Western Copper Corporation

Project Proposal

Carmacks Copper Project
Yukon Territory

Appendix D

EBA Engineering Consultants
Carmacks Copper Mine
Heap Leach Pad Liner Design Report (2005)

Heap Leach Pad Liner Design
Carmacks Copper Project
near Williams Creek, YT

EBA Project No. 1200133

May 2005

Heap Leach Pad Liner Design
Carmacks Copper Project
near Williams Creek, YT

Submitted To:

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Vancouver, BC

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1.0 INTRODUCTION

EBA Engineering Consultants Ltd. (EBA) have carried out a review of the existing Heap Leach Pad design for the Carmacks Copper Project, and prepared an alternative conceptual design for submission with the Project Description. Authorization to carry out this work was provided by Mr. Dan Cornett of Access Consulting Group (ACG) on behalf of Mr. Jonathan Clegg of Western Silver Corporation (Western Silver).

The purpose of this work was to assist Western Silver with the development of specific engineering components of the heap leach pad in support of the environmental assessment and permitting for the Carmacks Copper Project. The Carmacks Copper Project is located approximately 28 km northwest of Carmacks, Yukon at Latitude 62.35˚ North and Longitude 136.70˚ West.

Western Silver intends to submit their Project Description to the Yukon Government in early 2005.

2.0 AVAILABLE INFORMATION

Several key documents were either contained in EBA files, or forwarded to EBA for use in this study – they include:

- EBA Engineering Consultants Ltd., *Testpit and Laboratory Test Results – Heap Leach Pad Area, Carmacks Copper Project, NW of Carmacks YT.* EBA Report to Western Copper Holdings Ltd., December, 1997.

- Hallam Knight Piesold Ltd., *Western Copper Holdings Limited, Carmacks Copper Project, Detailed Report on Hydrogeological Summary and Preliminary Impact Assessment (Ref. No. 1783/3);* IEE Addendum No. 3, October 1995

- Hallam Knight Piesold Ltd., *Western Copper Holdings Limited, Carmacks Copper Project, Detailed QA/QC Program and Construction Specifications (Ref. No. 1783/5);* IEE Addendum No. 3, October 1995

- Hallam Knight Piesold Ltd., *Western Copper Holdings Limited, Carmacks Copper Project, Detailed Report on Initial Leach Pad Settlement Assessment (Ref. No. 1783/6);* IEE Addendum No. 3, October 1995

- Hallam Knight Piesold Ltd., *Western Copper Holdings Limited, Carmacks Copper Project, Detailed Terrain Hazard Mapping;* IEE Addendum No. 3, October 1995

- Hallam Knight Piesold Ltd., *Western Copper Holdings Limited, Carmacks Copper Project, Detailed Carmacks Copper Operating Plans;* IEE Addendum No. 3, October 1995

- Kilborn SNC·Lavalin, *Western Copper Holdings Limited, Carmacks Copper Project, Report on Detailed Design (Ref. No. 1784/2);* Revised 14 August 1996

- Kilborn SNC·Lavalin, *Western Copper Holdings Limited, Carmacks Copper Project, Report on Updated Detailed Design Criteria (Ref. No. 1784/5);* 3 July 1996

- Knight Piesold Ltd., *Western Copper Holdings Limited, Carmacks Copper Project, Report on Updated Detailed Design of the Heap Leach Pad and Events Pond (Ref. No. 1785/1);* 23 April 1997

- Knight Piesold Ltd., *Western Copper Holdings Limited, Carmacks Copper Project, Report on 1996 Geotechnical and Hydrogeological site Investigations (Ref. No. 1784/1);* June 1996

- Sitka Corp, *Carmacks Copper Project Design Criteria and Parameters*, October 1998 -

- Western Copper Holdings Limited, *Carmacks Copper Project, Hydrogeological and Water Management Issues*; IEE Addendum No. 4, December 1997

- Western Copper Holdings Limited, *Carmacks Copper Project, Basic Engineering Reports and Definitive Cost Estimate; IEE Addendum No. 4*, December 1997

- Western Copper Holdings Limited, Carmacks Copper Project, Technical Issue Response Document; 30 June 1997

3.0 YUKON GOVERNMENT GUIDELINES

EBA's conceptual design was prepared to comply with the Yukon Government guidelines and referenced standards. Table 1 below presents performance standards dated April 8, 2005 that were provided by Mr. Bill Dunn of the Department of Energy Mines and Resources.

Table 1
April 2005 Performance Standards for the Carmacks Copper Project

Issues	Performance Objective	Yukon Government Guideline/Standard
Liner Design	Prevent discharge of noncompliant waters	• Liner System (including materials; conceptual construction methods and conditions; operation and maintenance procedures) achieving a permeability at least equivalent to a synthetic liner over a 12” soil liner with permeability of 10^{-6} cm/sec • Leak detection and recovery system with contingency plans
Physical Stability of heap and associated earth works, such as berms constructed to constrain leachate	Minimize risk of liner damage	• Suitable design, criteria based on **Canadian Dam Association’s “Dam Safety Guidelines” (1999)**

4.0 METHODS

Knight Piesold Ltd (KP) prepared a design of the Heap Leach Pad in 1997 using the factual information identified in Section 2.0. EBA has undertaken a review of the 1997 design and KP reports in an effort to adequately understand the site, plans, resources and commitment by Western Silver. The primary purpose of this initial review was to identify any fatal flaws that may preclude the use of a lined heap leach pad system. The KP heap leach pad layout and design was used as the baseline work for EBA’s analyses.

EBA have reviewed the information and identified pertinent events and information that post-dates the April 23, 1997 report by KP. This new information includes:

- The Heap Leach pad site was cleared of forest and vegetation cover in 1996 and has been inactive since that time.
- Canadian Dam Association’s “Dam Safety Guidelines” (1999) were issued.
- 1998 Draft Report by Sitka
- Advancement in the design and manufacture of geosynthetic liner systems and geocomposites

In 2005 new seismic hazards will be introduced as part of the revised National Building Code of Canada. Further review of the influence of these new standards will be required, however, based on a cursory review of the information the assumptions made with respect to seismic loading are expected to satisfy the new code requirements. Further seismic analyses using the new guidelines will be necessary to confirm this.

5.0 DESIGN LIFE AND CONSEQUENCE CATEGORIES

The design life of the mine facilities is approximately 12 years, with long-term decommissioning, reclamation, and monitoring following.

The embankments at the Carmacks Copper site will not impound fluids for long periods during operations. The following measures will achieve this:

- Low-level outlets within the heap confining embankment will prevent fluid impoundment during normal operations.
- Solution in the events pond will be used as makeup water in the heap.
- Water in the sediment ponds will drain through decant structures in the abutments.

The Canadian Dam Safety Association (CDSA, 1999) categorizes the failure of a dam or similar impoundment structure using a consequence scale of Very High, High, Low or Very Low. The CDSA classification method requires that the project components be rated in terms of consequence to safety and/or failure hazard to socioeconomic, financial and environment in the event of a failure. EBA have reviewed terrain hazard mapping prepared by KP, subsurface soil conditions, seismic criteria and the proposed construction and operational plan. It is apparent that under static conditions conventional engineered measures can be implemented to address containment and overall stability. Under seismic loading the stability of the heap pile and impoundment has been shown to be feasible within ordinarily acceptable ranges of parameters and seismic criteria. Seismic loading magnitudes and overall influence are statistically based parameters for which the resulting loads can vary widely. Seismic events occur with little or no warning and therefore it is possible that a statistically low likelihood event might trigger a failure that could lead to a high consequence.

6.0 SEISMIC CRITERIA

6.1 General

Selection of appropriate seismic criteria is key to acceptable engineering design and analysis for the heap leach pad. The CDS guidelines provide the means for regulators and owners to choose an appropriate range of ground accelerations based on the available statistical information and for agreed upon consequences of a failure. Using the CDSA criteria, a corresponding range of seismic related forces acting on all forms of structures can be prescribed based on the estimated seismic generated horizontal ground accelerations. These forces are added to the static forces used in conventional slope stability analyses and the resulting analyses are termed pseudostatic.

To analyze the stability of an embankment under seismic load, Seed (1979) states that a pseudostatic analysis is appropriate provided that potentially liquefiable materials do not make up a significant portion of the structure or foundation. From the site investigations to date, there are few materials on site that could liquefy during an earthquake, making pseudostatic analysis appropriate. Furthermore, foundation treatment methods such as foundation drains are proposed by Western Silver to further reduce the possibility of liquefiable foundation soils.

For the pseudostatic analysis, Hynes-Griffin and Franklin (1984) describe another method to evaluate the appropriate seismic criteria prescribed by the CDSA Consequence based approach. The Hynes-Griffin and Franklin (1984) method determines the appropriate seismic coefficient from the maximum ground acceleration and the allowable displacement. This allowable displacement varies with the type of embankment or structure.

The following sections outline the approach that has been used to selecting the appropriate seismic design criteria for this project.

6.2 Maximum Credible and Design Basis Earthquakes

The heap confining embankment and events pond dam have a "high" consequence category as described by CDSA. According to the CDSA guidelines, these two embankments will be designed to withstand accelerations resulting from the greater of 50 percent of the Maximum Credible Earthquake (MCE) from a deterministic analysis or the acceleration from an earthquake with a 1000-yr return period from a probabilistic analysis. The lower end of the range of design criteria suggested by CDSA has been selected due to the relatively short design life of these two embankments and the resulting reduced risk exposure.

From the probabilistic analyses conducted by the Pacific Geoscience Centre (the maximum ground accelerations associated with the 475 and 1000-yr return period earthquakes are estimated to be 8.5 percent and 10.3 percent of gravity, respectively.

Using deterministic methods, the MCE is associated with the Fairweather-Yakutat source zone, with a maximum magnitude of 8.5. At an epicentral distance of 250 km, it is estimated to generate a maximum ground acceleration of 13.2 percent of gravity. Despite their shorter epicentral distances, three other potential source zones (Northern B.C., Denali-Shakwak, and Mackenzie) all generate lower maximum accelerations of 8.8 percent, 3.9 percent, and 2.1 percent of gravity, respectively. Thus, the maximum credible design acceleration from the deterministic analysis is 13.2 percent of gravity. EBA suggests that a design acceleration of 13.2 percent of gravity be used for stability analysis purposes, and that the corresponding Factor of Safety be greater than 1.

Using CDSA criteria, the heap confining embankment and events pond dam should be designed to withstand horizontal accelerations of 10.3 percent of gravity and satisfy a minimum factor of Safety of 1.15 or greater.

6.3 Allowable Deformations and Seismic Coefficients

For the pseudostatic analysis, Hynes-Griffin and Franklin (1984) describe a method for determining the appropriate seismic coefficient from the maximum ground acceleration and the allowable displacement from seismic activity. This allowable displacement varies with the type of embankment or structure.

For a conventional dam, Seed (1979) suggests that about one metre of crest displacement is usually acceptable.

Where geomembrane liners form a component of an embankment, such as in the heap confining embankment and events pond dam, a smaller crest displacement is allowed to reduce the possibility of influencing the integrity of the liner system during the design seismic event. For lined waste impoundments in the U.S., Seed and Bonaparte (1992) describe the current practice as using an allowable seismic displacement of 150 to 300 mm. A displacement of 150 mm will be allowed at the crest of the heap confining embankment and the events pond dam.

The following table summarizes the seismic coefficients from Hynes-Griffin and Franklin (1984) using the allowable crest displacements described above.

Table 2
Seismic Design Criteria

Embankment	Consequence Category	CDSA Max ground Accel., %g	Allowable Displ., m	Hynes-Griffin and Franklin (1984)Seismic Coef., %g
Heap confining Embankment	High	10.3	0.15	6.7
Events pond dam	High	10.3	0.15	6.7

Based on the above, the design seismic ground accelerations chosen in Section 6.2 satisfy CDSA (1999) guidelines and a strict interpretation of Hynes-Griffin and Franklin (1984).

