25,927	53,500
Jul-16	31	236,072	81.3	1.2%	94,054	19,203	24,731	50,121
Aug-16	31	236,072	61.8	1.0%	68,650	14,599	20,609	33,443
Sep-16	30	236,072	24.7	0.6%	60,532	5,833	11,966	42,733
Oct-16	31	258,226	1.6	0.0%	23,261	414	0	22,847
Nov-16	30	258,226	0.0	0.0%	0	0	0	0
Dec-16	31	258,226	0.0	0.0%	0	0	0	0
Jan-17	31	258,226	0.0	0.0%	0	0	0	0
Feb-17	28	258,226	0.0	0.0%	0	0	0	0
Mar-17	31	258,226	0.0	0.0%	0	0	0	0
Apr-17	30	258,226	5.6	0.0%	15,949	1,458	0	14,491
May-17	31	282,871	46.9	1.0%	155,518	13,260	20,609	121,650
Jun-17	30	282,871	77.1	1.3%	95,764	21,819	25,927	48,017
Jul-17	31	282,871	81.3	1.2%	94,054	23,010	24,731	46,314
Aug-17	31	315,199	61.8	1.0%	68,650	19,492	20,609	28,550
Sep-17	30	315,199	24.7	0.6%	60,532	7,788	11,966	40,778
Oct-17	31	315,199	1.6	0.0%	23,261	505	0	22,756
Nov-17	30	315,199	0.0	0.0%	0	0	0	0
Dec-17	31	315,199	0.0	0.0%	0	0	0	0
Jan-18	31	315,199	0.0	0.0%	0	0	0	0
Feb-18	28	315,199	0.0	0.0%	0	0	0	0
Mar-18	31	315,199	0.0	0.0%	0	0	0	0
Apr-18	30	335,503	5.6	0.0%	15,949	1,894	0	14,055
May-18	31	335,503	46.9	1.0%	155,518	15,727	20,609	119,182
Jun-18	30	335,503	77.1	1.3%	95,764	25,879	25,927	43,958
Jul-18	31	335,503	81.3	1.2%	94,054	27,291	24,731	42,033
Aug-18	31	337,429	61.8	1.0%	68,650	20,867	20,609	27,175
Sep-18	30	337,429	24.7	0.6%	60,532	8,337	11,966	40,229
Oct-18	31	337,429	1.6	0.0%	23,261	541	0	22,720
Nov-18	30	337,429	0.0	0.0%	0	0	0	0
Dec-18	31	337,429	0.0	0.0%	0	0	0	0
Jan-19	31	337,429	0.0	0.0%	0	0	0	0
Feb-19	28	337,429	0.0	0.0%	0	0	0	0
Mar-19	31	337,429	0.0	0.0%	0	0	0	0
Apr-19	30	320,368	5.6	0.0%	19,238	1,808	0	17,429
May-19	31	320,368	46.9	1.0%	187,591	15,017	20,609	151,965
Jun-19	30	320,368	77.1	1.3%	115,514	24,712	25,927	64,875
Jul-19	31	320,368	81.3	1.2%	113,452	26,060	24,731	62,661
Aug-19	31	291,352	61.8	1.0%	82,809	18,017	20,609	44,182
Sep-19	30	291,352	24.7	0.6%	73,016	7,198	11,966	53,851
Oct-19	31	291,352	1.6	0.0%	28,058	467	0	27,591
Nov-19	30	258,618	0.0	0.0%	0	0	0	0
Dec-19	31	258,618	0.0	0.0%	0	0	0	0
Jan-20	31	258,618	0.0	0.0%	0	0	0	0
Feb-20	29	258,618	0.0	0.0%	0	0	0	0
Mar-20	31	258,618	0.0	0.0%	0	0	0	0
Apr-20	30	258,618	5.6	0.0%	19,238	1,460	0	17,778
May-20	31	225,308	46.9	1.0%	187,591	10,561	20,609	156,421
Jun-20	30	225,308	77.1	1.3%	115,514	17,379	25,927	72,207
Jul-20	31	225,308	81.3	1.2%	113,452	18,327	24,731	70,394
Aug-20	31	193,152	61.8	1.0%	82,809	11,944	20,609	50,255
Sep-20	30	193,152	24.7	0.6%	73,016	4,772	11,966	56,278
Oct-20	31	159,887	1.6	0.0%	28,058	256	0	27,802
Nov-20	30	159,887	0.0	0.0%	0	0	0	0
Dec-20	31	159,887	0.0	0.0%	0	0	0	0
Jan-21	31	159,887	0.0	0.0%	0	0	0	0
Feb-21	28	159,887	0.0	0.0%	0	0	0	0
Mar-21	31	125,841	0.0	0.0%	0	0	0	0
Apr-21	30	125,841	5.6	0.0%	19,238	710	0	18,527
May-21	31	102,182	46.9	1.0%	187,591	4,790	20,609	162,192
Jun-21	30	71,381	77.1	1.3%	115,514	5,506	25,927	84,080
Jul-21	31	10,201	81.3	1.2%	113,452	830	24,731	87,891
Aug-21	31	10,201	61.8	1.0%	82,809	631	20,609	61,569
Sep-21	30	10,201	24.7	0.6%	73,016	252	11,966	60,798
Oct-21	31	10,201	1.6	0.0%	28,058	16	0	28,042
Nov-21	30	10,201	0.0	0.0%	0	0	0	0

Monthly Summary and Makeup Water Requirements

	Year Round - Crusher 1	From Stockpile		
Ore Production Rate =	29,500	11,800	41,300	tons/day
	16,389	6,556	22,944	m^3/day

Solution Application	66,480	m^3/day	
Solution Recovery	0	m^3/day	
As-Delivered Ore Water from Pit	0.09	m^3/ m3 ore	from Water Content Sheet
Optimal Operating Water	0.23	m^3/ m3 ore	from Water Content Sheet
As-Delivered Ore Water from Stockpile	0.13	m^3/ m3 ore	from Water Content Sheet

Phase 1
Phase 2
Phase 3

Date	# days per month	# Heap Days	Ore Water from Pit	Ore Water from Stockpile	Optimal Operating Water	Water Needed to Reach 13.3% WC	Environmental Contribution Water	Monthly Water Summary	Makeup Water Required
			(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)
Mar-13	31	30	41,792		111,166	(69,374)	0	(69,374)	69,374
Apr-13	30	29	40,387		107,429	(67,042)	5,901	(61,141)	61,141
May-13	31	30	41,792		111,166	(69,374)	36,514	(32,860)	32,860
Jun-13	30	29	40,387		107,429	(67,042)	5,415	(61,628)	61,628
Jul-13	31	30	41,792		111,166	(69,374)	2,981	(66,393)	66,393
Aug-13	31	30	41,792		111,166	(69,374)	-800	(70,174)	70,174
Sep-13	30	29	40,387		107,429	(67,042)	10,034	(57,008)	57,008
Oct-13	31	30	41,792		111,166	(69,374)	9,203	(60,171)	60,171
Nov-13	30	15	21,071		56,050	(34,979)	0	(34,979)	34,979
Dec-13	31	0	0		0	0	0	0	0
Jan-14	31	0	0		0	0	0	0	0
Feb-14	28	0	0		0	0	0	0	0
Mar-14	31	30	41,792	25,092	155,632	(88,748)	0	(88,748)	88,748
Apr-14	30	29	40,387	24,249	150,401	(85,765)	5,508	(80,257)	80,257
May-14	31	30	41,792	25,092	155,632	(88,748)	35,099	(53,649)	53,649
Jun-14	30	29	40,387	24,249	150,401	(85,765)	4,363	(81,401)	81,401
Jul-14	31	30	41,792	25,092	155,632	(88,748)	4,804	(83,944)	83,944
Aug-14	31	30	41,792	25,092	155,632	(88,748)	250	(88,498)	88,498
Sep-14	30	29	40,387	24,249	150,401	(85,765)	10,407	(75,358)	75,358
Oct-14	31	30	41,792	25,092	155,632	(88,748)	9,212	(79,536)	79,536
Nov-14	30	15	21,071	12,652	78,470	(44,747)	0	(44,747)	44,747
Dec-14	31	0	0	0	0	0	0	0	0
Jan-15	31	0	0	0	0	0	0	0	0
Feb-15	28	0	0	0	0	0	0	0	0
Mar-15	31	30	41,792	25,092	155,632	(88,748)	0	(88,748)	88,748
Apr-15	30	29	40,387	24,249	150,401	(85,765)	15,036	(70,729)	70,729
May-15	31	30	41,792	25,092	155,632	(88,748)	127,222	38,474	0
Jun-15	30	29	40,387	24,249	150,401	(85,765)	57,187	(28,578)	0
Jul-15	31	30	41,792	25,092	155,632	(88,748)	55,984	(32,764)	22,868
Aug-15	31	30	41,792	25,092	155,632	(88,748)	36,961	(51,787)	51,787
Sep-15	30	29	40,387	24,249	150,401	(85,765)	44,139	(41,626)	41,626
Oct-15	31	30	41,792	25,092	155,632	(88,748)	22,950	(65,798)	65,798
Nov-15	30	15	21,071	12,652	78,470	(44,747)	0	(44,747)	44,747
Dec-15	31	0	0	0	0	0	0	0	0
Jan-16	31	0	0	0	0	0	0	0	0
Feb-16	29	0	0	0	0	0	0	0	0
Mar-16	31	30	41,792	25,092	155,632	(88,748)	0	(88,748)	88,748
Apr-16	30	29	40,387	24,249	150,401	(85,765)	14,753	(71,012)	71,012
May-16	31	30	41,792	25,092	155,632	(88,748)	124,981	36,233	0
Jun-16	30	29	40,387	24,249	150,401	(85,765)	53,500	(32,265)	0
Jul-16	31	30	41,792	25,092	155,632	(88,748)	50,121	(38,627)	34,660
Aug-16	31	30	41,792	25,092	155,632	(88,748)	33,443	(55,305)	55,305
Sep-16	30	29	40,387	24,249	150,401	(85,765)	42,733	(43,032)	43,032
Oct-16	31	30	41,792	25,092	155,632	(88,748)	22,847	(65,901)	65,901
Nov-16	30	15	21,071	12,652	78,470	(44,747)	0	(44,747)	44,747
Dec-16	31	0	0	0	0	0	0	0	0
Jan-17	31	0	0	0	0	0	0	0	0
Feb-17	28	0	0	0	0	0	0	0	0
Mar-17	31	30	41,792	25,092	155,632	(88,748)	0	(88,748)	88,748
Apr-17	30	29	40,387	24,249	150,401	(85,765)	14,491	(71,274)	71,274
May-17	31	30	41,792	25,092	155,632	(88,748)	121,650	32,901	0
Jun-17	30	29	40,387	24,249	150,401	(85,765)	48,017	(37,747)	37,747
Jul-17	31	30	41,792	25,092	155,632	(88,748)	46,314	(42,434)	42,434
Aug-17	31	30	41,792	25,092	155,632	(88,748)	28,550	(60,198)	60,198
Sep-17	30	29	40,387	24,249	150,401	(85,765)	40,778	(44,987)	44,987
Oct-17	31	30	41,792	25,092	155,632	(88,748)	22,756	(65,992)	65,992
Nov-17	30	15	21,071	12,652	78,470	(44,747)	0	(44,747)	44,747
Dec-17	31	0	0	0	0	0	0	0	0
Jan-18	31	0	0	0	0	0	0	0	0
Feb-18	28	0	0	0	0	0	0	0	0
Mar-18	31	30	41,792	25,092	155,632	(88,748)	0	(88,748)	88,748
Apr-18	30	29	40,387	24,249	150,401	(85,765)	14,055	(71,710)	71,710
May-18	31	30	41,792	25,092	155,632	(88,748)	119,182	30,434	0
Jun-18	30	29	40,387	24,249	150,401	(85,765)	43,958	(41,807)	41,807
Jul-18	31	30	41,792	25,092	155,632	(88,748)	42,033	(46,715)	46,715
Aug-18	31	30	41,792	25,092	155,632	(88,748)	27,175	(61,573)	61,573
Sep-18	30	29	40,387	24,249	150,401	(85,765)	40,229	(45,536)	45,536
Oct-18	31	30	41,792	25,092	155,632	(88,748)	22,720	(66,028)	66,028
Nov-18	30	15	21,071	12,652	78,470	(44,747)	0	(44,747)	44,747
Dec-18	31	0	0	0	0	0	0	0	0
Jan-19	31	0	0	0	0	0	0	0	0
Feb-19	28	0	0	0	0	0	0	0	0
Mar-19	31	30	41,792	25,092	155,632	(88,748)	0	(88,748)	88,748
Apr-19	30	29	40,387	24,249	150,401	(85,765)	17,429	(68,336)	68,336
May-19	31	30	41,792	25,092	155,632	(88,748)	151,965	63,217	0
Jun-19	30	29	40,387	24,249	150,401	(85,765)	64,875	(20,890)	0
Jul-19	31	30	41,792	25,092	155,632	(88,748)	62,661	(26,087)	0
Aug-19	31	30	41,792	25,092	155,632	(88,748)	44,182	(44,566)	28,325
Sep-19	30	29	40,387	24,249	150,401	(85,765)	53,851	(31,913)	31,913
Oct-19	31	30	41,792	25,092	155,632	(88,748)	27,591	(61,157)	61,157
Nov-19	30	15	21,071	12,652	78,470	(44,747)	0	(44,747)	44,747
Dec-19	31	0	0	0	0	0	0	0	0
Jan-20	31	0	0	0	0	0	0	0	0
Feb-20	29	0	0	0	0	0	0	0	0
Mar-20	31	30	41,792	25,092	155,632	(88,748)	0	(88,748)	88,748
Apr-20	30	29	40,387	24,249	150,401	(85,765)	17,778	(67,987)	67,987
May-20	31	30	41,792	25,092	155,632	(88,748)	156,421	67,673	0
Jun-20	30	29	40,387	24,249	150,401	(85,765)	72,207	(13,558)	0
Jul-20	31	30	41,792	25,092	155,632	(88,748)	70,394	(18,354)	0
Aug-20	31	30	41,792	25,092	155,632	(88,748)	50,255	(38,493)	2,732
Sep-20	30	29	40,387	24,249	150,401	(85,765)	56,278	(29,487)	29,487
Oct-20	31	30	41,792	25,092	155,632	(88,748)	27,802	(60,946)	60,946
Nov-20	30	15	21,071	12,652	78,470	(44,747)	0	(44,747)	44,747
Dec-20	31	0	0	0	0	0	0	0	0
Jan-21	31	0	0	0	0	0	0	0	0
Feb-21	28	0	0	0	0	0	0	0	0
Mar-21	31	30	41,792	25,092	155,632	(88,748)	0	(88,748)	88,748
Apr-21	30	29	40,387	24,249	150,401	(85,765)	18,527	(67,238)	67,238
May-21	31	30	41,792	25,092	155,632	(88,748)	162,192	73,444	0
Jun-21	30	29	40,387	24,249	150,401	(85,765)	84,080	(1,684)	0
Jul-21	31	30	41,792	25,092	155,632	(88,748)	87,891	(857)	0
Aug-21	31	30	41,792	25,092	155,632	(88,748)	61,569	(27,179)	0
Sep-21	30	29	40,387	24,249	150,401	(85,765)	60,798	(24,967)	0
Oct-21	31	30	41,792	25,092	155,632	(88,748)	28,042	(60,706)	41,949
Nov-21	30	15	21,071	12,652	78,470	(44,747)	0	(44,747)	44,747

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Technical Memorandum

To:	File	**From:**	Ronson Chee
Company:		**Date:**	February 9, 2012
Re:	In-Heap Pond, Spillway and Event Ponds Sizing	**Doc #:**	114-320905X-5.3
CC:	Troy Meyer, P.E. (Tetra Tech)		

1.0 Introduction

This technical memorandum explains the methodology and assumptions used in sizing the In-Heap Pond and Event Ponds associated with the Eagle Gold Heap Leach Facility (HLF). The In-Heap Pond is the storage volume created within the pore space of the ore, directly upstream of the Confining Embankment (Embankment). The In-Heap Pond will store solution within the pore space of the ore to facilitate maximum recovery of solution and allow for leaching operations during the winters' freezing temperatures. The Event Ponds, which consist of two (2) adjacent ponds located down gradient of the Heap Leach Facility and Embankment, will store solution and/or storm event runoff in excess of the capacity of the In-Heap Pond. The capacity of the In-Heap Pond is a function of the height of the Embankment and the porosity of the ore. Incoming flows in excess of the In-Heap Pond capacity will spill over the Embankment into the Event Ponds via the Heap Leach Facility Spillway (Spillway). Accordingly, this memorandum will determine the necessary height and volume requirements of the Confining Embankment, the design dimensions of the Spillway, and the volume requirements of the Event Ponds.

The construction timing and phasing of HLF will ultimately have an effect on the contributing areas and precipitation volumes reporting to the In-Heap Pond. As the Eagle Gold Project advances and more detailed phasing information becomes available, the HLF design can be optimized. The design as presented is based on the current information/design criteria available at the time and may be subject to change as the information/design criteria is updated.

The In-Heap Pond and the Event Ponds were sized to provide storage for leaching solution and selected precipitation events. Sizing the In-Heap Pond assumed that all precipitation that falls upstream of the Embankment and downstream of the Phase 1 temporary diversion channels contributes to the In-Heap Pond. The Phase 1 configuration of the HLF was selected as the critical scenario as it will experience the largest lined area with the smallest amount of ore on the pad (largest runoff potential). As the pad increases to Phases 2 and 3, the entire pad will become hydraulically connected by the liner, however, this will not have significant effects on the In-Heap Pond volume. The contributing areas of Phases 2 and 3 will have minimal effects on the In-Heap Pond storage volume because the ore height will increase as well, this will significantly attenuate the infiltration of precipitation events.

2.0 In-Heap Pond Volume Requirements

The In-Heap Pond volume requirements were determined assuming a combination of events were to occur simultaneously. This approach was taken to ensure adequate storage volume under the worst of conditions. Thus, the In-Heap Pond will provide containment storage for the following summation of events:

- Minimum Operational Volume – the minimum operational volume is the minimum amount of solution required in the pond to supply the gold recovery plant for 48 hours at a nominal rate of 2,770 m^3/hr. Thus, the minimum operational volume required for the In-Heap Pond is 132,960 m^3.

- Snowmelt Runoff Volume – the snowmelt runoff volume is the volume required for snowmelt runoff. The estimated maximum average snowpack depth is 164 mm (Wardrop, 2011a) which is assumed to contribute uniformly over the Phase 1 contributing area of 399,200 m^2. Thus, the snowmelt volume reporting to the In-Heap Pond is 65,469 m^3 which assumes direct runoff (no initial losses).

- Heap Draindown Volume – in the event of a power loss (pumps stop operating), pump malfunction, or pump maintenance, the pond must be able to accommodate the draindown from the Heap. For conservatism, the largest possible draindown rate over a 72-hour period was used. The largest possible draindown volume is the nominal application rate of 2,770 m^3/hr (Wardrop, 2012) multiplied by 72 hours which equates to 199,440 m^3. It is assumed that the mine will be able to restore/repair or supply backup pumps within the 72 hours to prevent larger draindown volumes from accumulating. Should larger draindown volumes accumulate, they can be conveyed into the Events Ponds via the Spillway.

- Freeboard – 1.0 m of freeboard below the ultimate Embankment crest is required. The 1.0 m of freeboard is added above the corresponding stage-storage volume that provides the required total volume.

The summation of the In-Heap Pond volume requirements are summarized in Table 1 below. The In-Heap Pond must provide 397,869 m^3 of solution storage capacity, excluding freeboard.

Table 1. In-Heap Pond Volume Requirements Summary

Volume Requirement	Volume (m^3)
Minimum Operational Volume	132,960
Snowmelt Runoff Volume	65,469
Heap Draindown Volume	199,440
Total	**397,869**

The solution storage capacity of the In-Heap Pond excludes the runoff volume generated from the 100-year, 24-hour rainfall event. If the 100-year, 24-hour rainfall event were to occur simultaneously with the events described above, flows would be routed to the Event Ponds through the Spillway. If the 100-year, 24-hour rainfall event were to occur under normal conditions (not simultaneously with the events described), the runoff can be stored in the In-Heap Pond.

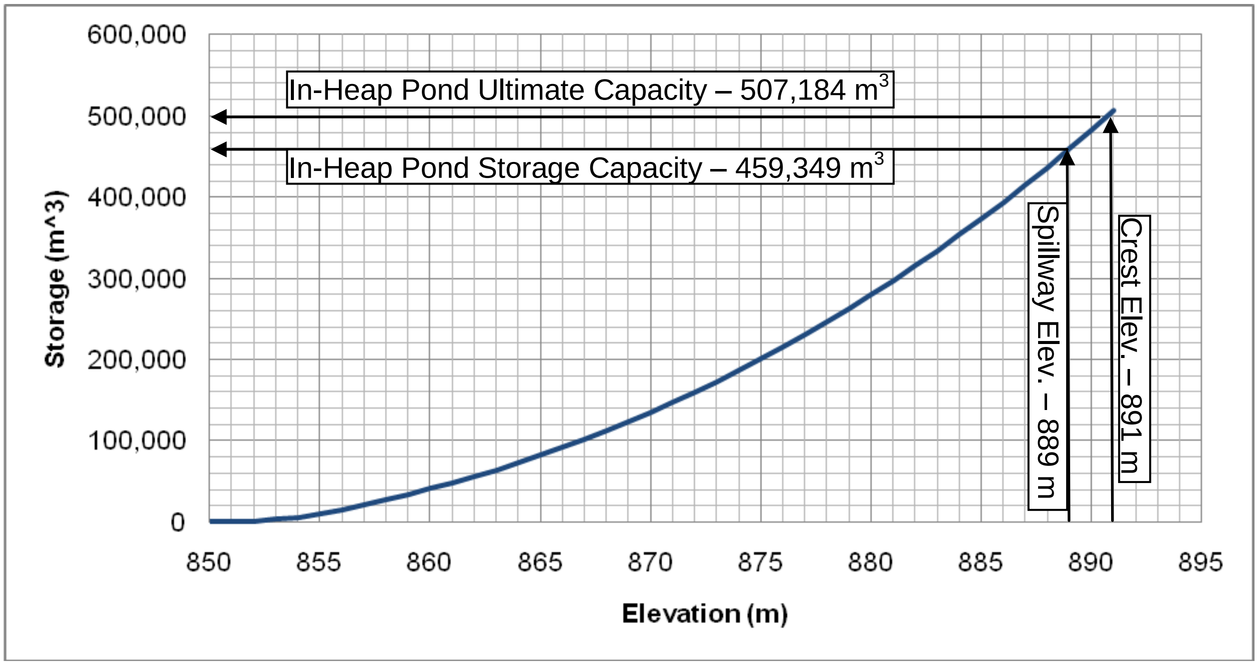
3.0 Confining Embankment Height

In order to determine the required ultimate Embankment height, a gross stage-storage curve based on the proposed Embankment and Heap Leach Pad grading was created. The actual net capacity (storage volume within the ore pore space) of the In-Heap Pond was determined by multiplying the gross storage volume by the ore storage solution factor of 0.1371 (0.1371 m^3 of solution/m^3 of ore). This factor was determined by assuming an initial ore moisture content of 5.0%, an ore bulk density of 1.8 tons/m^3, an ore specific gravity of 2.7, and an ore leaching moisture content of 13.3% (Wardrop, 2011b). Detailed calculations for derivation of the solution storage factor can be found in Attachment A.

The total required In-Heap Pond volume (397,869 m^3) was compared to the net stage-storage curve to obtain a corresponding elevation that will provide the necessary storage volume. Based on the net stage-storage curve for the In-Heap Pond, an elevation of 887 m (rounded to the next highest meter) will provide the required volume with an estimated net capacity of 414,652 m^3. To account for uncertainty in snowpack depth and snowmelt estimates, elevation 889 m was selected because it will provide an additional factor of safety to account for possible decreases in storage capacity. Decreases in storage capacity may also occur as a result of consolidation due to the loads from stacking as well as migration of fine particles into the pore space of the In-Heap Pond (i.e., agglomeration effects). However, these effects were not quantified for this level of design.

The final Embankment crest elevation will be at 891 m (2 m above the required elevation). The overflow spillway invert will be at elevation 889 m. The Spillway will be sized to allow up to 0.5 m of hydraulic head, allowing for 0.5 m of freeboard with respect to the crest of the Embankment. The ultimate storage capacity of the In-Heap Pond up to the final Embankment crest elevation (891 m) is estimated at 507,184 m^3. The In-Heap net stage-storage curve is presented in Figure 1 below (stage-storage calculations can be found in Attachment A). At its maximum, (downstream side of the embankment) the Embankment will have a height of approximately 63 m.

Figure 1. In-Heap Pond Stage-Storage – Net Volume



4.0 In-Heap Pond Spillway

The In-Heap Pond Spillway was designed in accordance with the Canadian Dam Association (CDA) guidelines for a "Very High" consequence dam. According to the CDA (2007) guidelines a dam classification of Very High suggests a design flood of 2/3 between the 1,000 year event and the Probable Maximum Flood (PMF). At this stage of the project, the 1,000 year rainfall event has not been firmly established for the site, thus, the design capacity for the spillway was increased to the full PMF. The PMF rainfall event for the Project site has been established by Knight Piesold (2011). According to the CDA, the PMF corresponds to an "Extreme" dam classification. As a result, the spillway capacity is conservatively estimated. Once the 1,000 year rainfall event has been established the spillway can be reduced to its required capacity.

The Spillway was sized to accommodate the PMF although the Event Ponds can only receive runoff volumes up to the 100 year, 24-hour event. The spillway's larger capacity was selected in order to eliminate the possibility of the Embankment being overtopped. Overtopping of the Embankment could compromise its stability resulting in Embankment failure. Failure of the Embankment would be an even more catastrophic event as it would result in a much larger volume of solution to be released. A higher capacity spillway will ensure that any flows in excess of the In-Heap Pond capacity can be routed downstream.

A worst case scenario was developed for calculating the largest peak flow to be experienced by the spillway. The scenario assumed that Phase 1 of the pad is constructed, Phases 2 and 3 are being cleared and grubbed for construction, and the pad is loaded with ore to elevation 889 m (roughly at the spillway invert - below the Embankment crest). The scenario also assumed that the ore is loaded to this elevation, and is completely saturated when the PMF occurs. The HLF temporary diversion channels (sized for the 100-year, 24-hour event) were assumed to have failed resulting in the entire upstream watershed contributing to the peak flow. Details of the hydrology calculations performed for sizing the Spillway are provided in the following sections.

4.1 Hydrology Methodology Overview

According to Knight Piesold (2011) the PMF can be modeled using a National Resource Conservation Service (NRCS) Type I curve with a 24-hour duration. Thus, the NRCS Method was selected to perform hydrologic calculations. The NRCS method described herein is based on two (2) components, the NRCS curve number approach (to determine initial losses and excess precipitation) and the unit hydrograph method (to derive the hydrograph resulting from excess rainfall).

The NRCS method was performed using the U.S. Army Corps of Engineer's Hydrologic Modeling System (HEC-HMS). HEC-HMS is a hydrologic modeling software package developed for general applications. HEC-HMS allows for the analysis of complex/integrated systems; i.e., multiple sub-basins, reservoir and channel routing, etc. Embedded in HEC-HMS are the NRCS method and unit hydrograph method.

4.2 Rainfall Distribution

The NRCS has developed synthetic hyetographs for the geographic U.S. for 24-hour storm events, called "type curves". The U.S. is divided into four (4) regions where specific "type curves" can be applied depending on the geographic location and on precipitation patterns. Since the project site is beyond the geographic borders of the U.S., the use of "type curves" in the Yukon Territory assumes that similar storm distribution patterns are prevalent at the project site. According to Knight Piesold (2011), a Type I curve is appropriate for the project site and can be used to model the PMF. A Type I storm is described as the Pacific maritime climate with wet winters and dry summers. According to Knight Peisold (2011), a PMF occurring from June to September was estimated to generate 256 mm of rainfall over a 24-hour period.

4.3 Rainfall Losses – Curve Number

The NRCS has developed a widely used curve number procedure for estimating runoff from storm events. The NRCS method incorporates this curve number procedure.

Rainfall initial losses depend primarily on soil characteristics and on land use (surface cover). The NRCS method uses a combination of soil conditions and land use to assign runoff factors (known as runoff curve numbers). Curve Numbers (CN) represent the runoff potential of a soil type (i.e., the higher the CN, the higher the runoff potential).

For practical purposes and due to the high likelihood of extreme rainfall events occurring during snowmelt, it was assumed that snowmelt has saturated the soil resulting in Antecedent Moisture Condition (AMC) III. AMC III increases the CN assuming a saturated soil condition. The worst case conditions were assumed in selecting the CN. It was assumed that the entire Heap Leach Pad has been cleared and grubbed (prior to construction of the pad) and the storm occurs. A CN of 86 was chosen to model this scenario, it corresponds with a hydrologic soil group of B and characterized as a "newly graded area" with no vegetation (Mays, 2005). Applying AMC III results in a CN of 93 which was used as input into the HEC-HMS model.

4.4 Rainfall Run-off Volume

The NRCS method determines rainfall runoff volume using the following relationship:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}$$

Where:

Q = the accumulated runoff volume in millimeters (mm);

P = the accumulated precipitation in millimeters (mm);

S = the maximum soil water retention parameter ($S = \frac{1000}{CN} - 10$) in millimeters (mm); and

CN = the curve number.

The calculated PMF runoff volume for the watershed upstream of the Heap Embankment was estimated to be 302,000 m^3 (see Attachment B).

4.5 Time of Concentration / Lag Time

The time of concentration (T_c) used for the NRCS method was determined using an average of the T_c's as recommended by Coulson (1991). Two methods for calculating T_c were taken from the Manual of Operational Hydrology in British Columbia; one (1), the Hathaway formula, which a function of basin slope, roughness and stream length; and two (2), the T_c based on the area and steepness of the basin taken from a chart. The calculated T_c's were then converted to lag time which is defined by the NRCS method as $0.6{*}T_c$. The average T_c value calculated was 33.6 minutes. This was used as input into the HEC-HMS model.

4.6 Spillway Peak Flow

The calculated peak flow for the PMF using HEC-HMS and the methods mentioned above is estimated to be 26.7 m^3/s. The summary of hydrology calculations can be found in Attachment B.

5.0 In-Heap Pond Spillway Hydraulics

Using the PMF peak flow calculated in the previous section, the flow capacity of the Spillway was estimated using two methods; the weir equation and Manning's equation for open channel flow.

5.1 Weir Equation

The spillway inlet capacity can be estimated using a weir equation as taken from Mays (2005) which is defined as:

$$Q = CLH^{3/2}$$

Where:

Q = the channel flow rate, in cubic meters per second (m^3/s);

C = the discharge coefficient (a conservative value of 3.0 was used);

L = the effective length of the crest, in meters (m); and

H = the head water on the crest, in meters (m).

Using a peak flow of 26.7 m^3/s, the selected dimensions to meet the required flows were a length of 5 m and a head water of 1.5 m. A spillway with a 2 m depth will allow a freeboard of 0.5 m.

5.2 Manning's Equation

The capacity of the shallowest sloping portion (inlet and outlet) of the Spillway was also estimated using Manning's Equation taken from Mays (2005):

$$Q = \frac{1}{n} AR^{2/3} S^{1/2}$$

Where:

Q = the channel flow rate, in cubic meters per second (m^3/s);

A = the cross sectional area of flow, in square meters (m^2);

R = the hydraulic radius of flow, in meters (m);

S = the longitudinal slope of the flow path for the channel, in millimeters over millimeters (m/m); and

n = Manning's roughness coefficient for the channel, (unitless).

Using the minimum slope on the spillway profile (0.5%), a roughness coefficient of 0.013 (concrete) and the rectangular dimensions calculated in Section 5.3, Manning's equation gives a flow depth of 1.15 m. This gives 0.85 m of freeboard in the Spillway.

5.3 Spillway Dimensions

Based on Spillway hydraulic calculations, the weir equation governs the Spillway dimensions. Thus, the Spillway will be rectangular with a width of 5.0 m and a depth of 2.0 m. The spillway will have an invert of 889 m. This configuration allows a 0.5 m of freeboard from the maximum anticipated water surface elevation to the crest of the Embankment.

6.0 Event Ponds Volume Requirements

The capacity of the Event Ponds are dependent on the events retained in the In-Heap Pond as described in section 2.0. The Event Ponds serve as an overflow containment area that provides additional storage in case the In-Heap Pond capacity is exceeded, or may also serve as a temporary storage area during ”wet” months when excess recycle water is available from leaching operations as determined in the heap leach facility water balance (Tetra Tech, 2012). The Event Ponds are sized to provide containment storage for the following:

- 100-year, 24-hour Rainfall – the Event Ponds will provide storage for the 100-year, 24-hour rainfall event. The 100-year, 24-hour rainfall event rainfall depth was estimated to be 103.2 mm (Knight Piesold, 2011) which is assumed to contribute entirely over the maximum contributing area (Phases 1, 2 and 3) of 1,281,000 m^3. Thus, the rainfall volume reporting to the Event Ponds is 132,200 m^3 which assumes direct runoff (i.e. no losses).

- Freeboard – 1.0 m of freeboard below the crest of the pond is required.

The Event Ponds must provide 132,200 m^3 of solution storage excluding freeboard. The configuration of the Event Ponds have a combined operational storage capacity of approximately 182,846 m^3 excluding freeboard. Event Pond 1 (closest to the Embankment) has a storage capacity of 92,153 m^3 and Event Pond 2 (farthest from the Embankment) has a storage capacity of 90,693 m^3. The combined ultimate storage capacity of the Event Ponds including 1.0 m of freeboard is 216,713 m^3. The stage-storage functions for both ponds are provided below (stage-storage calculations can be found in Attachment A).

7.0 Summary

The In-Heap Pond will provide a storage capacity of 459,349 m^3 at elevation 889m which also corresponds to the Spillway invert. The ultimate Confining Embankment crest elevation is 891m which corresponds to the ultimate storage capacity of 507,184 m^3. The Spillway is sized to accommodate the PMF peak flow with 0.5 m of dry freeboard from the Embankment crest, additionally. The Event Ponds have a combined storage capacity of 182,845 m^3 with 1.0 m of dry freeboard. The ultimate storage capacity of the Event Ponds is 216,713 m^3 (without any freeboard).