7.0 GEOTECHNICAL STABILITY

The targeted minimum factor of safety of embankments depends on the loading condition and the assigned consequence category of the embankment. The three primary loading conditions are steady-state static, seismic, and end of construction.

For the steady-state static loading condition at the “high” consequence structures-the heap confining embankment and events pond dam-the factor of safety will be at least 1.5 for all failure mechanisms.

For the seismic loading condition, the factor of safety for the heap confining embankment and events pond dam will be greater than 1.0 at the maximum credible earthquake and at least 1.15 using the maximum design earthquake seismic coefficients provided in Section 6.2.

For the end of construction static loading condition, all structures will have a factor of safety of at least 1.3, regardless of consequence category.

8.0 PERMAFROST

8.1 Regional Overview

The project site is in the southern region as defined by Brown (1970). In this region, the permafrost is discontinuous and its character is varied. Testholes across the site confirm this, with slightly more than half the holes drilled in the early and mid-1990's encountering permafrost. The upper and lower limits of the permafrost vary significantly, but the onsite thermistor instrumentation data suggest that in 1997 the permafrost did not extend deeper than about 25 m below ground surface and that its temperature was only a few tenths of a degree below freezing. Brown (1970) indicates the active layer-the zone subject to annual freeze-thaw cycles-usually ranges from about 1.5 m to 4 m depending on the thickness and character of the organic ground cover, slope aspect, and elevation.

8.2 Review of KP Subsurface Information (to 1996)

Of the 191 samples collected on the entire Carmacks Copper (Williams Creek) Project Site and tested, only 15 that were frozen had moisture contents greater than 17 percent. Of these, only 5 were deeper than 5 m, 2 were from a single test hole in the proposed heap area (DH95-C), 2 were from test holes on the north side of the proposed waste rock storage area (MW96-F and H), and 1 was east of the proposed development area (DH95-2).

8.3 Influence of Permafrost on Design and Construction

In 1996 the entire heap leach pad site was cleared of vegetation in preparation for subgrade preparation work. The removal of the insulating effect of the vegetative ground cover will undoubtedly have had a positive effect by causing the permafrost to degrade. These measures and other proposed measures would reduce the influence of thaw-unstable soils and the residual effects of the permafrost.

Thaw-unstable soils within the permafrost will be addressed. These soils contain sufficient ground ice to possibly cause unacceptable settlement or loss of shear strength as they melt. Without specific tests on individual soils to determine their thaw-instability, the natural moisture content can sometimes be used as an indicator. On other Yukon projects where foundation soils exhibit moisture contents greater than 17 percent of the dry soil weight, the foundation soils have been deemed potentially thaw-unstable. Site specific testing will be required to verify the applicable moisture content at this site, as it is dependant upon soil gradation and composition.

For the purposes of this report, however, it has been assumed that 17 percent moisture is an acceptable cutoff.

Prior to construction in an area, test holes will be drilled in a 50-m grid pattern through the soil cover to either the top of bedrock or to the base of permafrost estimated to be at maximum depth of 25 m, whichever is shallower. In the test holes, the upper 1.5 m will be sampled continuously to check for moisture content and suitable soil liner material. Below 1.5 m, samples will be collected approximately every 1.5 m or as required by material changes, and tested for moisture content. Where the soils are unfrozen, or frozen but with a moisture content not greater than 17 percent, construction can proceed without any special foundation treatment. Where there are frozen soils with a moisture content greater than 17 percent within 5 m of the ground surface, the potentially thaw-unstable soils will be excavated and the excavation backfilled with durable rock. This program and procedures will be described in more detail in the construction quality assurance plan.

It has been reported by others that less than 10 percent of the samples tested to 1997 have been frozen with moisture contents greater than 17 percent. Because of the advanced stripping and thawing, this proportion should have decreased significantly and thereby reduce the required amount of pad foundation preparation.

Also as noted above, there have been only three locations in the areas currently proposed for development where there have been frozen soils deeper than 5 m with water contents greater than 17 percent. Because these are below the active zone, the advanced stripping likely will not cause thawing in a single year at these depths. Also, because of their depth, they cannot simply be excavated.

Each area where there are deep soils that are potentially thaw-unstable will require further engineering analysis and/or testing to determine the proper treatment. For example, drill hole DH95-C in the proposed leach pad area encountered deep potentially thaw-unstable soil at depths of 6.1 m and 12.2 m. Surrounding drill holes did not encounter these soils, so for this example, the problem area could be considered to be local. In this case, a potential reduction in soil shear strength would not be critical to successful performance because the surrounding soils are stable and because the ore will be loaded in an uphill direction. However, without special treatment, local differential settlement could be enough to unacceptably strain the liner system. By subexcavating the subgrade several metres in this area and constructing a raft of heavily compacted durable rock fill, these potential differential settlements can be spread over a wider area to reduce the liner strains to an acceptable amount. For higher ice contents, the raft could be made more rigid to better spread the settlement by reinforcing it with geogrid or high-strength geotextile.

9.0 HEAP LEACH FACILITY

9.1 Design Basis

The heap will be designed to store approximately 13.3 million tonnes of ore at a dry density of 1.6 tonnes/m^3. The ore density may be higher in the later years of operation due to consolidation under load. The leach pad could be expanded beyond this capacity to the west or the height could be increased. Ore will be placed for eight years at a maximum rate of 9872 tonnes per day for up to 200 days per year. The 31.5 ha leach pad will be constructed in three stages ahead of ore placement. Ore will be placed in 8 m lifts at an overall slope of 2½h:1v using conveyors. It is anticipated that two years of residual leaching, three years of heap rinsing and eventual decommissioning will follow the eight years of ore placement.

The raffinate will be applied through a system of drip emitters at a rate of 0.204 litres/min/m^2. The total raffinate flow to the heap will be 540 m^3/hr for a design leaching cycle of 120 days. Solution will not be stored within the heap but will drain through perimeter piping and a low-level outlet to the process plant or the events pond.

9.2 General Arrangement

The ore will be placed on the valley-fill heap in 8-m lifts by haul trucks and leached in subsequent lifts, progressing up slope and atop previously leached lifts. Storage for excess solution and extreme precipitation events will be provided in an events pond located down gradient from the heap.

The proposed leach pad will be lined with a double composite liner system with a leak detection and recovery system (LDRS). The pad will be surrounded by a two metre high perimeter berm on the north and west sides and a perimeter bench on the east side. A confining embankment will form the lower limit of the leach pad to support the heap. With a crest elevation of 780 m, it will be about 22 m high and 350 m long.

There will be no in-heap solution storage behind this confining embankment. Solution from the heap will be collected by a network of corrugated polyethylene tubing (CPT) above the leach pad liner and conveyed by gravity flow to the process plant. There will a double lined spillway over the heap confining embankment to the events pond to convey solution during extreme precipitation events. Diversion ditches will collect and convey runoff from upslope of the heap leach facility to a sediment control pond, thereby reducing the quantity of water reporting to the heap and minimizing the pregnant leachate solution (PLS) dilution.

9.3 Geotechnical and Hydrogeological Properties

The geotechnical and hydrogeological properties of the foundation, zoned earthfill, liner, waste rock, ore, drainage layer, and overliner materials have been estimated from drilling and test pitting, site-specific laboratory results, published literature, and professional experience. The following documents form the basis for selecting the principal geotechnical and hydrogeological properties for final design of the leach pad and heap confining embankment:

- Knight Piésold, Ref. No. 1783/1, May 1995 'Report on Preliminary Design' – Laboratory test work and index test results, including foundation materials, pre- and post-leach ore, geosynthetic/soil interfaces, and geosynthetic/geosynthetic interfaces.

- Knight Piésold, Ref. No. 1784/1, June 1996 'Report on 1996 Geotechnical and Hydrogeological Site Investigations' – Laboratory test work and index test results, including permeability, coefficient of consolidation, coefficient of volume compressibility, and uniaxial compressive strength of bedrock.

- EBA Engineering Consultants Ltd. December 5, 1997. Submission of Testpit & Laboratory Test Results- Heap Leach Pad Area.

The complete site-specific test results are not repeated in this report. Table 3, on the next page, provides a list of all materials and interfaces to be considered during design and the principal geotechnical and hydrogeological parameters adopted for each. Where available, the range of test results is provided in parentheses. In general where test data are available, the selected parameters are at or near the lower bound of the test data. In the few exceptions to this, the parameters were selected after considering the variability of the data and experience in similar circumstances.

Table 3: Summary of Hydrogeological and Geotechnical Properties - Carmacks Copper Heap Leach Pad

Material	Bulk Unit Weight, γ_{bulk} (kN/m3)	Friction Angle, φ' (degrees)	Cohesion, c (kPa)	Permeability, k (m/s)	Coefficient of Consolidation, C_v (m^2/year)	Uniaxial Compressive Strength, UCS (MPa)
Ore Materials						
Crushed Ore (saturated)	19.6	37	0	1×10^{-8}	-	-
Crushed Ore (unsaturated)	16.7	37	0	1×10^{-8}	-	-
Foundation Materials						
Sands and gravels	21 (20.9 to 21.8)	36 (36 to 44)	0	$1x10^{-5}$ ($1x10^{-5}$ to $1x10^{-7}$)	150	-
Finer grained sand, silt and clay mixtures	21 (17.5 to 23.2)	33 (36 to 41)	0 (0 to 151)	$1x10^{-7}$ ($1x10^{-7}$ to $1x10^{-9}$)	20 (6 to 35)	-
Plastic clays	14 (14)	10 (10)	61 (61)	-	7 (3 to 13)	-
Frozen soils	21.7 (20.4 to 23.0)	-	-	1×10^{-15}	-	-
Weathered granodiorite	21 (21)	37 (37 to 44)	-	1×10^{-6}	-	5
Fresh granodiorite	25	45	15,000	2×10^{-7}	-	40 (35 to 75)
Fresh to weathered biotite gneiss	25	37 to 40	-	1×10^{-6} to 2×10^{-7}	-	56 (35 to 92)
Construction Materials						
Zoned earthfill	22.4	40 (36 to 44)	0	-	-	-
Waste rock	19.6	37	0	-	-	-
Overliner	-	-	-	1×10^{-4}	-	-
Soil liner	-	-	-	$1x10^{-8}$ ($1x10^{-9}$ to $1x10^{-11}$)	-	-
LDRS drain rock	-	-	-		-	-
Textured Geomembrane / Geonet		21*				
T. Geomembrane / Ore or overliner interface	-	26 (26 to 26)	0	-	-	-
Smooth Geomembrane / Geonet interface	-	n/a (6 to 25)	0	-	-	-

Notes:
1. Plastic clays not considered in hydrogeological modelling.
2. Sources: Site-specific laboratory testwork, references provided in Section 3.1, Knight Piésold Ref. No. 10178/6-1, professional experience, and external review comments.
3. Range of site-specific laboratory results presented in parentheses.
4. Omitted values not relevant to design analyses or not applicable to material.
5. Ore and overliner permeabilities to be confirmed, including stress dependancy and degradation during leaching.
6. * verification required

9.4 Foundation Preparations

9.4.1 Pad Grading

Most of the organics and topsoil were stripped from the foundation area in 1996. At the start of construction, any remaining windrows or piles will be removed and the area will be rough graded. Site preparation activities at this time will include:

- completion of the a drilling investigation to delineate any potential unstable soil (see Section 8.3) and assess suitability for soil liner material,
- removal or treatment of the unstable/unsuitable soils and controlled fill placement to subgrade elevation;
- in areas cut to subgrade elevation - scarification, moisture conditioning, and compaction of the subgrade level soils to depth of at least 300 mm,
- proof-roll of prepared subgrade
- construction of the liner

As part of the pre-construction investigation soil samples representative of the subgrade will be taken and tested for particle size, plasticity indices, and natural moisture content. There will also be enough control tests to relate the index properties and visual characteristics of the subgrade soils to the expected permeabilities. From the test results, the subgrade soils to be classified as follows:

- Soil Liner Material - permeability of 10^{-8} m/s or lower. This materials will satisfy the grading requirements for soil liner material, and will be suitable for compaction;

- Random fill – permeability greater than 10^{-8} m/s. These materials will be used selectively for site grading below the depth of any proposed soil liner or uses as appropriate in zoned earthfills based on grading requirements.

- Waste Materials – waste materials will include organic rich materials, potentially unstable materials or any other materials deemed deleterious. These materials will be excavated and hauled to a designated waste stockpile.

As described in Section 8.3, all potentially thaw-unstable materials within 5 m of the ground surface that are identified during the delineation program will be excavated. Excavations deeper than 1 m below final subgrade will be filled to 1 m below final subgrade with acceptable rock fill, then filled with soil liner material. These materials will be placed and compacted as subgrade. Further details will be provided in the QA/QC Manual.

Areas where potentially thaw-unstable materials are deeper than 5 m will be assessed individually and specific treatments developed.

9.4.2 Foundation Drainage

A foundation drainage system will be installed beneath the leach pad to intercept and remove near-surface and seasonal groundwater flows, to reduce the possibility of uplift pressures beneath the liner, and to provide another LDRS. The foundation drains will be installed at least 1.5 m below the prepared subgrade surface and will comprise perforated CPT surrounded by select drain gravel and wrapped in geotextile. The select drain gravel will provide continued foundation drainage in the event of blockage or collapse of the CPT. The drains will be located in the natural drainage swales and extended to intercept any springs, seeps, or damp spots identified during pad grading and mapping. These drains will convey any intercepted groundwater seepage under the embankment to a foundation drainage collection sump located at the toe of the confining embankment. Flow into the sump will be tested periodically for pH and conductivity. If its quality is acceptable, it will be discharged below the events ponds; otherwise, it will be discharged into the events pond.

Once the foundation drains are installed they will be covered by compacted soil liner material and the double composite liner system. The upslope ends of the main collection pipes extend beyond the limits of the pad area through solid CPT pipe so that they will remain accessible. The ends of the pipes will be capped to prevent animals from entering the pipe and to prevent icing. If blockage of the CPT is suspected an attempt will be made to pressure clean with water or mechanically clean the tubing. Given the redundant drainage provided by the surrounding drain gravel, no further attempt to recover the CPT installation will be made if cleaning is unsuccessful.

9.4.3 Perimeter Berm and Bench

The perimeter bench on the east side of the leach pad will be wide enough for the access road perimeter diversion ditch, perimeter piping and sumps, and the liner anchor trench. The perimeter berm on the north and west sides of the pad will incorporate the liner anchor trench and perimeter piping and sumps. The perimeter road and diversion ditch will be outside of this berm. The berm and bench will separate the surrounding diverted areas and the heaped ore. A channel, formed by the depression between the perimeter berm or bench and the sloping ore, will convey surface runoff from the heaped ore to the perimeter sumps. From there, it will be piped to the plant or events pond.

10.0 LINER SYSTEM

10.1 General

The entire leach pad and the uphill face of the confining embankment will be lined with a double composite liner with an integral LDRS. Three separate designs are envisioned with protection for the environment appropriate to the potential for leakage in any given zone: these zones have been designated as the upper works, lower works and trenches.

The upper works comprise the upper portion of the heap leach pad, at elevations greater than 830 m. In this zone, the base slope exceeds 7:1 with a consequence that pregnant leachate solution (PLS) flow velocities are high and hydraulic heads are low.

The lower works comprise the lower portion of the heap leach pad adjacent to the confining embankment. In this zone, PLS velocities are low and the hydraulic head will approach 1.0 m. Therefore, there is a potential for higher leakage rates through the primary liner in this area.

The trenches are constructed in the LDRS to move PLS laterally. In the trenches, PLS velocities will be high but, because these are the collector system for the LDRS, the hydraulic head will also be high. There is therefore a higher potential for leakage of the primary liner in this area.

Subject to the results of product specific laboratory testing of the liner system, the components of the liner system for the upper and lower works will generally comprise the following:

10.2 Upper Works:

The upper works liner system comprises (listed from the top down):

- High-permeability, durable overliner cushion layer with solution collection piping.
- 60 mil textured HDPE upper liner;
- Leak detection and recovery system (LDRS) comprising a high transmissivity tri-planar geocomposite;
- 60 mil textured HDPE lower liner;
- Subgrade (with foundation drains);

10.3 Lower Works:

The lower works liner system comprises (listed from the top down):

- High-permeability, durable overliner cushion layer with solution collection piping.
- 60 mil textured HDPE upper liner;

- Leak detection and recovery system (LDRS) comprising a high transmissivity tri-planar geocomposite;
- 60 mil textured HDPE lower liner;
- Compacted lower soil liner with a permeability not greater than 10^{-8} m/s;
- Subgrade (with foundation drains);

10.4 Trenches

The trench design profile comprises (listed from the top down):

- High-permeability, durable overliner cushion layer with solution collection piping.
- 60 mil textured HDPE upper liner;
- 12 oz nonwoven polypropylene geotextile
- Drainage layer comprising durable crushed ore or sand and gravel with permeability of at least 5×10^{-4} m/s and solution recovery piping;
- Leak detection and recovery system (LDRS) comprising a high transmissivity tri-planar geocomposite;
- 12 oz nonwoven polypropylene geotextile
- 60 mil textured HDPE lower liner;
- Subgrade;

The components of the various liner designs are further described as follows:

10.5 Subgrade

The subgrade will be suitable in-situ material that has been scarified and recompacted, or borrow material imported to backfill excavations of unsuitable material as described above. The design criteria for subgrade are:

- Random fill as defined in the technical specifications
- Maximum particle size equal to 75% of the approved layer thickness.

10.6 Soil Liner

<u>Lower Works</u>

Laboratory tests and correlations with index properties will confirm that the liner material meets the required permeability criterion of 10^{-8} m/s. These will be described in the QA/QC manual.

The soil liners will be compacted with a smooth drum vibratory roller in lifts of less than 150 mm, with careful inspection of the soil surface to ensure the removal of any stones larger than 10 mm under strict quality control. The liner installer will certify acceptance of the final surface as part of the QC and warranty process.

<u>Upper Works</u>

Beneath the upper works, subgrade preparation will require sufficient effort to remove any organic materials, provide a competent base and prevent rock fragments and gravel from puncturing the lower geomembrane liner. Product-specific laboratory testing under expected loads will dictate the maximum allowable particle size and final methods of subgrade preparation for the upper works

10.7 Textured HDPE Liner

Both geomembrane liners will be 60 mil textured HDPE. Careful manufacturing quality control and construction quality assurance will confirm the specifications are achieved.

10.8 Leak Detection and Recovery System

An LDRS will be constructed using a high flow triplanar geocomposite. The geocomposite utilizes a tri-planar structure with rigid vertical ribs that significantly increase the tensile strength and compressive resistance of the geocomposite. These ribs are also supported by structural planar ribs that reduce intrusion into the high flow drainage core. The LDRS will be subdivided into cells of appropriate size to allow for solution management in each pad area.

10.9 Geotextile

A 12 oz, non-woven, needle punched geotextile will be used as a separation and filtration layer in the trenches ("French drain").

10.10 Overliner

A maximum 1.0 m thick layer of processed, durable crushed ore or sand and gravel will cover the upper HDPE liner to protect it from puncture under ore loading and to promote the effective under-drainage and collection of PLS from the ore. The design criteria for the overliner are as follows:

- Maximum particle size of 19 mm to prevent liner puncture, unless specific testing shows a larger size is acceptable.
- Durable, hard rock resistant to acid degradation.
- Permeability of at least 5×10^{-4} m/s to enhance PLS recovery and to minimize hydraulic head on the upper liner.

Within the overliner, there will be a network of pipes to collect the solution within the overliner and transfer it to either the process plant or the events pond. This system of solution recovery piping also will reduce the hydraulic head on the upper liner. As within the LDRS, the overliner will be subdivided into cells of appropriate size to allow solution management above the liner.

10.11 Leakage Criteria

Previous leakage criteria used in the Yukon were reviewed to develop the liner design. These criteria require an allowable leakage rate into the LDRS of 100 L/day averaged over a twelve-month period, with a maximum of 300 L/day averaged over a 3-month period. Initially, because of the lack of a defined area in the criteria, we used the leakage rate to define the largest detection "cell" that could be allowed in the design.

At the outset of EBA's design, it was our belief that the design criteria could be accommodated with a conventional double-lined geomembrane system, commonly used in hazardous waste impoundments in low precipitation situations. However, when we apply accepted design standards for the geomembrane as proposed by Giroud and Bonaparte (1989) and updated by Maxxon and Feeney (1993), the leakage into the LDRS required a large number of cells to be constructed to remain below the leakage criteria.

The permeability of the various layers used in the design is as follows:

- Overliner: k> 5 x 10^{-4} m/s
- Textured HDPE liners: k< 1 x 10^{-10} m/s (permeability controlled by construction defects)
- LDRS: k> 1 x 10^{-4} m/s
- *Soil liner: k< 1 x 10^{-8} m/s

**There may be some issues with the performance of soil liners in a highly acidic permeant. A review of the literature indicates that between one and two orders of magnitude increase in permeability could be expected when soil is exposed to acid. In one case, despite some buffering of the acid by the soil itself, the permeability increased two orders of magnitude with a permeant of pH 2.3. To account for this potential impact, the permeability of the soil liner has been modelled using an increased k.*

10.12 Liner Terminations

All HDPE liners will be terminated in anchor trenches. These trenches will be either permanent trenches along the perimeter berm, bench and embankment, or temporary trenches on the edges of pad extensions. The design criteria for the trenches are:

- To ensure water cannot enter drainage systems by seeping through the trench backfill.
- To provide adequate anchoring resistance to withstand the pullout forces generated by gravity and thermal expansion and contraction of the HDPE geomembranes.

10.13 Frost Protection

To protect the soil portion of the leach pad liner from frost damage, the liner will be covered with at least 4.5 m of ore and overliner prior to winter. Additional frost protection may be provided by exothermally-generated heat from the leaching process, solution heating, and natural snow insulation.

10.14 Geotechnical Instrumentation

Geotechnical instruments will be used to monitor and confirm design assumptions and performance of the solution collection system, perimeter berms and heap confining embankments. They will include permanent surface movement monuments on the system embankment crest, and piezometers within the pad foundation, overliner and confining embankment. All piezometers will be monitored regularly but will not form a requirement for continued operation of the facility should they cease to function.

10.15 Leach Pad Settlement

Leach pad settlement could potentially result from several sources – thaw of ground ice in permafrost, and subsequent consolidation of thawed soils from overburden pressure; and elastic compression of coarser-grained soils, and consolidation of fine-grained soils due to vertical loads imposed by the heap.

The design criteria for the leach pad settlement are as follows:

- Differential settlements will not compromise the integrity of the liner system.
- Tensile strains of less than five percent in the synthetic and soil liner systems will be maintained.
- Positive drainage of foundation drains and LDRS and PLS collection pipes will be maintained by "overbuilding". All drainage grades and locations will be determined with an allowance for settlements of the foundations.
- Pipe joints will be capable of sustaining settlement-induced tensions without separation.

The initial Knight-Piesold settlement estimates of up to 1.2 m under the heap leach pad have been reviewed and are considered reasonable at this time. However, permafrost conditions have likely changed significantly over the past nine years – additional boreholes will be required to verify existing conditions, and to collect data to re-assess settlement potential from thawing permafrost. It is expected that some permafrost thaw has occurred, and therefore the estimates of total and differential settlements under the leach pad might be lower than initially predicted.