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ATTACHMENT A
IN-HEAP POND AND EVENT PONDS
STAGE-STORAGE CALCULATIONS

In-Heap Pond Ore Solution Storage Calculations

Equations:

$$\gamma_{dry} = \frac{\gamma_{bulk}}{1+w} \qquad V = V_a + V_w + V_s = 1 \qquad \text{assuming 1 m}^3 \text{ of ore}$$

$$W_w = wW_s \qquad V_v = V_a + V_w$$

$$V_w = \frac{W_w}{\gamma_w} \qquad n = \frac{V_v}{V}$$

$$V_s = \frac{W_s}{G_s\gamma_w} \qquad Se = wG_s$$

$$V_a = 1 - V_s - V_w$$

Initial Conditions (Ore as Delivered)

Specific Gravity G_s =	2.7	
Density of Water (γ_w) =	1000	kg/m^3
Water Content as Delivered (w) =	5.00%	
Ore Density (γ_{bulk}) =	1,800	kg/m^3
Ore Density (γ_{dry}) =	1714.29	kg/m^3
Weight of Water (W_w) =	85.71	kg/m^3
Volume of Water (V_w) =	0.0857	m^3
Volume of Solids (V_s) =	0.6349	m^3
Volume of Air (V_a) =	0.2794	m^3/m^3 of ore
porosity (n) =	0.3651	
void ratio (e) =	0.5750	
Saturation (S) =	23.48%	

Leaching Conditions/Ore Solution Storage Capacity

Water content (w) =	13.30%		
Weight of Water (W_w) =	228.00	kg/m^3	
Volume of Water (V_w) =	0.2280	m^3	
Volume of Solids (V_s) =	0.6349	m^3	
Volume of Air (V_a) =	**0.1371**	m^3/m^3 of ore	⇦ Ore Solution Storage Capacity at 13.3% initial water content
void ratio (e) =	0.5750	check - OK	(water required to bring to full saturation S=1)
Saturation (S) =	62.45%		

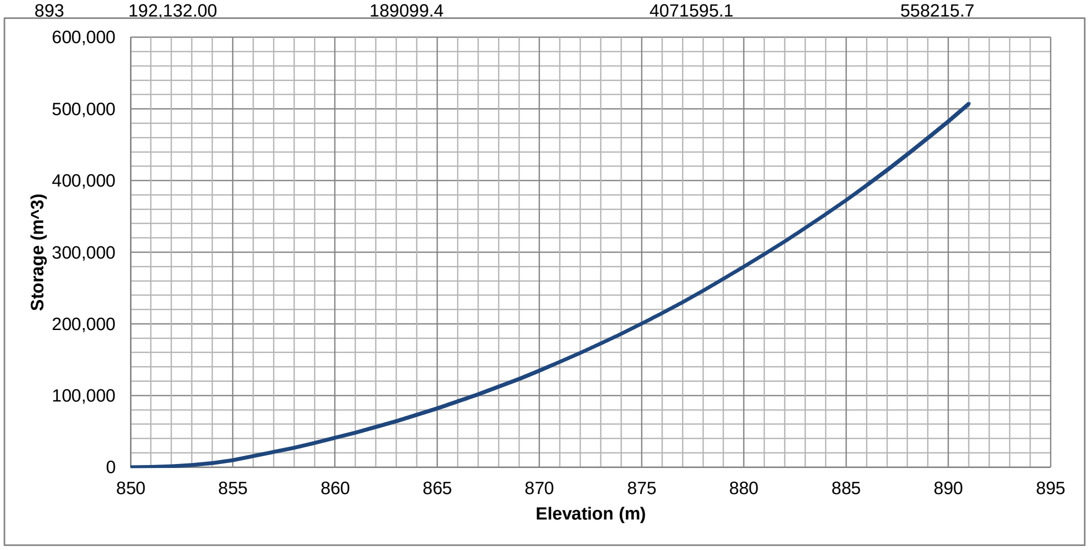
Ore properties provided by Wardrop (2011) in *Heap Leach Facility Design Criteria* , July, 2011

Attachment A to the Technical Memorandum titled
In-Heap Pond, Spillway and Event Pond Sizing

In-Heap Pond Stage-Storage

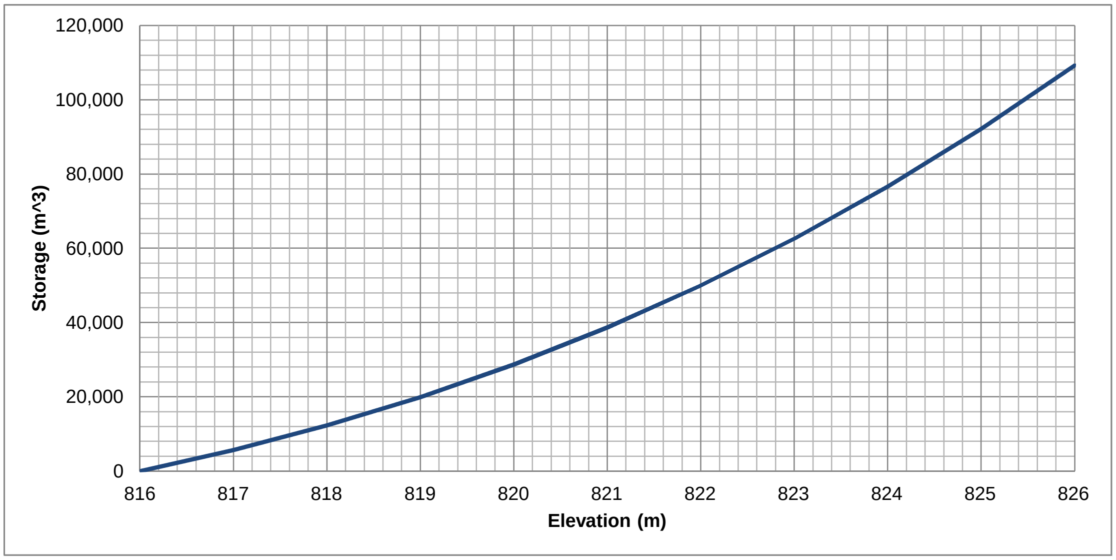
Elevation-Area Function

ELEV	AREA (m^2)	INCREMENTAL VOL (m^3)	GROSS TOTAL VOL (m^3)	NET TOTAL VOL (in Voids)
850	1,056.24	0.0	0.0	0.0
851	4,738.19	2897.2	2897.2	397.2
852	9,397.83	7068.0	9965.2	1366.2
853	15,328.82	12363.3	22328.6	3061.2
854	22,503.36	18916.1	41244.6	5654.6
855	38,575.42	30539.4	71784.0	9841.6
856	41,169.12	39872.3	111656.3	15308.1
857	43,839.58	42504.4	154160.7	21135.4
858	46,586.73	45213.2	199373.8	27334.1
859	49,410.57	47998.7	247372.5	33914.8
860	52,311.09	50860.8	298233.3	40887.8
861	55,313.72	53812.4	352045.7	48265.5
862	58,403.70	56858.7	408904.4	56060.8
863	61,581.05	59992.4	468896.8	64285.7
864	64,846.54	63213.8	532110.6	72952.4
865	68,197.83	66522.2	598632.8	82072.6
866	71,568.12	69883.0	668515.7	91653.5
867	75,031.01	73299.6	741815.3	101702.9
868	78,586.47	76808.7	818624.0	112233.4
869	82,234.53	80410.5	899034.5	123257.6
870	85,975.17	84104.9	983139.4	134788.4
871	89,810.77	87893.0	1071032.4	146838.5
872	93,728.77	91769.8	1162802.1	159420.2
873	97,720.13	95724.5	1258526.6	172544.0
874	101,783.00	99751.6	1358278.1	186219.9
875	105,918.84	103850.9	1462129.1	200457.9
876	109,749.78	107834.3	1569963.4	215242.0
877	113,636.79	111693.3	1681656.7	230555.1
878	117,580.75	115608.8	1797265.4	246405.1
879	121,580.35	119580.6	1916846.0	262799.6
880	125,635.81	123608.1	2040454.1	279746.3
881	129,695.04	127665.4	2168119.5	297249.2
882	133,802.01	131748.5	2299868.0	315311.9
883	137,957.84	135879.9	2435747.9	333941.0
884	142,162.46	140060.2	2575808.1	353143.3
885	146,784.74	144473.6	2720281.7	372950.6
886	152,066.10	149425.4	2869707.1	393436.8
887	157,424.23	154745.2	3024452.3	414652.4
888	162,975.38	160199.8	3184652.1	436615.8
889	168,653.13	165814.3	3350466.3	459348.9
890	174,488.42	171570.8	3522037.1	482871.3
891	180,181.02	177334.7	3699371.8	507183.9
892	186,066.74	183123.9	3882495.7	532290.2
893	192,132.00	189099.4	4071595.1	558215.7



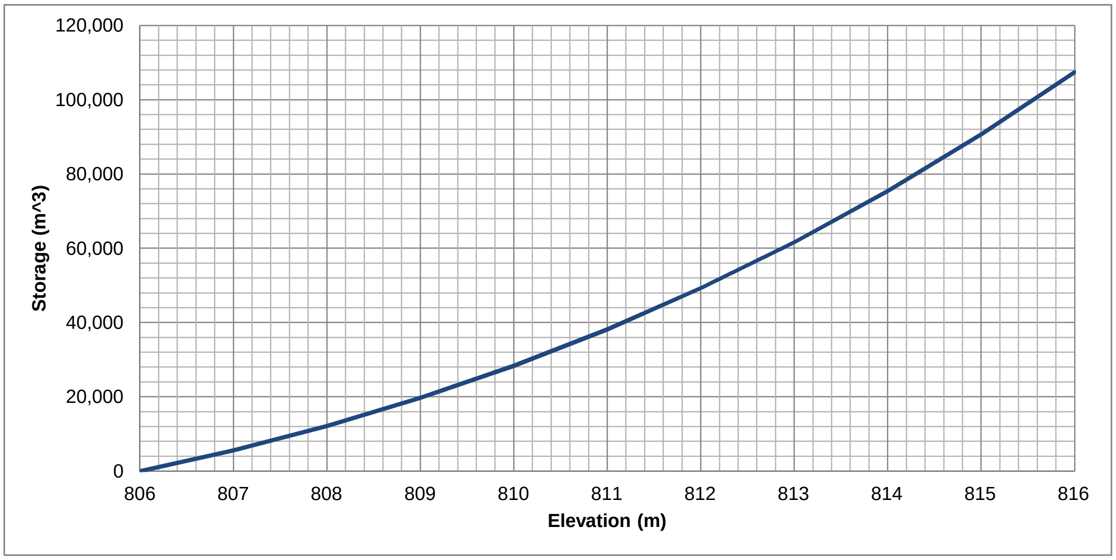
Event Pond 1 Stage-Storage

ELEV	AREA (m^2)	INCREMENTAL VOL (m^3)	TOTAL VOL (m^3)
816	5,173.41	0.0	0
817	6,111.88	5642.65	5643
818	7,124.70	6618.29	12261
819	8,211.88	7668.29	19929
820	9,373.41	8792.65	28722
821	10,609.30	9991.36	38713
822	11,919.55	11264.42	49978
823	13,304.15	12611.85	62590
824	14,763.11	14033.63	76623
825	16,296.42	15529.76	92153
826	17,904.09	17100.25	109253



Event Pond 2 Stage-Storage

ELEV	AREA (m^2)	INCREMENTAL VOL (m^3)	TOTAL VOL (m^3)
806	5,130.00	0.0	-
807	6,048.00	5589	5,589
808	7,038.00	6543	12,132
809	8,100.00	7569	19,701
810	9,234.00	8667	28,368
811	10,440.00	9837	38,205
812	11,718.00	11079	49,284
813	13,068.00	12393	61,677
814	14,490.00	13779	75,456
815	15,984.00	15237	90,693
816	17,550.00	16767	107,460



ATTACHMENT B
SPILLWAY SIZING HYDROLOGY
HEC-HMS RESULTS

TETRA TECH

Eagle Gold Hydrology Calculations Summary

Precipitation

P(100yr,24hr)=	103.20	mm	4.1	in
P(PMP, 24hr)=	256.0	mm	10.1	in

Hathaway Formula (roughness coefficient)

n= 0.3

Hydrology Calculations

	Watershed Data/HEC-HMS Input																			HEC-HMS Results			
			NRCS Parameters									Geometry			Time of Concentration/Lag Time								
	BASIN AREA		CN	CN(III)	S		Ia		Q(100,24)		Q(PMP,24)	Longest Watercourse		Average Watershed Slope	[1]NRCS T_{lag}	[2]Hathaway T_{lag}	[3]BCMOH T_{lag} Chart	[4]Kirpich	Tlag Avg	100-yr, 24 hr Volume	100-yr, 24-hr Peak Flow	PMP Volume	PMP Peak Flow
BASIN ID	(m^2)	(km^2)			(mm)	(in)	(mm)	(in)	(mm)	(in)	(mm)	(m)	(ft)	(%)	(min)	(min)	(min)	(min)	(min)	(1000 m^3)	(cms)	(1000 m^3)	(1000 m^3)
Ann Gulch / Spillway	1,281,018	1.28	86	93	18.0	1.628	3.596	0.326	84.375	2.603	235.622	1800	5906	19.2	14.7	24.1	43.2	7.1	22.3	108	9.9	302	26.7

Avg Tc **33.6**

Notes:
1) NRCS Lag time equation
2) Hathaway formula from BCMOH for small interior basins
3) Tc estimated from the British Columbia Manual of Operational Hydrology Figure 1. Steep Slope Time of Concentration p. 150
4) Kirpich formula small imperameable catchment from BCMOH

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Technical Memorandum

To:	File	**From:**	Ronson Chee
Company:		**Date:**	December 23, 2011
Re:	Dublin Gulch Diversion Channel Design	**Doc #:**	114-320905X-5.3
CC:	Troy Meyer, P.E. (Tetra Tech)		

1.0 Introduction

This technical memorandum provides the hydrologic and hydraulic design information for the Dublin Gulch Diversion Channel (Channel) for the Eagle Gold Project. Construction of the Heap Leach Facility will encroach on the natural drainage of Dublin Gulch, this will necessitate diversion of Dublin Gulch around the Leaching Facilities (Heap Leach Pad, Confining Embankment, and Event Ponds). The diversion channel capacity and armoring is sized primarily for the 100-year, 24-hour storm event with adequate freeboard requirements. The channel will also have the capacity to convey the 500-year, 24-hour event without freeboard. Armoring will not be sized to accommodate the 500-year event.

In general, the Channel will redirect flow in Dublin Gulch to the south of the proposed Heap Leach Facility and will rejoin with Dublin Gulch prior to the existing natural junction with Haggart Creek. The Dublin Gulch Diversion Channel is a large structure, and will consist of the following major components:

- A turf reinforced armored “upper channel reach” with a slope of 1.0%.
- A series of concrete armored stepped “drop structures”. A drop structure consists of: an approach inlet channel (1.0% slope); a drop chute channel (50.0% slope), an energy dissipation pool (0.0% slope); and an outlet channel (1.0% slope).
- A turf reinforced armored “lower channel reach” with a slope of 1.0% with intermediate drop structures.

The Channel will receive the majority of its flow at the inlet (flow from Dublin Gulch), in addition, it will also intercept overland flow and run-on from Eagle Pup and Stuttle Gulch. The extents of the Dublin Gulch Diversion Channel is limited to immediately downstream of the Event Ponds. The remainder of the channel is subject to the final placement of facilities downstream and is to be completed by others as determined by Victoria Gold.

The design and armoring material presented in this memorandum is based on the available site geotechnical information. The design as presented is subject to change as more site geotechnical investigation is conducted.

2.0 Hydrologic Design Flows

Hydrologic peak flows for the 100-year, 24-hour and 500-year, 24-hour events were calculated by Knight Piesold (2011) and are summarized in Table 1 below.

Table 1. Dublin Gulch Diversion Channel Design Flows

Contributing Basin/Gulch	100-year, 24-hour Design Flow (m^3/s)	500-year, 24-hour Design Flow (m^3/s)
Dublin Gulch	8.5	10.3
Eagle Pup Gulch	1.5	4.9
Stuttle Gulch	0.6	2.6
Total Design Flow	**10.6**	**17.8**

For practical purposes the total design flow was used to size the entire length of the channel.

3.0 Channel Armoring Methodology and Calculations

The Dublin Gulch Diversion Channel was designed using the appropriate hydraulic equations for each component of the Channel. Vendor supplied software was used to perform the hydraulic calculations to compute flow depth and analyze the stability of the channel armoring.

3.1 Pyramat® Turf Reinforcement Mat Armoring

Pyramat® armoring was selected to armor the 1.0% slopes of the channel. Pyramat® by Propex Geosynthetics is a three-dimensional, woven polypropylene geotextile turf reinforcement mat. It allows vegetation to grow through the mat which increases its stability. Accordingly, two (2) channel conditions were analyzed to ensure that the Pyramat® armoring would be stable throughout all construction phases; an un-vegetated condition (short term recently constructed) and a fully vegetated condition (long term). Erosion Control Design Package (EC-DESIGN) 2003 was used to perform hydraulic calculations to ensure stability of the Pyramat® Armoring for all conditions. EC-DESIGN is a vendor supplied hydraulics program created by SI Geosolutions, Inc. and is created specifically to determine the stability of the Pyramat® turf reinforcement. The program has embedded shear strength and velocity calculations that determine the factory of safety (FS) of Pyramat® for various flow conditions and channel configurations. As recommended by the vendor, Propex Geosynthetics would need to verify calculations and evaluate site specific applicability of Pyramat® for the Channel. See Attachment B for product information.

The 1.0% portions of the Channel to be armored with Pyramat® will consist of the following from bottom to top:

- A prepared subgrade;

- A prepared seedbed and applied seeding; and
- Pyramat® Turf Reinforcement Mat.

3.1.1 Un-Vegetated Condition

The un-vegetated condition of the channel will govern the stability of the Pyramat® under peak flow conditions. A recently constructed channel using Pyramat® will allow the water to flow faster due to a smoother surface which increases the velocity and shear stress on the mat. For the un-vegetated condition a Manning's "n" roughness coefficient of 0.02 was used. This is the recommended minimum roughness coefficient according to Propex Geosynthetics. The stability analysis of the mat was conducted for the 100-year and 500-year events. The un-vegetated condition is a short lived condition. Construction procedures and scheduling will have to ensure that vegetation is established before channel is put into use. Detailed Calculations can be found in Attachment A.

3.1.2 Fully Vegetated Condition

The fully vegetated condition governs the depth of the channel. A higher vegetation growth in the channel will increase the Manning's "n" value which increases the flow depth in the channel. A fully vegetated condition increases the stability of the Pyramat®. It allows the Pyramat® to resist higher velocities and withstand higher shear stresses. The NRCS vegetated channel classification method which is embedded in the EC-Design software was used to estimate the vegetation condition in the channel. A vegetation retardance class of C was estimated (Mays, 200.

3.1.3 Pyramat Armoring Results

The calculated FS for velocity and shear stress stability using the EC-DESIGN software for the 100-year and 500-year events are presented in Table 2 below. A minimum FS of 1.5 is recommended for stability of the Pyramat®. Program output calculations can be found in Attachment A.

Table 2. Pyramat Stability Analysis Results

Channel Condition	Velocity FS		Shear Stress FS	
	100-year	500-year	100-year	500-year
Un-vegetated	1.0	0.9	2.4	1.8
Fully Vegetated	2.6	1.9	2.8	2.2

As shown, Pyramat® does not meet the required velocity FS for the un-vegetated condition for both storm events. For all other conditions and events the Pyramat® exceeds the recommended minimum FS. The stability analysis shows that the un-vegetated (recently constructed) condition is the most critical stage of the design. Thus, construction staging is critical to ensure that the Pyramat® is seeded and has adequate time to establish some vegetation. In order to determine the minimum vegetation threshold required for stability, an additional analysis was conducted using a vegetation retardance class of E, which is the lowest vegetation class established by the NRCS. A vegetation retardance class of E yields a velocity FS of 2.15 for the 100-year event and a velocity FS of 1.8 for the 500-year event. This shows that only a minimal amount of vegetation needs to be established to ensure stability of Pyramat® in the channel.

3.2 Armorflex® Articulated Concrete Block Armoring

Armorflex® Articulated Concrete Block was selected to armor the 50.0% slopes of the Drop Structure. Armorflex® by Armortech/Contech is a flexible matrix of concrete blocks that are laced longitudinally with steel revetment cables. The Armorflex® is an open cell concrete block that also allows for vegetation to be established. More specifically, the 70-T block class by Armortech was selected to as the armament material (see Attachment C for product information) which is suited for high velocity applications.

Appropriate installation is required for Armorflex armoring methods to be effective and is subject to the ground conditions encountered at the site. The proposed design and stability calculations will need to be verified by Contech to ensure stability in final design.

Armorflex® Design Software by Armortec Erosion Control Solution (Armortec, 2002) was used to perform hydraulic calculations to ensure stability of Armorflex®. Armorflex® Design Software is a vendor supplied program and is created specifically to determine the stability of Armorflex® in open channel flow. Embedded in the program are stability calculations that take into account the over-turning moments created by the moving water on a concrete block.

The armor on the 50.0% slopes will consist of the following from bottom to top:

- A prepared subgrade;
- A site specific geotextile;
- A minimum of 250 mm of angular drainage rock;
- A geogrid; and
- Armorflex® articulated concrete block

The calculated FS for overturning stability using the Armorflex® Design Software for the 100-year and 500-year events are presented in Table 3 below. A minimum FS of 1.5 is recommended for stability of the Armorflex®.

Table 3. Armorflex® Stability Analysis Results

Channel Armoring	FS	
	100-year	500-year
Armorflex® 70-T Block Type	1.5	1.1

As shown, the Armorflex® meets the required FS of 1.5 for the 100-year storm event but does not meet the required FS for the 500-year event. A FS of 1.1 suggests that the channel armoring may not necessarily result in failure, but may be subject to damage. Additional design features may be incorporated into the design to increase the FS such as; reducing the bed slope, reducing the channel sideslopes, and increasing the channel bottom width.

4.0 Channel Dimensions Results

4.1 Upper Channel Reach

The channel will primarily be in cut with the exception of a few fill areas. The upper channel reach will have the following dimensions.

- A bottom width of 4.0 m;
- A depth of 1.5 m;
- 2H:1V side-slopes; and
- A slope of 1.0%.

The upper channel reach will also receive overland flow along the length of the channel. At about mid-length of the upper reach, the channel will intercept flow from Eagle Pup basin. At the intersection with Eagle Pump the diversion channel will require additional armament where the Eagle pup flow enters the channel.

4.2 Drop Structure

The Drop Structure segment of the Channel is designed to facilitate the large elevation change across the site, and will be armored due to the steep slopes and high velocities in the channel. The Drop Structure will create various flow regimes due to the alternating slopes resulting in a complex water surface profile. The Drop Structure will require two types of armoring in order to effectively protect the Channel and dissipate the energy.

4.2.1 Inlet and Outlet Channel

The inlet and outlet channel portion of the drop structure will have the following dimensions:

- A bottom width that transitions from 4.0 m to 8.0 m;
- A depth that transitions from 1.5 m to 2.0 m;
- Side-slopes that range from 2H:1V to 3H:1V; and
- A slope of 1.0%.

4.2.2 Drop Chute

The drop chute portion of the Channel will have the following dimensions:

- A bottom width of 8.0 m;

- A depth of 2.0 m;
- 3H:1V side-slopes; and
- A slope of 50.0%.

4.2.3 Energy Dissipation Pool

Energy Dissipation Pools are used to reduce the energy gained within the water from the 50% slopes. Energy is reduced through a hydraulic jump, which will occur when there is an abrupt change in channel slope (from steep to shallow), and as the water changes from super critical flow to subcritical flow. Additionally, the jump must be fully dissipated, with the flow returning to subcritical prior to entering the outlet channel. The pools are designed such that the hydraulic jump calculated is retained within the pool and will have dissipated sufficient energy to return the flow back to the subcritical flow. Preliminary calculations were performed to determine the height and length of the jump which was used to determine the dimensions of the Energy Dissipation Pool to ensure that water does not "jump" out of the pool/channel.

The Energy Dissipation Pools will have the following dimensions:

- A bottom width of 8.0 m;
- A depth of 3.0 m;
- 3H:1V side-slopes; and
- A slope of 0.0%.

4.3 Lower Channel Reach

The lower channel reach is similar to the upper channel reach but will have two intermediate drop structures to account for abrupt elevation changes along the channel alignment. The channel will primarily be in cut with the exception of a few fill areas. The lower channel reach will ultimately discharge into Haggart Creek, however, this part of the channel was not designed. The lower channel reach will have the following dimensions.

- A bottom width of 4.0 m;
- A depth of 1.5 m;
- 2H:1V side-slopes; and
- A slope of 1.0%.

4.4 Channel Dimensions Design Summary

The calculated flow depths and channel dimensions are summarized in Table 4 below. The flow depths shown are for the vegetated condition which gives the deepest flow depth.

Table 4. Dublin Gulch Diversion Channel Summary

Diversion Channel Segment		Armoring	Channel Slope (%)	Bottom Width (m)	Side slope (H:V)	100-yr Flow Depth (m)	500-yr Flow Depth (m)	Channel Design Depth (m)
Upper Channel Reach		Pyramat®	1.0	4.0	2:1	1.02	1.30	1.5
Drop Structure	Inlet Channel	Pyramat®	1.0	[1]4.0 - 8.0	[2]2:1 – 3:1	[1]1.02 – 0.74	[1]1.3 – 0.94	[3]1.5 - 2.0
	Drop Chute	Armorflex®	50.0	8.0	3:1	0.59	0.86	2.0
	Energy Dissipation Pool	Armorflex®	0.0	8.0	3:1	[4]2.4	[4]2.7	3.0
	Outlet Channel	Pyramat®	1.0	[1]4.0 - 8.0	[2]2:1 – 3:1	[1]1.02 – 0.74	[1]1.3 – 0.94	[3]1.5 - 2.0
[4]Lower Channel Reach		Pyramat®	1.0	4.0	2:1	1.02	1.30	1.5

1) Channel transitions between a 4.0 m and 8.0 m bottom width.
2) Channel side-slope transitions between 2H:1V and 3H:1V.
3) Channel depth transition from 1.5 m to 2.0 m.
4) Flow depth in Energy Dissipation pool is estimated by adding the hydraulic jump depth to an assumed full pool depth of 1 m.

The final channel design depth shows that the channel depth accommodates the required flow depths for the 100-year and 500-year events. The final channel design depths were selected based on the 100-year, 24-hour flow depth in addition to a freeboard of 0.3 m and an estimated super elevation effect of water of 0.1 m. The channel depths for the drop chute and energy dissipation pools were increased to account for increases in flow depth cause by possible air entrainment and wave action effects. These effects were only estimated for this level of design.

5.0 Conclusion and Considerations

Except for small fill areas along the channel, the majority of the channel was designed to be in cut in order to minimize construction complications. Final design of the Dublin Gulch Diversion Channel will include finalizing and verifying with the suppliers the selection of artificial erosion protection and subgrade preparation for the specified armoring materials. This is highly dependent on soil conditions along the channel alignment.

Special construction considerations must be taken into account during construction of the diversion channel. The channel must be completed prior to construction of the Confining Embankment, Heap Leach Pad and Ponds. The entire channel must have adequate vegetation growth established prior to redirecting flows into the channel.

The final design of the channel is subject to further site investigation. The proposed armament materials and channel design are based on the available data, and additional data such as soil conditions and geology along the channel alignment are needed to finalize the design. Rock encountered along the alignment may eliminate the need for armament and can alter the geometry of the channel or drop structure design.

REFERENCES

Armortec Erosion Control Solutions (2002). *ArmorFlex Design Manual Abridged version 2002.*

Knight Piesold Ltd., Aurala, C. (2011). *Email Correspondence Subject: Eagle Gold Design Precipitation Events.* Email dated July 4, 2011.

Mays, Larry W. (2005). *Water Resources Engineering,* 2005 Edition, John Wiley & Sons, Inc., 2005, pgs. 85-139, 596-599.

ATTACHMENT A
ARMORING STABILITY AND HYDRAULIC CALCULATIONS

ECDesign® 2003 Channel Analysis Report

Project Information

Last Update: 1/2/2012 11:44:44 AM

Project Name: DGDC - 100 year, 24-hour Event
Description:
City: **State:** **Units:** Metric
Notes:

Channel Design

Channel Name: 100yr-unvegetated **Units:** Metric **Design Life:** 1200 months

Design Criteria

Flow Rate (Q)

Vegetation and Soil

Vegetated	No
Vegetation Class	
Soil Filled	Yes

Channel Geometry

Bed Slope (m/m)	0.01
Req. Freeboard (m)	
Channel Length (m)	1000
Bottom Width (m)	4
Channel Depth (m)	1.5

Flow/Velocity

Discharge (m^3/s)	10.6
Flow Duration (hrs)	24
Avg. Velocity (m/s)	3.07

Channel Side Slopes

Left (H:1 V)	2
Right (H:1 V)	2

Channel Bend true

Bend Radius (m)	33
Outside Bend	Right

Required Factor of Safety

1.5

Results

Avg. Flow Depth (m): 0.64

Lining Materials		Velocity (m/s) Computed	Velocity (m/s) Maximum Allowed	Velocity (m/s) Safety Factor	Shear Stress (Pascals) Computed	Shear Stress (Pascals) Maximum Allowed	Shear Stress (Pascals) Safety Factor	Pass	Quantity (SM)
Left	PYRAMAT	2.770	3.530	1.270	48.940	181.800	3.710	N	3,354.100
Bottom	PYRAMAT	3.420	3.530	1.030	74.680	181.800	2.430	N	4,000.000
Right	PYRAMAT	3.010	3.530	1.170	58.050	181.800	3.130	N	3,354.100

Calculation Results

Flow Depth (m)	0.640	**Left Wetted Perimeter (m)**	1.440
Flow Area (m)	3.390	**Bottom Wetted Perimeter (m)**	3.990
		Right Wetted Perimeter (m)	1.440
		Total Wetted Perimeter (m)	6.870
Hydraulic Radius (m)	0.490	**Avg. Velocity (m/s)**	3.070
Composite 'n'	0.0200	**Avg. Discharge (m^3/s)**	10.600

Channel Lining Material Costs

Channel Name: 100yr-unvegetated **Units:** Metric

Lining Materials		Estimated Cost per Square (m)			
		Quantity *	**Unit Cost**	**% Waste**	**Material Cost**
Left	PYRAMAT	3354.1	0.00	0	0.00
Bottom	PYRAMAT	4000	0.00	0	0.00
Right	PYRAMAT	3354.1	0.00	0	0.00

* Quanities do not reflect seam overlaps

Total Material Cost: **0.00**

Installation Costs

Description	Quantity	Unit Cost	Costs per (SM)

Total Installation Cost: **0.00**

ECDesign® 2003 Channel Analysis Report

Project Information

Last Update: 1/2/2012 11:46:14 AM

Project Name: DGDC - 100 year, 24-hour Event
Description:
City: **State:** **Units:** Metric
Notes:

Channel Design

Channel Name: 100yr-vegetated **Units:** Metric **Design Life:** 1200 months

Design Criteria
Flow Rate (Q)

Vegetation and Soil

Vegetated	Yes
Vegetation Class	C
Soil Filled	No

Channel Geometry

Bed Slope (m/m)	0.01
Req. Freeboard (m)	
Channel Length (m)	1000
Bottom Width (m)	4
Channel Depth (m)	1.5

Flow/Velocity

Discharge (m^3/s)	10.6
Flow Duration (hrs)	24
Avg. Velocity (m/s)	1.7

Channel Side Slopes

Left (H:1 V)	2
Right (H:1 V)	2

Channel Bend true

Bend Radius (m)	33
Outside Bend	Right

Required Factor of Safety
1.5

Results

Avg. Flow Depth (m): 1.02

Lining Materials		Velocity (m/s) Computed	Maximum Allowed	Safety Factor	Shear Stress (Pascals) Computed	Maximum Allowed	Safety Factor	Pass	Quantity (SM)
Left	PYRAMAT	1.540	4.800	3.120	79.210	333.070	4.200	Y	3,354.100
Bottom	PYRAMAT	1.880	4.800	2.550	118.830	333.070	2.800	Y	4,000.000
Right	PYRAMAT	1.670	4.800	2.870	93.970	333.070	3.540	Y	3,354.100

Calculation Results

Flow Depth (m)	1.020	**Left Wetted Perimeter (m)**	2.290
Flow Area (m)	6.180	**Bottom Wetted Perimeter (m)**	3.990
		Right Wetted Perimeter (m)	2.290
		Total Wetted Perimeter (m)	8.570
Hydraulic Radius (m)	0.720	**Avg. Velocity (m/s)**	1.700
Composite 'n'	0.0468	**Avg. Discharge (m^3/s)**	10.600

Channel Lining Material Costs

Channel Name: 100yr-vegetated **Units:** Metric

Lining Materials		**Estimated Cost per Square (m)**			
		Quantity *	**Unit Cost**	**% Waste**	**Material Cost**
Left	PYRAMAT	3354.1	0.00	0	0.00
Bottom	PYRAMAT	4000	0.00	0	0.00
Right	PYRAMAT	3354.1	0.00	0	0.00

* Quanities do not reflect seam overlaps

Total Material Cost: **0.00**

Installation Costs

Description	**Quantity**	**Unit Cost**	**Costs per (SM)**

Total Installation Cost: **0.00**

ECDesign® 2003 Channel Analysis Report

Project Information

Last Update: 1/3/2012 3:40:18 PM

Project Name: DGDC - 100 year, 24-hour Event
Description:
City: **State:** **Units:** Metric
Notes:

Channel Design

Channel Name: 100yr-vegetated-8m **Units:** Metric **Design Life:** 1200 months

Design Criteria

Flow Rate (Q)

Vegetation and Soil

Vegetated	Yes
Vegetation Class	C
Soil Filled	No

Channel Geometry

Bed Slope (m/m)	0.01
Req. Freeboard (m)	
Channel Length (m)	1000
Bottom Width (m)	8
Channel Depth (m)	1.5

Flow/Velocity

Discharge (m^3/s)	10.6
Flow Duration (hrs)	24
Avg. Velocity (m/s)	1.6

Channel Side Slopes

Left (H:1 V)	3
Right (H:1 V)	3

Channel Bend true

Bend Radius (m)	33
Outside Bend	Right

Required Factor of Safety

1.5

Results

Avg. Flow Depth (m): 0.74

Lining Materials		Velocity (m/s)			Shear Stress (Pascals)			Pass	Quantity
		Computed	Maximum Allowed	Safety Factor	Computed	Maximum Allowed	Safety Factor		(SM)
Left	PYRAMAT	1.300	4.800	3.690	60.690	333.070	5.490	Y	4,743.420
Bottom	PYRAMAT	1.830	4.800	2.620	120.340	333.070	2.770	Y	8,000.000
Right	PYRAMAT	1.680	4.800	2.860	101.080	333.070	3.300	Y	4,743.420

Calculation Results

Flow Depth (m)	0.740	**Left Wetted Perimeter (m)**	2.330
Flow Area (m)	7.530	**Bottom Wetted Perimeter (m)**	8.000
		Right Wetted Perimeter (m)	2.330
		Total Wetted Perimeter (m)	12.660
Hydraulic Radius (m)	0.590	**Avg. Velocity (m/s)**	1.600
Composite 'n'	0.0500	**Avg. Discharge (m^3/s)**	10.600

Channel Lining Material Costs

Channel Name: 100yr-vegetated-8m **Units:** Metric

Lining Materials		Estimated Cost per Square (m)			
		Quantity *	**Unit Cost**	**% Waste**	**Material Cost**
Left	PYRAMAT	4743.42	0.00	0	0.00
Bottom	PYRAMAT	8000	0.00	0	0.00
Right	PYRAMAT	4743.42	0.00	0	0.00

* Quanities do not reflect seam overlaps

Total Material Cost: 0.00

Installation Costs

Description	Quantity	Unit Cost	Costs per (SM)

Total Installation Cost: 0.00

ECDesign® 2003 Channel Analysis Report

Project Information

Last Update: 1/2/2012 11:46:43 AM

Project Name: DGDC - 100 year, 24-hour Event
Description:
City: **State:** **Units:** Metric
Notes:

Channel Design

Channel Name: 500yr-unvegetated **Units:** Metric **Design Life:** 1200 months

Design Criteria

Flow Rate (Q)

Vegetation and Soil

Vegetated	No
Vegetation Class	
Soil Filled	Yes

Channel Geometry

Bed Slope (m/m)	0.01
Req. Freeboard (m)	
Channel Length (m)	1000
Bottom Width (m)	4
Channel Depth (m)	1.5

Flow/Velocity

Discharge (m^3/s)	17.8
Flow Duration (hrs)	24
Avg. Velocity (m/s)	3.59

Channel Side Slopes

Left (H:1 V)	2
Right (H:1 V)	2

Channel Bend true

Bend Radius (m)	33
Outside Bend	Right

Required Factor of Safety

1.5

Results

Avg. Flow Depth (m): 0.85

Lining Materials		Velocity (m/s)			Shear Stress (Pascals)			Pass	Quantity
		Computed	Maximum Allowed	Safety Factor	Computed	Maximum Allowed	Safety Factor		(SM)
Left	PYRAMAT	3.240	3.530	1.090	65.440	181.800	2.780	N	3,354.100
Bottom	PYRAMAT	4.000	3.530	0.880	99.420	181.800	1.830	N	4,000.000
Right	PYRAMAT	3.530	3.530	1.000	77.630	181.800	2.340	N	3,354.100

Calculation Results

Flow Depth (m)	0.850	**Left Wetted Perimeter (m)**	1.910
Flow Area (m)	4.880	**Bottom Wetted Perimeter (m)**	4.000
		Right Wetted Perimeter (m)	1.910
		Total Wetted Perimeter (m)	7.820
Hydraulic Radius (m)	0.620	**Avg. Velocity (m/s)**	3.590
Composite 'n'	0.0200	**Avg. Discharge (m^3/s)**	17.800

Channel Lining Material Costs

Channel Name: 500yr-unvegetated **Units:** Metric

Lining Materials		Estimated Cost per Square (m) Quantity *	Unit Cost	% Waste	Material Cost
Left	PYRAMAT	3354.1	0.00	0	0.00
Bottom	PYRAMAT	4000	0.00	0	0.00
Right	PYRAMAT	3354.1	0.00	0	0.00

* Quanities do not reflect seam overlaps

Total Material Cost: 0.00

Installation Costs

Description	Quantity	Unit Cost	Costs per (SM)

Total Installation Cost: 0.00

ECDesign® 2003 Channel Analysis Report

Project Information

Last Update: 1/2/2012 11:47:12 AM

Project Name: DGDC - 100 year, 24-hour Event
Description:
City: **State:** **Units:** Metric
Notes:

Channel Design

Channel Name: 500yr-vegetated **Units:** Metric **Design Life:** 1200 months

Design Criteria

Flow Rate (Q)

Vegetation and Soil

Vegetated	Yes
Vegetation Class	C
Soil Filled	No

Channel Geometry

Bed Slope (m/m)	0.01
Req. Freeboard (m)	
Channel Length (m)	1000
Bottom Width (m)	4
Channel Depth (m)	1.5

Flow/Velocity

Discharge (m^3/s)	17.8
Flow Duration (hrs)	24
Avg. Velocity (m/s)	2.06

Channel Side Slopes

Left (H:1 V)	2
Right (H:1 V)	2

Channel Bend true

Bend Radius (m)	33
Outside Bend	Right

Required Factor of Safety

1.5

Results

Avg. Flow Depth (m): 1.3

Lining Materials		Velocity (m/s) Computed	Velocity (m/s) Maximum Allowed	Velocity (m/s) Safety Factor	Shear Stress (Pascals) Computed	Shear Stress (Pascals) Maximum Allowed	Shear Stress (Pascals) Safety Factor	Pass	Quantity (SM)
Left	PYRAMAT	1.880	4.800	2.550	103.460	333.070	3.220	Y	3,354.100
Bottom	PYRAMAT	2.270	4.800	2.110	151.280	333.070	2.200	Y	4,000.000
Right	PYRAMAT	2.040	4.800	2.350	122.740	333.070	2.710	Y	3,354.100

Calculation Results

Flow Depth (m)	1.300	**Left Wetted Perimeter (m)**	2.910
Flow Area (m)	8.590	**Bottom Wetted Perimeter (m)**	4.000
		Right Wetted Perimeter (m)	2.910
		Total Wetted Perimeter (m)	9.820
Hydraulic Radius (m)	0.870	**Avg. Velocity (m/s)**	2.060
Composite 'n'	0.0441	**Avg. Discharge (m^3/s)**	17.800

Channel Lining Material Costs

Channel Name: 500yr-vegetated **Units:** Metric

Lining Materials		Estimated Cost per Square (m)			
		Quantity *	**Unit Cost**	**% Waste**	**Material Cost**
Left	PYRAMAT	3354.1	0.00	0	0.00
Bottom	PYRAMAT	4000	0.00	0	0.00
Right	PYRAMAT	3354.1	0.00	0	0.00

* Quanities do not reflect seam overlaps

Total Material Cost: **0.00**

Installation Costs

Description	Quantity	Unit Cost	Costs per (SM)

Total Installation Cost: **0.00**

ECDesign® 2003 Channel Analysis Report

Project Information

Last Update: 1/3/2012 3:41:02 PM

Project Name: DGDC - 100 year, 24-hour Event
Description:
City: **State:** **Units:** Metric
Notes:

Channel Design

Channel Name: 500yr-vegetated-8m **Units:** Metric **Design Life:** 1200 months

Design Criteria

Flow Rate (Q)

Vegetation and Soil

Vegetated	Yes
Vegetation Class	C
Soil Filled	No

Channel Geometry

Bed Slope (m/m)	0.01
Req. Freeboard (m)	
Channel Length (m)	1000
Bottom Width (m)	8
Channel Depth (m)	1.5