Mitigative measures include the potential use of (more flexible) PVC liners in specific areas, based on the results of an additional site assessment.

11.0 LIMITATIONS

The recommendations provided in this report have provided fundamental geotechnical parameters and recommendations for one conceptual design of the heap leach pad facility for the Carmacks Copper Project. A double geomembrane liner system is being considered at the current time as the only configuration that would satisfy a permitting process in this setting. Alternative development concepts are being considered, the final design will reflect the impacts of capital and operating costs, leakage rates, stability under seismic loading, laboratory tests of the liner system, and constructability; among others. Additional geotechnical data and specific analyses will be required to finalize the design.

Respectfully submitted,
EBA Engineering Consultants Ltd.

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PROFESSIONAL ENGINEER
YUKON
R. TRIMBLE
TERRITORY

May 5/05

Richard Trimble, P. Eng
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PERMIT TO PRACTICE
Signature
Date May 5/05 EBA Engineering
PERMIT NUMBER: PP003
Association of Professional Engineers of Yukon

This report incorporates and is subject to these "General Conditions".

1.0 USE OF REPORT AND OWNERSHIP

This geotechnical report pertains to a specific site, a specific development and a specific scope of work. It is not applicable to any other sites nor should it be relied upon for types of development other than that to which it refers. Any variation from the site or development would necessitate a supplementary geotechnical assessment.

This report and the recommendations contained in it are intended for the sole use of EBA's client. EBA does not accept any responsibility for the accuracy of any of the data, the analyses or the recommendations contained or referenced in the report when the report is used or relied upon by any party other than EBA's client unless otherwise authorized in writing by EBA. Any unauthorized use of the report is at the sole risk of the user.

This report is subject to copyright and shall not be reproduced either wholly or in part without the prior, written permission of EBA. Additional copies of the report, if required, may be obtained upon request.

2.0 NATURE AND EXACTNESS OF SOIL AND ROCK DESCRIPTIONS

Classification and identification of soils and rocks are based upon commonly accepted systems and methods employed in professional geotechnical practice. This report contains descriptions of the systems and methods used. Where deviations from the system or method prevail, they are specifically mentioned.

Classification and identification of geological units are judgmental in nature as to both type and condition. EBA does not warrant conditions represented herein as exact, but infers accuracy only to the extent that is common in practice.

Where subsurface conditions encountered during development are different from those described in this report, qualified geotechnical personnel should revisit the site and review recommendations in light of the actual conditions encountered.

3.0 LOGS OF TEST HOLES

The test hole logs are a compilation of conditions and classification of soils and rocks as obtained from field observations and laboratory testing of selected samples. Soil and rock zones have been interpreted. Change from one geological zone to the other, indicated on the logs as a distinct line, can be, in fact, transitional. The extent of transition is interpretive. Any circumstance which requires precise definition of soil or rock zone transition elevations may require further investigation and review.

4.0 STRATIGRAPHIC AND GEOLOGICAL INFORMATION

The stratigraphic and geological information indicated on drawings contained in this report are inferred from logs of test holes and/or soil/rock exposures. Stratigraphy is known only at the locations of the test hole or exposure. Actual geology and stratigraphy between test holes and/or exposures may vary from that shown on these drawings. Natural variations in geological conditions are inherent and are a function of the historic environment. EBA does not represent the conditions illustrated as exact but recognizes that variations will exist. Where knowledge of more precise locations of geological units is necessary, additional investigation and review may be necessary.

5.0 SURFACE WATER AND GROUNDWATER CONDITIONS

Surface and groundwater conditions mentioned in this report are those observed at the times recorded in the report. These conditions vary with geological detail between observation sites; annual, seasonal and special meteorologic conditions; and with development activity. Interpretation of water conditions from observations and records is judgmental and constitutes an evaluation of circumstances as influenced by geology, meteorology and development activity. Deviations from these observations may occur during the course of development activities.

6.0 PROTECTION OF EXPOSED GROUND

Excavation and construction operations expose geological materials to climatic elements (freeze/thaw, wet/dry) and/or mechanical disturbance which can cause severe deterioration. Unless otherwise specifically indicated in this report, the walls and floors of excavations must be protected from the elements, particularly moisture, desiccation, frost action and construction traffic.

7.0 SUPPORT OF ADJACENT GROUND AND STRUCTURES

Unless otherwise specifically advised, support of ground and structures adjacent to the anticipated construction and preservation of adjacent ground and structures from the adverse impact of construction activity is required.

8.0 INFLUENCE OF CONSTRUCTION ACTIVITY

There is a direct correlation between construction activity and structural performance of adjacent buildings and other installations. The influence of all anticipated construction activities should be considered by the contractor, owner, architect and prime engineer in consultation with a geotechnical engineer when the final design and construction techniques are known.

9.0 OBSERVATIONS DURING CONSTRUCTION

Because of the nature of geological deposits, the judgmental nature of geotechnical engineering, as well as the potential of adverse circumstances arising from construction activity, observations during site preparation, excavation and construction should be carried out by a geotechnical engineer. These observations may then serve as the basis for confirmation and/or alteration of geotechnical recommendations or design guidelines presented herein.

10.0 DRAINAGE SYSTEMS

Where temporary or permanent drainage systems are installed within or around a structure, the systems which will be installed must protect the structure from loss of ground due to internal erosion and must be designed so as to assure continued performance of the drains. Specific design detail of such systems should be developed or reviewed by the geotechnical engineer. Unless otherwise specified, it is a condition of this report that effective temporary and permanent drainage systems are required and that they must be considered in relation to project purpose and function.

11.0 BEARING CAPACITY

Design bearing capacities, loads and allowable stresses quoted in this report relate to a specific soil or rock type and condition. Construction activity and environmental circumstances can materially change the condition of soil or rock. The elevation at which a soil or rock type occurs is variable. It is a requirement of this report that structural elements be founded in and/or upon geological materials of the type and in the condition assumed. Sufficient observations should be made by qualified geotechnical personnel during construction to assure that the soil and/or rock conditions assumed in this report in fact exist at the site.

12.0 SAMPLES

EBA will retain all soil and rock samples for 30 days after this report is issued. Further storage or transfer of samples can be made at the client's expense upon written request, otherwise samples will be discarded.

13.0 STANDARD OF CARE

Services performed by EBA for this report have been conducted in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practising under similar conditions in the jurisdiction in which the services are provided. Engineering judgement has been applied in developing the conclusions and/or recommendations provided in this report. No warranty or guarantee, express or implied, is made concerning the test results, comments, recommendations, or any other portion of this report.

14.0 ENVIRONMENTAL AND REGULATORY ISSUES

Unless stipulated in the report, EBA has not been retained to investigate, address or consider and has not investigated, addressed or considered any environmental or regulatory issues associated with development on the subject site.

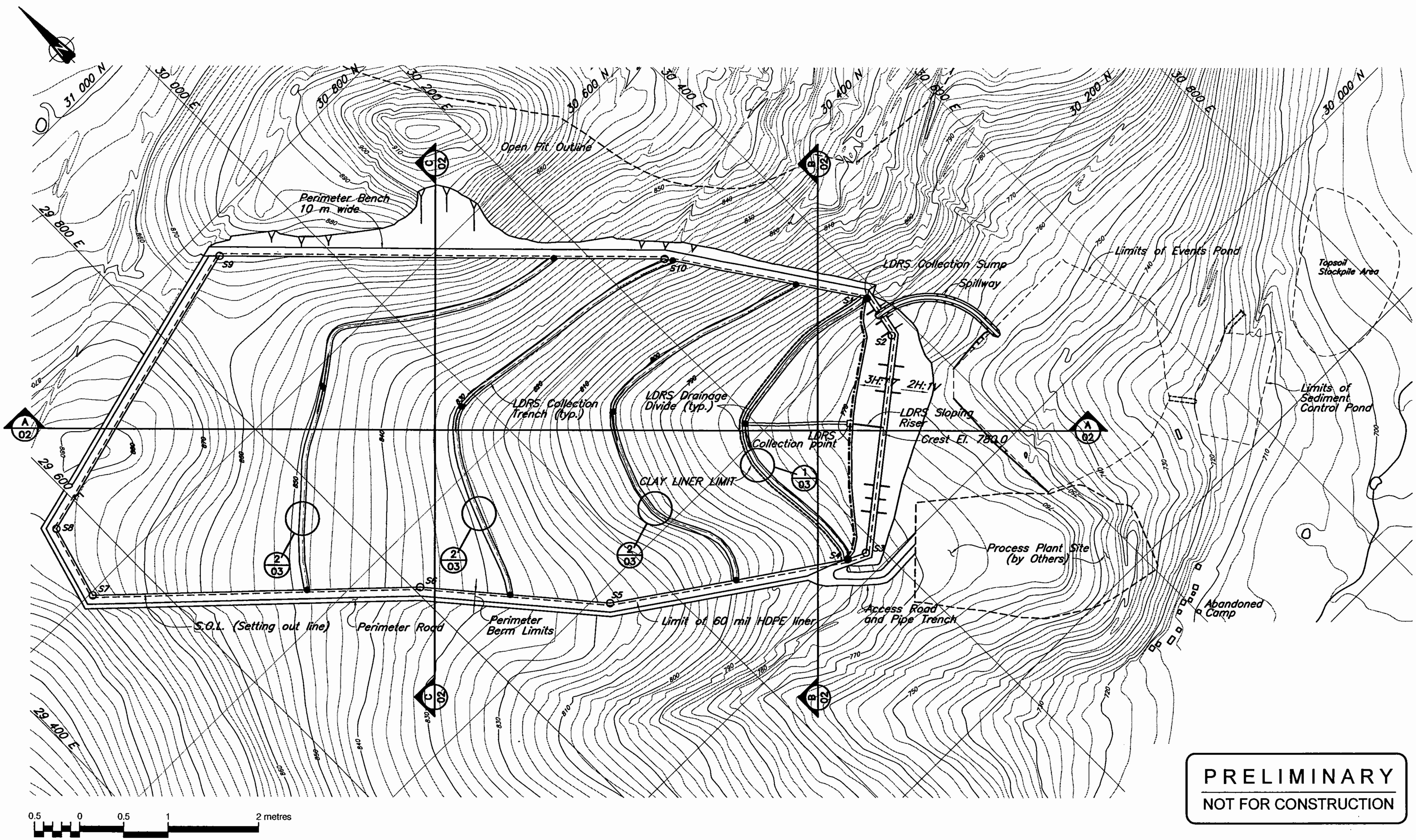
15.0 ALTERNATE REPORT FORMAT

Where EBA submits both electronic file and hard copy versions of reports, drawings and other project-related documents and deliverables (collectively termed EBA's instruments of professional service), the Client agrees that only the signed and sealed hard copy versions shall be considered final and legally binding. The hard copy versions submitted by EBA shall be the original documents for record and working purposes, and, in the event of a dispute or discrepancies, the hard copy versions shall govern over the electronic versions. Furthermore, the Client agrees and waives all future right of dispute that the original hard copy signed version archived by EBA shall be deemed to be the overall original for the Project.

The Client agrees that both electronic file and hard copy versions of EBA's instruments of professional service shall not, under any circumstances, no matter who owns or uses them, be altered by any party except EBA. The Client warrants that EBA's instruments of professional service will be used only and exactly as submitted by EBA.

The Client recognizes and agrees that electronic files submitted by EBA have been prepared and submitted using specific software and hardware systems. EBA makes no representation about the compatibility of these files with the Client's current or future software and hardware systems.