Flow/Velocity

Discharge (m^3/s)	17.8
Flow Duration (hrs)	24
Avg. Velocity (m/s)	1.99

Channel Side Slopes

Left (H:1 V)	3
Right (H:1 V)	3

Channel Bend — true

Bend Radius (m)	33
Outside Bend	Right

Required Factor of Safety

1.5

Results

Avg. Flow Depth (m): 0.94

Lining Materials		Velocity (m/s)			Shear Stress (Pascals)			Pass	Quantity
		Computed	Maximum Allowed	Safety Factor	Computed	Maximum Allowed	Safety Factor		(SM)
Left	PYRAMAT	1.620	4.800	2.960	78.980	333.070	4.220	Y	4,743.420
Bottom	PYRAMAT	2.260	4.800	2.120	154.060	333.070	2.160	Y	8,000.000
Right	PYRAMAT	2.090	4.800	2.300	131.560	333.070	2.530	Y	4,743.420

Calculation Results

Flow Depth (m)	0.940	**Left Wetted Perimeter (m)**	2.980
Flow Area (m)	10.220	**Bottom Wetted Perimeter (m)**	8.010
		Right Wetted Perimeter (m)	2.980
		Total Wetted Perimeter (m)	13.970
Hydraulic Radius (m)	0.730	**Avg. Velocity (m/s)**	1.990
Composite 'n'	0.0466	**Avg. Discharge (m^3/s)**	17.800

Channel Lining Material Costs

Channel Name: 500yr-vegetated-8m **Units:** Metric

Lining Materials		Estimated Cost per Square (m)			
		Quantity *	**Unit Cost**	**% Waste**	**Material Cost**
Left	PYRAMAT	4743.42	0.00	0	0.00
Bottom	PYRAMAT	8000	0.00	0	0.00
Right	PYRAMAT	4743.42	0.00	0	0.00

* Quanities do not reflect seam overlaps

Total Material Cost: **0.00**

Installation Costs

Description	Quantity	Unit Cost	Costs per (SM)

Total Installation Cost: **0.00**

ARMORFLEX DESIGN REPORT

ArmorFlex Blocks by ARMORTEC Erosion Control Solutions

4301 Industrial Drive
Bowling Green, Kentucky 42101
Phone (270) 843-4659
Toll free (800) 305-0523
Fax (270) 783-8952

Design Report Printed from Armorflex Design Software
Report Type: Summary

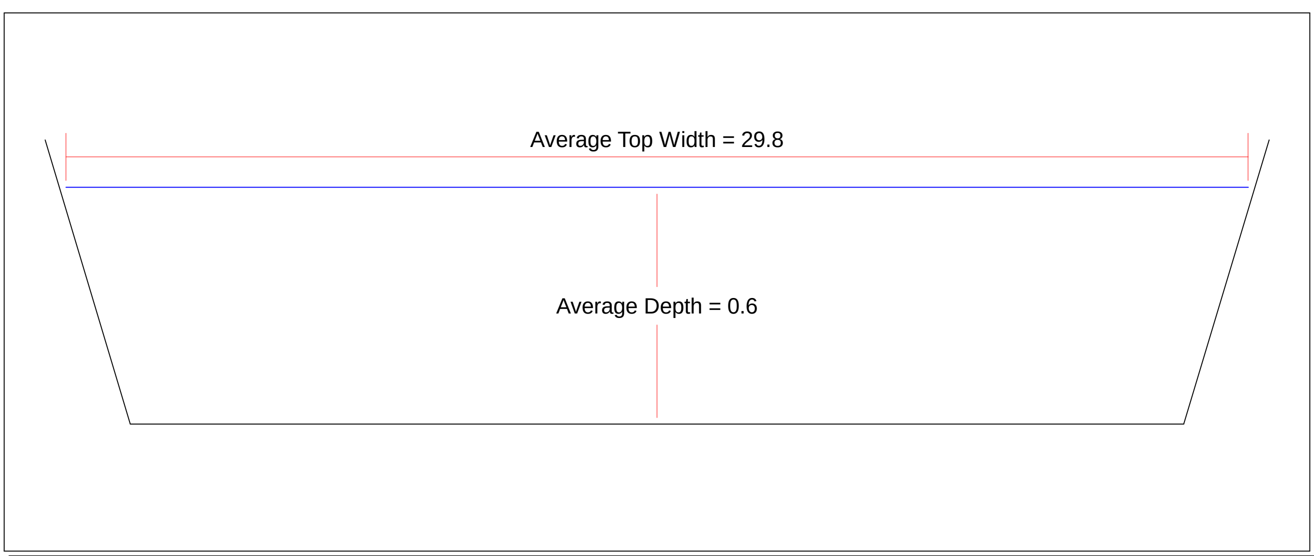
Company: Tetra Tech
Designer: Ronson Chee
Project No.: Project No. not supplied
Report Date: 1/2/2012

Client: Victoria Gold
Waterway: Dublin Gulch Diversion Channel
Location: Eagle Gold
Event: 100-year, 24-hour

Flow Scenario: Trapezoidal Channel
Block Cell Type: Open Cell Block
Block Taper Type: Tapered Block

Design Input for Factor of Safety Calculations

Left Side Slope (_H:1V) = 3
Right Side Slope (_H:1V) = 3
Channel Bottom Width (ft) = 26.248
Channel Bed Slope (ft/ft) = .5
Bend Coefficient = 1
Discharge (cfs) = 374
Projection Height (in.) = 0.2
Vertical Exageration for Plot = 10



Graphical Output of Normal Depth Calculations

Output from Factor of Safety Calculations

Block Type	n-Value	Depth (ft)	Velocity (ft/s)	Froude No.	Shear (psf)	Factor of Safety
40-T	0.031	0.59	22.66	5.2	16.44	1
50-T	0.031	0.59	22.66	5.2	16.44	1.2
60-T	0.031	0.59	22.66	5.2	16.44	1.3
70-T	0.031	0.59	22.66	5.2	16.44	1.5

ARMORFLEX DESIGN REPORT

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Company: Tetra Tech
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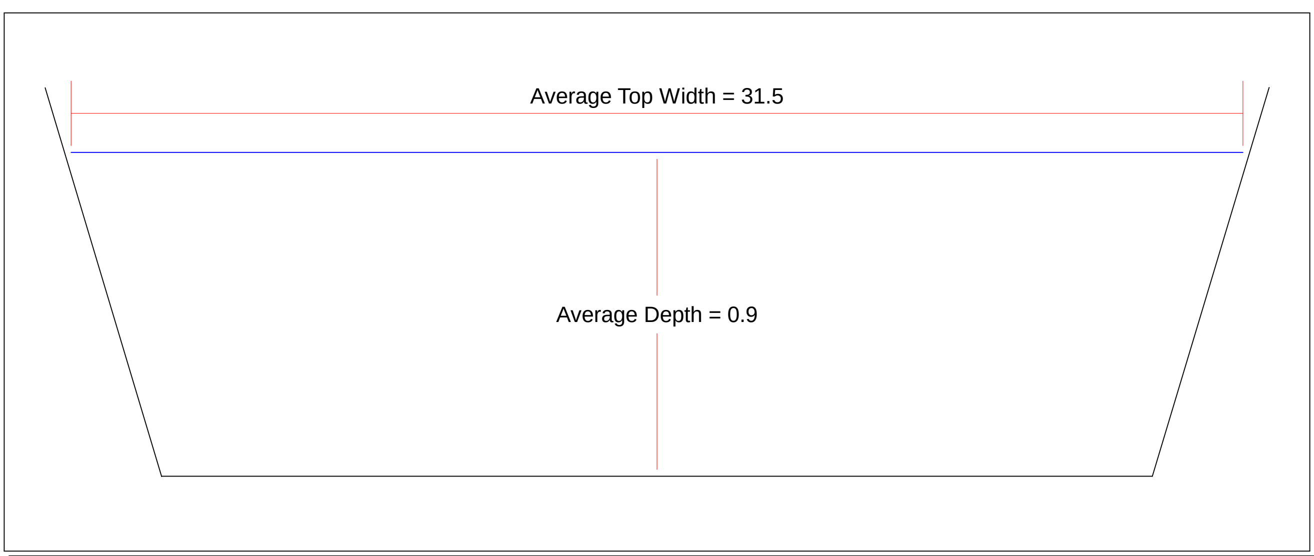
Report Date: 1/2/2012

Client: Victoria Gold
Waterway: Dublin Gulch Diversion Channel
Location: Eagle Gold
Event: 500-year, 24-hour

Flow Scenario: Trapezoidal Channel
Block Cell Type: Open Cell Block
Block Taper Type: Tapered Block

Design Input for Factor of Safety Calculations

Left Side Slope (_H:1V) = 3
Right Side Slope (_H:1V) = 3
Channel Bottom Width (ft) = 26.3
Channel Bed Slope (ft/ft) = .5
Bend Coefficient = 1
Discharge (cfs) = 628
Projection Height (in.) = .2
Vertical Exageration for Plot = 10



Graphical Output of Normal Depth Calculations

Output from Factor of Safety Calculations

Block Type	n-Value	Depth (ft)	Velocity (ft/s)	Froude No.	Shear (psf)	Factor of Safety
40-T	0.036	0.86	25.13	4.76	24.14	0.8
50-T	0.036	0.86	25.13	4.76	24.14	0.9
60-T	0.036	0.86	25.13	4.76	24.14	1
70-T	0.036	0.86	25.13	4.76	24.14	1.1

ATTACHMENT B
PYRAMAT TURF REINFORCEMENT MAT
PRODUCT INFORMATION
DATA SHEETS

PYRAMAT® HIGH PERFORMANCE TURF REINFORCEMENT MATS

Pyramat® High Performance Turf Reinforcement Mats (HPTRMs) feature our patented woven technology composed of a unique, three-dimensional matrix of polypropylene yarns. These yarns are designed in a uniform, dimensionally stable and homogenous configuration of pyramid-like structures, and they feature our patented X3® fiber technology specially created to lock soil in place. HPTRMs exhibit extremely high tensile strength as well as superior interlock and reinforcement capacity with both soil and root systems. They stand up to the toughest erosion applications where high loading and/or high survivability conditions are required, including maintenance access, steep slopes, arid and semi-arid environments, pipe inlets and outlets, structural backfills, utility cuts, potential traffic areas, abrasion, high-flow channels and/or areas where greater factors of safety are desired. Pyramat's superior characteristics provide a longer design life than our first and second generation standard TRMs, and meet the definition of HPTRM as defined by the U.S. EPA Storm Water Fact Sheet, "Turf Reinforcement Mats" (EPA 832-F-99-002) and FHWA FP-03 Specifications Section 713.8.

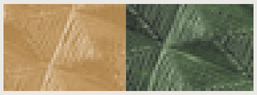
FEATURES & BENEFITS

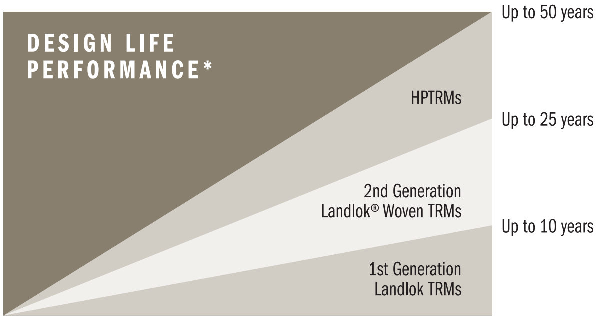
- A unique, patented matrix of pyramids formed with X3 fibers that gridlocks soil in place under unvegetated, partially vegetated and high-flow conditions
- Ideal for extended ultraviolet (UV) exposure, utility cuts, maintenance equipment traffic, pipe inlets and outlets and other high loadings
- X3 cross-sectional area for additional tensile strength, flexibility and seedling emergence
- Holds seed and soil in place on channels and slopes while vegetation grows
- Provides permanent reinforcement to enhance vegetation's natural ability to filter soil particles and prevent soil loss during storm events
- Promotes infiltration which leads to groundwater recharge
- Vegetation solution providing more pleasing aesthetics than conventional methods (i.e. rock riprap and concrete paving)
- Greater flexibility to maintain intimate contact with subgrade, resulting in rapid seedling emergence and minimal soil loss
- Can be used in arid and semi-arid environments
- Completely interconnected yarns that provide superior UV resistance throughout the HPTRM
- Meets requirement of 5 mm^2 or less mesh size to prevent wildlife entanglement in any sensitive habitats
- Superior product testing, performance and design life

Outperforms and is more cost-effective than conventional erosion control methods, including:

- Large rock riprap
- Grouted riprap
- Gabions
- Concrete paving
- Hard roadside shoulders
- Articulated concrete blocks
- Fabric formed revetments

PYRAMAT® HPTRMs PRODUCT FAMILY TABLE

PRODUCT		FUNCTIONAL LONGEVITY	COLOR	FIBER TYPE	# OF NETS	FP-03, SECTION 713 COMPLIANCE
	PYRAMAT®	PERMANENT	TAN OR GREEN	POLYPROPYLENE X3® FIBER TECHNOLOGY	0 (WOVEN)	EXCEEDS TYPE 5C



*Design life performance may vary depending upon field conditions and applications.

PROPEX® | THE ADVANTAGE CREATORS.™
GEOSYNTHETICS

PYRAMAT® HIGH PERFORMANCE TURF REINFORCEMENT MATS

APPLICATION SUGGESTIONS FOR PYRAMAT® HPTRMs

	APPLICATION	FUNCTIONAL LONGEVITY	PRODUCT STYLE	INSTALLED COST[1]	ANCHOR SUGGESTIONS[2]
SLOPES	1H:1V OR STEEPER	PERMANENT	PYRAMAT®	\$12.00 - 18.00/yd² \$14.35 - 21.53/m²	2.5 ANCHORS/yd² 3 ANCHORS/m²
CHANNELS	SHEAR STRESS UP TO 15 lb/ft² (718 N/m²) VELOCITY UP TO 25 ft/sec (7.6 m/sec)				
BANKS	MODERATE WAVE ACTION = 1-2 ft (30 - 60 cm)				
CRITICAL STRUCTURES	PIPE INLETS & OUTLETS				

NOTES: 1. Installed cost estimates range from large to small projects according to material quantity. The estimates include material, seed, labor and equipment. Costs vary greatly in different regions of the country. 2. For anchor size and style, please see our HPTRM Installation Guidelines.

KEY PHYSICAL PROPERTIES OF PYRAMAT® HPTRMs

- Construction: Patented three-dimensional woven matrix makes it 10 times stronger than first generation TRMs, with performance unequaled in turf reinforcement.
- Tensile Strength: 4000 lb/ft (58.4 kN/m) tensile strength meets U.S. EPA definition of a High Performance Turf Reinforcement Mat.
- UV Resistance: Patented UV protection package provides superior resistance to the damaging effects of ultraviolet radiation.

SEVEN STEPS FOR SUCCESSFUL TRM SELECTIONS*

1. SELECT APPLICATIONS
2. DETERMINE FUNCTIONAL LONGEVITY
3. ANTICIPATE CLIMATE (ARID, SEMI-ARID, OR TEMPERATE)
4. UNDERSTAND TRADITIONAL SOLUTION
5. PREDICT NON-HYDRAULIC STRESSES (MAINTENANCE STRESSES)
6. KNOW VEGETATION TYPE
7. CALCULATE HYDRAULIC STRESSES

*See Propex Engineering Bulletins or EC-DESIGN® software for more information.

PYRAMAT® HPTRM PROPERTY TABLE[1] ENGLISH & METRIC VALUES

	PROPERTY	TEST METHOD	VALUE[2]	PYRAMAT®
PHYSICAL	MASS PER UNIT AREA	ASTM D-6566	MARV	13.5 oz/yd^2 455 g/m^2
	THICKNESS	ASTM D-6525	MARV	0.4 in 10.2 mm
	LIGHT PENETRATION	ASTM D-6567	TYPICAL	10%
	COLOR	VISUAL	–	GREEN, TAN
MECHANICAL	TENSILE STRENGTH	ASTM D-6818	MARV	4000 x 3000 lb/ft 58.4 x 43.8 kN/m
	TENSILE ELONGATION	ASTM D-6818	MaxARV	65%
	RESILIENCY	ASTM D-6524	MARV	80%
	FLEXIBILITY/STIFFNESS	ASTM D-6575	TYPICAL	0.534 in-lbs 615000 mg-cm
ENDURANCE	FUNCTIONAL LONGEVITY	OBSERVED	TYPICAL	PERMANENT
DURABILITY	UV RESISTANCE[4]	ASTM D-4355	MINIMUM	90% @ 6000 HOURS
PERFORMANCE	SEEDLING EMERGENCE[3]	ECTC DRAFT METHOD #4	TYPICAL	296%
PACKAGING	ROLL WIDTH	MEASURED	TYPICAL	8.5 ft 2.6 m
	ROLL LENGTH	MEASURED	TYPICAL	90 ft 27.4 m
	ROLL WEIGHT	CALCULATED	TYPICAL	76 lb 34 kg
	ROLL AREA	MEASURED	TYPICAL	85 yd^2 71 m^2

NOTES: 1. The listed property values are effective 06/2009 and are subject to change without notice. 2. MARV indicates Minimum Average Roll Value calculated as the typical minus two standard deviations. Statistically, it yields a 97.7% degree of confidence that any sample taken during quality assurance testing will exceed the reported value. Maximum Average Roll Values (MaxARV) is calculated as typical plus two standard deviations. 3. Calculated as percent increase in average plant biomass with tall fescue grass seed in sand 14 days after seeding versus a non-RECP protected control specimen. 4. All components must meet UV resistance values.

PYRAMAT® HPTRM PERFORMANCE VALUES ENGLISH & METRIC UNITS

MATERIAL	FUNCTIONAL LONGEVITY	SHORT-TERM MAXIMUM SHEAR STRESS AND VELOCITY						MANNING'S "n"		
		VEGETATED[5]		PARTIALLY[6]		UNVEGETATED[7]		0"-6"	6"-12"	12"-24"
PYRAMAT®	PERMANENT	15 lb/ft^2 718 N/m^2	25 ft/sec 7.6 m/sec	10 lb/ft^2 478 N/m^2	20 ft/sec 6.1 m/sec	6.0-8.0 lb/ft^2 285-383 N/m^2	15 ft/sec 4.6 m/sec	0.035	0.028	0.017

NOTES: 5. Maximum permissible shear stress has been obtained through fully vegetated (70% to 100% density) testing programs featuring specific soil types, vegetation classes, flow conditions and failure criteria. Achieved after 14 weeks of vegetative establishment versus the industry standard of two full growing seasons. These conditions may not be relevant to every project nor are they replicated by other manufacturers. Please contact Propex for further information. 6. Maximum permissible shear stress has been obtained through partially vegetated (30% to 70% density) testing programs featuring specific soil types, vegetation classes, flow conditions and failure criteria. These conditions may not be relevant to every project nor are they replicated by other manufacturers. Please contact Propex for further information. 7. Maximum permissible shear stress has been obtained through unvegetated (0% to 30% density) testing programs featuring specific soil types, vegetation classes, flow conditions and failure criteria. These conditions may not be relevant to every project nor are they replicated by other manufacturers. Please contact Propex for further information.

For downloadable documents like construction specifications, installation guidelines, case studies and other technical information, please visit our web site at **geotextile.com.** These documents are available in easy-to-use Microsoft® Word format.

PROPEX® | THE ADVANTAGE CREATORS.™

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INSTALLATION GUIDELINES

FOR LANDLOK® TRMs AND PYRAMAT® HPTRMs

BEFORE YOU BEGIN

Thank you for purchasing high quality Landlok® Turf Reinforcement Mats (TRMs) and Pyramat® High Performance Turf Reinforcement Mats (HPTRMs) from Propex. We're committed to offering the best erosion control products in the industry.

It is important to follow these installation guidelines for a successful project. (Note: Construction shall be performed in accordance with the specific project bid documents, construction drawings, and specifications.) In addition, we suggest that a pre-installation meeting be held with the construction team and a representative from Propex. This meeting shall be scheduled by the contractor with at least two weeks notice. Also, Propex suggests that installation monitoring of our TRMs and HPTRMs be performed by a qualified independent third party.

SITE PREPARATION

- Grade and compact area of TRM/HPTRM installation as directed and approved by Engineer. Subgrade shall be uniform and smooth. Remove all rocks, clods, vegetation or other objects so the installed mat will have direct contact with soil surface.
- Prepare seedbed by loosening the top 2-3 in (50-75 mm) minimum of soil.
- Incorporate amendments such as lime and fertilizer and/or wet the soil, if needed.
- Do not mulch areas where mat is to be placed.

SEEDING

- Apply seed to soil surface before installing mat. Disturbed areas shall be reseeded.
- When soil filling, first install the mat, apply seed and then soil-fill per guidelines (see page 8).
- Consult project plans and/or specifications for seed types and application rates.

INSTALLATION ON STABLE SOIL SLOPES

- Excavate a 12 x 6 in (300 x 15 mm) minimum longitudinal anchor trench 2-3 ft (600-900 mm) over crest of slope (see Figure 2).
- Install top end of mat into trench and secure to bottom using suggested ground anchoring devices (see Tables 1 and 2 on page 7) spaced every 12 in (300 mm) minimum. Backfill and compact soil into trench (see Figure 2).
- Unroll mat down slope. Landlok® 1051 shall have the geotextile on bottom.
- Overlaps shall be 6 in (150 mm) minimum and anchored every 18 in (450 mm) minimum along the overlap. Secure using suggested ground anchoring devices shown in Table 1 for appropriate frequency and pattern. Overlaps are shingled away from prevailing winds (see Figure 1).
- Unroll mat in a manner to maintain direct contact with soil. Secure mat to ground surface using ground anchoring devices (see Table 1). Anchors shall be placed in accordance with the Anchor Pattern Guide on page 7.
- Excavate a 12 x 6 in (300 x 150 mm) key anchor trench at toe of slope (see Figure 3).
- Place bottom end of mat into key anchor trench at toe of slope and secure to bottom of trench using suggested ground anchoring devices (see Tables 1 and 2) spaced every 12 in (300 mm) minimum. Backfill and compact soil into trench (see Figure 3).
- If the potential for standing and/or flowing water exists at the toe of slope, the key anchor trench at the toe detail (see Figure 3) is not sufficient. Consult the project engineer for the appropriate detail.
- Irrigate as necessary to establish/maintain vegetation. Do not over-irrigate.

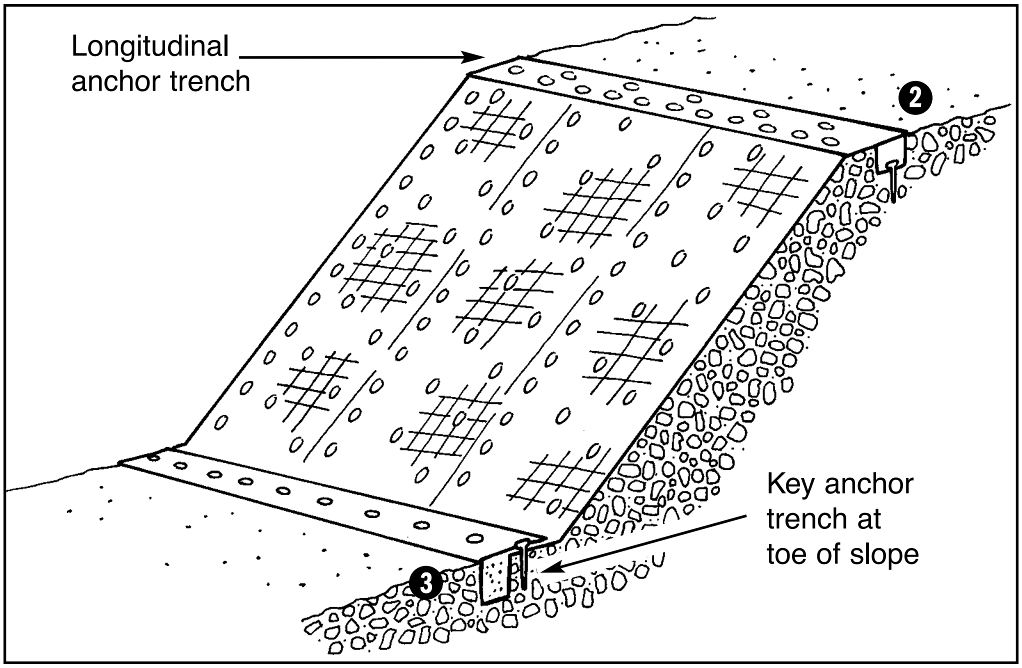


FIGURE 1
Installation of permanent turf reinforcement mat on slope

- Overlaps 6 in (150 mm) minimum
- Space anchors 18 in along overlaps down the slope
- Anchor pattern shall be in accordance with the "Anchor Pattern Guide" found on page 7

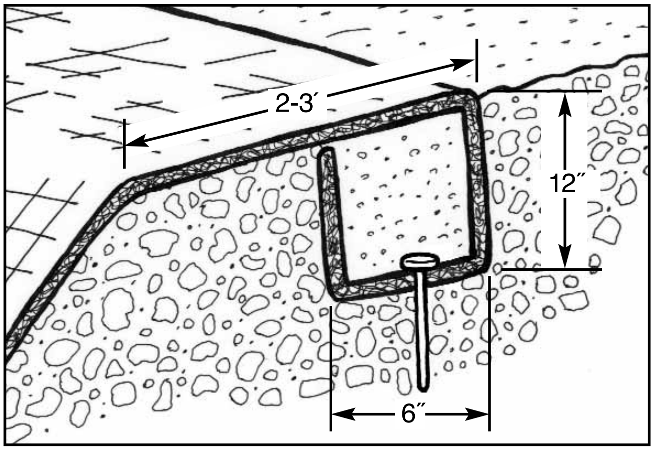


FIGURE 2

Longitudinal anchor trench at top of slope

- Space anchors 12 in (300 mm) along bottom of trench

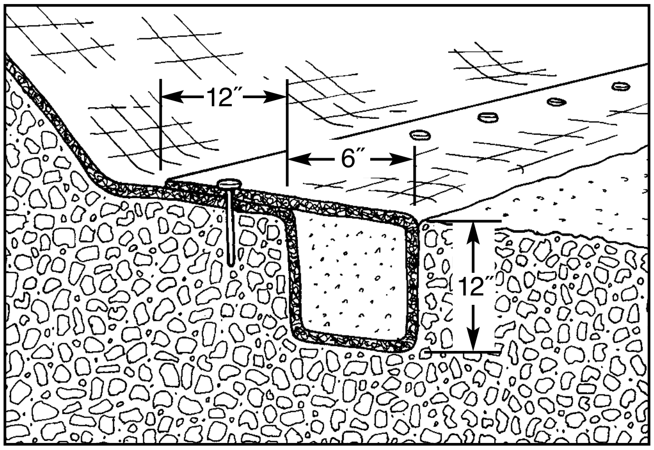


FIGURE 3

Key anchor trench at toe of slope

INSTALLATION IN STORM WATER CHANNELS

- Figure 4 shows general installation layout and details for TRMs and HPTRMs in storm water channels.
- Excavate an initial anchor trench 12 in (300 mm) minimum deep and 12 in (300 mm) minimum wide across the channel at downstream end of project (see Figure 5). Deeper initial anchor trench is needed in channels that have the potential for scour.
- Excavate longitudinal anchor trenches 12 in (300 mm) minimum deep and 6 in (150 mm) minimum wide along both sides of the installation to bury edges of mat (see Figure 6). The trench shall be located 2-3 ft (600-900 mm) over crest of slope.
- Place roll end into the initial anchor trench and secure with anchoring devices at 12 in (300 mm) minimum intervals (see Figure 5). Position adjacent rolls and secure in anchor trench in same manner. Backfill and compact soil into trench.
- Unroll mat in the upstream direction over the compacted trench.
- Continue installation as described above, overlapping adjacent rolls as follows:
  - Roll edge: 6 in (150 mm) minimum with upslope mat on top. Secure with one row of ground anchoring devices on 12 in (300 mm) minimum intervals (see Figure 7).
  - Roll end: 12 in (300 mm) minimum with upstream mat on top. Secure with two rows of ground anchoring devices staggered 12 in (300 mm) minimum apart on 12 in (300 mm) minimum intervals (see Figure 8).
- Fold and secure mat rolls snugly into intermittent check slots. Lay mat in the bottom and fold back against itself. Anchor through both layers of blanket or mat at 1 ft (300 mm) intervals then backfill and compact soil (Figure 9). Continue rolling upstream over the compacted slot to the next check slot or terminal anchor trench. Check slots are placed at 25 to 30 ft (7.6 to 9.1 m) intervals perpendicular to flow.

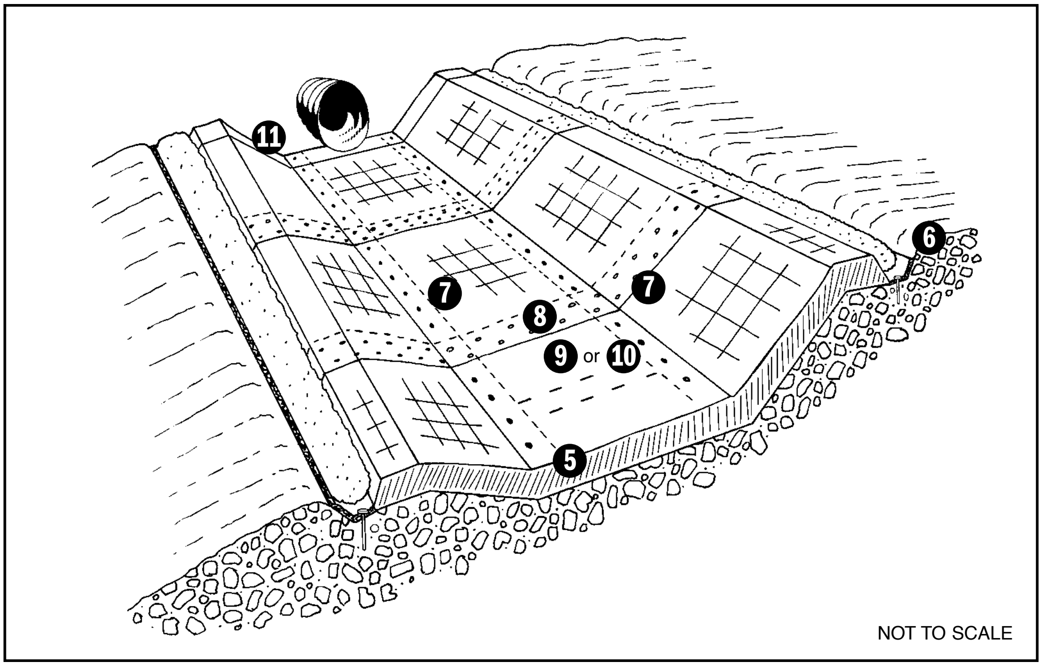


FIGURE 4 Installation of TRMs & HPTRMs in storm water channels

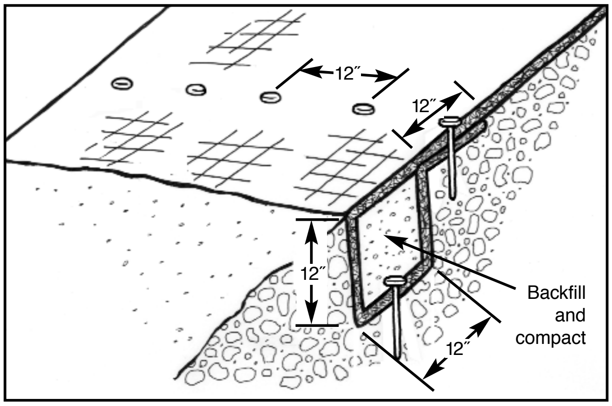


FIGURE 5 Initial anchor trench (downstream) detail

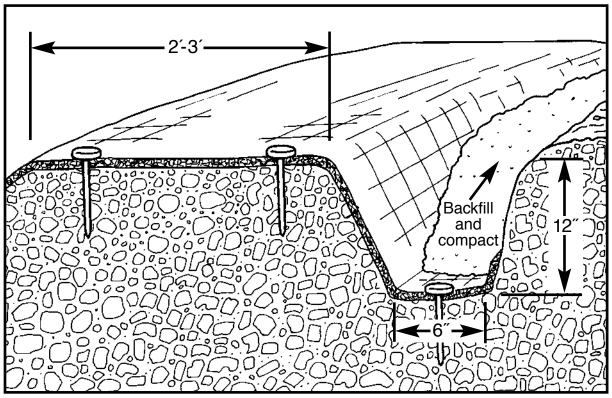


FIGURE 6 Longitudinal anchor trench detail (trapezoidal channel)

- An alternate method to the intermittent check slot is the simulated check slot. This method includes placing two staggered rows of anchors on 4 in (100 mm) centers at 30 ft (9.1 m) intervals (see Figure 10).
- Excavate terminal anchor trench 12 in wide x 12 in deep (300 x 300 mm) minimum across the channel at the upstream end of the project (see Figure 11). Deeper terminal anchor trench is needed in channels that have the potential for scour.
- Anchor, backfill and compact upstream end of mat in 12 x 12 in (300 x 300 mm) minimum terminal anchor trench (see Figure 11). Unroll mat in downstream direction over compacted trench with a minimum 2 ft (600 mm) lap. Secure with anchors in accordance with Figure 8.
- Secure mat using suggested ground anchoring devices (see Tables 1 and 2 on page 7) for appropriate frequency and pattern (see Anchor Pattern Guide on page 7).
- Seed and fill with soil for enhanced performance. See Soil Filling Section on page 8.
- When using Landlok® 1051, seed after installing mat and then fill with soil.
- Irrigate as necessary to establish/maintain vegetation. Do not over irrigate.

NOTE: If you encounter roll with factory overlap, install factory seam such that it shingles in the direction of the flow of water. Place anchoring devices in accordance with Figure 8 "Overlap at roll end" on page 5.

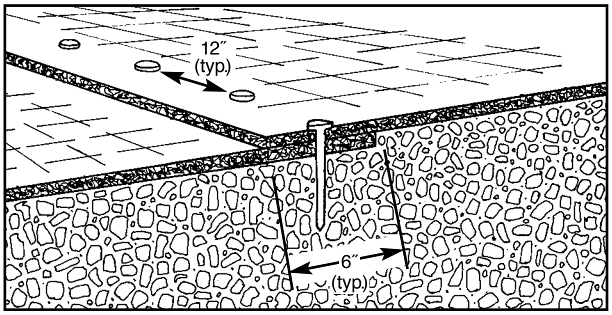


FIGURE 7 Overlap at roll edge

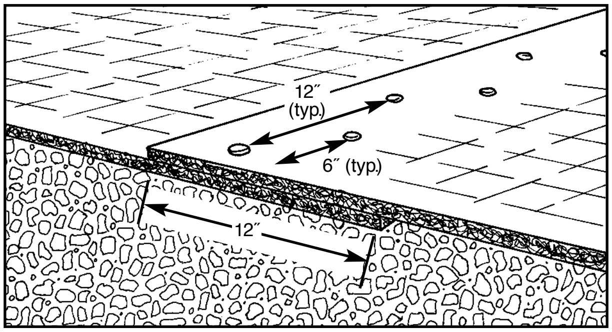


FIGURE 8 Overlap at roll end

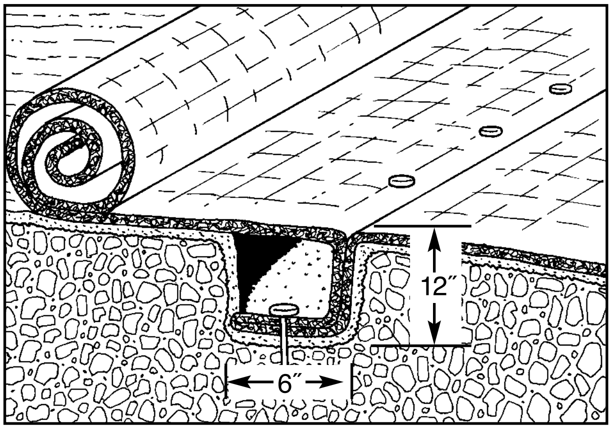


FIGURE 9 Intermittent check slot (channels)

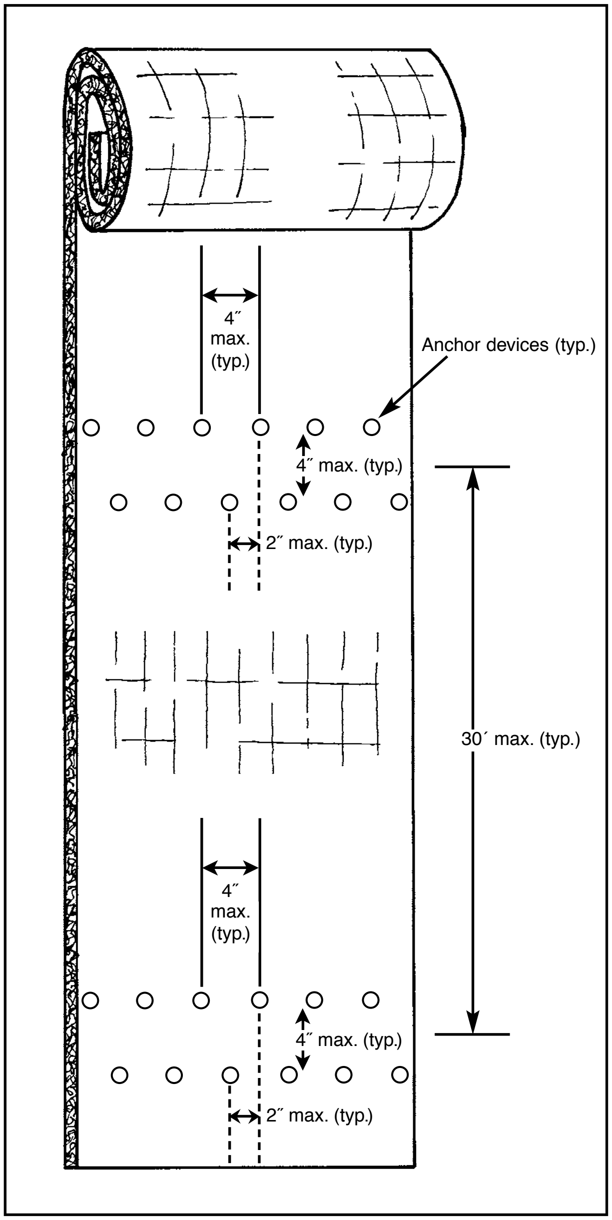


FIGURE 10 Simulated check slot

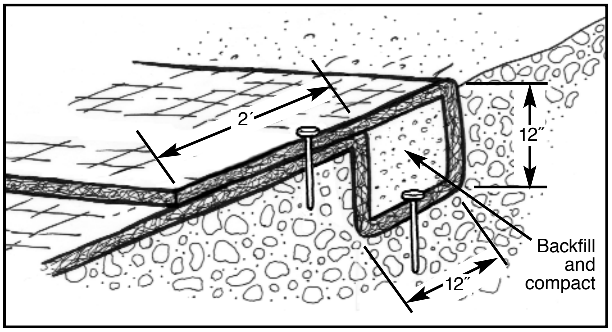


FIGURE 11 Terminal anchor trench (upstream) detail

SPECIAL TRANSITION GUIDELINES

- Rock Riprap
  - Excavate an anchor trench 12 x 12 in (300 x 300 mm) minimum at the transition between the mat and the rock riprap.
  - Place roll end into anchor trench and secure with suggested anchoring devices at 12 in (300 mm) minimum intervals. Position adjacent rolls and secure in anchor trench in same manner.
  - Backfill the anchor trench with rock riprap.
  - Place rock riprap as specified, extending approximately 3 ft (1 m) minimum beyond the anchor trench onto the mat.
- Concrete
  - Alternative 1: Concrete Apron
    - Place ready mixed concrete directly onto a 3 ft (0.9 m) wide minimum strip of mat.
  - Alternative 2: Concrete Backfill
    - Excavate an anchor trench 12 x 12 in (300 x 300 mm) minimum at the edge of the concrete structure.
    - Place roll end into anchor trench and secure with suggested anchoring devices at 12 in (300 mm) minimum intervals. Position adjacent rolls and secure in anchor trench in same manner.
    - Backfill trench with concrete slurry.
  - Alternative 3: Bolt to Structure (HPTRMs Only)
    - Cast threaded dowel in fresh ready mix concrete or install expanding bolt into cured concrete. Then affix HPTRM with washer (minimum 2 in or 50 mm diameter) or batten strip and bolt.
- Pipe Inlets/Outlets (HPTRMs Only)
  - Review the construction drawings and project specifications to evaluate the required area to be treated.
  - Excavate an anchor trench 12 x 12 in (300 x 300 mm) minimum above the pipe to bury end of HPTRM roll. The trench shall be located a minimum 2-3 ft (600-900 mm) above the pipe inlet/outlet.
  - Backfill and compact soil into trench.
  - Cut HPTRM to meet project requirements, slope length and pipe diameter.
  - Unroll HPTRM down the slope and secure around pipe circumference with ground anchoring devices spaced 6 in (150 mm) minimum. Also, the HPTRM can be secured around the pipe in a 12 x 12 in (300 x 300 mm) minimum trench filled with concrete slurry.

GROUND ANCHORING DEVICES

- Ground anchoring devices are used to secure the mat to the soil using the suggested anchor device (see Tables 1 and 2 on page 7) at a minimum frequency and pattern shown on the Anchor Pattern Guide on page 7.
- U-shaped wire staples or metal geotextile pins can be used to anchor mat to the ground surface. Wire staples should be a minimum thickness of 8 gauge (4.3 mm). Metal pins should be at least 0.20 in (5 mm) diameter steel with a 1 1/2 in (38 mm) steel washer at the head of the pin. Wire staples and metal pins should be driven flush to the soil surface. All anchors should be between 6-24 in (150-600 mm) long and have sufficient ground penetration to resist pullout. Longer anchors may be required for loose soils. Heavier metal stakes may be required in rocky soils.

TABLE 1: SUGGESTED GROUND ANCHORING DEVICE SELECTION*

		DEGRADABLE STAKES	WIRE STAPLES	METAL PIN/WASHERS OR NAIL/WASHERS	PERCUSSION DRIVEN ANCHORS
PRODUCT	LANDLOK® ECBs	●	●		
	LANDLOK® TRMs		●	●	
	PYRAMAT®		●	●	●
APPLICATION	SLOPES	●	●	●	●
	BANKS			●	●
	CHANNELS		●	●	●

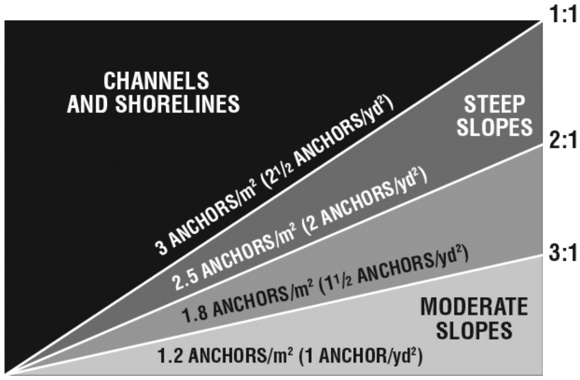
TABLE 2: SUGGESTED LENGTHS OF GROUND ANCHORING DEVICES*

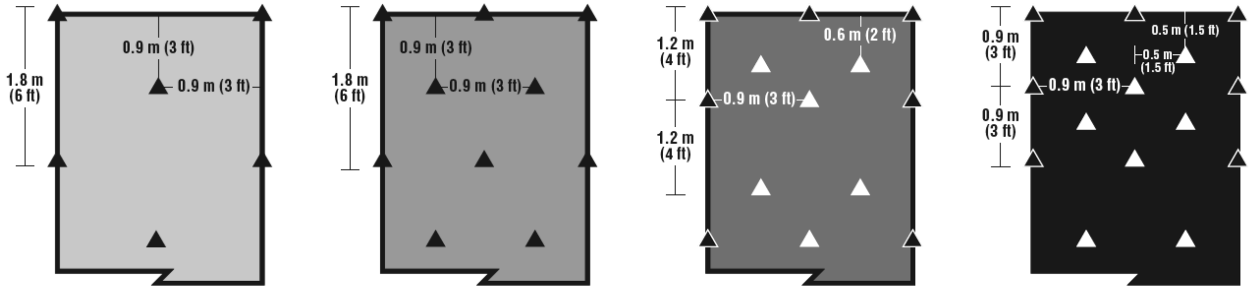
		6-INCH	12-INCH	18-INCH	24-INCH
SOIL TYPES	ROCKY	●			
	CLAYEY	●	●		
	SILTY		●	●	
	SANDY			●	●

*The performance of ground anchoring devices is highly dependent on numerous site/project specific variables. It is the sole responsibility of the project engineer and/or contractor to select the appropriate anchor type and length. Anchoring shall be selected to hold the mat in intimate contact with the soil subgrade and resist pullout in accordance with the project's design intent.

ANCHOR PATTERN GUIDE

- The shaded areas in the diagram provide anchor suggestions based on slope gradient and/or anticipated flow conditions. When the correct number of anchors has been determined, refer to the four illustrations below to establish anchor pattern. Increased anchoring may be required depending upon site conditions.





1.2 ANCHORS/m² (1 ANCHOR/yd²)

1.8 ANCHORS/m² (1 1/2 ANCHORS/yd²)

2.5 ANCHORS/m² (2 ANCHORS/yd²)

3 ANCHORS/m² (2 1/2 ANCHORS/yd²)

SOIL FILLING

- Soil filling is suggested for optimum performance.
- After seeding, spread and lightly rake $\frac{1}{2}$-$\frac{3}{4}$ in (12-19 mm) minimum of fine site soil or topsoil into the mat and completely fill the voids using backside of rake or other flat tool.
- If equipment must operate on the mat, make sure it is of the rubber-tired type. No tracked equipment or sharp turns are allowed on the mat.
- Avoid any traffic over the mat if loose or wet soil conditions exist.
- Smooth soil-fill in order to just expose the top netting of matrix. Do not place excessive soil above the mat.
- Broadcast additional seed and install a Landlok® ECB above the soil-filled mat (if desired).
- Hydraulically-applied mulch or seed may be used as an alternate to soil-fill on select applications. Consult manufacturer's technical representative for more information.
- Consult manufacturer's technical representative or local distributor for installation assistance, particularly if unique conditions apply (sandy soils and infertile environments).

MAINTENANCE

All slopes, channels, banks and other transition structures shall be maintained to assure the expected design life of the reinforced vegetated system. Here are a few tips that should prove helpful:

- Monitoring
  - Should be conducted semi-annually and after major storm events. This should include: observing the condition of the vegetation; testing the irrigation system; checking condition of all permanent erosion control systems; observing sediment and debris deposits that need removal.
- Vegetation
  - Repair and maintenance of various types of vegetation shall be consistent with their original design intent, including:
    - Grass/Turf Areas: applications shall be maintained for adequate cover and height.
    - Mowing: grasses shall be mowed according to normal maintenance schedules as determined by local jurisdictions or maintenance agreements; operations shall not start until vegetation achieves a minimum height of 6 in (150 mm); mower blades shall be greater than 6 in (150 mm) above the mat.
    - Unvegetated Areas: shall be re-seeded and soil-filled (if applicable).
- Sediment and Debris Deposits
  - Accumulation of sediment and debris can reduce the hydraulic capacity of channels, clog inlet and outlet structures and can damage existing vegetation. Sediment and debris removal is a vital part of system maintenance.
    - Removal: shall be done carefully to avoid damage. When excavation is within 12 in (300 mm) minimum of matting, removal shall be done by hand or with a visual "spotter." If equipment must operate on the mat, make sure it is of the rubber-tired type. No tracked equipment or sharp turns are allowed on the mat.
      - Alternatively, "stake chasers" or some other form of permanent visual markers can be utilized to provide a visual marker for maintenance activities.
- Damaged Sections
  - Missing or damaged sections of the matting should be replaced per the installation guidelines.
    - Repairing Rips or Holes: these should be patched with identical matting material. First, carefully cut out the damaged section with a knife. Then replace and compact soil to the elevation of the surrounding subgrade and plant seed. Cut a piece of replacement material a minimum of 12 in (300 mm) larger than the rip or tear. Use ties to attach the replacement material to the existing material. At overlaps, the upstream and upslope material should be on top. Secure the replacement material with ground anchoring devices spaced every 6 in (150 mm) around the circumference of the repair and at the frequency and spacing shown in the Anchor Pattern Guide on page 7. Seed and soil fill replacement area.

PROPEX | THE ADVANTAGE CREATORS.™
GEOSYNTHETICS

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Chattanooga, TN 37422

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PYRAMAT® high performance turf reinforcement mat (HPTRM) is a three-dimensional, lofty, woven polypropylene geotextile that is available in green or tan which is specially designed for erosion control applications on steep slopes and vegetated waterways. The matrix is composed of polypropylene monofilament yarns ***featuring X3® technology*** woven into a uniform configuration of resilient pyramid-like projections. The material exhibits very high interlock and reinforcement capacity with both soil and root systems, demonstrates superior UV resistance, and enhances seedling emergence.

PYRAMAT conforms to the property values listed below[1] and is manufactured at a Propex facility having achieved ISO 9001:2000 certification. Propex performs internal Manufacturing Quality Control (MQC) tests that have been accredited by the Geosynthetic Accreditation Institute – Laboratory Accreditation Program (GAI-LAP).

		MARV[2]	
PROPERTY	TEST METHOD	ENGLISH	METRIC
ORIGIN OF MATERIALS			
% U.S. Manufactured Inputs		100%	100%
% U.S. Manufactured		100%	100%
PHYSICAL			
Mass/Unit Area	ASTM D-6566	13.5 oz/yd²	457.7g/m²
Thickness	ASTM D-6525	0.4 in	10.2 mm
Light Penetration (% Passing)	ASTM D-6567	15% (Max)	15% (Max)
Color	Visual	Green or Tan	
MECHANICAL			
Tensile Strength (Grab)	ASTM D-6818	4000 x 3000 lb/ft	58.4 x 43.8 kN/m
Elongation	ASTM D-6818	40 x 35%	40 x 35%
Resiliency	ASTM D-6524	80%	80%
Flexibility	ASTM D-6575	0.534 in-lb (avg)	29.6 mg-cm (avg)
ENDURANCE			
UV Resistance % Retained 6000 hrs	ASTM D-4355	90%	90%
UV Resistance % Retained 10000 hrs	ASTM D-4355	85%	85%
PERFORMANCE			
Velocity[3] (Fully Vegetated)	Large Scale	20 ft/sec	6.1 m/sec
Velocity[3] (65 – 70% Vegetated)	Large Scale	16 ft/sec	4.9 m/sec
Velocity[3] (20 – 30% Vegetated)	Large Scale	12 ft/sec	3.7 m/sec
Shear Stress[3] (Fully Vegetated)	Large Scale	16 lb/ft²	766 Pa
Shear Stress[3] (65 – 70% Vegetated)	Large Scale	12 lb/ft²	575 Pa
Shear Stress[3] (20 – 30% Vegetated)	Large Scale	5 lb/ft²	239 Pa
Manning's "n"[4] (Unvegetated)	Calculated	0.028	0.028
Seedling Emergence[4]	ECTC Draft Method #4	296%	296%
ROLL SIZES		8.5 ft x 90 ft	2.6 m x 27.4 m

NOTES:

1. The property values listed are effective 04/2011 and are subject to change without notice.
2. MARV indicates minimum average roll value calculated as the typical minus two standard deviations. Statistically, it yields a 97.7% degree of confidence that any sample taken during quality assurance testing will exceed the value reported.
3. Maximum permissible velocity and shear stress has been obtained through vegetated testing programs featuring specific soil types, vegetation classes, flow conditions, and failure criteria. These conditions may not be relevant to every project nor are they replicated by other manufacturers. Please contact Propex for further information.
4. Calculated as typical values from large-scale flexible channel lining test programs with a flow depth of 6 to 12 inches.

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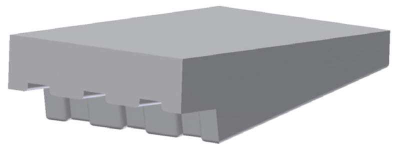
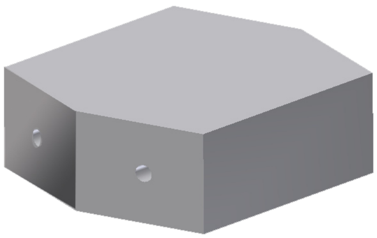
ATTACHMENT C
ARMORFLEX ARTICULATED CONCRETE BLOCK
PRODUCT INFORMATION
DATA SHEETS

CONTECH CONSTRUCTION PRODUCTS INC.

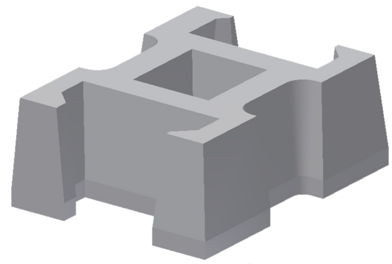
Armortec Product Details



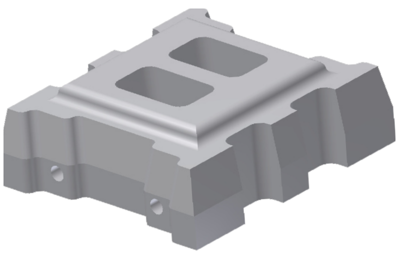
ArmorWedge®



ArmorRoad®

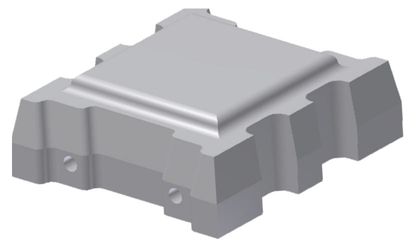


ArmorLoc®

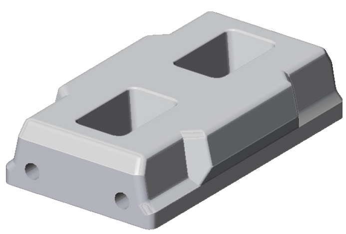


ArmorFlex® - Open Cell

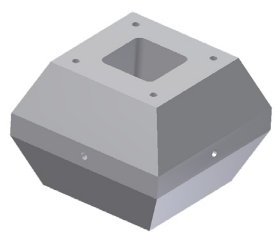
A-Jacks®



ArmorFlex® - Close Cell



ArmorStone®

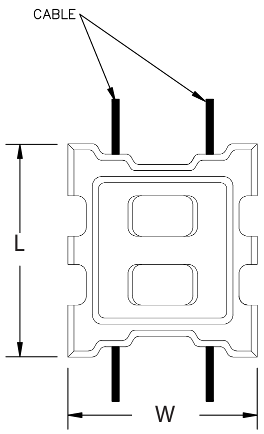


ArmorFlex® OS

MANUFACTURING SPECIFICATION
ASTM D6684-04

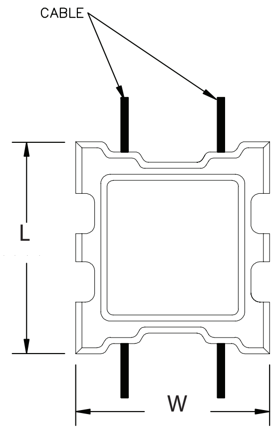
ARMORTEC
Erosion Control Solutions

ArmorFlex® (not to scale)



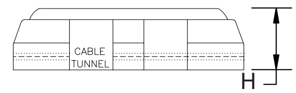
TOP VIEW

Open Cell Block

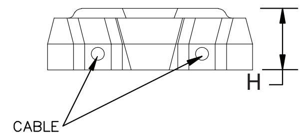


TOP VIEW

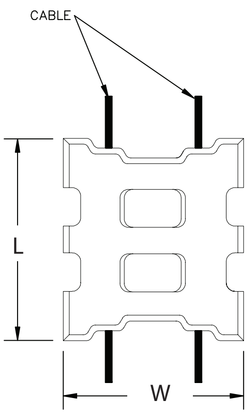
Close Cell Block



SIDE VIEW

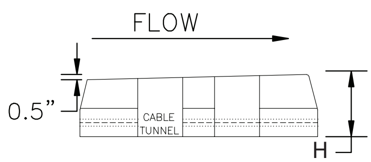


END VIEW

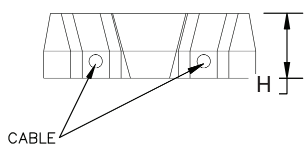


TOP VIEW

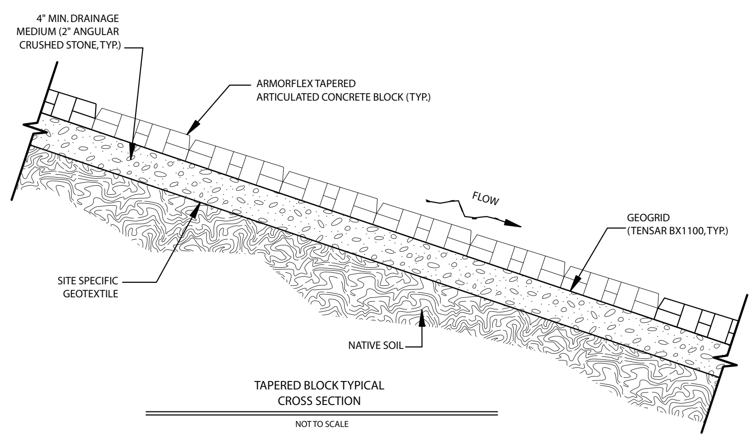
Tapered Series



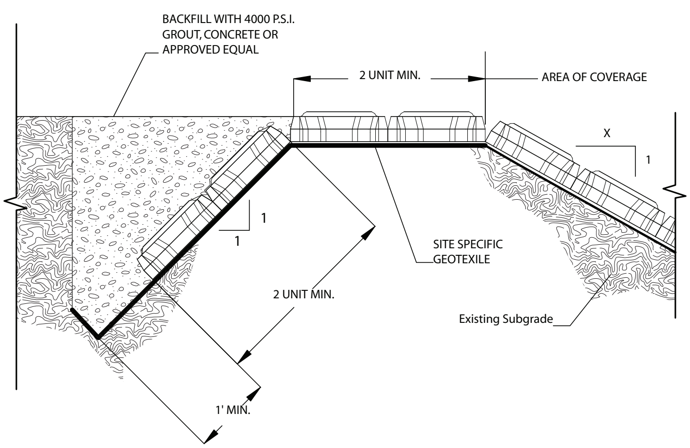
SIDE VIEW



END VIEW



Tapered Series - Cross Section

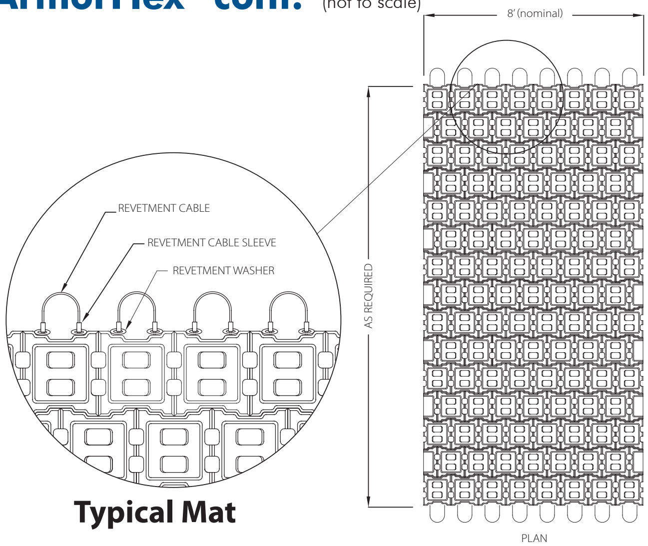


Top of Slope - Standard Detail

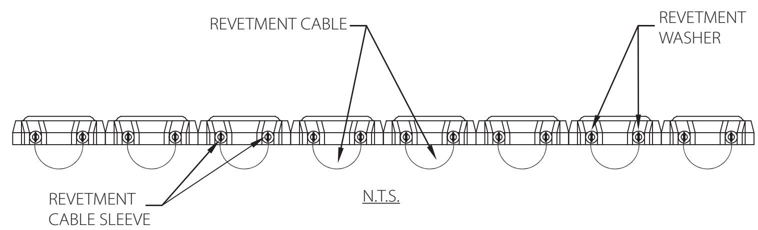
ArmorFlex Unit Specification

Concrete Block Class	Open/Closed Cell	Nominal Dimensions			Gross Area/ (sq. ft.)	Block Weight		Open Area %
		L	W	H		lbs	lbs/sq. ft.	
30s	Open	13.0	11.6	4.75	0.98	31-36	32-37	20
50s	Open	13.0	11.6	6.00	0.98	45-52	45-53	20
40	Open	17.4	15.5	4.75	1.77	62-71	35-40	20
50	Open	17.4	15.5	6.00	1.77	81-94	46-53	20
70	Open	17.4	15.5	8.50	1.77	120-138	68-78	20
40L	Open	17.4	23.6	4.75	2.58	90-106	35-41	20
70L	Open	17.4	23.6	8.50	2.58	173-201	67-78	20
45s	Closed	13.0	11.6	4.75	0.98	39-45	40-45	10
55s	Closed	13.0	11.6	6.00	0.98	53-61	54-62	10
45	Closed	17.4	15.5	4.75	1.77	78-89	43-50	10
55	Closed	17.4	15.5	6.00	1.77	94-108	53-61	10
85	Closed	17.4	15.5	8.50	1.77	145-167	82-98	10
45L	Closed	17.4	23.6	4.75	2.58	108-126	42-49	10
85L	Closed	17.4	23.6	8.50	2.58	209-243	81-94	10
High Velocity Application Block Classes								
40-T	Open	17.4	15.5	4.75	1.77	62-71	35-40	20
50-T	Open	17.4	15.5	6.00	1.77	81-94	46-53	20
70-T	Open	17.4	15.5	8.50	1.77	120-138	68-78	20

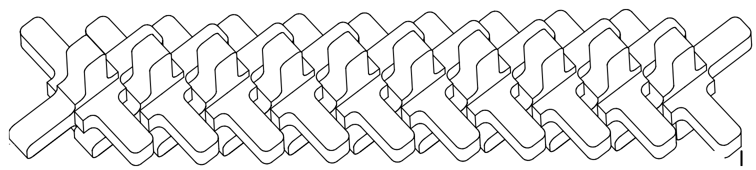
ArmorFlex® cont. (not to scale)



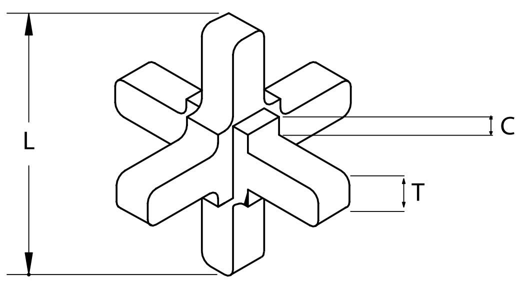
Typical Mat



A-Jacks® (not to scale)



A-Jacks Placement Profile

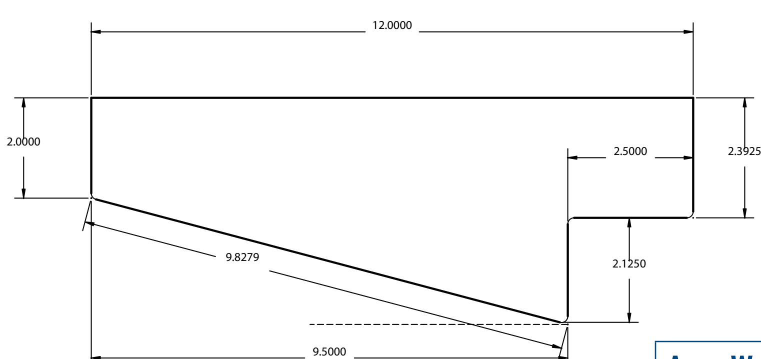


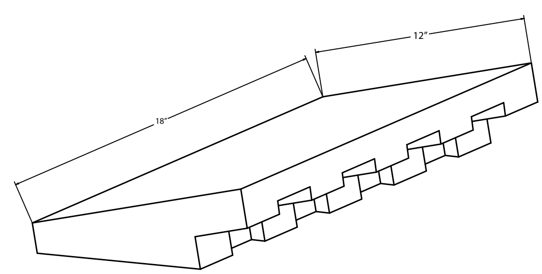
A-Jacks Unit

A-Jacks Unit Specification

A-JACKS	L(IN)	T(IN)/H(IN)	C(IN)	VOL(FT^3)	WT (LBS)
AJ-24	24	4	1.84	0.56	78
AJ-48	48	7.36	3.68	4.49	629
AJ-72	72	11.04	5.52	15.14	2.120
AJ-96	96	14.72	7.396	35.87	5.022
AJ-120	120	18.40	9.20	70.69	9.699

ArmorWedge® (not to scale)



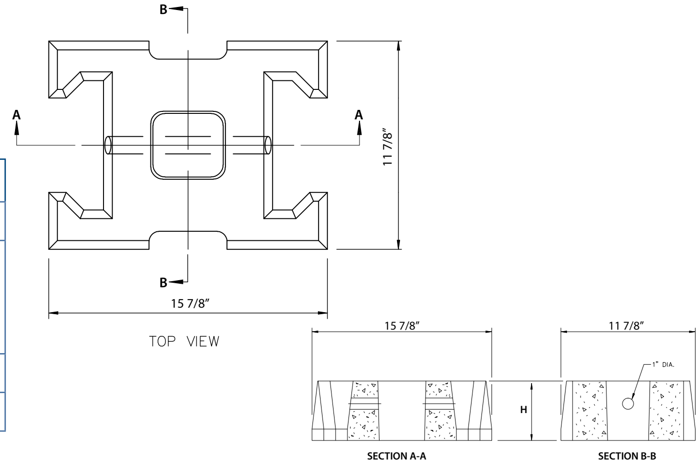


Side View - Typical Block

ArmorWedge Unit Specification

UNIT DIMENSION	UNIT WEIGHT (LBS)	SYSTEM WEIGHT (LBS)	UNIT COVERAGE (SF)	COMPRESSIVE STRENGTH (PSI)	MAXIMUM ABSORPTION (LBS/FT^3)
12x18	40-52	36-40	1.1875	4000	12

ArmorLoc® (not to scale)

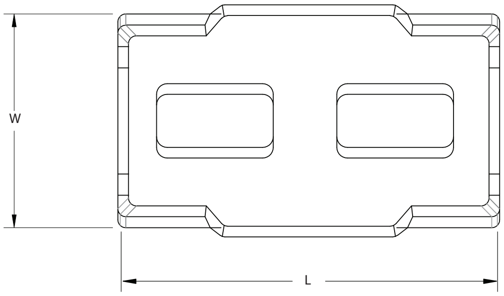


TOP VIEW

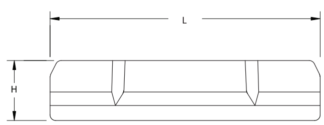
ArmorLoc Unit Specification

BLOCK	TECHNICAL DATA			DIMENSIONS AND WEIGHTS				
	Specific Weight lbs/ft³	Compressive Strength psi	Max Absorption, Avg. of 3 units 10 lbs/ft³	Thick-n2ess Inches (H)	Gross Area/ Grid ft²	Weights/ Grid lbs	Weights/ Area lbs/ft²	Open Area %
3510	130-150	4000 min	10	4	1.0	30-35	30-35	25
4511	130-150	4000 min	10	5.25	1.1	44-50	40-45	20

ArmorStone® (not to scale)



PLAN

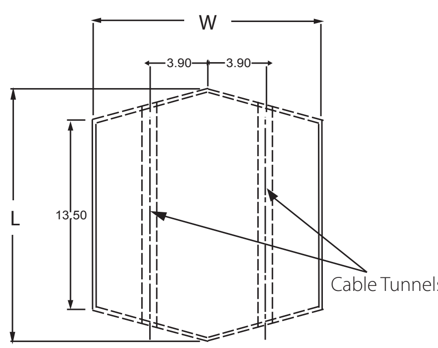


FRONT

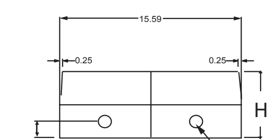
ArmorStone Unit Specification

CONCRETE BLOCK CLASS		NOMINAL DIMENSIONS (IN.)			UNIT COVERAGE (SF)	BLOCK		OPEN AREA %
		L	W	H		UNIT WEIGHT LBS.	SYSTEMS WEIGHTS LBS./SF	
AS 40	Open	18.00	10.00	4.50	1.25	50-54	40-44	25
AS 55	Closed	18.00	10.00	4.50	1.25	61-66	49-53	5

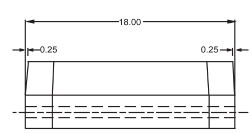
ArmorRoad® (not to scale)



TOP VIEW



FRONT VIEW



SIDE VIEW

ArmorRoad Unit Specification

BLOCK	TYPE	L	W	H	SF COVERAGE PER UNIT	WEIGHT LBS PER UNIT	SF PER TRUCK LOAD
Mat	Closed	18.00	15.60	6.00	1.74	105-109	750
Individual	Closed	18.00	15.60	6.00	1.74	100-104	750

Armortec Minimum Physical Requirements per ASTM 06684-04

MIN. DENSITY (IN AIR) LBS/FT³		MIN. COMPRESSIVE STRENGTH PSI		MAX WATER ABSORPTION LBS/FT³	
Ave. of 3 Units	Individual Unit	Ave. of 3 Units	Individual Unit	Ave. of 3 Units	Individual Unit
130	125	4,000	3,500	9.1	11.7

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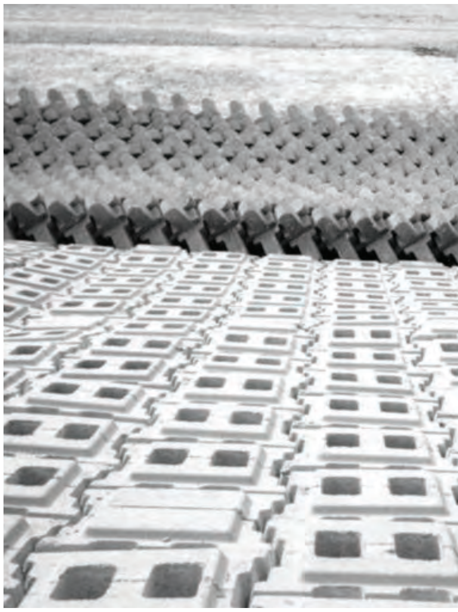
Engineered Hard Armor Solutions



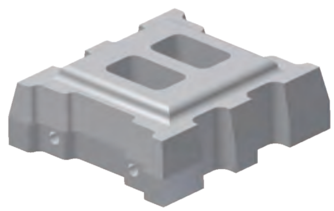
ARMORTEC Erosion Control Solutions

Channel Lining | Shoreline Protection | Dam & Spillway Protection | Levee Stabilization | Bridge Scour

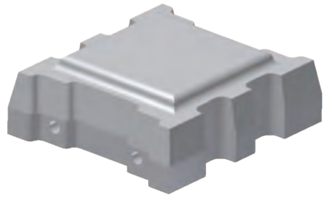
ArmorFlex: Articulating Concrete Block Mats

The industry leader since 1978, ArmorFlex® articulating concrete block (ACB) mats make a flexible matrix of concrete blocks with uniform size, shape and weight used for hard armor erosion control. ArmorFlex blocks have specific hydraulic capacities and are laced longitudinally with galvanized steel, stainless steel or polyester revetment cables which provide ease of handling and installation.

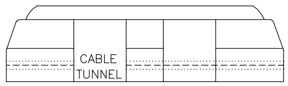
Open-Cell Block



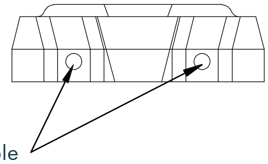
Closed-Cell Block



Side View



End View



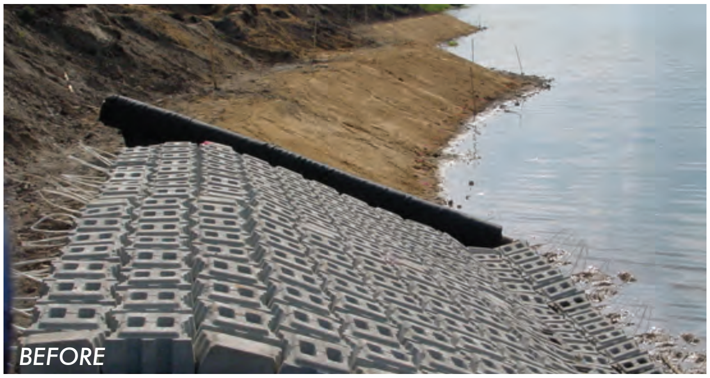
Applications

- Channel Lining
- Shoreline Protection
- Boat Ramps & Access Roads
- Dam Overtopping Protection
- Pipeline & Cable Protection
- Bridge Abutment Protection
- Retention Basins
- Levee Stabilization
- Bridge Scour Protection

ArmorFlex has proven to be an aesthetic and functional alternative to dumped stone riprap, gabions, structural concrete and other hard armor erosion protection systems. ArmorFlex is easy to install and has a low life-cycle cost when compared to other permanent solutions. These two benefits can drastically reduce the cost to install and maintain the system. ArmorFlex mats are installed on a prepared subgrade utilizing conventional construction equipment and site-specific filter fabric. While both block types provide protection and stability, only the open-cell specifically offers the void space necessary for revegetation.

Research Proven Performance

Armortec has carried out extensive research into wave and open channel flow conditions on ArmorFlex in the United States and the Netherlands. Design manuals and computer programs are available to assist in the proper ArmorFlex block selection for your hydraulic conditions.

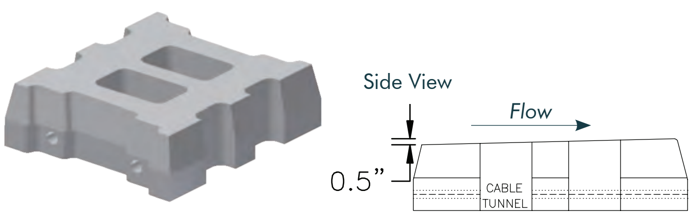


ArmorFlex: Articulating Concrete Block Mats

Tapered Series

Armortec's unique Tapered Series ArmorFlex block design offers superior protection for embankment dams, spillways, high velocity channels and down chutes. The essential design component of ArmorFlex Tapered series is the 0.5 inch taper that virtually eliminates destabilizing impact flow forces, thereby providing a high factor of safety. The ArmorFlex Tapered block system has been successfully tested under hydraulic jump conditions at Colorado State University. Each Tapered series design incorporates a four inch rock drainage layer beneath the system.