FIGURES



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DATE: APRIL 3/05
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PROJECT No.: 1200133
ACAD FILENAME: 1200133R01A.dwg

PERMIT

SEAL

CARMACKS COPPER PROJECT
Carmacks, Yukon

HEAP LEACH PAD

SITE PLAN

REVISION ISSUE
A

DRAWING No.
1200133-01

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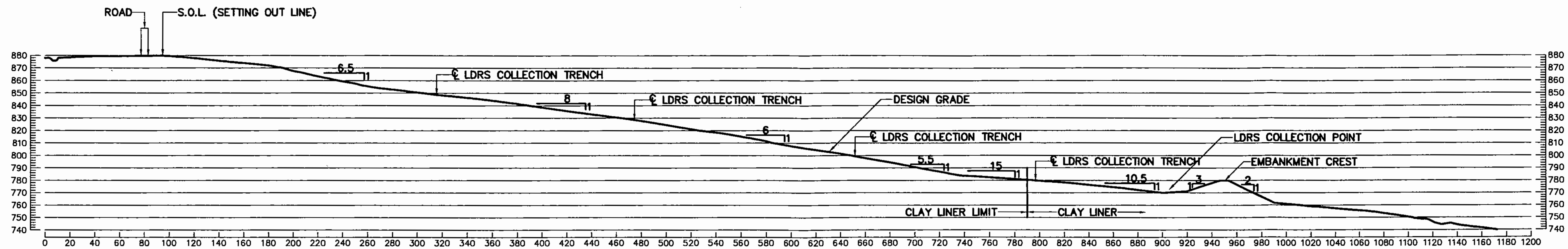

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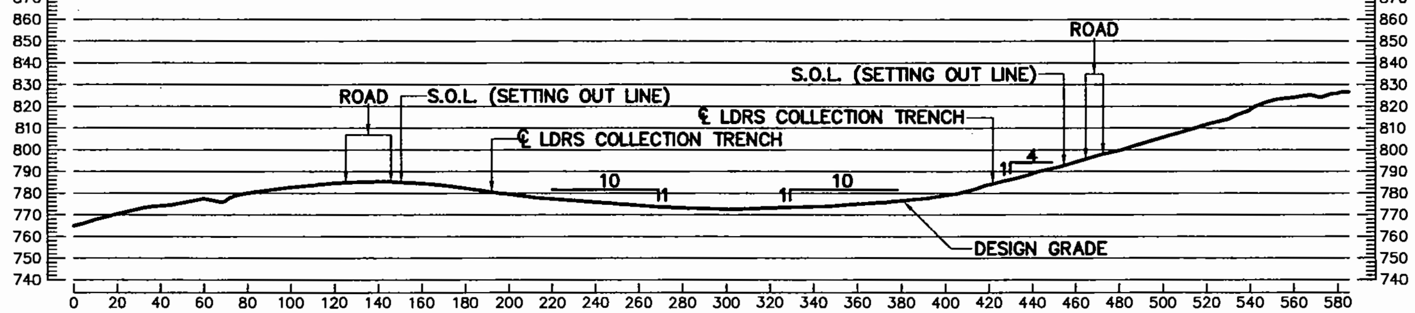

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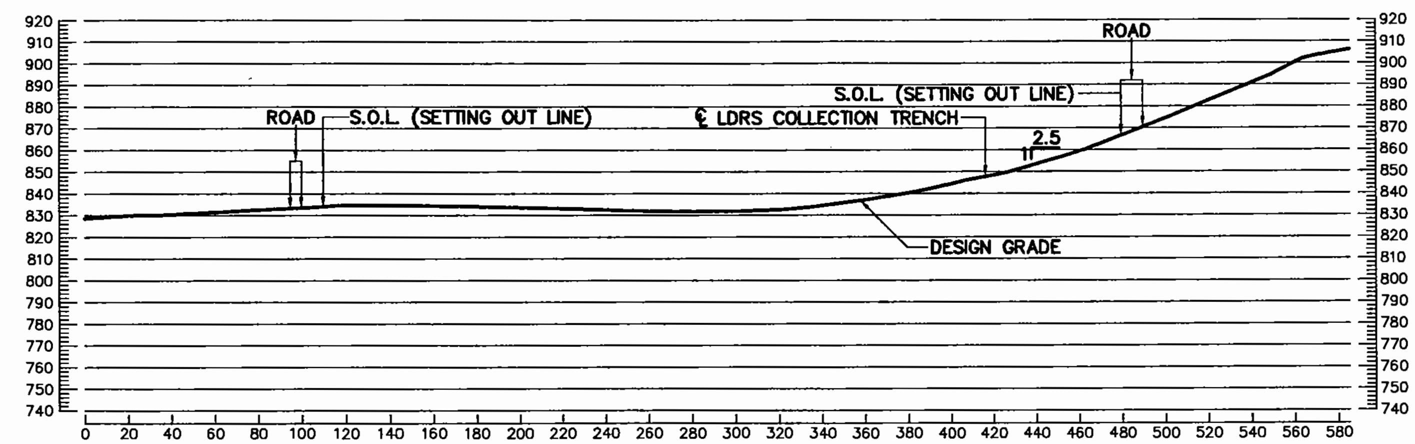

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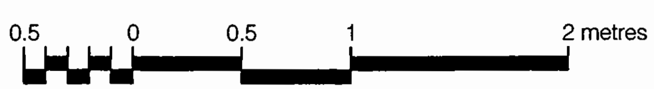

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Carmacks, Yukon

HEAP LEACH PAD

SECTIONS

REVISION ISSUE: A

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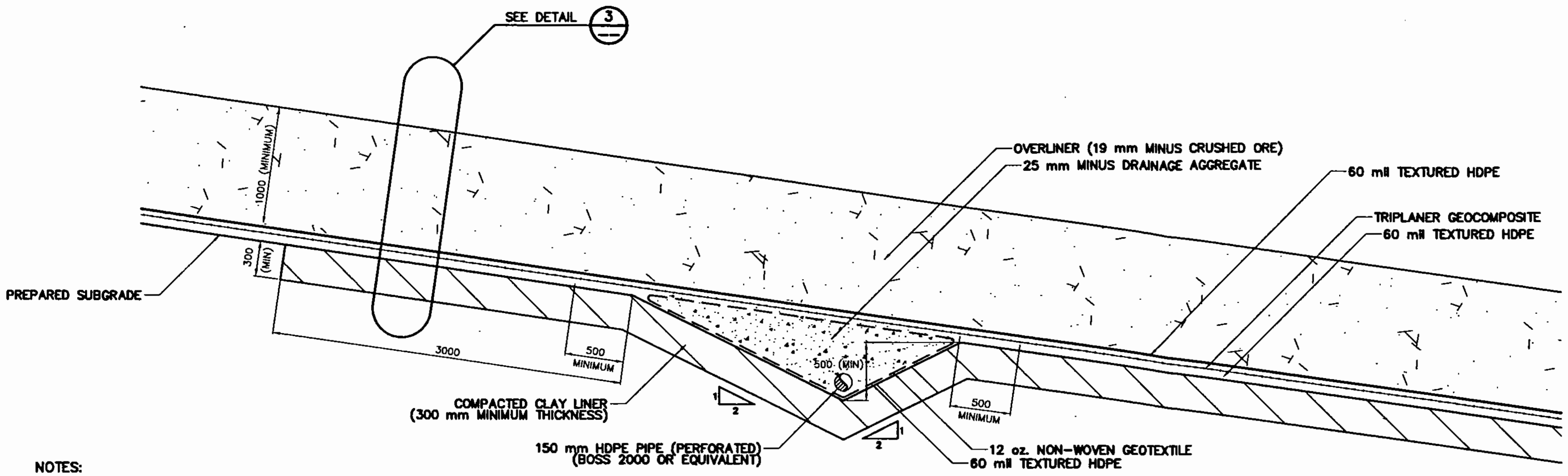
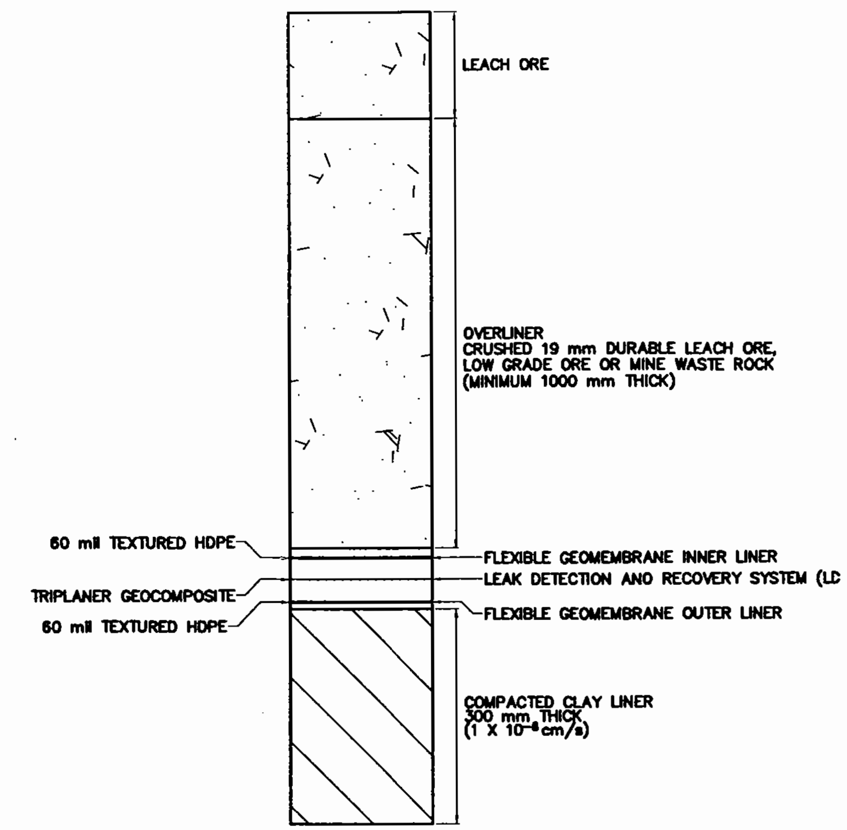


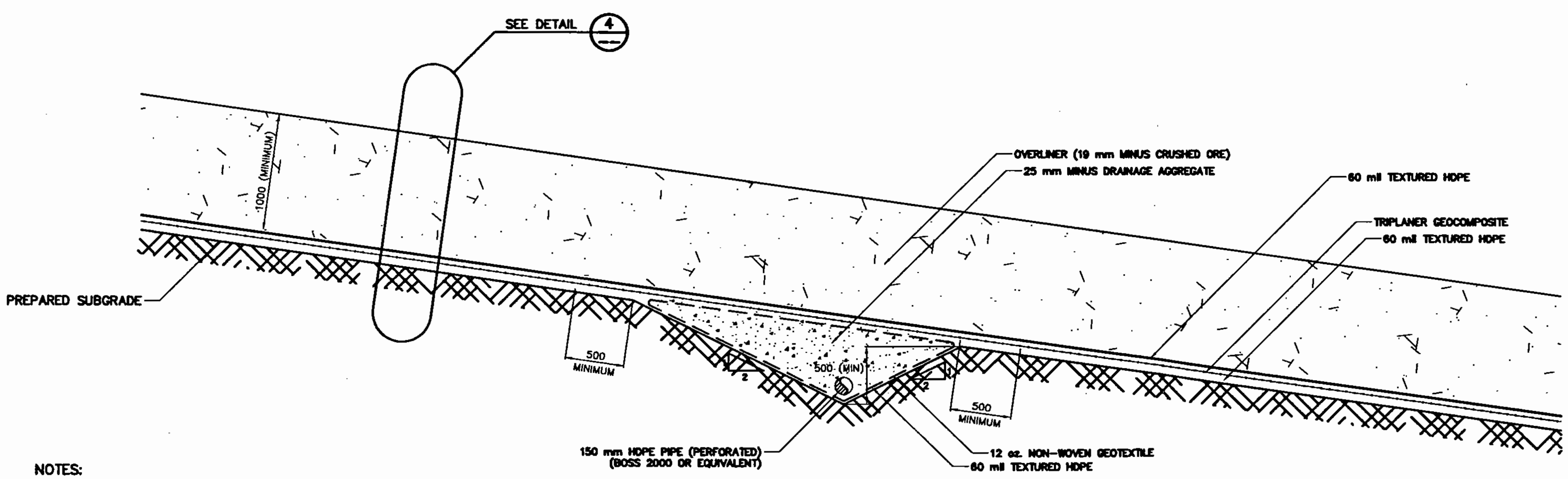
NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. MATERIAL AND LINER LAYER ARE SHOWN SEPARATED FOR CLARITY.
3. CROSS-SLOPE VARIES BETWEEN 3 (HORIZONTAL) AND 15 (HORIZONTAL) TO 1 (VERTICAL) APPROXIMATELY.