Tapered-Cell Block



Block and A Half

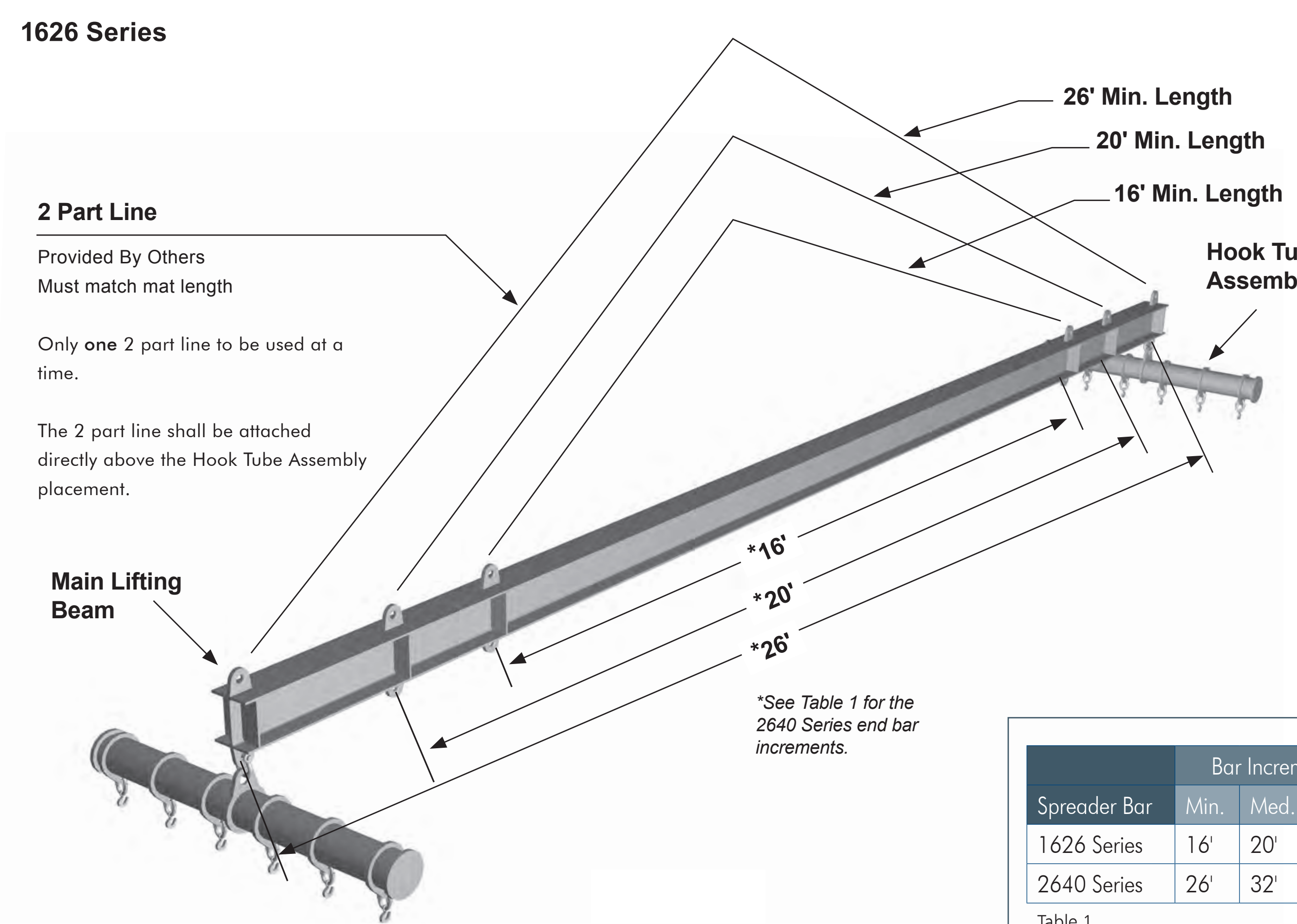
The latest innovation in ACB technology is the ArmorFlex Block and a Half®. This new product introduction increases the factor of safety for the overall system while maintaining the ease of installation and overall benefits of the typical ArmorFlex systems.

Block and A Half Block

ArmorFlex: Installation

The proper installation of ArmorFlex ACB mats is important to achieving the intended hydraulic performance and maintaining stability against the erosive forces of flowing water. An ACB revetment system consists of a suitably prepared and compacted subgrade, a suitable site-specific filter fabric and properly sized ACB mattresses placed in "intimate contact" with the filter fabric and subgrade. Each individual site will vary, so it is important to follow the engineering project drawings as designed and sealed by a registered Professional Engineer; particularly as they relate to standard termination details. Please refer to the Armortec Installation Guide for further instructions on proper material handling.

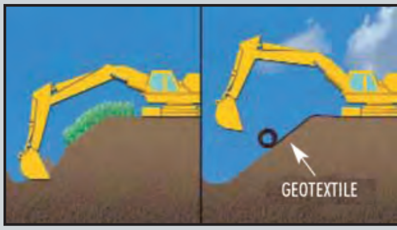
Spreader Bar Rigging Detail



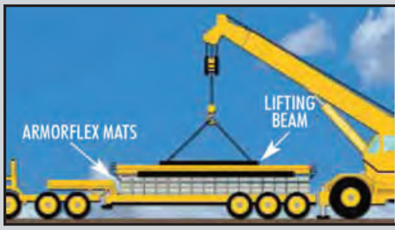
	Bar Increments		
Spreader Bar	Min.	Med.	Max.
1626 Series	16'	20'	26'
2640 Series	26'	32'	40'

Table 1

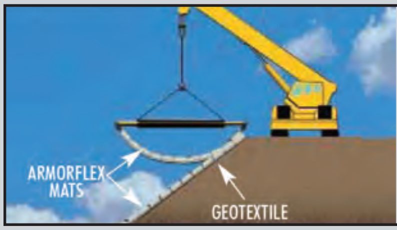
ArmorFlex: Installation



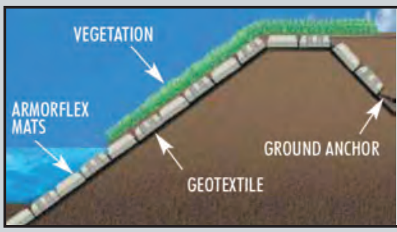
Step 1:
ArmorFlex arrives on-site as a system of factory-assembled mats. ArmorFlex is placed on a site specific geotextile which has been placed on a prepared subgrade using conventional construction equipment.



Step 2:
Mats are supplied on flat bed trailers. Mats can be handled with a spreader bar which can be rented from CONTECH.



Step 3:
Above normal waterline mats may be topsoiled and seeded to give a vegetated effect.



Step 4:
Proper toe trench requires a minimum of two rows of block buried below predicated soil depth. Tapered series block or mats subject to wave attack are required to have a bedding layer of crushed stone or gravel.

See Armortec Installation Guide for complete information on how to properly install ArmorFlex.

ArmorRoad

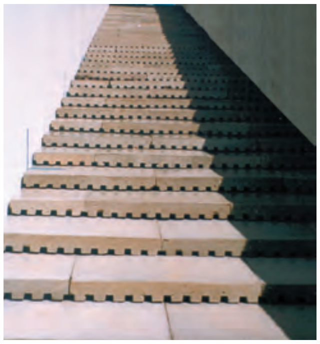
ArmorRoad® was developed in the field with input from contractors, construction managers and owners. The result is a flexible product that is efficient to install, aesthetically pleasing and able to withstand heavy traffic loads in harsh environments. ArmorRoad does not require the sand backfill typically required of standard pavers due to its unmatched durability with 8,000 PSI and 6” thickness. In addition, should a problem occur in the subgrade, ArmorRoad can be removed quickly and reinstalled.

Applications

- Durable Driving Surface
- Temporary Road Application
- Heaving and Expanding Subgrade Condition

ArmorRoad Block

ArmorWedge



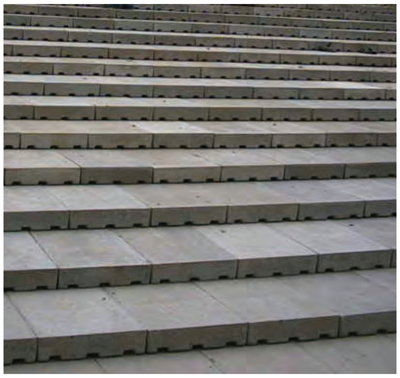
ArmorWedge® is a concrete step overlay protection system for embankment dams and spillways that are subject to high forces associated with overtopping flow. Researchers at Colorado State University assessed the stability of the blocks by comparing the downward (positive) forces of the block weight and the pressure of the flowing water to the uplift (negative) forces. The ArmorWedge system was tested up to and including the facility discharge capacity of 40 cf/s/ft. This discharge capacity had associated water velocities of 35 ft/sec and a shear stress of 22 lbs/sqft. Even at these levels the ArmorWedge system remained stable. An effective drainage system - allowing water to be removed from beneath the system - is essential to the design of the overlay.

The practicality of ArmorWedge lies in the cost effective ease of installation. This is particularly true for projects where the use of large machinery is deemed impractical due to confined, hard to reach jobsites or environmental impact on the surrounding area. ArmorWedge is typically installed by hand over site-specific filter fabric and subsequent drainage medium on a well compacted surface.

Applications

- Dam Overtopping
- High Velocity Channels
- Primary and Secondary Spillways

ArmorWedge Block



ArmorLoc

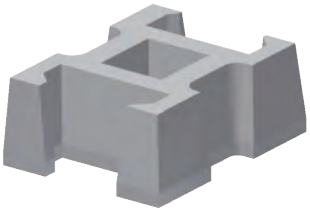
ArmorLoc® concrete interlocking blocks are specifically designed to control erosion. The ArmorLoc system provides easy and economical installation when equipment is not feasible or cannot be used due to confined or hard to reach areas. ArmorLoc is installed manually over site-specific filter fabric on a prepared surface. It improves the landscape and promotes drainage from the smallest erosion control job to the largest commercial project.

ArmorLoc is available in two sizes and weight classifications that provide excellent performance during light wave and open-channel flow conditions. The unique interlocking design of ArmorLoc keys each block into four adjacent blocks to hold it firmly in position and resist lateral movement.

Applications

- Retention Basins
- Shoreline Protection
- Drainage Ditch Lining
- Outfall Protection
- Bridge Abutment Protection

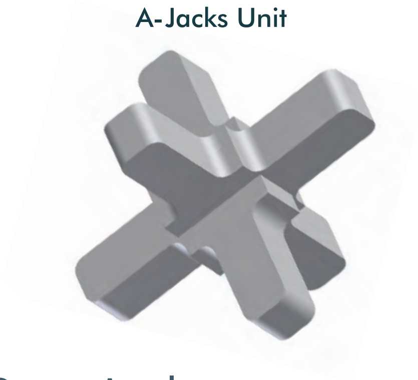
ArmorLoc Block



A-Jacks: Concrete Armor Units

A-Jacks® are high stability concrete armor units designed to interlock into a flexible, highly permeable matrix. A-Jacks can be installed either randomly or in a uniform pattern. The voids formed within the A-Jacks matrix provide approximately 40% open space in the uniform placement pattern. These voids provide habitat for fish and other marine life when applied as a reef, revetment or as a soil support system in river applications. In addition, the voids may be backfilled with suitable soils and planted with a variety of vegetation including grasses, shrubs and trees above the normal base flow.

A-Jacks Unit



Applications

- Drop Structures
- Weirs
- Energy Dissipation
- Bridge Scour Protection
- Streambank/Toe Stabilization

Streambank Applications

Streambank erosion often produces steep banks with little or no vegetation. These unprotected banks are even more susceptible to erosion due to over steepening, loss of ground cover, groundwater discharge and stream erosion at the base of the bank. A-Jacks concrete armor units provide an alternative which when used with bio stabilization technique, develops a cost-effective solution.

Bridge Scour Applications

The ability of the A-Jacks system to dissipate energy and resist the erosive forces of flowing water allows this system to protect channel boundaries from scour and erosion. Extensive laboratory research was performed on both model and full scale units in order to evaluate the hydraulic properties of the A-Jacks units. An A-Jacks Design Manual for the hydraulic design of open-channel conveyance ways and pier scour countermeasure is available upon request.

Energy Dissipation

A-Jacks ability to dissipate energy in channel, spillway or culvert outfall applications relies on the inherent roughness of the units. For A-Jacks, the design value for Manning's roughness coefficient is $n=0.1$. This value was determined from extensive full and quarter scale laboratory testing. The ability of A-Jacks to increase roughness creates a hydraulic jump when flow encounters the units. Creating the hydraulic jump effectively releases the energy associated with high velocity and/or steep embankment flow conditions. By releasing the energy, the erosive forces associated with the hydraulic jump are also greatly diminished. As the flow travels downstream through the A-Jacks matrix, the energy grade line slope continues to be reduced until the desired flow conditions are obtained downstream of the A-Jacks units.

CONTECH Construction Products Inc. provides site solutions for the civil engineering industry. CONTECH's portfolio includes bridges, drainage, retaining walls, sanitary sewer, stormwater, erosion control and soil stabilization products.

For more information, call one of CONTECH's Regional Offices located in the following cities:

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Colorado (Denver)	720-587-2700
Florida (Tampa)	727-544-8811
Georgia (Atlanta)	770-409-0814
Maine (Scarborough)	207-885-9830
Maryland (Baltimore)	410-740-8490
Oregon (Portland)	503-258-3180
Texas (Dallas)	972-590-2000

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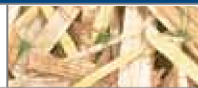
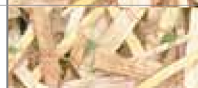
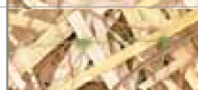
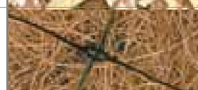
Armortec 8/11 MC

RECYCLED PAPER

CONTECH CONSTRUCTION PRODUCTS INC.

EROSION CONTROL PRODUCT SELECTION GUIDE [1]

	PRODUCT		Functional Longevity	Slopes			Channels		Bank/Shoreline Stabilization		Culvert Outlets	Installed Costs[2] ($/SY)
				≤1:1[3]	≤2:1	≤3:1	Typical Velocity (ft/s)	Typical Shear Stress (lb/ft^2)	Wave Potential	No Wave Potential		
TEMPORARY BLANKETS	**Landlok** (S1)		12 months			✔	5-6	2.0				1.00 to 1.75
	Landlok (S2)		18 months			✔	5-6	1.5				1.25 to 1.75
	Landlok (CS2)		24 months		✔		5-6	2.0				1.75 to 2.25
	Landlok (C2)		36 months		✔ (≤1.5:1)		5-6	2.3				2.00 to 2.75
PERMANENT TURF REINFORCEMENT MATS	**Landlok 450**		Permanent	✔			8 to 18	2 to 10				6.00 to 8.00
	Landlok 300		Permanent	✔			6 to 20	2 to 12		✔	✔	10.00 to 15.00
	Pyramat		Permanent (up to 50 years)	✔			6 to 25	2 to 15		✔	✔	15.00 to 20.00
	ArmorMax Anchored Reinforced System		Permanent (up to 50 years)	✔			6 to 25	2 to 18		✔	✔	20.00 to 25.00
HARD ARMOR	**Armorflex** ACB Revetment System		Permanent	✔			4" - 11 - 15 6" - 13 - 29 9" - 17 - 37	4" - 14 - 31 6" - 19 - 37 9" - 22 - 48	✔		✔	82.50 to 112.50 90.00 to 127.50 97.50 to 135.00
	Armorloc Hand Placed ACB Revetment System		Permanent	✔			4" - 10 6" - 12	4" - 8 6" - 11	✔		✔	52.50 to 82.50 75.00 to 97.50
	A-Jacks		Permanent	✔			24" - 22.0 48" - 31.1 72" - 38.1 96" - 44.0	24" - 38 48" - 76 72" - 114 96" - 152	✔		✔	30 to 45/ea. 375 to 525/ea. 900 to 1350/ea. 1650 to 2250/ea.
	Gabions		Permanent	✔			16	20	✔		✔	Basket:: 100 to 125/cy. Mattress:: 30 to 60/cy.

NOTES: 1. The above design recommendations should only be used as a "quick" reference tool for general project situations. Final selection of an appropriate product should be done by an experienced engineer and should consider site-specific parameters such as climate, soil, geometry, vegetation selection, irrigation, and installation conditions.
2. Installed cost estimates range from large to small projects according to material quantity. The estimates include E.C. material, seed, labor and equipment.
3. For slopes steeper than 2H:1V, mechanical anchoring should be investigated

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Golden, CO 80401
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Technical Memorandum

To:	File	From:	Diana Cook, Justin Knudsen
Company:	Victoria Gold Corp.	Date:	May 3, 2011
Re:	Dublin Gulch – Seismic Peak Ground Accelerations for Design	Project #:	114-201045x
CC:	Troy Meyer		

EXECUTIVE SUMMARY

This document provides a summary of the seismic hazard analysis performed for Victoria Gold Corporation's Dublin Gulch property. This seismic hazard analysis includes results from both deterministic and probabilistic methods. Deterministic analyses were performed using five equally weighted attenuation relationships to evaluate seismic hazards for the Goldstrike property resulting from a maximum credible earthquake (MCE). A MCE, by definition, has no specific recurrence interval and is the largest reasonably conceivable earthquake that appears possible along a recognized fault or within a geographically defined tectonic province, under the presently known or presumed tectonic framework. Theoretically, no ground motion should occur which exceeds that of the MCE. A deterministic analysis therefore allows for a more conservative approach to the determination of risks associated with identified seismic hazards. Data published by Natural Resources Canada (NRCAN) were used in the probabilistic analysis to estimate the probability of exceedance of peak ground accelerations (PGA) at the site for various return periods.

Considering the level of conservatism inherent in a deterministic analysis, and the added conservatism discussed in Section 3.1.5, Tetra Tech recommends a design PGA of 0.27g for high hazard facilities, based on an MCE of moment magnitude 7.0 generated in the Ogilvie Mountains area. This PGA is anticipated to reflect the current tectonic environment with greater accuracy than a low probability value based on the very short historic seismic record available, such as a 5,000-year event would produce. For facilities requiring a PGA based on a return period of 1,000 years or less, the mean National Building Code of Canada (NBCC) values from Table 3 may be used.

1.0 Introduction

This document provides a summary of a seismic hazard analysis for the Eagle Gold Project at Dublin Gulch, located approximately 85 km by road north of Mayo, and 20km northwest of Elsa in the Yukon Territory of Canada. For the purposes of this seismic hazard study, the site is assumed to be centered at approximately 64.0º N Latitude and 135.9º W Longitude. Access to the site is by way of

Silver Trail Highway from Mayo, and South McQuesten River Road, which connects to the highway approximately 39 km northeast of Mayo. The purpose of this document is to provide ground motions that may be used in design procedures for facilities on the mine site. This updated seismic hazard analysis includes results from both deterministic and probabilistic analyses.

2.0 Tectonic Setting and Seismicity

Regionally, the site is contained within the northern Canadian Cordillera, which encompasses an area stretching between approximately Latitudes 55 to 70 N, and Longitudes 110 to 150 W. More specifically, the site is located in the Selwyn Basin and lies within the Tombstone Thrust and Robert Service Thrust fault zones (Figure 1), where more than 100 km of structural overlap was accommodated during the Cretaceous (Mair et al., 2006). Indeed, much of the mineralization present in the Selwyn Basin is related to collision-related deformation of the Yukon Tanana terrane onto the ancient continental margin before 100 Ma, followed by the intrusion of granitic magma around 93 Ma. This new assembly experienced lateral displacement along the Tintina fault (located south of the site) during the Late Cretaceous (post-85 Ma). During recent geologic history, however, the Dublin Gulch area has generally been categorized as an area of low seismicity (Colpron, 2011), compared to other areas in Yukon Territory. These more seismically active areas include, but are not limited to, the Wernecke and Mackenzie Mountains to the east-northeast, the Ogilvie Mountains to the west-northwest, the Richardson Mountains to the north, and the Denali Fault zone to the southwest.

According to Natural Resources Canada (NRCAN), the northern Rocky Mountains region, which runs along the border of the Yukon and Northwestern Territory before extending westward, is one of the most seismically active areas in Canada. Earthquakes along the mountain front north and east of the Ogilvie, Wernecke, and Mackenzie Mountains are related to the northeastward push by the Yakutat microplate against the St. Elias Mountains in the south.

The largest earthquake recorded in the northern Rocky Mountains area was a magnitude 6.9 earthquake that occurred on December 23, 1985 in the Mackenzie Mountains of the Northwest Territories, at a distance of approximately 622 km from the site, according to the available records. Other 6-plus magnitude earthquakes have occurred in the Richardson Mountains of the Yukon Territory and include: M=6.2 in May, 1940; M=6.5 in June, 1940, and M=6.6 in March, 1955.)

3.0 Seismic Hazard Analysis

Seismic hazard analyses are typically conducted using one of two readily available methods: (1) deterministic analysis or (2) probabilistic analysis. Evaluating potential ground motions from a maximum credible earthquake (MCE), which is by definition the largest reasonably conceivable earthquake possible along a known fault or seismogenic source zone, is a deterministic method. Probabilistic analyses are commonly used where the earthquakes occur in conjunction with known structures with known activity rates and in areas of diffuse historic seismicity where large regions of similar historic seismicity can be assigned characteristic ground motions based on the rate of historic earthquakes. This type of analysis results in probabilities of occurrence or non-exceedance versus time or return period. Both approaches were used in this study.

3.1 Deterministic Analysis

There are four key elements to performing deterministic seismic hazard analyses (DSHA) for use in project design, namely: (1) determination of seismogenic structures or seismogenic source zones; (2) definition of the associated characteristic earthquake magnitudes; (3) the distances between the project site and the seismogenic sources, and; (4) selection of an appropriate attenuation function to represent the decay of earthquake ground motions with distance and estimate the ground motions at the project site. The following sections outline these key elements and discuss the assumptions and methodologies employed in determining each of these elements for the current study.

3.1.1 Earthquake Database

The deterministic seismic hazard analysis conducted for this study included a review of earthquake records from the Canadian National Earthquake Database (NEDB). A search of the NEDB was performed for a 150 km radius around a central Latitude and Longitude of 64.0^{o} N and 135.9^{o} W for Dublin Gulch (Figure 1). The search was restricted to earthquakes with moment magnitudes of 4 or greater, as magnitudes below this will have little impact on engineering works. Due to the historic, and even current, paucity of seismograph stations in the region, the database only includes earthquakes that have occurred since 1985.

The results of the earthquake search included local magnitude (M_L), short-wave body magnitude (m_b), and moment magnitude (M_w). These magnitude measurement scales are essentially equal for moment magnitudes of 6 or less (Idriss, 1985). They were therefore not converted to reflect the same magnitude scale. The search yielded 19 shallow earthquakes (18 km depth or less) that occurred within a 150 km radius, 16 between magnitudes 4 and 4.9, and three between magnitude 5 and 5.1. The two largest earthquakes in the 150 km record were located east and southeast of the site, at distances of 89 km and 148 km, respectively. These earthquakes were both magnitude 5.1, and occurred on November 25, 1997 and September 24, 2005 in the region of the Mackenzie Mountains.

3.1.2 Seismogenic Source Zones

Assessment of seismic hazards for the Eagle Gold site requires consideration of potential earthquake source zones, either identifiable seismogenic faults or larger areas with common seismogenic characteristics. Once source zones have been identified, maximum earthquakes can be assigned for each source zone. In the following sections, potential fault sources and source zones are identified. Considering that ground motions resulting from earthquakes with source-to-site distances greater than about 150 km are relatively small, the study area for the hazard assessment was restricted to those sources lying within 150 km of the project site.

Typically deterministic studies are restricted to assessing faults or source zones (large areas with common seismogenic characteristics) that have shown to or are suspected of having displaced Quaternary-age (less than approximately 1.8 million years) deposits, and are therefore generally considered "active." According to Maurice Colpron of the Yukon Geological Survey (Personal Communication, 2011), the Dublin Gulch area is not known to contain Quaternary-age faults. The site-specific search therefore included all known faults near Dublin Gulch, and was performed using Geographic Information System (GIS) shapefiles provided through the Yukon Geological Survey. The search resulted in 3,145 individual fault listings within 150 km of the site. The majority of these faults do not appear to have been studied in detail, and few details are available concerning type, orientation, total length, or age. Many of the individual fault segments are part of larger fault systems. The catalog (Table 1) has therefore been restricted to named faults in the area, most of which have been studied and can be tracked in a literature search, and includes the longest individual segments and the segments closest to the site for each fault source. The faults listed in Table 1 are labeled in Figure 1.

Table 1: Named Faults Near Dublin Gulch

FAULT ID	FAULT TYPE	FAULT NAME	LENGTH (km)	NEAREST DISTANCE TO SITE (km)
10666	Fault, defined, thrust, upright	CALLISON LAKE NORMAL FAULT	8.2	72.1
8011	Fault, defined, thrust, upright	CALLISON LAKE THRUST FAULT	1.2	62.6
7388	Fault, defined, thrust, upright	CALLISON LAKE THRUST FAULT	7.6	68.4
27530	Fault, approximate, thrust, upright	DAWSON THRUST	53.2	44.2
7650	Fault, defined, normal/reverse	FOREST FAULT	7.6	116.6
28896	Fault, defined, normal/reverse	FOREST FAULT	4.0	109.5
13467	Fault, assumed, movement undefined	JOSEPHINE CREEK FAULT	0.4	53.6
13697	Fault, approximate, movement undefined	JOSEPHINE CREEK FAULT	2.1	53.7
11592	Fault, assumed, movement undefined	KATHLEEN LAKES FAULT	12.0	73.7
27133	Fault, defined, thrust, upright	LOWER LAKE CREEK THRUST	9.4	66.7
8457	Fault, defined, thrust, upright	LOWER RAE CREEK THRUST	2.7	90.0
9113	Fault, defined, thrust, upright	LOWER RAE CREEK THRUST	0.7	82.8
11707	Fault, assumed, thrust, upright	Moose Lake Thrust	2.1	89.3
23905	Fault, extrapolated, thrust, upright	Moose Lake Thrust	19.7	96.8
8663	Fault, defined, thrust, upright	NORTH FORK THRUST FAULT	0.3	123.2
9347	Fault, defined, thrust, upright	NORTH FORK THRUST FAULT	13.5	123.5
12284	Fault, assumed, thrust, upright	Robert Service Thrust	58.7	32.0
12838	Fault, approximate, thrust, upright	ROBERT SERVICE THRUST	3.2	15.2
13138	Fault, assumed, movement undefined	Sideslip Lake Fault	13.2	101.8
23915	Fault, approximate, movement undefined	Sideslip Lake Fault	2.0	95.4
13665	Fault, approximate, normal/reverse	SPRAGUE CREEK FAULT	7.5	35.3
13692	Fault, approximate, normal/reverse	SPRAGUE CREEK FAULT	0.8	34.7
26882	Fault, extrapolated, dextral	TINTINA FAULT	14.9	82.5
26631	Fault, extrapolated, dextral	TINTINA FAULT	39.2	104.3
13679	Fault, approximate, thrust, upright	TOMBSTONE STRAIN ZONE UPPER BOUNDARY	21.6	6.8
27690	Fault, approximate, thrust, upright	TOMBSTONE THRUST	29.2	29.3
8273	Fault, defined, thrust, upright	UPPER LAKE CREEK THRUST	1.1	67.3
8009	Fault, defined, thrust, upright	UPPER LAKE CREEK THRUST	4.8	76.6
10298	Fault, assumed, thrust, upright	WERNICKE FAULT	0.2	129.3
9856	Fault, defined, thrust, upright	WERNICKE FAULT	8,409	133.7

The youngest known fault in the vicinity of the site which is large enough to generate strong ground motions appears to be the Tintina Fault, which is a major strike-slip fault that stretches from British Columbia to Alaska, and is estimated to have last moved during the Eocene (approximately 54-36 million years ago). Other large historic faults in the area include the Dawson, Tombstone, and Roberts Service thrust faults, which are Cretaceous in age (approximately 141-65 million years ago). The remaining faults listed in Table 1 appear to be related to collisional-deformation during the Cretaceous. It must be emphasized that these structures were active during a very different seismogenic framework than exists in the area today, and they are therefore not included in the DSHA. While earthquakes have occurred in the area near Dublin Gulch in the recent past, they do not appear to be associated with specific geologic structures exhibiting offset or displacement at the ground surface.

A review of potential earthquake source zones in the project area, based on recent earthquake history and the existing seismogenic framework, confirms seismic activity related to movement in nearby mountain ranges. Areas that show a concentration of earthquake activity in the 26 years covered in NRCAN's database include the Wernecke/Mackenzie (these mountains are grouped as a single seismic source for the purposes of this study), and Ogilvie Mountains, as noted previously in Section 2.0. An MCE associated with one of these sources is likely to control the design PGA. For purposes of the DSHA, the distance to these two source zones was assumed to approximately coincide with the base of the mountains at their nearest point to the site. A typical method for assigning a MCE to site-specific area sources is to add 0.5 to 1.0 magnitude to the maximum magnitude in the earthquake history. However, due to the short span of the earthquake history, and considering the earthquake magnitudes on a regional scale (Section 2.0), a conservative magnitude of 7.0 was assigned to these sources, summarized below:.

- Wernecke/Mackenzie Source Zone, M = 7.0, Epicentral Distance (D) = 60 km
- Ogilvie Source Zone, M = 7.0, D = 30 km

3.1.3 Attenuation Relations

Seismic hazard analyses require an attenuation relationship to represent the decay of earthquake ground motions with distance, or a combination of weighted attenuation relationships, in order to produce estimated ground motions for use in project design. Most recently published applicable attenuation relationships are based on a database of worldwide strong motion recordings provided by the Pacific Earthquake Engineering Research (PEER) Center for their Next Generation Attenuation (NGA) project. In general, these attenuation relationships are considered applicable to the western United States and other tectonically active regions that experience shallow crustal faulting (Campbell and Bozorgnia, 2007). These attenuation relationships are therefore also considered applicable to the Dublin Gulch Property.

The NGA project supported five teams in the development of new empirical ground motion models for the estimation of peak ground accelerations and 5%-damped pseudo-acceleration response spectra. Each team was given access to the same database and a set of criteria dictating the limiting parameters of the final product, but otherwise the researchers were allowed to make their own interpretations. The five NGA model development teams included Abrahamson and Silva (2008); Boore and Atkinson (2008); Campbell and Bozorgnia (2008); Chiou and Youngs (2008); and Idriss (2008). In general, the NGA models provide median and aleatory uncertainty values for peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), and response spectral acceleration (PSA) and displacement (SD) for oscillator periods ranging from 0.01 to 10.0 seconds. The models are valid for earthquake magnitudes ranging from 4.0 to 8.0, and distances ranging from 0 to 200 km. A Microsoft Excel spreadsheet published by Dr. Linda Al Atik in September, 2009, includes the five NGA attenuation models and allows for the calculation of averages weighted at the user's discretion. The spreadsheet is available through the PEER website (http://peer.berkeley.edu/ngawest/index.html) and, according to Dr. Atik, has been compared with success to other calculation files (Al Atik, Linda, 2010).

3.1.4 Attenuation Model Input

The NGA spreadsheet requires earthquake magnitude, and specific fault parameters be input, namely geometry-related values such as: site-to-source distances, fault rupture depth, fault rupture width, and dip of rupture plane; depths to 1.0 km/s and 2.5 km/s shear wave velocity horizons; average shear wave velocity in the upper 30 m of the foundation materials (V_{s30}); sense of fault movement (i.e., normal, reverse, strike-slip, or unspecified); and, whether the site lies on the hanging wall (hanging wall factor) or foot wall, in the case of normal or reverse faults. This last factor is meant to account for the fact that ground motions on the hanging wall are typically higher than those observed on the footwall of a fault.

According to Cassidy et al. (2005), large earthquakes that have occurred in the Mackenzie Mountains are generally shallow thrust faults that often do not have a surface expression. This is assumed to be the case for earthquakes in the Wernecke and Ogilvie Mountains as well, based on the tectonic framework of the area and the northeasterly movement of the Yukatat microplate. Both source zones were modeled assuming movement on a thrust fault dipping to the southwest. This is a conservative assumption, as this puts the Dublin Gulch Property on the hanging wall for both source zones. In addition, a shallow dip of 30 degrees and a depth of 6 km were assumed, based on the earthquake history and the gentle to moderate dip described for historic shallow thrust faults in the area (Mair et al., 2006). Empirical relationships by Wells and Coppersmith (1994), which relate segment lengths to magnitude, and magnitude to rupture area, were used to back-calculate segment lengths, and then, based on the segment lengths, estimate rupture widths. Depths to 1.0 km/s and 2.5 km/s shear wave velocity horizons are not known, but as this is the expected case for most analyses, default values are provided by the NGA models; these values were used for the Dublin Gulch Property. The average shear wave velocity in the upper 30 m of the foundation (V_{s30}) was assumed to be 750 m/s, a typical value for the National Building Code of Canada (NBCC) "firm ground" soil class C.

3.1.5 Notes on Deterministic Model Conservatism

Conservative assumptions for this portion of the study mainly relate to magnitude and source orientation, and the method of analysis itself (deterministic using the MCE). MCE events are the largest possible earthquake that could reasonably be associated with a seismogenic structure under the presently known or presumed tectonic framework with no consideration given to the probability that such an event will occur. The MCE is also conservatively assumed to occur at the closest point of the source zone to the project site. Theoretically, no ground motion should occur which exceeds that of the MCE.

The value used for shear wave velocity in the upper 30 m of the foundation was based on a regional average used in the NBCC. Considering the shallow depth to bedrock at the site, the actual value may well be higher, which would lead to a lower anticipated PGA. However, raising it to 1100 m/s (i.e., soft rock), only reduces the PGA by 0.01g to 0.03g from the PGA values reported (Table 2).

3.1.6 Deterministic Peak Ground Accelerations

A primary result of deterministic seismic hazard analysis is an estimate of peak ground acceleration (PGA) that can be expected at the site given the various geological and seismological parameters of the region. Table 2 presents a comparison of PGA estimates for the two sources expected to contribute the greatest potential ground motions within the 150 km radius of the Eagle Gold property. All PGA estimates presented in Table 2 reflect an average value derived from applying equal weights to the 2008 NGA models discussed previously.

Table 2: Deterministic Peak Ground Acceleration Estimates for Dublin Gulch

Seismic Source	PGA (g)	Distance from Site (km)	MCE
Wernecke/Mackenzie Source Zone	0.10	60	7.0
Ogilvie Source Zone	0.27	30	7.0

3.2 Probabilistic Analysis

A consequence of deterministic analyses is that there is no consideration of probability or risk associated with the identified hazards. Ground motions associated with MCE are discrete values, whereas ground motions derived from probabilistic analyses, by definition, have likelihoods of occurrence associated with them. For instance, interpolation of the NRCAN 2005 National Building Code of Canada Seismic Hazard data indicates a ground motion of 0.19g at Dublin Gulch (Table 3) has a 10 percent chance of exceedance in 50 years. This equates to a risk of an earthquake occurring that exceeds 0.19g approximately every 475 years. A probabilistic analysis is included in order to assist Victoria Gold Corp. with quantification of the risks associated with seismic hazards at the Eagle Gold property.

The probabilistic analysis was initially conducted using the NRCAN 2005 National Building Code Seismic Hazard website http://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index-eng.php to determine the peak horizontal ground acceleration (PGA) for various return periods. The website is a tool developed by NRCAN to calculate probabilistic response spectra with different hazard levels for spectral periods up to 2.0 seconds at any location (the equivalent spectral response period for peak ground acceleration is 0.0 seconds). The on-line calculator uses an average shear wave velocity in the upper 30 meters (V_{s30}) that corresponds to NBCC 2005 soil class

C (360-750 m/s). The median (50th percentile) peak ground accelerations for 10%, 5%, and 2% probabilities of exceedance in 50 years were calculated using the on-line calculator. For comparison, Stephen Halchuk of NRCAN (Personal Communication, 2011) provided mean values for the same probabilities. Both are shown in Table 3.

Table 3: Probabilistic Ground Motions for Dublin Gulch
NRCAN 2005 National Building Code of Canada Seismic Hazard Interpolation

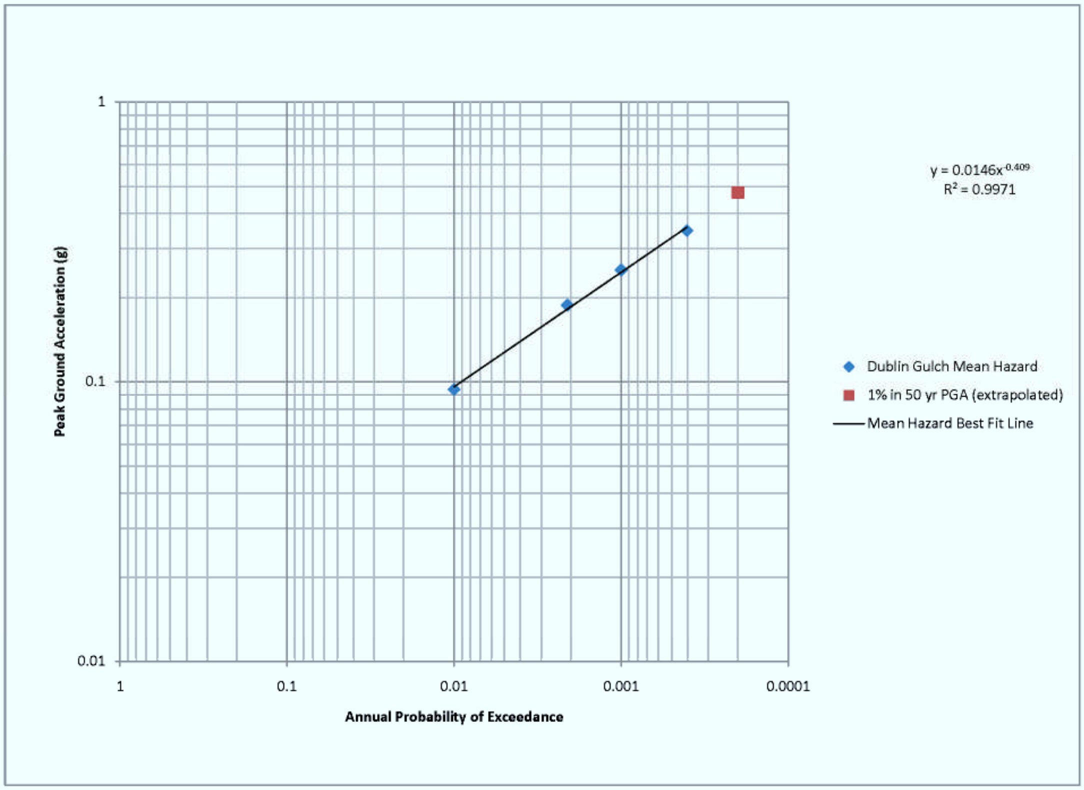
Probability of Exceedance in 50 years (%)	Approximate Equivalent Return Period (yrs)	Median Peak Ground Acceleration (g)	Mean Peak Ground Acceleration (g)
10%	475	0.14	0.19
5%	975	0.18	0.25
2%	2475	0.25	0.35

4.0 Discussion

The design PGA should be chosen based on regulatory requirements and the level of tenable risk to the project. The Yukon Water Board Licensing Guidelines reference use of the MCE, but also defer to the 2007 Canadian Dam Association (CDA) guidelines for water management structures. The CDA requires a mean (rather than median) 1 percent in 50 yrs probability of exceedance (approximate return period of 5,000 yrs) PGA for high hazard water management structures. For the moment, it is unclear which regulatory guidelines will take precedent for the various engineering works at the Eagle Gold mine site. However, a PGA corresponding to a 5,000-year event is not available through NRCAN, as this probability category is beyond the intended purpose of the NBCC models (Personal Communication with Stephen Halchuck, 2011). Extrapolating a value for this low probability ground motion, according to NRCAN recommendations, results in a PGA of 0.48g (Figure 2). This mean value is larger than the MCE ground motions, which implies an event with a larger moment magnitude than the MCE may reasonably be expected to occur. This is contrary to the definition of an MCE. The large difference between the extrapolated 5,000-yr PGA and the MCE PGA can be explained by the fact that the 5,000-yr extrapolation is not constrained by the current tectonic environment. In addition, NRCAN emphasizes that low probability values extrapolated from their models can be given little credence, and should only be used as a screening tool to determine if a site-specific seismic hazard assessment is warranted. The site specific hazard study has shown that there is a very short earthquake history for the area, and, in particular, there have been very few earthquakes large enough to formulate a meaningful site-specific earthquake forecast model, which would be required to perform a site-specific probabilistic analysis. Such an analysis would result in a prediction for a 5,000-year event based on only 26 years of available earthquake history.