LOWER LDRS COLLECTION DITCH 1/01



TYPICAL LINER SYSTEM – LOWER LDRS COLLECTION DITCH 3
SCALE: N.T.S.



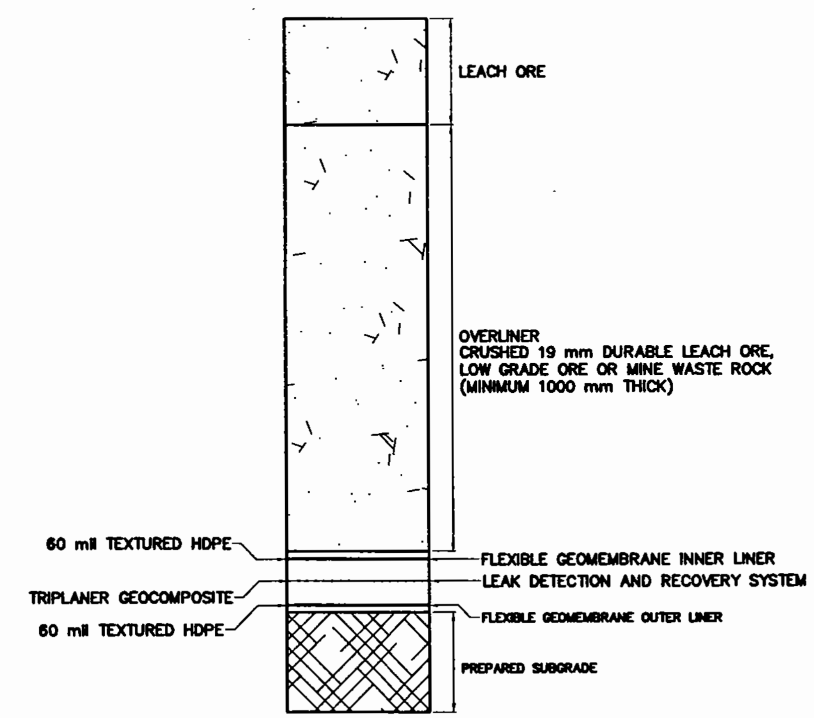
NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. MATERIAL AND LINER LAYER ARE SHOWN SEPARATED FOR CLARITY.
3. CROSS-SLOPE VARIES BETWEEN 3 (HORIZONTAL) AND 15 (HORIZONTAL) TO 1 (VERTICAL) APPROXIMATELY.

UPPER LDRS COLLECTION DITCH 2/01

PRELIMINARY
NOT FOR CONSTRUCTION

0.5 0 0.5 1 2 metres



TYPICAL LINER SYSTEM – UPPER LDRS COLLECTION DITCH 4
SCALE: N.T.S.

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CARMACKS COPPER PROJECT
Carmacks, Yukon

HEAP LEACH PAD

SECTIONS and DETAILS

REVISION ISSUE: A

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APPENDIX A

High Flow Geocomposite

TENAX TENDRAIN™ CLASS 1

AASHTO Class 1 Highly UV Stable Double-Sided Geocomposite

TENDRAIN™ C1 geocomposite is comprised of a tri-axial geonet structure consisting of thick supporting ribs with diagonally placed top and bottom ribs and with thermally bonded, nonwoven highly UV resistant, AASHTO Class 1 Ultra-Vera™ geotextiles on both sides. The product is capable of providing high Transmissivity in a soil environment under high normal loads and will have properties conforming to the values and test methods listed below.

Property	Test Methods	Units	Value	Qualifier	Test Frequency
Resin					
• Density	ASTM D 1505	g/cm3	0.94	MAV	lot
• Melt Flow Index	ASTM D 1238	g/10min	1.0	MAX	lot
Geonet Core[4]					
Structure	Tri-axial				
• Tensile Strength - MD	ASTM D 4595	lb/ft (kN/m)	1200 (17.5)	MAV	50,000 sf
• Creep Reduction Factor[1] @ 20°C	GRI-GC8	-	1.2		
• Retained thickness[2] @ 40°C	ASTM D1621	%	65		
• Thickness[3]	ASTM D 5199	mil (mm)	300 (7.6)	MAV	50,000 sf
• Carbon Black	ASTM D 4218	%	2-3	Range	50,000 sf
Top Filter Geotextile[4,5]					
• U.V. Resistance (500 hrs)	ASTM G 154	%	95	MAV	Per formula
• Color			Orange		
• Serviceability Class	AASHTO M-288		Class 1		
• Grab Tensile	ASTM D 4632	lbs (N)	202 (900)	MARV	100,000 sf
• Tear Strength	ASTM D 4533	lbs (N)	79 (350)	MARV	100,000 sf
• Puncture Resistance	ASTM D 4833	lbs (N)	79 (350)	MARV	100,000 sf
• CBR Puncture Strength	ASTM D 6241	lbs (N)	449 (2000)	MARV	100,000 sf
• AOS	ASTM D 4751	US Std. Sieve (mm)	80 (0.18)	MaxARV	500,000 sf
• Permittivity	ASTM D 4491	Sec-1	0.5	MARV	500,000 sf
Bottom Friction Geotextile[4,5]					
• U.V. Resistance (500 hrs)	ASTM G 154	%	95	MAV	Per formula
• Color			Orange		
• Serviceability Class	AASHTO M-288		Class 1		
• Grab Tensile	ASTM D 4632	lbs (N)	202 (900)	MARV	100,000 sf
• Tear Strength	ASTM D 4533	lbs (N)	79 (350)	MARV	100,000 sf
• Puncture Resistance	ASTM D 4833	lbs (N)	79 (350)	MARV	100,000 sf
• CBR Puncture Strength	ASTM D 6241	lbs (N)	449 (2000)	MARV	100,000 sf
Geocomposite					
• Peel Adhesion[6] - MD	F904 Modified	lb/in (g/in)	1.0 (454)	MAV	100,000 sf
• Labeling	Product code, geotextile type, roll dimensions, finished product lot and roll number.				
Hydraulic Behavior of Geocomposite					
• Transmissivity[7] - MD					
Gradient / Load			25,000 psf (1200 kPa)		
0.33	ASTM D 4716		5.0*10-4		
0.1	GRI - GC8	m2/sec	7.6*10-4	MAV	200,000 sf
0.02			1.0*10-3		

Qualifiers:
MARV = Minimum Average Roll Value (MARV)
MAV = Minimum Average Value
MAX = Maximum Value
MaxARV = Maximum average roll value

NOTES:
1. Creep Reduction Factor is based on 10,000 hour test duration, extrapolated to 30 years and using a compressive load of 25,000 psf.
2. Retained thickness is measured through 5,000 hours duration, under a compressive load of 15,000 psf and temperature of 400C.
3. Thickness measured by manufacturer per ASTM D5199 with a 2.22 in. diameter presser foot and 2.9 psi pressure.
4. Geotextile and geonet properties listed are prior to lamination.
5. Geotextiles meet ASSHTO Standard Specification M 288-00 strength requirements of class 1, and filter geotextile meets the highest filter requirements.
6. Peel adhesion is tested by the manufacturer per modified ASTM F904, with a 2-inch wide (5 longitudinal ribs) by 10-inch long strip. The geotextile bonded to either side of the geonet is pulled apart at a peeling rate of 12 in/min., for at least 4 inches of peeling distance. The reported value for each laminated side is the average of the "peak" values from 5 tested sam ples. The 5 samples are cut evenly distributed along the roll width with a 1-foot margin from both edges of the roll.
7. Geocomposite transmissivity measured by manufacturer per ASTM D4716 with testing boundary conditions as follows: steel plate / uniform sand / geo composite / 60 mil HDPE geomembrane / steel plate, and sealing period of 100 hours according to GRI-GC8.

TENAX
Engineered for Life

Sales/Technical Service
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• 410.522.7000 (Phone) • 410.522.7015 (Fax) • 800.356.8495 (Toll-Free)
www.tenaxus.com E-Mail:info@us.tenax.com

2/2004 DRSAM003

TENAX® Corporation

TENAX TENDRAIN GEONET (TD7)

HIGH PERFORMANCE TRI-PLANAR GEONET

TENAX TD7 is an innovative high performance tri-planar geonet consisting of thick supporting ribs with diagonally placed top and bottom ribs. The three sets of intersecting strands form unique flow conduits that provide extremely high flow capacity, high compressive resistance and enhanced tensile properties. TENAX TD7 is manufactured from the extrusion of high density polyethylene resin and carbon black. TENAX TD7 is inert to chemical and biological attack and is stabilized against UV degradation.

Typical applications:

- leachate collection/detection systems in landfills - landfill cap drainage layer - gas venting media

TECHNICAL CHARACTERISTICS	TEST METHOD	UNIT	Value	QUALIFIER
MD TENSILE STRENGTH	ASTM D4595	lb/ft (kN/m)	1200 (17.5)	a, Note 1
COMPRESSION BEHAVIOR				
% retained thickness				
@50,000 psf (short term)	ASTM D1621	%	50	a, Note 2
@25,000 psf (10,000 hours)		%	65	
Creep Reduction Factor	GRI-GC8	-	1.2	Note 3
DENSITY	ASTM D1505	g/cm^3	0.94	a
MELT FLOW INDEX	ASTM D1238	g/10 min	1.0	b
CARBON BLACK CONTENT	ASTM D4218	%	2.0 ~ 3.0	c
THICKNESS	ASTM D5199	mils (mm)	300 (7.6)	a, Note 4
TRANSMISSIVITY-MD @ i = 0.1 at 15,000 psf	ASTM D4716	m^2/sec	4.0×10^{-3}	a, Note 5
ROLL WIDTH	-	ft	6.7	c, Note 6
ROLL LENGTH	-	ft	200	c, Note 6
ROLL AREA	-	ft^2	1340	c

QUALIFIER:
a) Minimum Average Value b) Typical value c) Maximum

NOTES:

1. Tensile properties tested by manufacturer every 50,000 square feet of product per ASTM D4595 with a specimen width of 8.0 in. and cross-head speed of 0.4 in/min.
2. Short term compression behavior tested by manufacturer every 50,000 square feet of product per ASTM D1621 with a 4 in.x4 in. specimen and a constant rate of strain of 0.04 in./min.
3. Creep Reduction Factor is based on 10,000-hour test duration, extrapolated to 30 years and using a compressive load of 25,000 psf.
4. Thickness measured by manufacturer every 50,000 square feet of product per ASTM D5199 with a 2.22 in. diameter presser foot and 2.9 psi pressure.
5. Geonet transmissivity measured by manufacturer every 200,000 square feet of product as per ASTM D4716 with testing boundary conditions as follows: steel plate / geonet / 60 mil HDPE geomembrane / steel plate and seating period of 100 hours.
6. Roll dimensions are measured at the time of manufacture.