Figure 2: Extrapolation of NRCAN Mean Probabilistic PGA Values to a 5,000-year Event

(0.0002 Annual Probability of Exceedance)



Considering the level of conservatism inherent in a deterministic analysis, and the added conservatism discussed in Section 3.1.5, Tetra Tech recommends a design PGA of 0.27g based on an MCE of moment magnitude 7.0.

5.0 Conclusions and Recommendations

This memorandum provides a summary of a seismic hazard analysis performed for Victoria Gold Corporation's Eagle Gold project, located near Dublin Gulch in the Yukon Territory, Canada. Peak ground accelerations were developed for both deterministic analyses and probabilistic analyses. A Microsoft Excel spreadsheet that applies a weighted average to five attenuation relationships developed through the PEER NGA project was used for the deterministic analyses, while data published by NRCAN was used to determine probabilistic site ground motions and their corresponding probabilities of occurrence. These results are summarized in Tables 2 and 3.

Tetra Tech recommends a design PGA of 0.27g for high hazard facilities, as determined using an MCE of moment magnitude 7.0. For facilities requiring a PGA based on a return period of 1,000 years or less, the mean NBCC values from Table 3 may be used.

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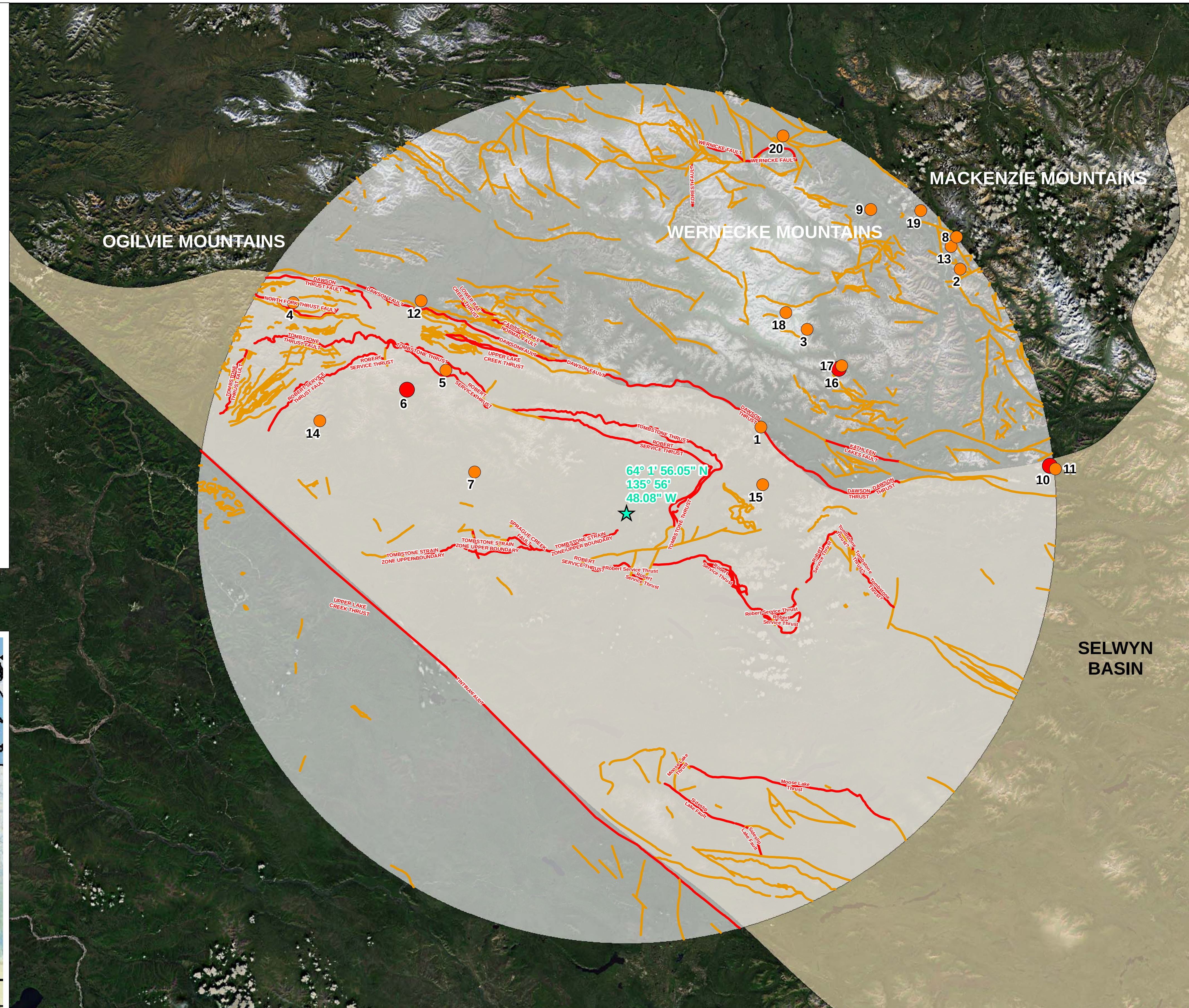
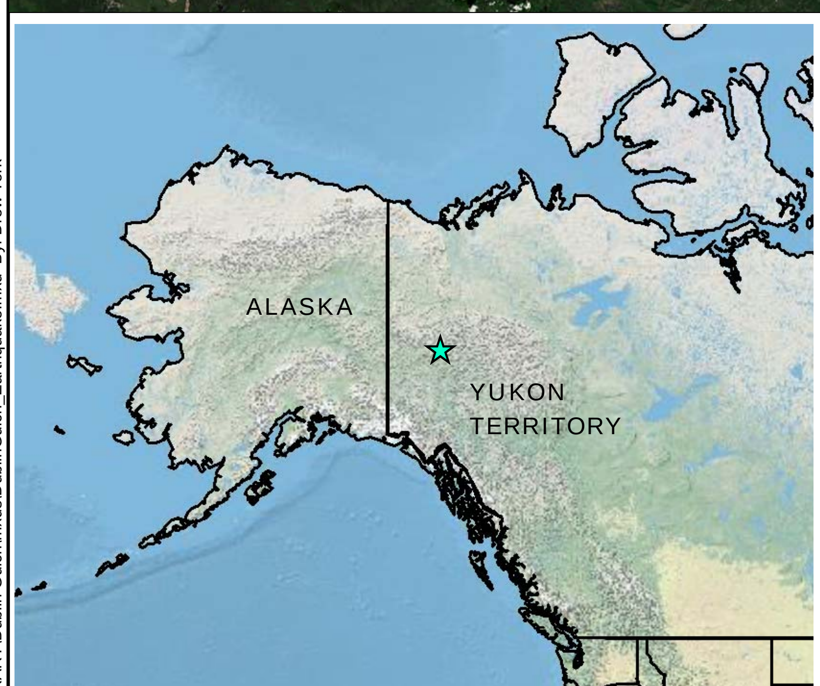
Dublin Gulch Earthquake Catalog				
Point ID	Date	Lat	Long	Mag
1	1985/08/09	64.32	-135.01	4.0ML
2	1987/03/02	64.83	-133.59	4.3ML
3	1991/06/13	64.63	-134.7	4.8ML
4	1991/10/22	64.61	-138.46	4.8MB
5	1993/04/16	64.44	-137.31	4.0ML
6	1996/07/21	64.37	-137.58	5.0ML
7	1996/10/16	64.13	-137.05	4.9ML
8	1996/11/27	64.93	-133.62	4.0ML
9	1997/01/04	65.01	-134.26	4.9mb
10	1997/11/25	64.22	-132.92	5.1ML
11	1997/11/25	64.21	-132.88	4.8ML
12	1998/04/07	64.65	-137.53	4.3ML
13	2000/02/23	64.9	-133.66	4.1ML
14	2001/06/04	64.25	-138.19	4.0Mw
15	2003/12/06	64.14	-134.98	4.0ML
16	2005/09/24	64.51	-134.46	5.1Mw
17	2005/10/21	64.52	-134.44	4.0Mw
18	2007/06/23	64.68	-134.86	4.3Mw
19	2010/08/07	65.01	-133.89	4.5Mw
20	2010/12/12	65.23	-134.93	4.0Mw





LEGEND

FAULTS WITHIN 150KM RADIUS[1]

NAMED FAULT

UNNAMED FAULT

150 KM BUFFER

AREA OF INTEREST

SELWYN BASIN

EARTHQUAKE MAGNITUDE[2]

5.0–5.9

4.0–4.9

Source: (1) Yukon Geological Survey – http://www.geology.gov.yk.ca/databases_gis.html
(2) Earthquakes Canada Online Bulletin – http://earthquakescanada.nrcan.gc.ca/stnsdata/

N

SCALE IN KILOMETERS
0 25 50

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TETRA TECH
350 Indiana Street, Suite 500
Golden, Colorado 80401
(303) 217-5700 (303) 217-5705 fax

DUBLIN GULCH – FAULTS IN 150 KM RADIUS & EARTHQUAKE DATA

Project: EAGLE GOLD	Project no.: 114-201045X	
Location: YUKON TERRITORY	Date: APRIL 2011	FIGURE 1

REVISION A

APRIL 2011 T:\GIS\TEMPORARY\Dublin Gulch\mxds\DublinGulch_Earthquake.mxd By: Drew York

120 West Park Drive. Suite 204
Grand Junction, Colorado 81505
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Technical Memorandum

To:	File	From:	Troy Meyer, P.E., P Eng
Company:	Tetra Tech	Date:	24 October 2011
Re:	Eagle Gold HLF – Embankment Filter Design	Project #:	114-201045X
CC:			

Introduction

This memo presents the detailed design for the filter zone that is located between the subliner material and the embankment structural fill.

The Eagle Gold Heap Leach Facility (HLF) has been designed as a fully lined pad with internal solution pond. The feasibility-level design of the HLF embankment consists of the following elements, from the top down:

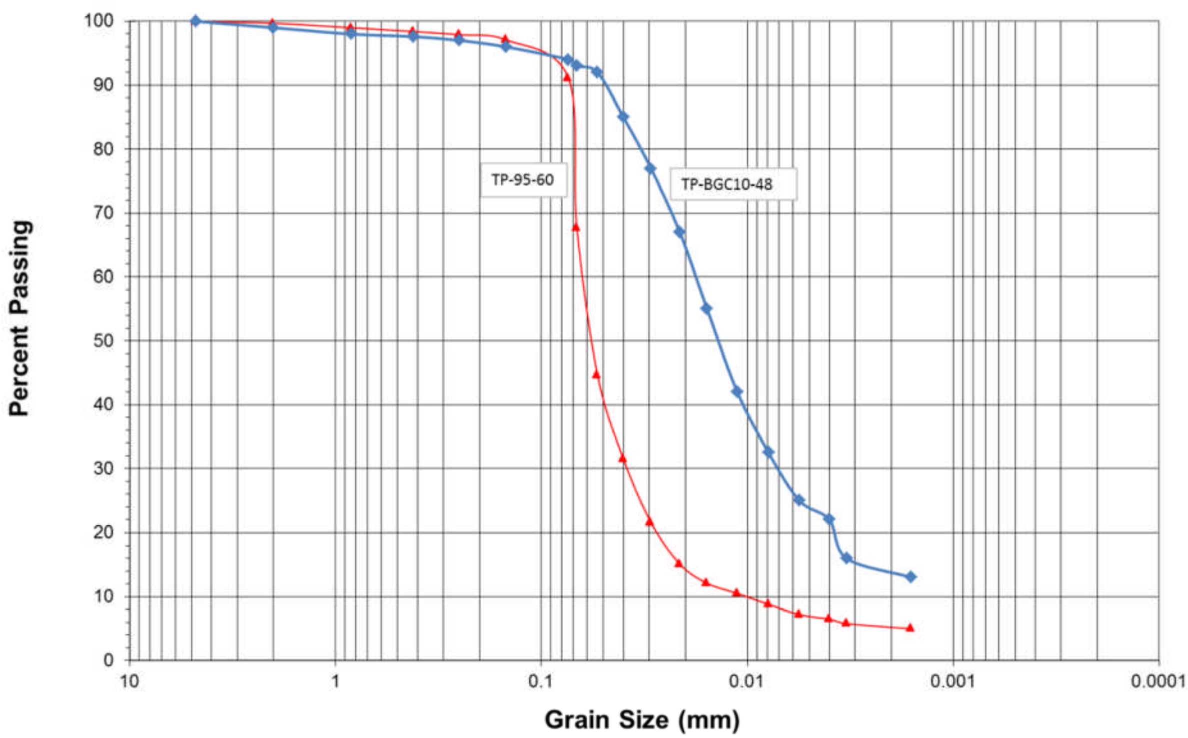
1. A zone of crushed and agglomerated ore,
2. Over Drain Fill (ODF) consisting of a 1000 mm thickness of permeable granular material (minus 38 mm crushed durable gravel) and perforated pipes.
3. A linear low density polyethylene (LLDPE) upper geomembrane liner.
4. A high load drainage net
5. A linear low density polyethylene (LLDPE) upper geomembrane liner.
6. A low permeability compacted liner bedding fill above the prepared foundation subgrade. The liner bedding material must be filter compatible with the filter zone, preventing migration of the solids (piping) in the unlikely event of hydraulic head transferring through an opening in the liner system.
7. A filter zone.
8. Embankment structural fill.

Filter Design

The filter design and resulting filter zone material specification is based on U.S. Dept. of Agriculture National Resource Conservation Service (NRCS 1994) filter selection criteria and the grain size distribution of the tailings material from laboratory test work conducted by BGC (2010) and Golder Associates (1995). The following steps were performed:

Step 1 – Plot grain size curve of base soil (see Figure 1). These gradations represent the range of expected materials from the identified borrow sources.

Figure 1 - Base Soil Gradation Curves



Step 2 – Proceed to Step 4 if the base soil contains no gravel (+4.76 mm)

Step 3 – (this step is skipped since tailings contains no gravel)

Step 4 – Determine base soil category based on fines content (percent finer than 0.074 mm).

Base soil contains 91 – 94% fines.

Soil category 1 – Fine Silt and Clays

Step 5 – Determine maximum D_{15} of the filter for filtration requirements.

Base soil category 1: $D_{15,max} \leq 9 \times D_{85}$ but not less than 0.2mm

$D_{15,max} \leq 0.495$ mm

Step 6 and 7 – Determine minimum D_{15} of the filter for permeability requirements.

Step 6 does not apply. The minimum D_{15} is not a requirement for the HLF embankment since the seepage from a potential leak through the double liner system is expected to be is very small compared to the capacity of the drain Also, since $D_{15,max}/D_{15,min} < 5$, adjustment of filter band width (Step 7) does not apply. Based on the gradational data, the following was chosen:

$D_{15,min} \geq 0.1$ mm

Step 8 – Coefficient of Uniformity (C_u) to prevent gap grading.

Coefficient of uniformity (C_u) of less than 6 will be specified for the drain material to ensure gap-graded materials and not used. For the purposes of providing a specification, the following was chosen:

$D_{60,min} \geq 0.5$ mm

Step 9 – Determine minimum D_5 & maximum D_{100} of the Filter Material from NRCS Table 26-5.

The D_5 value is not a factor since the tailings permeability is 10^{-7} cm/s and overdrain permeability is 10^{-3} cm/s. The D_{100} was chosen from the table as follows:

$D_{50,min} = .075$ mm

$D_{100,max} \leq 3$ inches (75 mm)

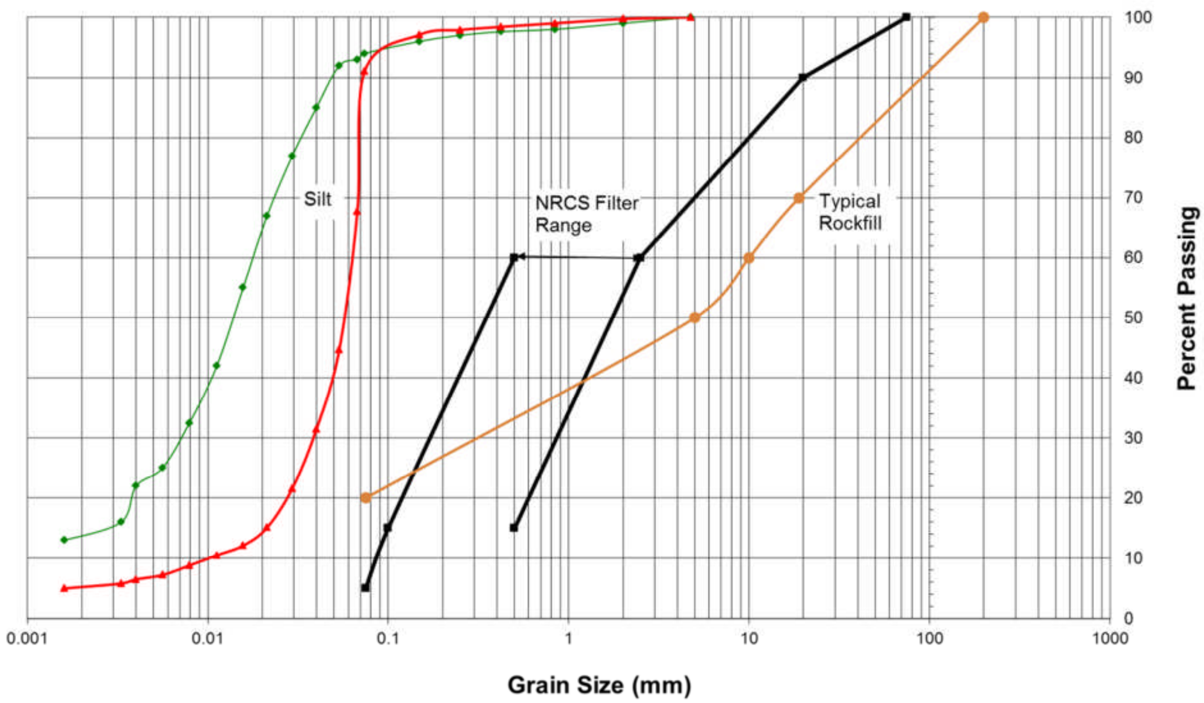
Step 10 – Determine maximum D_{90} for the Filter Material based on $D_{10,min}$ and NRCS Table 26-6

$\Rightarrow$ from Table 26-6 $D_{90,max} = 20$ mm

Step 11 – Connect the points to form the design for the filter band. Extrapolate to complete the curves.

Figure 2 presents the filter design based on NRCS criteria with gradation curves for representative liner bedding material.

Figure 2 - Filter Design Results



Based on the filter design, the suggested gradational specifications for the filter are shown in Table 1.

Table 1 – Specification for Filter Material

US Sieve Size	**% Passing by Weight**
3 inch (75 mm)	100
No. 4	70-100
No. 40	15-60
No. 200	0-5

Note: Coefficient of uniformity (C_u) shall be less than 6.

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Technical Memorandum

To:	File	**From:**	Troy Meyer, P.E., P Eng
Company:	Tetra Tech	**Date:**	10 February 2012
Re:	Eagle Gold HLF – Drain Pipe Design	**Project #:**	114-201045X
CC:			

Introduction

This section presents the heap leach drain pipe sizing results for the Eagle Gold Projects' (Project) Heap Leach Pad. The designed heap leach pad will contain a piping network that will be distributed throughout the limits of the facility and will accommodate the volumes from a 100-year, 24-hour storm in addition to 150% of the flow generated from the applied leaching solution. The drain pipes are located within the free-draining Overliner Drain Fill (ODF) material placed above the pad liner and will collect and convey the stormwater and Pregnant Leach Solution (PLS) to the designated pond. Drain pipe sizing calculations are provided in Attachment 1.

The following parameters were used in the Heap Leach Pad drain pipe sizing calculations. These parameters were based on the design criteria provided by Wardrop (2011) and on standard engineering practice.

- The 100-year, 24-hour storm event precipitation is 103.5 mm.
- The leaching solution application flow rate is 2770 m^3/hr.
- Total flow capacity to be accounted for in the pads' drainage pipe system was comprised of the 100-year, 24-hour storm event plus 1.5 times the design leaching application flow rate (4155 m^3/hr). Collector pipe spacing is based on the unit leaching rate (10 l/hr/m^2)
- Manning's "n" of 0.014 was selected for the perforated ADS N12 pipes to account for the roughness of the manufactured perforations and for wear over time.
- Flow conditions assumed pipes were 85% full for maximum conveyance under gravity.
- Drainage sections were divided and grouped in terms of similar average slopes over the length of the entire section area.
- The ODF permeability is equivalent to 2 x 10^{-2} cm/sec or greater under a 91 m maximum ore heap load.
- The drain system design conservatively assumed no moisture absorption or retention losses in the ore material.

The calculations assumed that the ODF has been placed over the entire pad footprint and that the 103.5 mm of precipitation is evenly distributed throughout the 24-hours of the storms duration; therefore resulting in no peak flow occurrences. Pipe capacities were calculated using Manning's equation shown below (Chow, 1959).

$$\mathbf{Q = (1/n) \times A \times (R)^{2/3} \times (S)^{1/2}}$$ where:

Q = discharge (cms)

n = Manning's roughness coefficient

A = cross sectional area of the pipe (m^2)

$\mathbf{(R)^{2/3}}$ = hydraulic radius at 85% capacity (m)

$\mathbf{(S)^{1/2}}$ = pipe slope (m/m)

Note: Pipe design slopes were obtained from the design contours within the Heap Leach Pad limits.

In order to maximize the efficiency of the ore's drainage and to minimize the potential for leakage through the pads' liner system, the hydraulic head above the liner was designed to be less than a maximum height of 1.0 m. (within the ODF layer). Drain pipe spacing was determined by the equation below for estimating the peak hydraulic head on the pads' liner system between drain pipes (McWhorter and Sunada, 1977).

$$\mathbf{H = (L/2) \times (W / K)^{0.5}}$$ where:

H = maximum desired hydraulic head on the liner of 2.0 m

L = drain pipe spacing (to be determined)

W = unit application rate of (10 l/hr/m2)

K = hydraulic conductivity (permeability) of pad drain material (2×10^{-2} cm/sec)

Calculating the hydraulic head using the assumed values indicates that a minimum secondary pipe spacing of 17 m on center will limit hydraulic head on the liner to less than 1.0 m.

The ODF shall consist of free-draining granular material with 38-mm maximum particle size and a maximum of 5 percent fines passing the No. 200 ASTM sieve size (0.075-mm). The material shall be free of organic matter and soft, friable particles in quantities objectionable to the Engineer. The minimum in place hydraulic conductivity of the ODF will be 2×10^{-4} m/s. The design criteria specified an ODF permeability and maximum ore heap load to ensure both reasonable spacing of the drain pipes and fully drained heap conditions.

The Heap Leach Pads' drain pipes will consist of 450, 375, 250, and 100 mm diameter corrugated, dual-wall, perforated ADS N-12 pipes. The pipe network to be constructed consists of a series of 100 mm secondary drain pipes, spaced every 50 m on center and arranged in a "herringbone" pattern around the larger pipes that will convey the collected fluid, i.e., PLS and stormwater flows. The larger pipes mentioned consist of 250 mm collector pipes transmitting their flows to the 375 mm primary pipes which ultimately drain their collected and transmitting fluid to the 450 mm header pipes.

For design purposes the Heap Leach Pad was divided into East and West Regions. The dividing line for these two (2) Regions was based on the design ground contours. Each East and West Region was again further subdivided into smaller Sections based on similar slopes from the design contours. Pipes were individually sized by Section to account for the different slopes from each contributing Collector, Primary, and Header area. The East Region consisted of Header, Primary, and Collector Sections. The West region consisted of Header and Primary Sections. The contributing land areas from each of these Regions and Sections are summarized in the table below.

Table 1.1 – Heap Leach Pad Land Area Breakdown

Heap Leach Region	Total Region Area (m^2)	Heap Leach Section	Total Section Area (m^2)
East	525,874	Header	146,524
		Primary	38,423
		Collector	162,270
West	717,253	Header	67,080
		Primary	599,838

The total combined (100-year, 24-hour storm plus 150% of the applied leaching solution) flows for each of these section areas are summarized in the table below.

Table 1.2 – Summary of Total Combined Flows

Heap Leach Region	Header Section Total Flow (m^3/hr)	Primary Section Total Flow (m^3/hr)	Collector Section Total Flow (m^3/hr)
East	5,652	5,020	4,855
West	7,031	6,742	N/A

Note: For reference, see Attachment 1.

The results summarized in Table 1.3 below are the minimum quantities of pipes needed to satisfy the stated design criteria for each Region of the Heap Leach Pad and assumes that 100 mm secondary drainage pipes will be arranged in a herringbone pattern and spaced 50 m on center with one another.

Table 1.3 – Summary of Minimum Pipe Quantities

Heap Leach Region	Quantity of 450mm Header Pipes	Quantity of 375mm Primary Pipes	Quantity of 250mm Collector Pipes
East	2	3	5
West	2	3	N/A

Table 1.4 – Summary of Total Average Pipe Lengths Required

Heap Leach Construction Phase	Average Required Length (m) of 450mm Header Pipe	Average Required Length (m) of 375mm Primary Pipe	Average Required Length (m) of 250mm Collector Pipe	Average Required Length (m) of 100mm Secondary Pipe
Phase 1	1,652	1,559	N/A	8,515
Phase 2	N/A	279	1,117	8,152
Phase 3	N/A	814	2,174	6,624

References

Chow, V.T. *Open-Channel Hydraulics*, McGraw Hill, New York, 1959.

McWhorter, David B. & Sunada, Daniel K. *Ground-water Hydrology and Hydraulics*. Water Resources Publication, 1977.

TETRA TECH

Client:	Eagle Gold Feasibility Study	Job No.:	114-201045X
Subject:	Heap Leach Facility	By:	TLM
Details:	Pipe Sizing Calculations	Date:	February 10, 2012

Avg. Solution Outflow Rate (m^3/hr):	2770.00	(From Design Criteria, Wardrop 2012)
100-Year 24-Hour Precipitation (mm):	103.50	(From Design Criteria, Wardrop 2012)

Header	Area (m^2)
East:	146524.00
West:	67080.00
Total Header Area:	**213604.00**

Header Pipe East Slope (m/m)	Header Pipe West Slope (m/m)
0.13	0.13

Location	Area (m^2)
East:	525,874
West:	717,253
Total Area (m^2):	**1243126.32**

Header Pipes - N 12 Perforated Pipe

150% PLS Solution + 100_Year Storm

Area	**PLS Flow (m^3/hr)**	**Storm Flow (m^3/hr)**	**Combined Storm Flow (m^3/hr)**	**Total Flow (m^3/hr)**	**Total Flow (m^3/s)**	**No. Pipes**	**Pipe Diameter (mm)**
East	4,155	631.9	1497.4	5652.4	1.6	2	450
West	4,155	289.3	2876.1	7031.1	2.0	2	450

Primary	Area (m^2)
East:	38423.00
West:	599838.00
Total Primary Area:	**638261.00**

Primary Pipe East Slope (m/m)	Primary Pipe West Slope (m/m)
0.13	0.16

Primary Pipes - N 12 Perforated Pipe

150% PLS Solution + 100_Year Storm

Area	**PLS Flow (m^3/hr)**	**Storm Flow (m^3/hr)**	**Combined Storm Flow (m^3/hr)**	**Total Flow (m^3/hr)**	**Total Flow (m^3/s)**	**No. Pipes**	**Pipe Diameter (mm)**
East	4,155	165.7	865.5	5020.5	1.4	3	375
West	4,155	2586.8	2586.8	6741.8	1.9	3	375

Collector	Area (m^2)
East:	162270.00
Total Collector Area:	**162270.00**

Collector Pipe East Slope (m/m)
0.29

Collector Pipes - N 12 Perforated Pipe

150% PLS Solution + 100_Year Storm

Area	**PLS Flow (m^3/hr)**	**Storm Flow (m^3/hr)**	**Total Flow (m^3/hr)**	**Total Flow (m^3/s)**	**No. Pipes**	**Pipe Diameter (mm)**
East	4,155	699.8	4854.8	1.3	5	250

TETRA TECH

CLIENT:	Eagle Gold Feasibility Study		
PROJECT:	Heap Leach Facility	JOB NO:	114-320905X
SUBJECT:	Pipe Spacing Calculations	BY:	TLM
DETAILS	Pipe Spacing - McWhorter & Sunada, 1977	Date:	February 10, 2012

Pipe Spacing

Units:	Metric	
Application Rate:	0.24	m^3/day/m^2
Maximum Desired Head on the Liner:	1.0	m
Hydraulic Conductivity:	17.28	m/day
Pipe Spacing:	17.0	m
Average Hydraulic Head on Liner:	0.4	m

Application Rate Converter

Application Rate:	0.0041	gpm/ft^2
	10.00	$l/hr/m^2$
	0.0100	$m^3/hr/m^2$
	0.24	$m^3/day/m^2$
	8.47	$ft^3/day/ft^2$

Hydraulic Conductivity Converter

Hydraulic Conductivity	0.0200	cm/s
	56.69	ft/day
	17.28	m/day

$$H = \frac{L}{2} * \left(\frac{W}{K} \right)^{0.5}$$

Where:

H = Maximum Hydraulic Head on Liner at Midpoint between Pipes
L = Drain Pipe Spacing
W = Application Rate
K = Hydraulic Conductivity

1) Permeability in drain layer of 2 x 10^{-2} cm/s for a fully loaded scenario

Results		
Required Pipe Spacing:	17.0	meters
Design Pipe Spacing:	17.0	meters

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Technical Memorandum

To:	File	**From:**	Kanyembo Katapa
Company:		**Date:**	August 30, 2011
Re:	Heap Leach Facility Settlement Assessment	**Doc #:**	
CC:	Marvin Silva, Joel Carrasco		

1.0 Introduction

This Technical Memorandum provides a summary of Tetra Tech's one dimensional settlement assessment of the Heap Leach Facility (HLF) at the proposed Eagle Gold Mine Project (Project).

The assessment of the foundation conditions has been carried out as part of the preliminary design of the HLF. The construction of the HLF would apply loads to the foundation soils which would result in total and differential settlements. These settlements will impact the performance of the proposed liner system and collection pipe network at the base of the HLF pad. In addition, the settlements would impact the stability of the confining embankment and the performance of other facilities directly associated with the embankment such as the conveyor belt and the in-heap pond.

This assessment has been conducted using data collected from previous geotechnical site investigations and site information obtained by others. Tetra Tech did not conduct any additional geotechnical investigations prior to completing this settlement assessment.

2.0 Site Conditions

2.1 Current Site Conditions and Proposed Development

The planned development at the Eagle Gold mine calls for construction of a heap leach facility in the Ann Gulch area (see Figure 1). The primary structures of the HLF include the heap leach pad and the confining embankment. The heap leach pad covers an area of approximately 128 hectares. The impoundment area is sloping down at a slope of approximately 18%. A confining embankment at the down-slope toe of the leach pad will be constructed to provide lateral containment of the pregnant leach solution and form the in-heap pond.

The change in elevation from the toe of the confining embankment to the top of the heap pad is approximately 380 m, with a maximum thickness at any location of approximately 150 m. The maximum changes in pressure applied to the foundation soil and embankment are subsequently expected to be in the order of 2,700 kPa.

It is understood that current site conditions consist of a thin cover of organic soil and vegetation underlain by colluvium and variably weathered bedrock. Some areas of the site have been disturbed from previous mining activities. Alluvial sediments, glacial till or 'placer' tailings (fill) are present in or near the valley bottom close to Dublin Gulch. Ground conditions throughout the project site are highly variable.

It is recommended that all surficial organic soil must be removed prior to construction. Therefore, the presence of the organic layer, although noted on borehole logs, is neglected in this settlement assessment.

2.2 Existing Subsurface Conditions

Subsurface conditions at the HLF site were inferred from geotechnical investigations previously carried out by others across the proposed location of the HLF (BGC 2009 and 2010). Borehole and test pit data from these investigations were provided to Tetra Tech. The previous geotechnical investigations consist of 15 boreholes and 31 test pits. Groundwater monitoring wells were installed in four of the 15 boreholes.

Based on the previous site investigation results, the subsurface soil stratigraphy across the HLF generally consists of the following:

- Organic Layer
- Placer Tailings (Fill)
- Colluvium
- Highly Weathered to Moderately Weathered Bedrock
- Slightly Weathered Bedrock

The previous site investigation reports characterized these subsurface layers as follows:

Organic cover consists of vegetative rootmat, moss, silt and sand, and organic matter. The thickness was typically in the order of 0.2 m and was not present in previously disturbed areas.

The fill materials were encountered in the south areas of the HLF site in the lower elevations. The fill consists of placer tailings material from previous mining activities. The bulk of the fill is loose to compact sand and gravel with variable amounts of silt, cobbles and boulders. The fill is also characterized with considerable variability in material gradation and compactness. Average SPT N-values for tests performed on placer tailings ranged from 4 to 32.

Colluvium material was generally encountered on sloping ground and near the ground surface below the organic cover and was not present in previously disturbed areas. The colluvium consists of loose to compact angular gravel with occasional cobbles in a silt and sand matrix, derived from transported weathered metasedimentary bedrock. The colluvium may also include variable amounts of organics, which are often observed in distinct layers within the colluvium.

A horizon of variably weathered rock was observed across the entire site. The bedrock encountered at the HLF site is generally classified as metamorphosed sedimentary rock, with a variably deep weathering profile. Bedrock at the mine site has been subdivided into three broad categories – Type 1, Type 2 and Type 3 based on the rockmass quality and inferred engineering behavior. A summary of the typical characteristics on the three rock types are summarized in the Table 1.

Table 1 Bedrock Characteristics

Parameter	Type 1	Type 2	Type 3
Unconfined Compressive Strength	>25 MPa	5 to 25 MPa	1 to 5 MPa
Geological Strength Index	>20	15 to 25	15 to 25
Weathering Grade	Slightly to Moderately Weathered	Moderately to Highly Weathered	Highly Weathered

Groundwater

Seven groundwater wells have been installed in the general area of the HLF. Data from these wells show an average groundwater depth of approximately 12 m in the upland and 2.5 m in the valley bottom.

Permafrost

Frozen ground was encountered in the upper part of the HLF footprint. Frozen ground was typically encountered within colluvial gravels, and gravels and sands with depths varying between 0.6 m to 2.8 m, and occasionally included excess ice. Frozen ground was not encountered in the valley bottom nor on the southern edge of the proposed heap leach pad, but localized pockets of frozen ground may be present in these areas. One thermistor string with multiple temperature-measuring beads was installed in the HLF area to the depth of 10 m. Temperature data measured in August, September and October of 2009 do not show freezing temperatures within the 10 m zone, indicating an absence of permafrost at this location. Permafrost has been confirmed using thermistor data in other locations of the mine site. The thermistor data show the permafrost to be warm, at close to 0° C.

Existing Laboratory Tests Data

Laboratory testing conducted on select samples included: moisture content tests, Atterberg limits, grain size analysis (sieves and hydrometers), specific gravity, and modified proctor. Not all samples tested were from the HLF area. However, in certain cases, the test results are considered to be representative of similar soils within the HLF area.

Moisture content test results were used in this evaluation to estimate the total moisture content of frozen soil.

The laboratory test data are contained in the respective references noted above.

3.0 Settlement

3.1 Thaw Consolidation

Frozen soil was encountered within the colluvium layer at the HLF site. The colluvium is classified as gravel in a silt and sand matrix. Gravels are generally not considered to be highly susceptible to frost heave and thaw consolidation settlement. The potential for thaw consolidation settlement due to the silt and sand matrix was investigated using the oedometer test data for silts and sand from several sites along the Mackenzie River presented by (Andersland & Ladanyi, 2004). Moisture content data from the tests pits and boreholes in which frozen soil was reported was analyzed to determine the representative moisture content of the frozen soil. An estimate of the thaw settlement was determined by comparing the representative moisture content to the plot of moisture content versus thaw settlement, presented by Andersland and Ladanyi (2004), shown in Illustration 1.0..

The average moisture content from boreholes and test pits from which frozen soil was reported was calculated to be 12.2 %. The thaw settlement corresponding to this moisture content is very low (Illustration 1.0). Thaw consolidation at the HLF was considered to be negligible. It is therefore, not expected that thaw consolidation will contribute significantly to the HLF total and differential settlements.

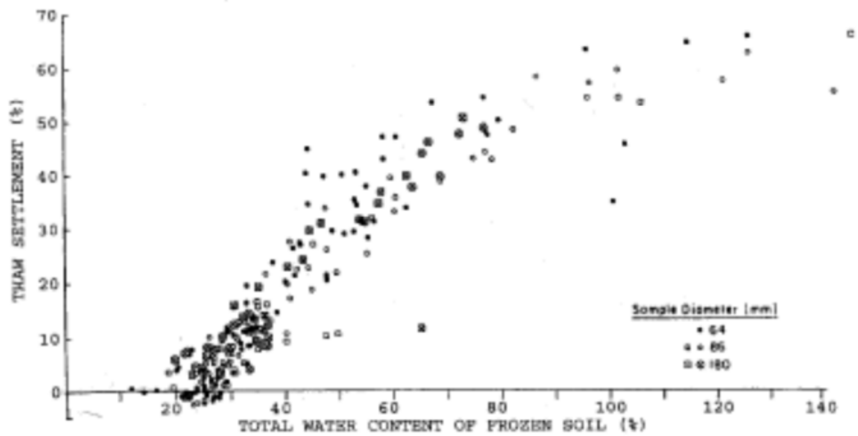


Illustration 1.0 Thaw settlement versus total water content for some Mackenzie Valley soils (From Andersland & Ladanyi 2004)

3.2 Elastic Settlement Prediction Method

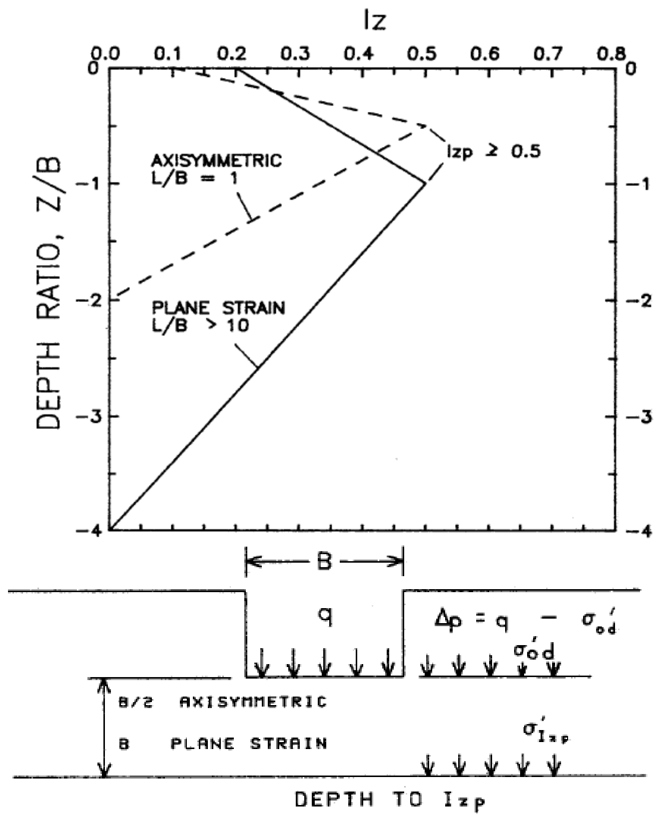
Foundation elastic settlements were calculated using the Schmertmann strain influence method. Originally proposed by Schmertmann (1970) and modified by Schmertmann, Hartmann, and Brown (1978), this method was developed to estimate foundation settlements in cohesionless soils. This procedure provides settlement compatible with field measurements in many different areas. The analysis assumes that the distribution of vertical strain is compatible with a linear elastic half space subjected to a uniform pressure. To use this method, the subsurface is broken into sub layers. Each layer has a constant value of strain and soil modulus. The soil parameters used in the settlement calculation were selected based on the results of the geotechnical investigations referenced above. Settlement is calculated by summing the influence of all layers, as calculated by equation (1).

$$\Delta H = C_1 C_2 \Delta p \sum_{i=1}^{n} \left(\frac{I_{zi}}{E_{si}}\right) \Delta z_i \qquad (1)$$

where:

C_1 = embedment correction factor = 1 – 0.5($\sigma'_{od}/\Delta p$) ≥ 0.5
C_2 = creep correction factor = 1 + 0.2 log (10t)
σ'_{od} = overburden pressure at foundation level or depth d, tsf
ΔP = net foundation pressure increase = q – σ'_{od}, tsf
t = lapsed time in years
I_{zi} = influence factor of soil layer i
E_{si} = elastic modulus of soil layer i, 0.2B, tsf
Δz_i = depth increment i, inches

Schmertmann developed the diagram shown in Illustration 2.0 to determine the appropriate strain influence factor, I_z, for each layer within the profile. Two distributions are shown: one for square or circular footings (L/B=1, axisymmetric), and a second for strip footings (L/B>10, plane strain). Both are triangular distributions, and the one for square or circular footings begins at a value of 0.1 at the base of the footing, while the one for strip footings begins at a value of 0.2 at the base of the footing.



Z = DEPTH BELOW FOOTING BOTTOM, FT
B = FOOTING WIDTH, FT
Iz = DEPTH INFLUENCE FACTOR
Izp = PEAK DEPTH INFLUENCE FACTOR

Illustration 2.0 Strain Influence Factors for Schmertmann's Approximation (From Engineer Manual No. 1110-1-1904, USACOE, 30 Sep 90, page 3-8)

The maximum strain factor, I_{zp}, occurs at a depth equal to B/2 for square footings and B for strip footings, and is calculated using equation (2)

$$I_{zp} = 0.5 + 0.1\sqrt{\frac{\Delta P}{\sigma'_{Izp}}} \quad (2)$$

where:

σ'_{Izp} = initial effective stress at the depth of maximum strain influence

Axisymmetric: $\sigma'_{Izp} = 0.5B\gamma' + D\gamma'$
Plane strain: $\sigma'_{Izp} = B\gamma' + D\gamma'$

where:

γ' = effective unit weight, tcf
B = footing width, ft
D = excavated or embedded depth, ft

In this evaluation, the plane strain distribution was used. The strain factors I_z, any given depth within the soil (i.e. not peak strain factor) profile were computed using equations (3) and (4).

For z_f = 0 to B $\quad I_z = 0.2 + \left(\frac{z_f}{B}\right)\left(I_{zp} - 0.2\right) \quad (3)$

For z_f = B to 4B $\quad I_z = 0.333 I_{zp}\left(4 - \frac{z_f}{B}\right) \quad (4)$

where:

z_f = depth from bottom of leach pad to midpoint of layer

I_z = strain influence factor

Values of soil modulus, E_s, for the fill (placer tailings) and the colluvium were established using the following relationships (NAVFAC 1986):

Table 2 Soil Modulus/SPT-N Value Relationship

Soil Type	E_s/N
silts, sands silts, slightly cohesive silt-sand mixtures	4
clean, fine to med, sands & slightly silty sands	7
coarse sands & sands with little gravel	10
sandy gravels and gravel	12

where:

Es = Soil modulus in tons per square foot (tsf)

N = corrected blow count from standard penetration tests (SPT)

For this evaluation, based on the grain size analyses results and the descriptions of the subsurface profile at the HLF site, the fill was classified as coarse sand with little gravel and the colluvium was classified as sandy gravel. This classification of the fill and colluvium resulted in Es/N values of 10 and 12 respectively.

The results of SPTs performed in the fill material show that the blow counts range from 4 to 36. The lowest blow counts recorded were N = 3.5 at a depth of 8 m. Based on the lowest blow count, the N value for the fill was assumed to be 4. SPTs were not performed in the colluvium. Based on the soil classification and the characterization of the colluvium consistency, the N-value for the colluvium was assumed to be equal to 8.

Values of rock modulus were established using the following relationship (Marinos and Hoek, 2001):

$$E_m = \sqrt{\frac{\sigma_{ci}}{100}} \times 1000 \times 10^{\frac{(GSI-10)}{40}}$$

where:

E_m = modulus of deformation in mega pascals (MPa)

σ_{ci} = the uniaxial compressive strength of the intact rock pieces

GSI = Geological Strength Index

A stress increase approximately equal to the height of the leach material multiplied by the unit weight of the ore was applied to the heap leach foundation soils to model the ultimate loading due to the heap. The height of the leach material at each borehole location was obtained from the proposed site plans (Tetra Tech 2011). The borehole locations at which the settlement assessment was performed are shown on Figure 1. The unit weight of the ore is 18 kN/m^3 (BGC, 2011). The estimated settlement at each of the borehole locations within the HLF is presented in Table 3.

Table 3 Settlement Evaluation Results Summary

Area	Borehole ID	Consolidation Settlement (mm)
HLF Impoundment	BH-BGC10-AG3	716
	BH-BGC10-AG5	533
	BH-BGC10-1	82
	BH-BGC10-2	1,357
	BH-BGC10-3	809
	BH-BGC10-4	2,142
	BH-BGC10-5	42
	BH-BGC10-6	673
	DH-BGC09-AG3	123
	MW09-AG2[A]	1,898
HLF Embankment	BH-BGC10-16	67
	BH-BGC10-17	297
	BH-BGC10-23	572
	DH-BGC09-DG-1	766

[A]The boring log for this borehole was not available, only depth to rock was reported. Material above the rock was assumed to be colluvium based on logs from the surrounding boreholes.

3.3 Criteria for Determining Acceptable Settlements

The maximum allowable strain on the liner system is controlled by the strain tolerance of the LLDPE and the GCL components. The allowable yield strain for the proposed 60 mil double-side textured LLDPE is 12 percent and the elongation at break is 250 percent. There is additional concern when a geomembrane is exposed to tension perpendicular to seams. In these cases, a general rule-of-thumb is that the allowable strain on the geomembrane is about half the value of the un-seamed sheet material (Giroud et al. 1995). For this reason, horizontal seams are not allowed on side slopes. Tensile stresses applied to a geomembrane parallel to the seams are generally not a large concern, provided that the seams are good quality, and were installed in accordance with the specifications. For these reasons, strains of up to 12 percent will be considered acceptable for the proposed LLDPE geomembrane.

For GCL materials, the yield strain is not typically included on standard specifications. For these materials, the yield strain is typically controlled by the geotextile layers on the top and bottom of the clay. Geotextiles generally have yield strains in excess of 50 percent. The bentonite component of GCLs also has a high strain tolerance, and can heal cracks (if they occur) over time. If the GCL were to experience such large strains, thinning of the bentonite layer (and a corresponding increase in permeability) would likely be the primary concern. A second concern would be the GCL panel overlap. To avoid separation of panels caused by strain on the liner system, project specifications include required overlaps twice as large as typical manufacturer recommended overlaps.

In summary, the least strain-tolerant component of the liner system is the LLDPE geomembrane. Accordingly, the maximum acceptable strain on the liner system is 12 percent, which is the allowable yield strain of the LLDPE component.

The settlement calculations show that the differential settlement on the foundation liner system caused by the ore material will be most critical between MW09-AG2 and BH-BGC10-1. Settlement at the two boreholes was calculated to be 1,898 mm and 82 mm respectively. The two boreholes are 150 m apart. This differential settlement will produce an increase in the liner system length of 0.011 m, which is equivalent to a strain of 0.007 percent. Therefore, according to the criteria set previously, the liner system will not be damaged by differential settlement induced by the weight of the ore material.

4.0 Summary and Conclusions

Subsurface soil information from previous geotechnical investigations has been used in this settlement assessment in spite of limitations in terms of the amount of data. Subsurface layer moduli were conservatively inferred from the limited SPT-N value data and from empirical correlations.

Based on these conservative estimates of strength parameters, the settlement calculations show that significant settlement may be expected in areas with deep fill (placer tailings) and colluvium. Settlement within the slightly weathered bedrock was considered negligible.

Ground settlements of up to 2,000 mm can be expected in area with large thicknesses of fill and/or colluvium. In areas of shallow bedrock and low imposed loads from the ore, settlements as low as 50 mm can be expected. Settlements at the embankment can be expected to range from 760 mm to 60 mm. The settlement could potentially impact the performance of the embankment, the in-heap pond and the conveyor belt.

The results of this settlement evaluation support the previous subgrade preparation recommendations by BGC, 2011:

1. Surficial organics should be removed from all foundation subgrades.
2. Frozen materials containing excess ice should be removed from all foundation and replaced with structural fill as required to build the grades up to design elevations.
3. Shallow colluvium should be removed to expose Type 3 (or better) rock. Where colluvium is exposed as HLF subgrade, it should be proof-rolled to identify weak zones for removal and replacement with structural or rock fill.
4. All placer tailings should be removed from all foundation subgrades to expose Type 3 (or better) rock. Structural or rock fill can be used to build the grade up to design foundation elevations.

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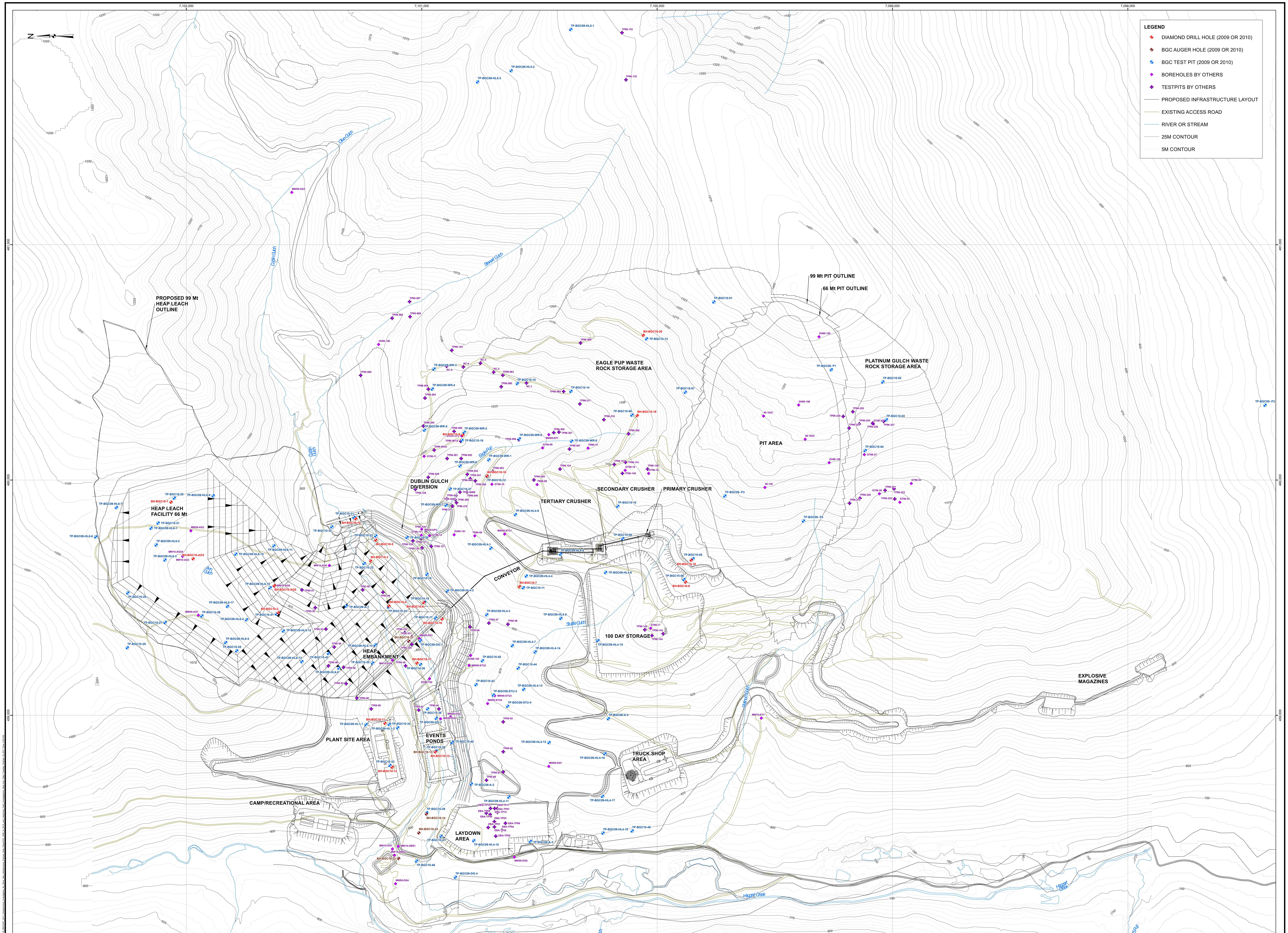
FIGURES



NOTES:
1. PROPOSED INFRASTRUCTURE LAYOUT RECEIVED FROM WARDROP AUG.10, 2011.
2. EXISTING ACCESS ROAD LOCATIONS RECEIVED FROM WARDROP AUG.10, 2011.

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
A	OCT 2011	ISSUED FOR COMMENT	LL	DW	PQ

SCALE: 1:5,000
DATE: OCT 2011
DRAWN: LL
DESIGNED: DW
CHECKED: DW
APPROVED: PQ

PROFESSIONAL SEAL: DRAFT

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT: VICTORIA GOLD CORPORATION

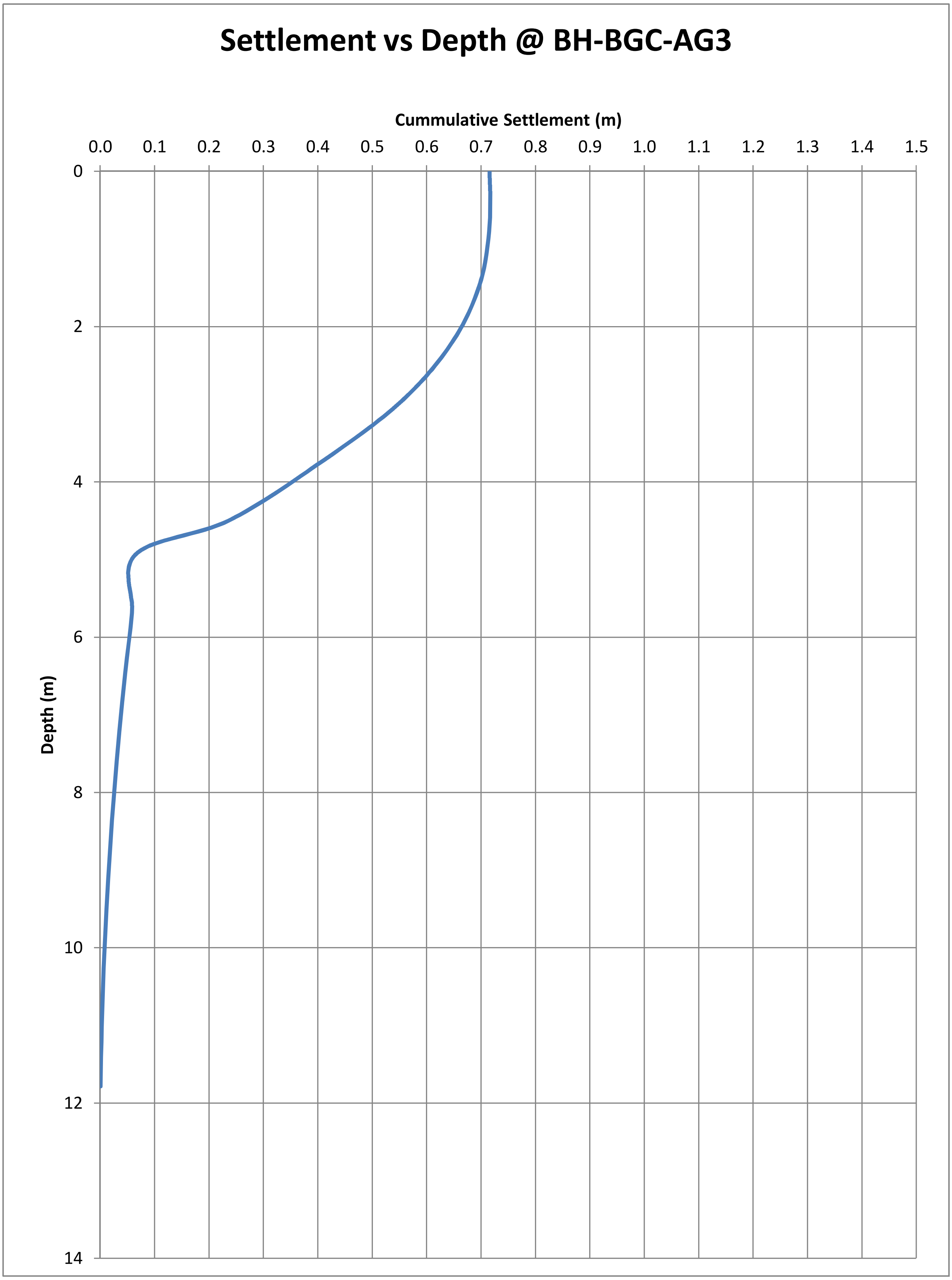
PROJECT: EAGLE GOLD PROJECT 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FACTUAL DATA REPORT

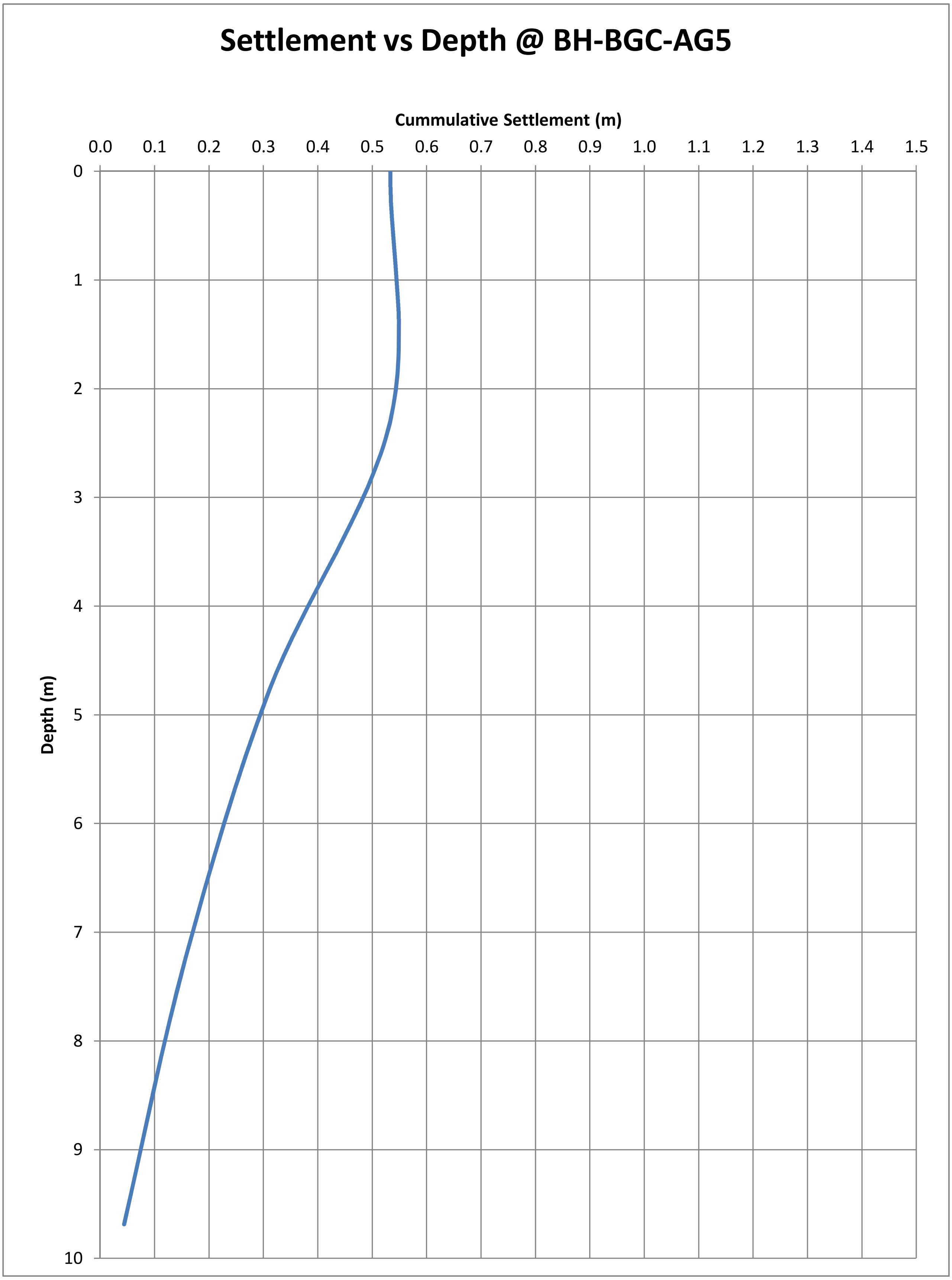
TITLE: HISTORICAL FIELD INVESTIGATIONS (1995 TO 2010)

PROJECT No.: 0792-006
DWG No.: 03
REV.: A

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FACTUAL DATA REPORT" DATED OCTOBER 2011

APPENDIX A
SETTLEMENT VERSUS DEPTH PLOTS


Settlement vs Depth @ BH-BGC-AG3
Cummulative Settlement (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
2
4
6
8
10
12
14


Settlement vs Depth @ BH-BGC-AG5
Cummulative Settlement (m)
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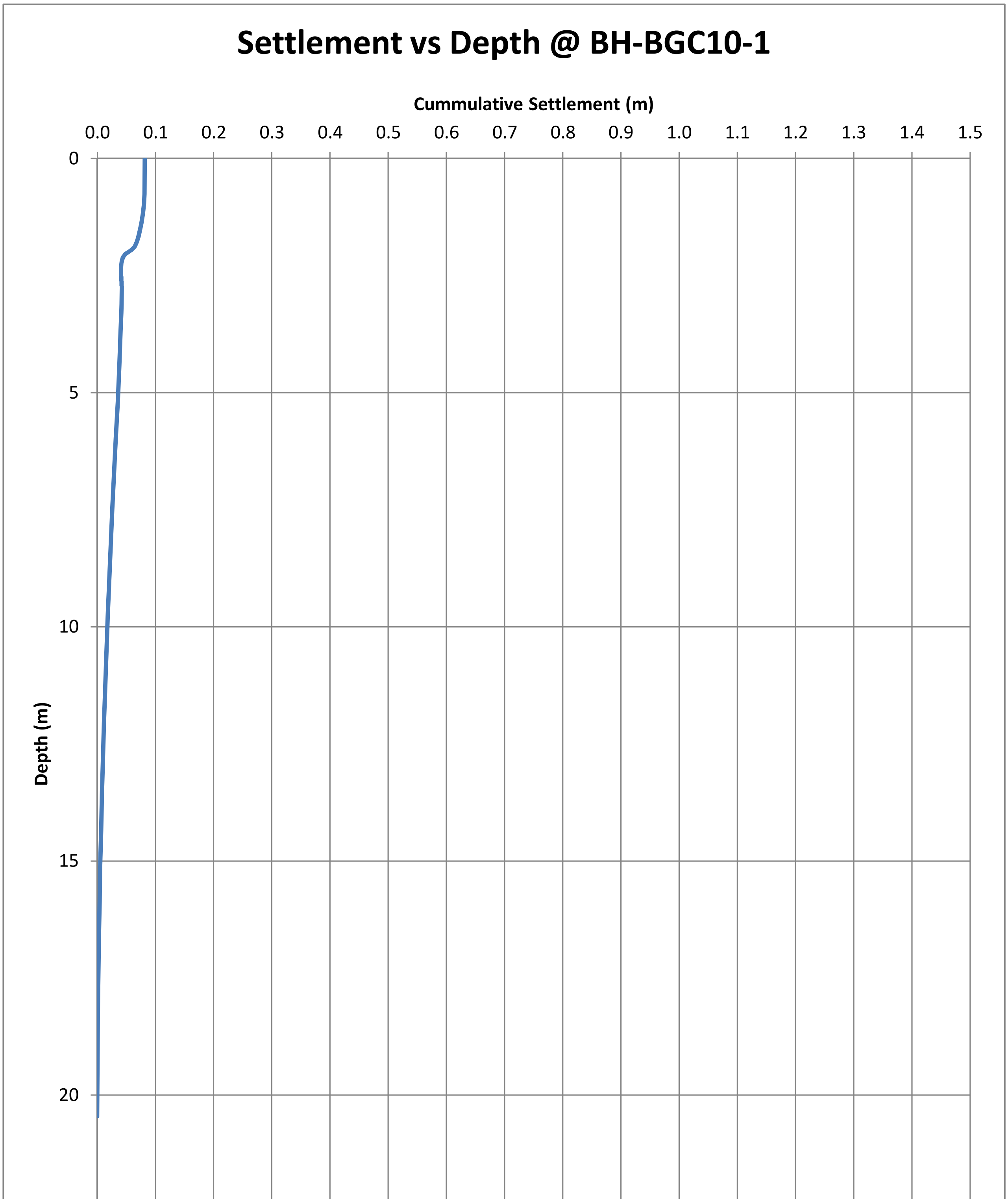
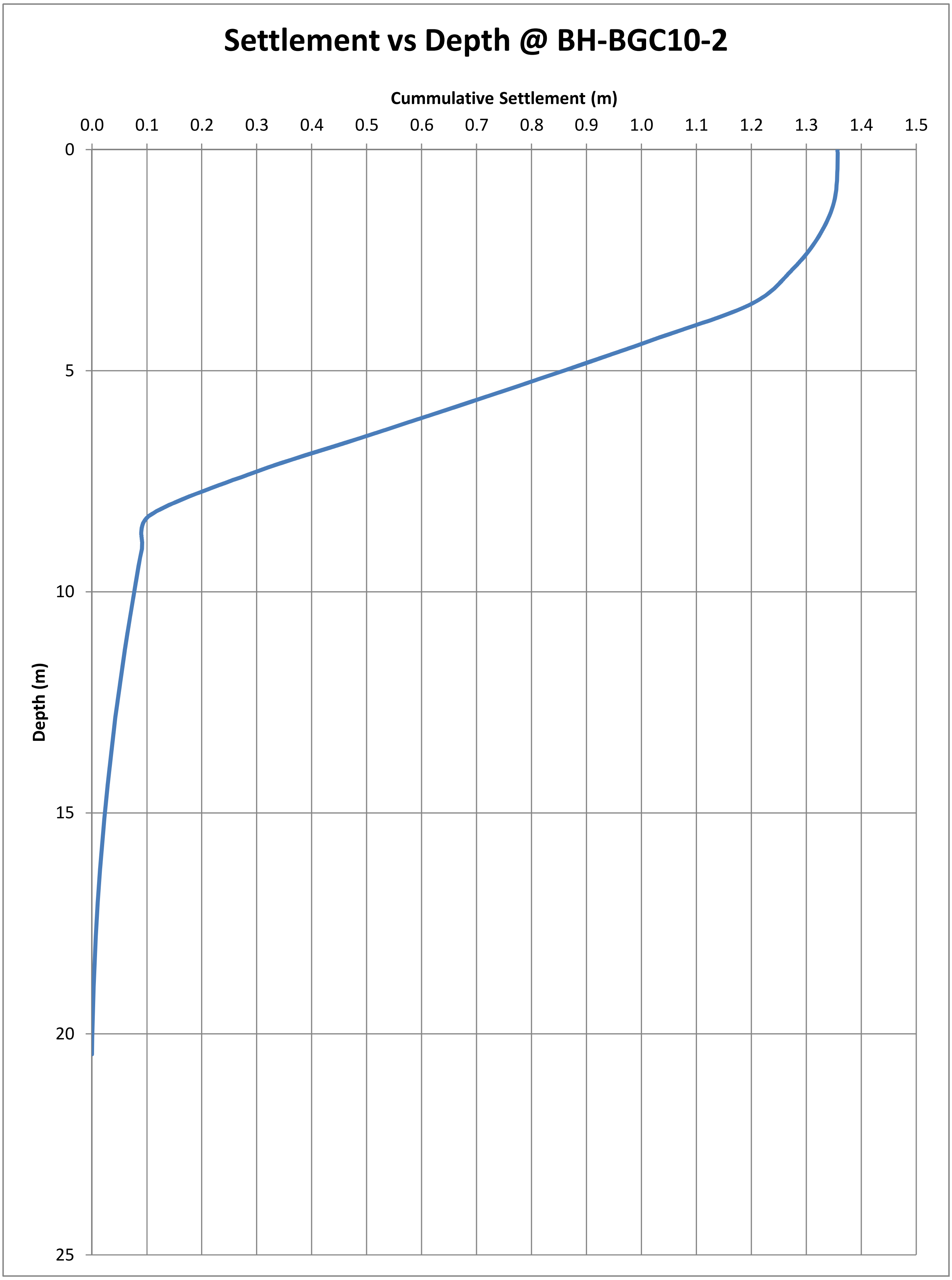
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
1
2
3
4
5
6
7
8
9
10


Settlement vs Depth @ BH-BGC10-1
Cummulative Settlement (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
5
10
15
20
25



Settlement vs Depth @ BH-BGC10-2
Cummulative Settlement (m)
0.0
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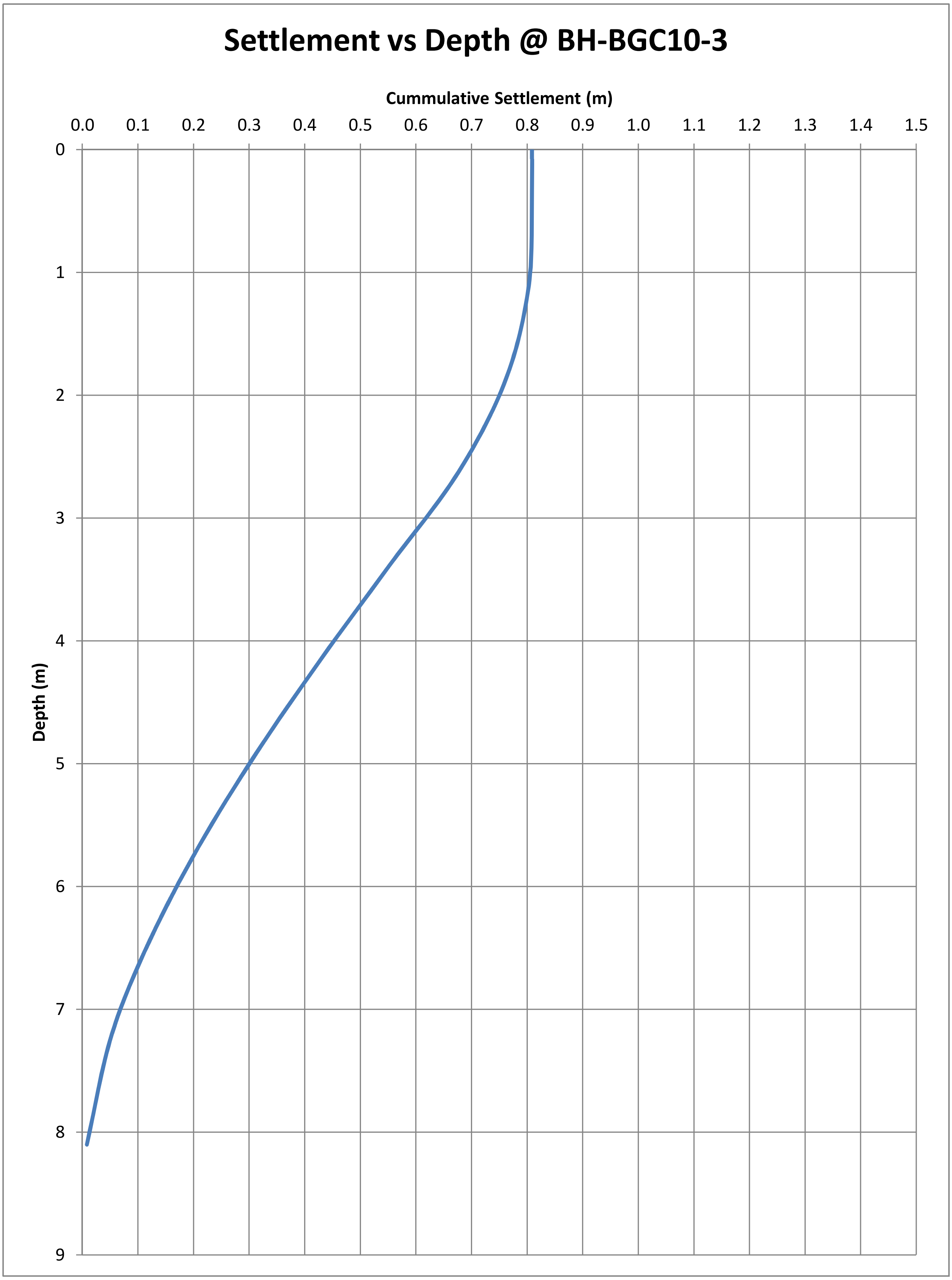
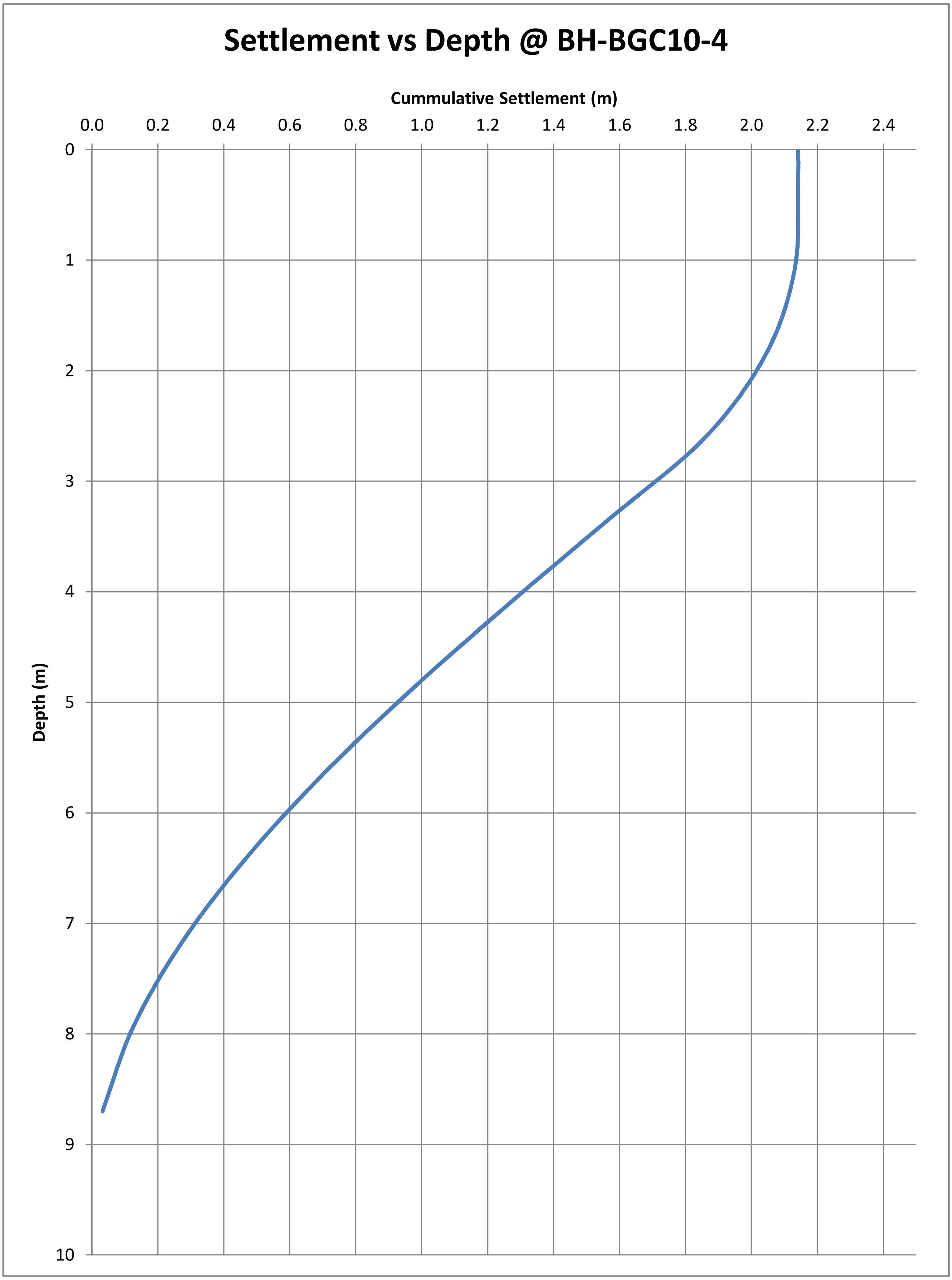
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
5
10
15
20
25