TENAX Engineered for Life
Sales/Technical Service
4800 East Monument Street • Baltimore, Maryland 21205 • 410.522.7000 • 410.522.7015 (fax) • 800.356.8495
Manufacturing/Quality Assurance
200 Miller Sellers Drive • Evergreen, Alabama 36401
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Technical Note

Determining Creep Reduction Factor for Tri-Planar Geonets

In this technical note, the Creep Reduction Factor, RF_{cr} is determined to account for the creep that takes place in a tri-planar geonet between the seating period of 100 hours and two time periods. The first case is to represent the initial phase of waste placement, and the second case is to represent the stage of the landfill prior to final closure where waste thickness reaches its maximum height. The reduction factor for creep is then applied to the transmissivity value determined from performance test with a 100-hour seating to account for the creep deformation.

The attached Figure shows the results of 10,000-hour compressive creep tests on tri-planar geonets under normal loads of 2,000psf and 25,000psf respectively. The creep curve under 2,000psf load is used to determine the creep reduction fact for the first case (initial phase of waste placement), and the creep curve under 25,000psf is used for the second case (landfill prior to final closure). The following Equation[1] is used to determine the Creep Reduction Factor, RF_{cr}

$$RF_{cr} = \left[\frac{(t_{CO}/t_{virgin}) - (1 - n_{virgin})}{(t_{cr}/t_{virgin}) - (1 - n_{virgin})}\right]^3$$

Where:

T_{CO} = thickness after load application for 100 hours

t_{virgin} = initial thickness under 2.9 psi (ASTM D5199)

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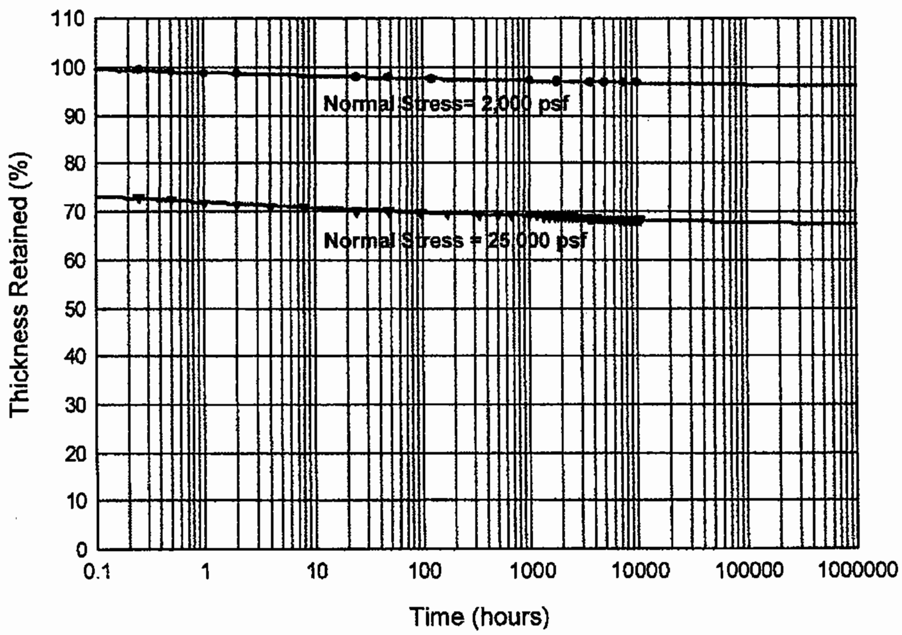
[1] Giroud, J.P., Zhao, A., and Richardson, G., 2000, "Effect of Thickness Reduction on Geosynthetic Hydraulic Transmissivity", Special Issue on Liquid Collection Systems, *Geosynthetics International*, Vol.7, Nos. 4-6, pp. 433-452.

Compressive Creep Data on Tendrain



t_{CR}= thickness at the time period of interest

n_{virgin} = initial porosity

<u>Case 1: Using the compressive creep curve under 2000psf load (Initial Phase of Waste Placement):</u>

t_{CO} = t_{100h} = 7.62*0.975 = 7.43 mm

t_{vingin} = 7.62 mm (300 mils)

t_{CR} (use 4,000 hours) = t_{4000h} = 7.62*0.965 = 7.35 mm

n_{virgin} = 0.75 (calculated)

Substituting these values in the above Equation:

RC_{CR} = **1.05**

<u>Case 2: Using the compressive creep curve under 25,000psf load (Landfill Prior to Final Closure):</u>

t_{CO} = t_{100h} = 7.62*0.7 = 5.32 mm

t_{vingin} = 7.62 mm (300 mils)

t_{CR} (use 17,000 hours) = t_{17000h} = 7.62*0.675 = 5.14 mm

n_{virgin} = 0.75 (calculated)

Substituting these values in the above Equation:

RC_{CR} = **1.20** (for landfill prior to final closure)

GSE

Product Data Sheet

GSE STANDARD PRODUCTS

GSE Nonwoven Geotextiles

GSE Nonwoven Geotextiles is a family of polypropylene, staple fiber, nonwoven needle punched geotextiles. Manufactured using an advanced manufacturing and quality system, these products are the most uniform and consistent nonwoven needle punched geotextile currently available in the industry. GSE combines a fiber selection and approval system with in-line quality control and a state-of-the-art laboratory to ensure that every roll shipped meets customer specifications. The company has performed extensive performance testing to evaluate suitability of its nonwovens for various applications. GSE Nonwoven Geotextiles are available in a range of weights to meet your specific project needs. *These product specifications meet or exceed GRI GT12.*

Product Specifications

TESTED PROPERTY	TEST METHOD	FREQUENCY	NW4	NW6	NW8	NW10	NW12	NW16
Product Code			GEO 0408002	GEO 0608002	GEO 0808002	GEO 1008002	GEO 1208002	GEO 1608002
Mass per Unit Area, oz/yd² (g/m²)	ASTM D 5261	90,000 ft²	4 (135)	6 (200)	8 (270)	10 (335)	12 (405)	16 (540)
Grab Tensile Strength, lb (N)	ASTM D 4632	90,000 ft²	120 (530)	170 (755)	220 (975)	260 (1,155)	320 (1,420)	390 (1,735)
Grab Elongation, %	ASTM D 4632	90,000 ft²	50	50	50	50	50	50
Puncture Strength, lb (N)	ASTM D 4833	90,000 ft²	60 (265)	90 (395)	120 (525)	165 (725)	190 (835)	240 (1,055)
Trapezoidal Tear Strength, lb (N)	ASTM D 4533	90,000 ft²	50 (220)	70 (310)	95 (420)	100 (445)	125 (555)	150 (665)
Apparent Opening Size, Sieve No. (mm)	ASTM D 4751	540,000 ft²	70 (0.212)	70 (0.212)	80 (0.180)	100 (0.150)	100 (0.150)	100 (0.150)
Permittivity, sec⁻¹	ASTM D 4491	540,000 ft²	1.50	1.50	1.50	1.20	0.80	0.70
Permeability, cm/sec	ASTM D 4491	540,000 ft²	0.22	0.30	0.30	0.30	0.29	0.27
Water Flow Rate, gpm/ft² (l/min/m²)	ASTM D 4491	540,000 ft²	120 (4,885)	110 (4,480)	110 (4,480)	85 (3,460)	60 (2,440)	50 (2,035)
UV Resistance (% retained after 500 hours)	ASTM D 4355	per formulation	70	70	70	70	70	70
Roll Length, ft (m)			600 (182)	600 (182)	600 (182)	300 (91)	300 (91)	300 (91)
Roll Width, ft (m)			15 (4.6)	15 (4.6)	15 (4.6)	15 (4.6)	15 (4.6)	15 (4.6)
Roll Area, ft² (m²)			9,000 (836)	9,000 (836)	9,000 (836)	4,500 (418)	4,500 (418)	4,500 (418)

NOTES:

- The property values listed are in weaker principal direction. All values listed are Minimum Average Roll Values (MARV) except apparent opening size in mm and UV resistance. Apparent opening size (mm) is a Maximum Average Roll Value. UV is a typical value.

DS037 R04/17/03

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Asia/Pacific	GSE Lining Technology Company Ltd.	Bangkok, Thailand		66-2-937-0091	Fax: 66-2-937-0097

GSE Nonwoven Geotextiles

GSE NONWOVEN NEEDLEPUNCHED GEOTEXTILES

GSE nonwoven needlepunched geotextiles are manufactured at our state-of-the-art needlepunching plant in Kingstree, South Carolina. The plant has been certified according to ISO 9002 quality system.

GSE manufactures 4 to 20 oz/yd^2 geotextiles designated as NW4, NW6, NW8, NW10, NW12, NW16 and NW20. The most common function and usage of these products is indicated in Figure 1 below. However, the actual selection of the product depends on the specific needs of a project. For example, while NW16 is commonly used for geomembrane protection, it can also be used in filtration and separation because of specific design needs.

ASPHALT OVERLAY

It is common for asphalt pavements to crack prematurely because of design flaws, material limitations or environmental reasons. A fresh layer of asphalt is the most common remedy for this problem. However, reflective cracking – the propagation of cracks from old cracked surface into the new surface – limits the performance of the fresh asphalt overlay. To prevent reflective cracking, a nonwoven needlepunched geotextile must be placed above the cracked surface before placing the new layer. The geotextile works as a sealant and stress-absorbing layer. There are comprehensive design and construction methods available for this purpose. GSE NW4 is ideal for preventing reflective cracking. Figure 2 shows the use of geotextiles to prevent reflective cracking.

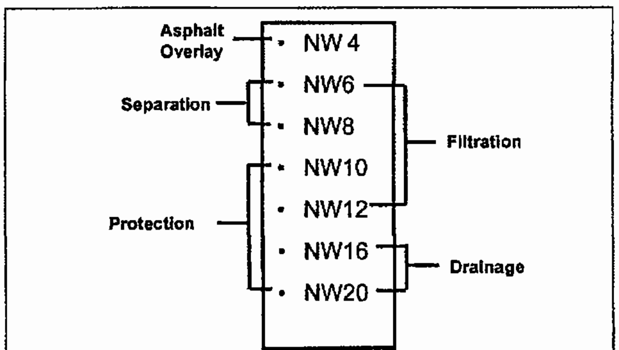

Figure 1 - Most common usage of GSE nonwoven needlepunched geotextiles

SEPARATION

Intermixing of two dissimilar materials always leads to the deterioration of their engineering performance. For example, contamination of aggregate by fine particles always leads to a decrease in the permeability of the aggregate. The separation function refers to the use of geotextiles to maintain physical separation between two adjacent materials. This is demonstrated graphically in Figure 3. GSE geotextiles are ideal for this purpose because of their strength, durability, flexibility and a highly porous structure.

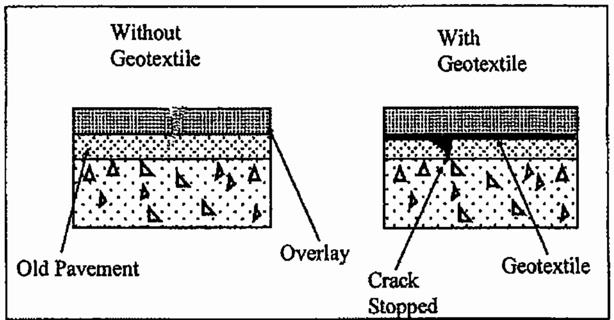

Figure 2 - Use of GSE nonwoven geotextiles to prevent reflective cracking

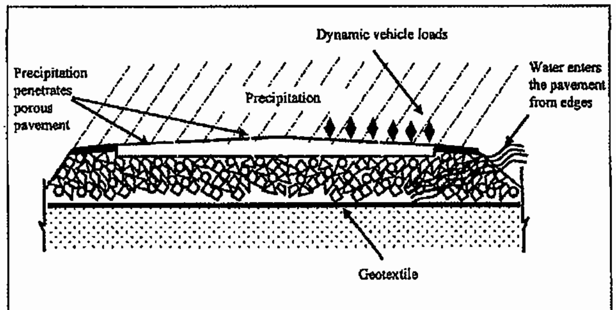

Figure 3 - Use of GSE geotextiles for separation

FILTRATION

When used as filters, GSE nonwoven needlepunched geotextiles allow the passage of liquid while preventing the loss of soil particles. GSE offers a range of products with opening size to meet filtration needs for different types of soils. For relatively coarse soils, lower mass products – NW4, NW6 and NW8 are recommended. For fine soil particles, it is better to use heavier mass geotextiles such as NW10, NW12 or NW16. Depending on the needs of a specific project, GSE has a geotextile available which will perform the intended design function.