Settlement vs Depth @ BH-BGC10-3
Cummulative Settlement (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
1
2
3
4
5
6
7
8
9


Settlement vs Depth @ BH-BGC10-4
Cummulative Settlement (m)
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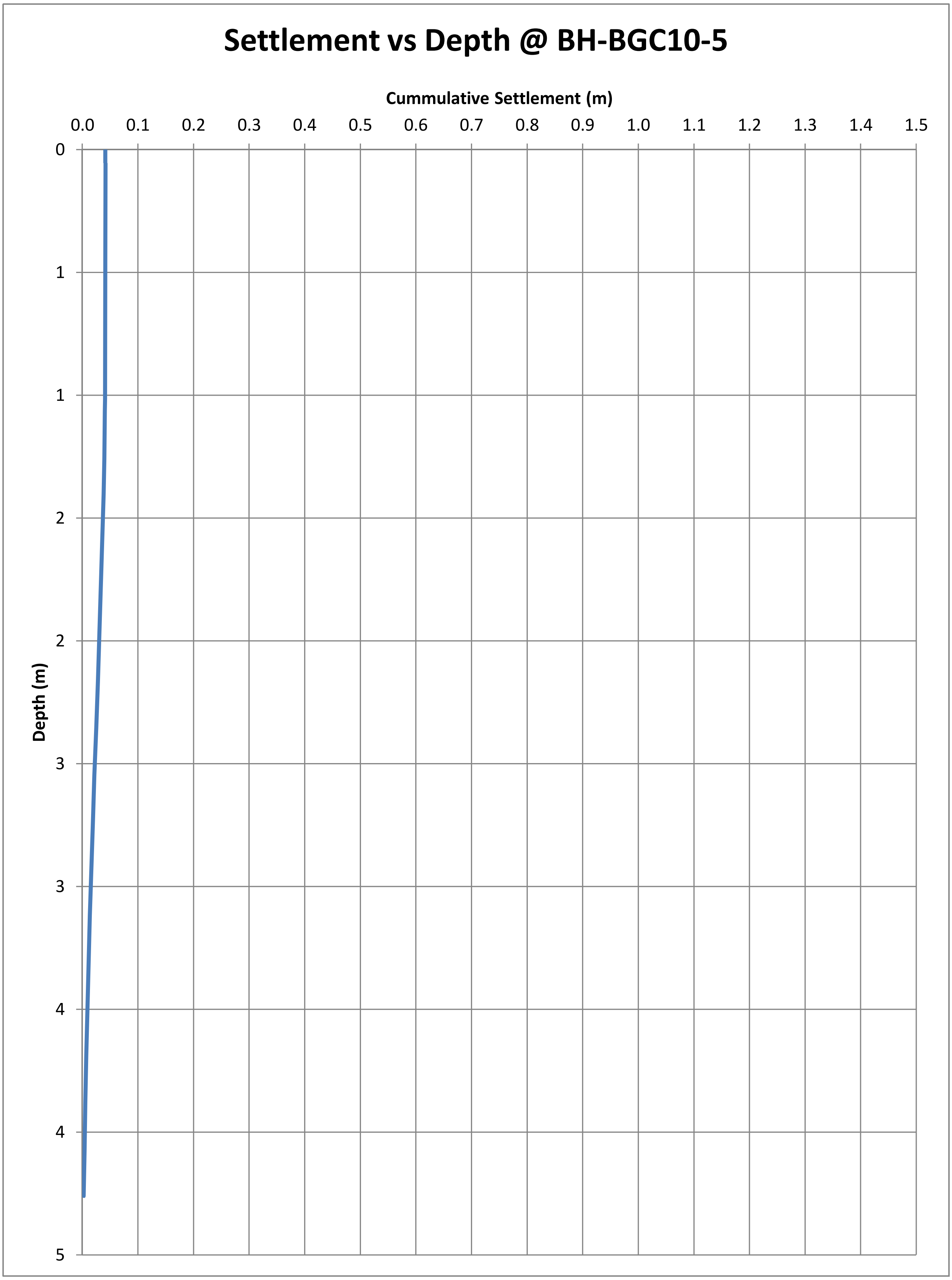
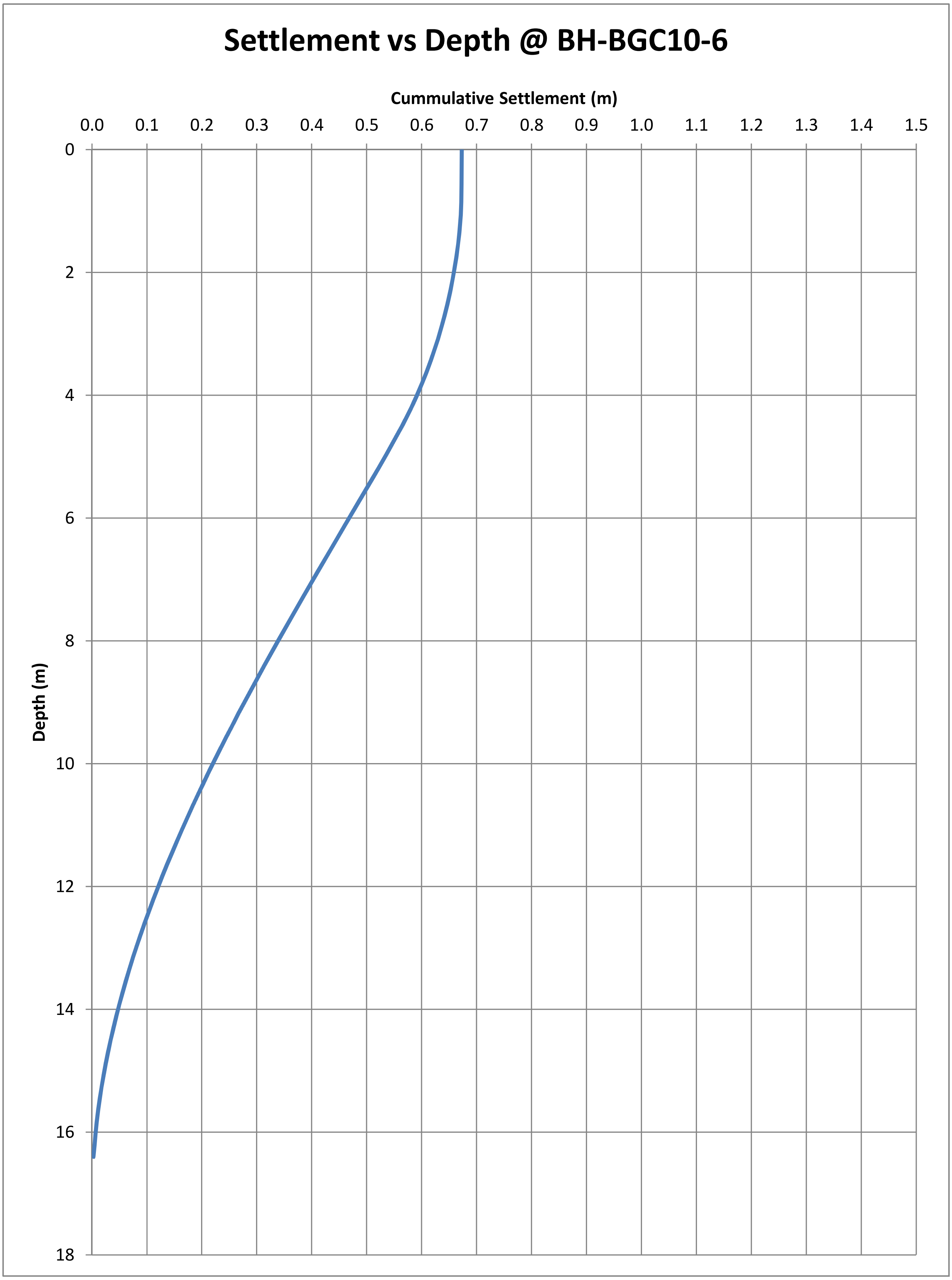
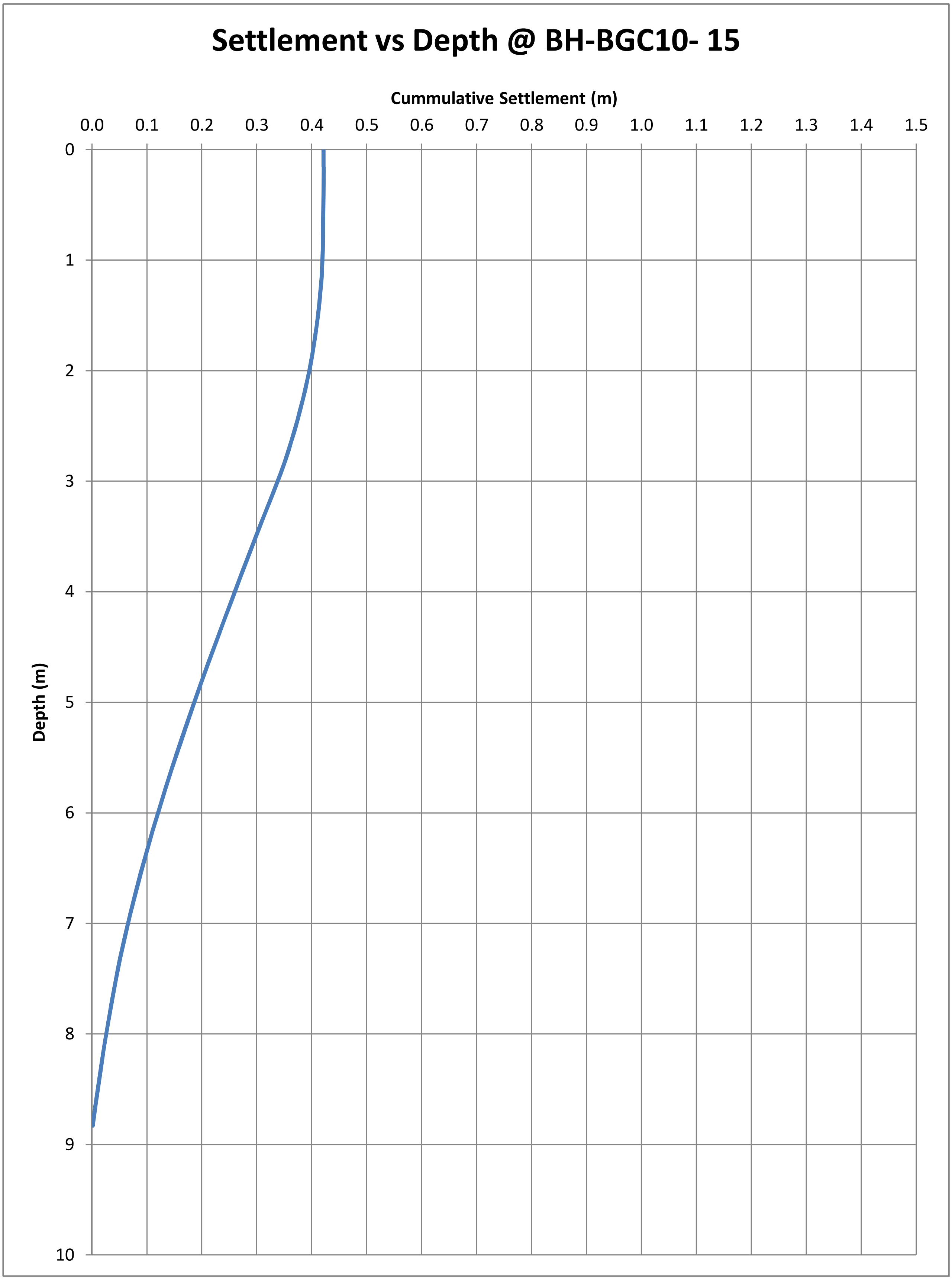
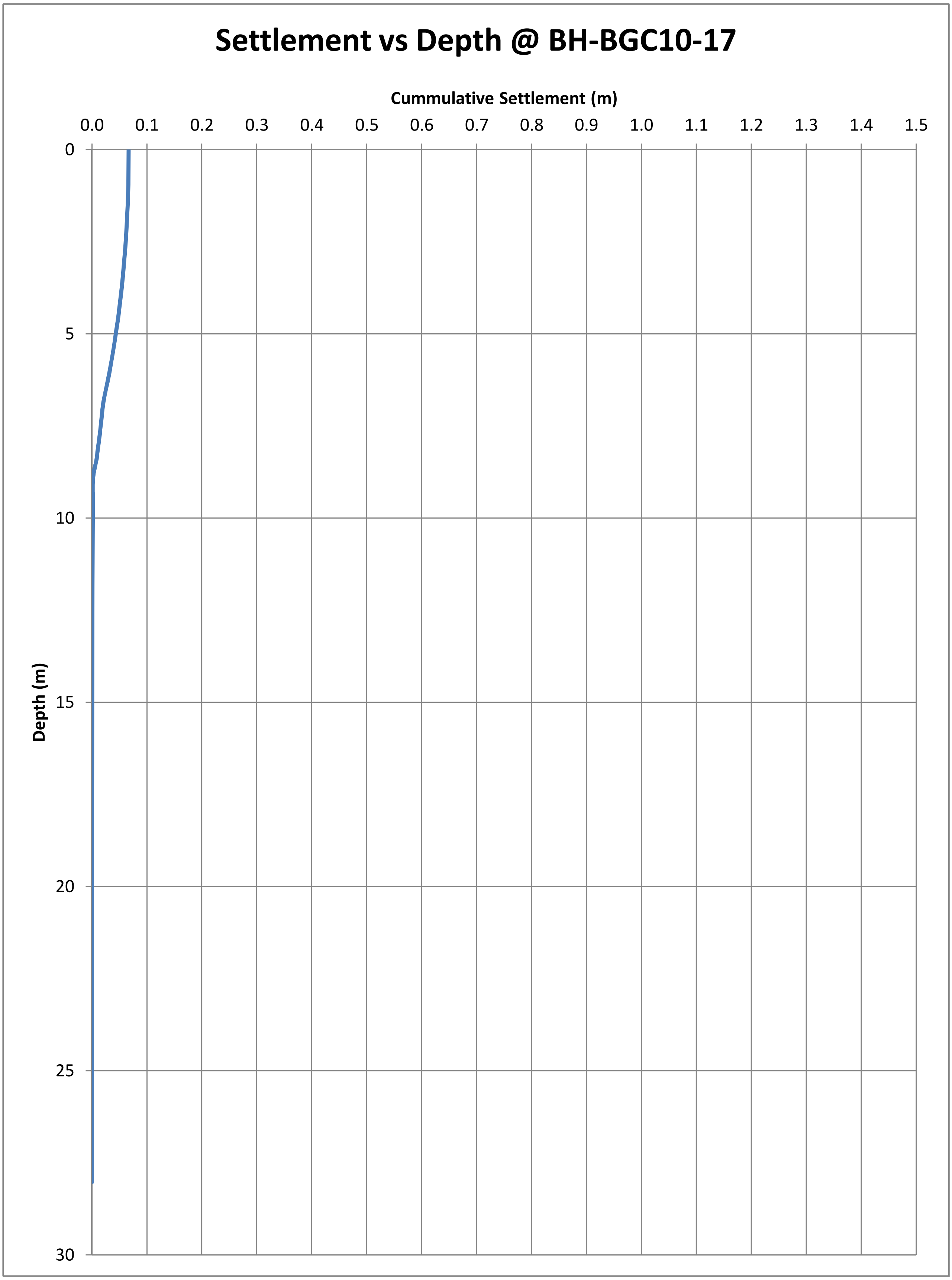
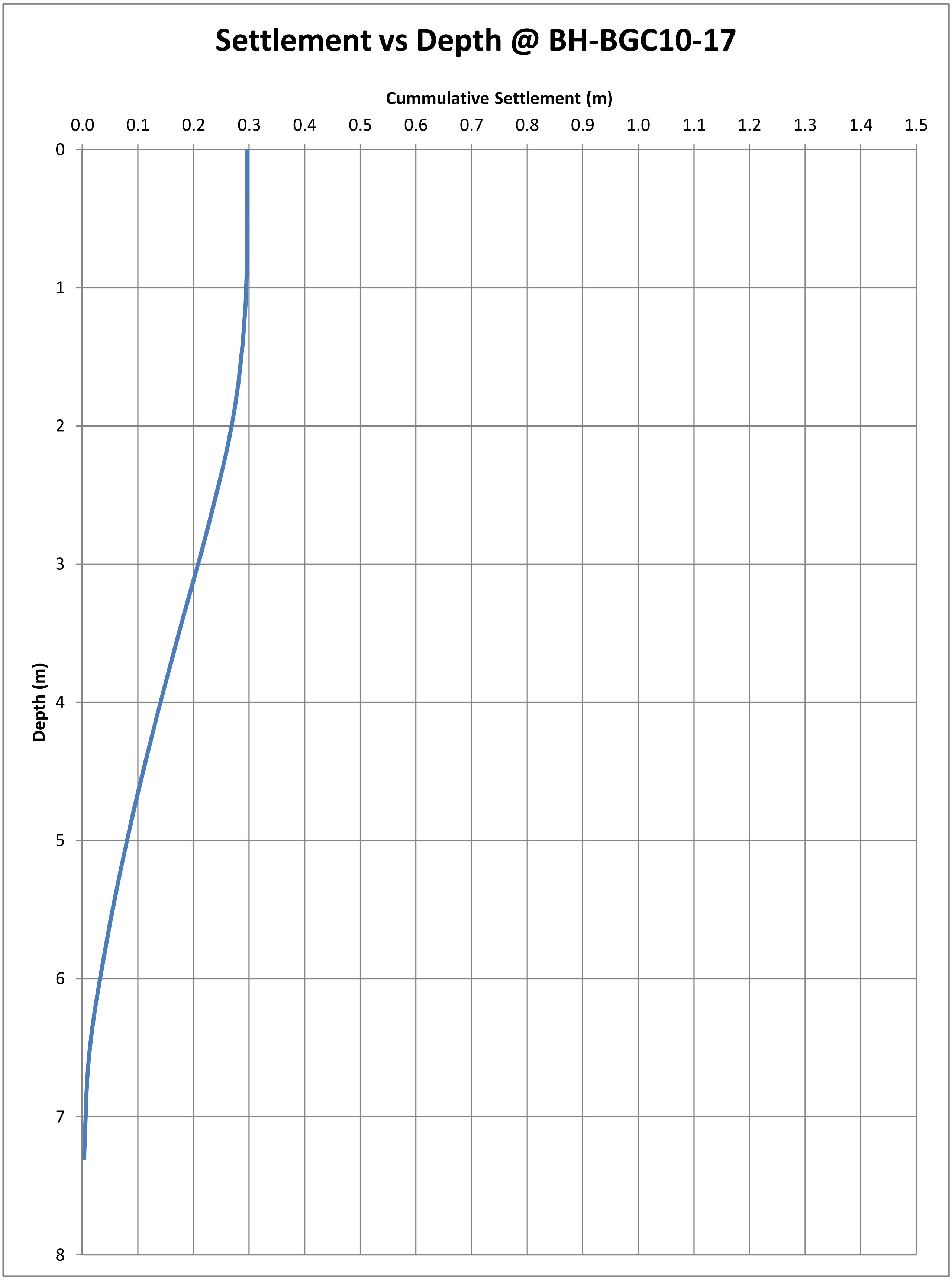
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
Depth (m)
0
1
2
3
4
5
6
7
8
9
10


Settlement vs Depth @ BH-BGC10-5
Cummulative Settlement (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
1
1
2
2
3
3
4
4
5

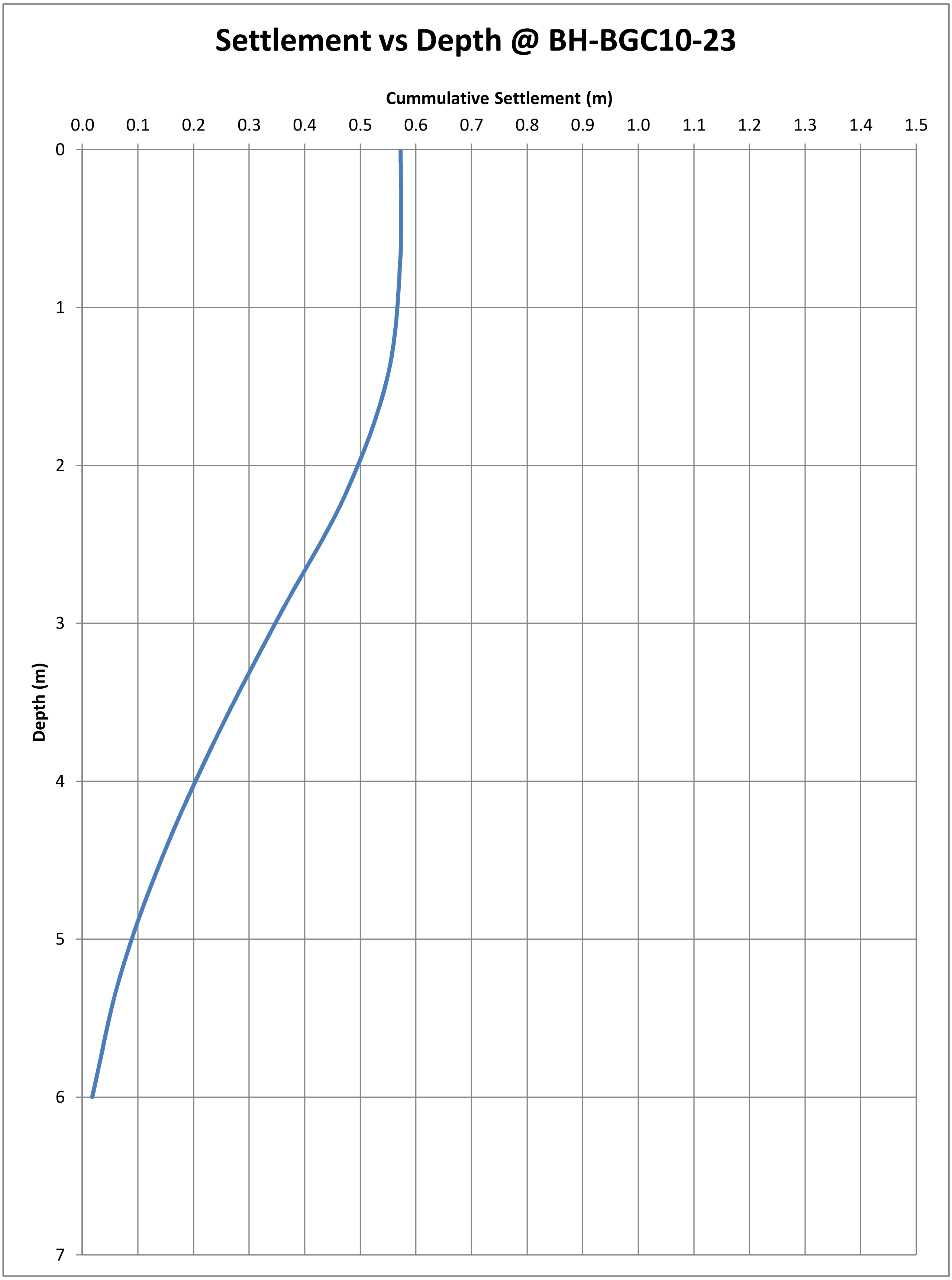

Settlement vs Depth @ BH-BGC10-6
Cummulative Settlement (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
2
4
6
8
10
12
14
16
18


Settlement vs Depth @ BH-BGC10- 15
Cummulative Settlement (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
1
2
3
4
5
6
7
8
9
10


Settlement vs Depth @ BH-BGC10-17
Cummulative Settlement (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
5
10
15
20
25
30



Settlement vs Depth @ BH-BGC10-17
Cummulative Settlement (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
1
2
3
4
5
6
7
8



Settlement vs Depth @ BH-BGC10-23
Cummulative Settlement (m)
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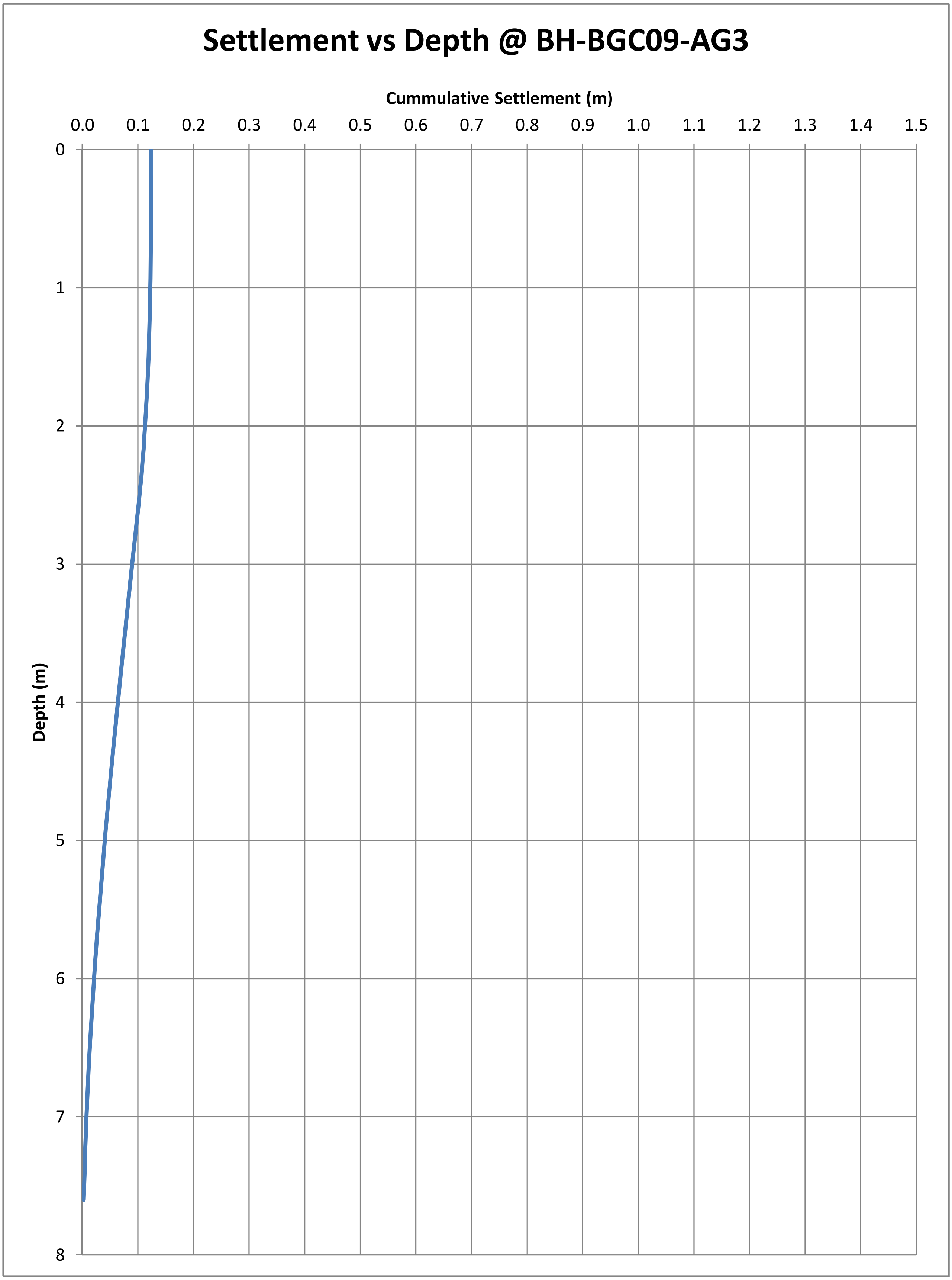
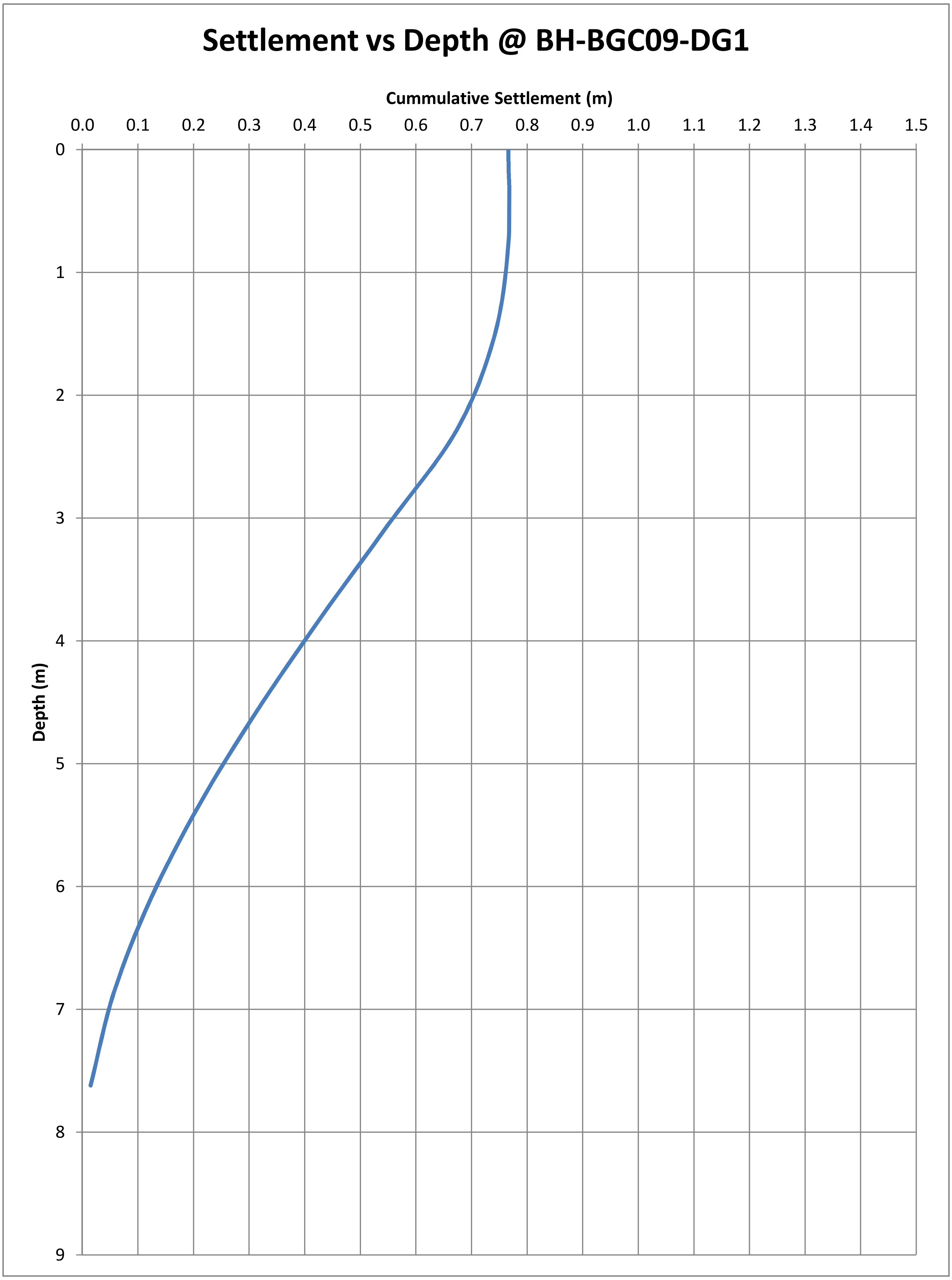
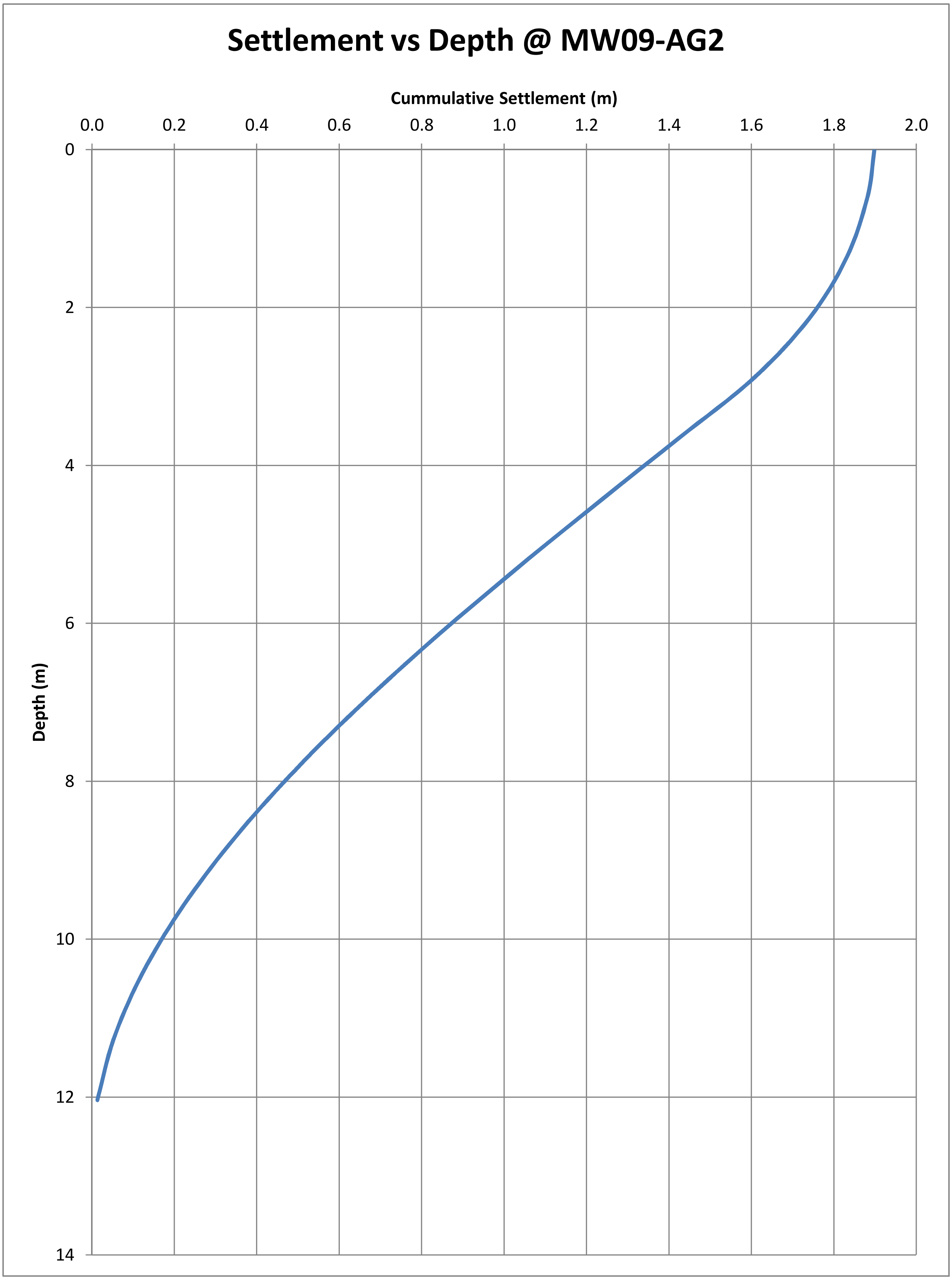
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
1
2
3
4
5
6
7


Settlement vs Depth @ BH-BGC09-AG3
Cummulative Settlement (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
1
2
3
4
5
6
7
8


Settlement vs Depth @ BH-BGC09-DG1
Cummulative Settlement (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
Depth (m)
0
1
2
3
4
5
6
7
8
9


Settlement vs Depth @ MW09-AG2
Cummulative Settlement (m)
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Depth (m)
0
2
4
6
8
10
12
14

Borehole ID	Change in Stress	Material	Thickness, (m)		Classification
BH-BGC09-AG2					
BH-BGC09-AG3		Colluvium	1.6		SW/GW
		--	4.8		SP
			3.1		Quartzite
		Rock	3.02		Biotite Schist
BH-BGC10-1		--	1.8		--
		Moderately Weathered Rock	18.6		
BH-BGC10-2		--	7.3		--
		Very Weathered Rock	13.1		--
BH-BGC10-3		Fill	7.2		
		Colluvium	0.9		
		Moderately Weathered Rock	42.6		
BH-BGC10-4		Fill	8.7		
		Moderately Weathered Rock	22.3		
BH-BGC10-5		Fill	4.3		
		Moderately Weathered Rock	16.7		
BH-BGC10-6		Till	16.4		
		Moderately Weathered Rock	12.5		
BH-BGC10-16		--	8.8		
		Moderately Weathered Rock	12.2		
BH-BGC10-17		Fill	7.3		
		Moderately Weathered Rock	30		
BH-BGC10-23		Fill	6		
BH-BGC10-23		Placer Taillings	4.5		SW/GW
		Colluvium?	1.6		GP
		Weathered Bedrock	1.5		CL
		Metasedimentary	5.2		Rock

Area	Borehole ID	Consolidation Settlement (mm)
HLF Impoundment	BH-BGC10-AG3	716
	BH-BGC10-AG5	533
	BH-BGC10-1	82
	BH-BGC10-2	1,357
	BH-BGC10-3	809
	BH-BGC10-4	2,142
	BH-BGC10-5	42
	BH-BGC10-6	673
	DH-BGC09-AG3	123
	MW09-AG2	1,898
HLF Embankment	BH-BGC10-16	67
	BH-BGC10-17	297
	BH-BGC10-23	572
	DH-BGC09-DG-1	766