- Continued -

PROTECTION

Geomembrane liners are very sensitive to damage and puncture during construction as well as over the life of a project. Therefore, geomembranes must be protected both from top and bottom. GSE nonwoven needlepunched geotextiles are ideal for this purpose because of their cushioning ability. Depending on soil size and overburden loads, one of the many geotextiles offered by GSE can be selected to ensure that geomembrane performance is not compromised.

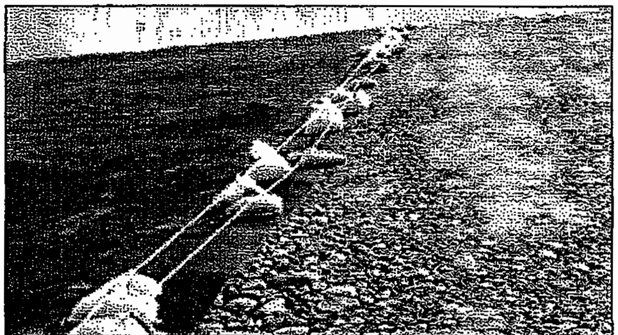

Figure 4 - Use of GSE geotextiles for geomembrane protection

DRAINAGE

Liners are used typically to prevent infiltration of liquids into environmentally sensitive areas. In certain cases, trapped gases and vapors must be vented to prevent uplifting of the liner. GSE nonwoven needlepunched geotextiles are ideal for gas and vapor drainage from under the liners. The high porosity of GSE geotextiles facilitates drainage while providing added benefit of cushion for the liners. A general configuration of this application is provided in Figure 5 below.

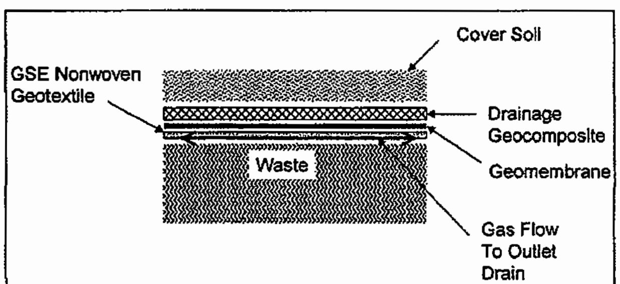

Figure 5 - Use of GSE geotextiles for geomembrane protection

ADDITIONAL INFORMATION

If you have an upcoming project please give us a call. We will prepare a scope of work outlining the necessary materials and construction support for the effective completion of a specific project. GSE has a staff of product managers, estimators, and project managers to assist with your project from conception through project completion.

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Asia/Pacific	GSE Lining Technology Company Ltd.	Bangkok, Thailand		66-2-937-0091	Fax: 66-2-937-0097

This application sheet is also available on our website at:

www.gseworld.com

GSE

Product Data Sheet

GSE STANDARD PRODUCTS

GSE HD Textured

GSE HD Textured is the textured version of GSE HD. It is a high quality, high density polyethylene (HDPE) geomembrane with one or two coextruded, textured surfaces, and consisting of approximately 97.5% polyethylene, 2.5% carbon black and trace amounts of antioxidants and heat stabilizers; no other additives, fillers or extenders are used. The resin used is specially formulated, virgin polyethylene and is designed specifically for flexible geomembrane applications. GSE HD Textured has excellent resistance to UV radiation and is suitable for exposed conditions. This product allows projects with greater slopes to be designed since frictional characteristics are enhanced. *These product specifications meet or exceed GRI GM13.*

Product Specifications

TESTED PROPERTY	TEST METHOD	FREQUENCY	MINIMUM VALUE				
Product Code			HDT 030A000	HDT 040A000	HDT 060A000	HDT 080A000	HDT 100A000
Thickness, mil (mm) or per project specs	ASTM D 5994	every roll	27 (0.69)	36 (0.91)	54 (1.4)	72 (1.8)	90 (2.3)
Density, g/cm^3	ASTM D 1505	200,000 lb	0.94	0.94	0.94	0.94	0.94
Tensile Properties (each direction)[1]	ASTM D 6693, Type IV	20,000 lb					
Strength at Break, lb/in-width (N/mm)	Dumbell, 2 ipm		45 (8)	60 (11)	90 (16)	120(21)	150 (27)
Strength at Yield, lb/in-width (N/mm)			63 (11)	84 (15)	130 (23)	173 (30)	216 (38)
Elongation at Break, %	G.L. = 2.0 in (51 mm)		150	150	150	150	150
Elongation at Yield, %	G.L. = 1.3 in (33 mm)		13	13	13	13	13
Tear Resistance, lb (N)	ASTM D 1004	45,000 lb	21 (93)	28 (125)	42 (187)	56 (249)	70 (311)
Puncture Resistance, lb (N)	ASTM D 4833	45,000 lb	54 (240)	72 (320)	108 (480)	144 (641)	150 (667)
Carbon Black Content, %	ASTM D 1603	20,000 lb	2.0	2.0	2.0	2.0	2.0
Carbon Black Dispersion	ASTM D 5596	45,000 lb	+Note 1	+Note 1	+Note 1	+Note 1	+Note 1
Asperity Height	GRI GM 12	second roll	+Note 2	+Note 2	+Note 2	+Note 2	+Note 2
Notched Constant Tensile Load[2], hrs	ASTM D 5397, Appendix	200,000 lb	400	400	400	400	400
REFERENCE PROPERTY	**TEST METHOD**	**FREQUENCY**	**NOMINAL VALUE**				
Oxidative Induction Time, minutes	ASTM D 3895, 200° C; O_2, 1 atm	200,000 lb	>100	>100	>100	>100	>100
Roll Length (approximate), ft (m)	Standard Textured		830 (253)	700 (213)	520 (158)	400 (122)	330 (101)
Roll Width, ft (m)			22.5 (6.9)	22.5 (6.9)	22.5 (6.9)	22.5 (6.9)	22.5 (6.9)
Roll Area, ft^2 (m^2)			18,674 (1,735)	15,750 (1,463)	11,700 (1,087)	9,000 (836)	7,425 (690)

NOTES:

- +Note 1: Dispersion only applies to near spherical agglomerates. 9 of 10 views shall be Category 1 or 2. No more than 1 view from Category 3.
- +Note 2: 10 mil average. 8 of 10 readings ≥7 mils. Lowest individual ≥ 5 mils.
- GSE HD Standard Textured is available in rolls weighing about 4,000 lb (1,800 kg).
- [1]The combination of stress concentrations due to coextrusion texture geometry and the small specimen size results in large variation of test results. Therefore, these tensile properties are minimum average values.
- [2]NCTL for HD Textured is conducted on representative smooth membrane samples.
- All GSE geomembranes have dimensional stability of ±2% when tested with ASTM D 1204 and LTB of <-77° C when tested with ASTM D 746.

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This product data sheet is also available on our website at:

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Application Sheet

GSE HD Geomembranes

GSE HD and GSE HD Textured geomembranes are high quality HDPE geomembranes that provide the following benefits:

- **Excellent chemical resistance**
- **Outstanding stress crack resistance**
- **Lowest permeability**
- **History of proven performance**
- **Meet or exceed all aspects of GRI GM 13**

GSE HD is available with either a black or white upper surface. GSE HD Textured is available as either single or double sided textured geomembrane with either a black or white upper surface.

CHEMICAL RESISTANCE

The chemical resistance of HDPE is the best of any available geomembranes. GSE HD is chemically resistant to a wide variety of chemicals including aromatic and halogenated hydrocarbons. They have been used successfully for years as primary and secondary landfill liners, in secondary containment applications and as liners for mining leach pads.

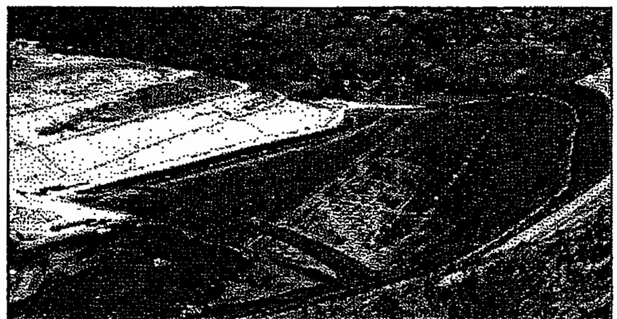
GSE HD can be used to line decorative ponds.

STRESS CRACK RESISTANCE

GSE HD is manufactured from resins specially designed to provide outstanding resistance to stress cracking. The appendix to ASTM D 5397, Single Point Notched Constant Tensile Load, is the test method most commonly specified for determination of stress crack resistance. GRI GM 13 requires a minimum of 300 hours to failure. GSE requires that every lot of resin used to manufacture GSE HD geomembranes has a minimum of 400 hours.

PERMEABILITY

Permeability of HDPE geomembranes is the lowest of any available geomembranes. This coupled with outstanding chemical and stress crack resistance combine to maximize the integrity of containment for any application.



GSE HD textured is used on slopes to increase the capacity of a landfill.

BENEFITS OF A ROUGHENED SURFACE

Perhaps the most important attribute textured geomembranes offer is the ability to improve geosynthetic profile stability which ultimately maximizes the available volume that can be contained by the geomembrane. The ability to line steeper slopes allows increases in design capacity providing cost savings. Further, the white upper surface of GSE White has the same physical properties as the black with the added benefit of a light reflective layer. This light reflective layer reduces heat gain, thereby reducing wrinkling, subgrade desiccation and worker fatigue.

IN-LINE TEXTURING DECREASES LEAD TIME

GSE HD Textured is manufactured using coextrusion technology – the same technology used by GSE for over fifteen years to produce GSE UltraFlex, GSE Conductive and GSE White geomembranes. GSE HD Textured meets the increasing need for textured HDPE geomembranes because it is an in-line one-step texturing process. Availability to GSE customers is increased and lead times are minimized.

PROVEN RELIABILITY

GSE HD geomembranes have a long history of reliability and proven performance. Hundreds of millions of square feet of GSE HD and GSE HD Textured have been sold and

- *Continued* -

installed. They have been used in wide ranging containment applications including potable water, decorative ponds, animal waste containment, landfills, canal linings and secondary containment. In addition to their exceptional performance, GSE HD products have excellent weldability under a variety of conditions. Extrusion and fusion welding can be performed with ease and confidence.

PREMIUM RAW MATERIAL

GSE HD products are made from high quality high density polyethylene resins. To these resins, carbon black, antioxidants and UV stabilizers are added to assure long term performance and UV resistance even in exposed conditions. The absence of leachable additives to all GSE geomembranes allows them to maintain excellent resistance to brittleness that may occur over time when plasticizers are used.

GSE HD can be used to line pregnant solution ponds in mining applications.

GSE QUALITY ASSURANCE SYSTEMS

All GSE geomembrane production involves three levels of quality assurance. First, raw material suppliers must comply with GSE specifications on incoming resin. Before the resin is unloaded from the railcar, GSE verifies the raw material test results that are submitted by our suppliers by performing selected conformance tests. The second level of QA begins during actual production. As each roll is produced it is electronically monitored for pinholes. Finally, GSE HD products undergo a rigorous Quality Assurance program after production to ensure the mechanical properties are intact and meet or exceed GSE current quality standards. All GSE laboratories are certified to both ISO and GAI-LAP standards.

ADDITIONAL INFORMATION

If you have an upcoming project, please give us a call. We will provide you with recommendations, an estimate for material and installation and contacts for a GSE approved installer.

AP032 R06/12/03

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