INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY REPORT

Prepared for
Indian and Northern Affairs Canada

Prepared by
UMA Engineering Ltd.
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UMA Job No. 41 01 4440 044 00

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Our File: 41 01 4440 044 00

Indian and Northern Affairs Canada
300 – 300 Main Street
Whitehorse, Yukon
Y1A 2B5

Attention: **Mr. Brett Hartshorne**

Dear Sir:

Reference: **Abandoned Clinton Creek Asbestos Mine – Environmental Liability**

We are pleased to submit 5 copies of our report summarizing the environmental liability associated with the abandoned Clinton Creek Asbestos Mine, Yukon Territory. Since abandonment of the Clinton Creek Asbestos Mine in the Yukon Territory, concerns have been raised with respect to the physical condition of the site, in particular downstream hazards associated with landslide dams created from unstable waste rock and tailings piles. In areas of significant relief such as the mine site location, flooding from failures of channel blockages can be especially dangerous and their occurrence can be unrelated to normal precipitation events that would be expected to produce flooding conditions. Existing and future conditions at the abandoned Clinton Creek Asbestos Mine potentially expose individuals, property and the environment to some degree of risk associated with flooding, downstream sedimentation and transport of asbestos fibres.

For the purposes of this report, environmental liability is considered to be the cost associated with the implementation of remedial measures to mitigate i) the risks associated with a catastrophic breach of the waste rock and tailings piles and ii) the environmental concerns from chronic erosion and redistribution of tailings and waste rock downstream of the mine site. Based on the monitoring completed to date, the preferred remedial option to address the landslide blockage at the Clinton Creek Waste Rock Dump is channel stabilization for which the capital construction costs are estimated to be in the range of $2,500,000 to $6,000,000 depending on whether stabilization of the waste rock dump is required. Given the possibility of conditions worsening at the outlet before long term remedial measures are implemented, the stabilization of the creek channel could be staged to allow the most immediate concern (the condition of the outlet) to be addressed prior to construction of the works for the entire length of the channel. Construction of the first two gabion drop structures, as a minimum, would significantly reduce the immediate threat of a breach. In this regard, a 30m long section of the channel immediately downstream of the Hudgeon lake outlet was stabilized with a gabion drop structure in the fall of 2002. The estimated capital costs to mitigate the concerns associated with the tailings piles range from $5,500,000 to stabilize the tailings and construct a stabilized creek channel to about $30,000,000 to remove a sufficient amount of the tailings to restore natural creek drainage.

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If we can be of further assistance, please contact Mr. Ken Skaftfeld, P.Eng. or Mr. Gil Robinson, M.Sc., P.Eng.

Sincerely,

UMA ENGINEERING LTD.

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1.0 INTRODUCTION

This report summarizes the results of our assessment of the environmental liability associated with the abandoned Clinton Creek Asbestos Mine, Yukon Territory. Significant environmental and physical hazards associated with continued degradation of the Clinton Creek channel through the waste rock dump and the Wolverine Creek channel through the tailings piles have been identified (UMA 2000). Of particular concern are i) the chronic redistribution of asbestos laden tailings and waste rock from the mine site into the Clinton Creek channel and ii) potential risks to human life and property downstream of the mine associated with a sudden breach of the Hudgeon Lake outlet. In areas with significant relief, such as the Clinton Creek valley, flooding from failures of channel blockages can be especially dangerous and unrelated to precipitation events that would normally be expected to produce flooding conditions.

For the purposes of this report, environmental liability is considered to be the cost associated with the implementation of remedial measures to mitigate the chronic erosion and redistribution of tailings and waste rock and risks associated with a catastrophic breach of the Hudgeon Lake outlet. This report presents the preliminary design of channel stabilization measures at the waste rock dump and the conceptual design of remedial measures to mitigate erosion and re-distribution of tailings. The conceptual design of remedial measures to mitigate the hazards associated with a breach of the waste rock dump was presented previously (UMA 2001).

2.0 HISTORICAL SUMMARY

The abandoned Clinton Creek Asbestos Mine is located about 100 km northwest of Dawson City in the Yukon Territory, 9 km upstream of the confluence of Clinton Creek and the Forty Mile River. The mine consists of three open pits (Porcupine, Creek and Snowshoe), two waste rock dumps (Porcupine Creek and Clinton Creek) along the south side of Clinton Creek, and a tailings pile on the west side of Wolverine Creek (Drawing 01).

Over 60 million tonnes of waste rock from the open pits was deposited over the south slope of the Clinton Creek valley at what is referred to as the Clinton Creek waste rock dump. From 1968 until depletion of economic reserves in 1978, the Cassiar Mining Corporation extracted approximately 12 million tonnes of serpentine ore from the bedrock. The ore was transported by an aerial tramway to the mill located on a ridge along the west side of Wolverine Creek, a tributary of Clinton Creek. Over the same period of time, about 10 million tonnes of asbestos tailings from the milling operation were deposited over the west slope of the Wolverine Creek valley (Wolverine Creek tailings piles). Since closure of the asbestos mine, concerns have been raised with respect to the physical condition of the site, in particular downstream hazards associated with channel blockages resulting from landslides of the Clinton Creek waste rock dump and Wolverine Creek tailings piles.

3.0 CLINTON CREEK WASTE ROCK DUMP

Four remediation alternatives to mitigate the hazards associated with a breach of the waste rock dump were presented in UMA's Conceptual Design Report (UMA 2001). These alternatives and their estimated construction cost were valley restoration ($30M), conveying creek flow around the waste rock dump via a tunnel ($20M), conveying creek flow via an alternate alignment across the middle of the waste rock dump ($14M) and conveying creek flow over the waste rock dump within a stabilized channel along the existing alignment ($7M). Valley restoration and tunnelling were not contingent on stabilizing the waste rock pile. The two channel stabilization alternatives included approximately 600,000 m^3 of waste rock excavation to achieve a stable waste rock geometry. It was also pointed out however, that if continued monitoring confirmed that movement rates of the waste rock were sufficiently small or if movements had terminated, the need to stabilize the waste rock dump should be re-evaluated. Based on the observed waste rock movements and the comparatively lower capital costs, channel stabilization has since been selected as the preferred remedial option to address the hazards associated with a breach of the waste rock at the Hudgeon Lake outlet and to reduce the chronic erosion of waste rock material.

Subsequent to preparation of the Conceptual Design Report, waste rock movement monitoring and a detailed survey of the Hudgeon Lake outlet were carried out in June 2001. Over the two-year period from July 1999 to June 2001, annual horizontal movements ranging from 1 to 11cm were observed, or an average annual rate of 7cm. Over the same time period, the average rate of vertical settlement appears to be in the order of 7 cm. The movements confirm previous observations that waste rock pile movements are small (in comparison to movements prior to 1986) and are perhaps decreasing with time. The horizontal movements for monitoring monument #19 are summarized on Figure 3-1. The movements can either be interpreted as small constant strain rates or strain rates that are decreasing with time, and as such are referred to as creep movements (as compared with the large movements observed prior to 1986). There are no signs to indicate strain rates are increasing, observations that would be expected if large movements of the waste rock were imminent. These creep movements may continue at similar strain rates for many more years, and in particular, the horizontal movements may be susceptible to channel erosion (i.e. down cutting) along the north edge of the waste rock. A location plan of all the monitoring monuments and survey benchmarks, a table of the benchmark co-ordinates and the monitoring results for all the waste rock movement monitors are included in Appendix-A.

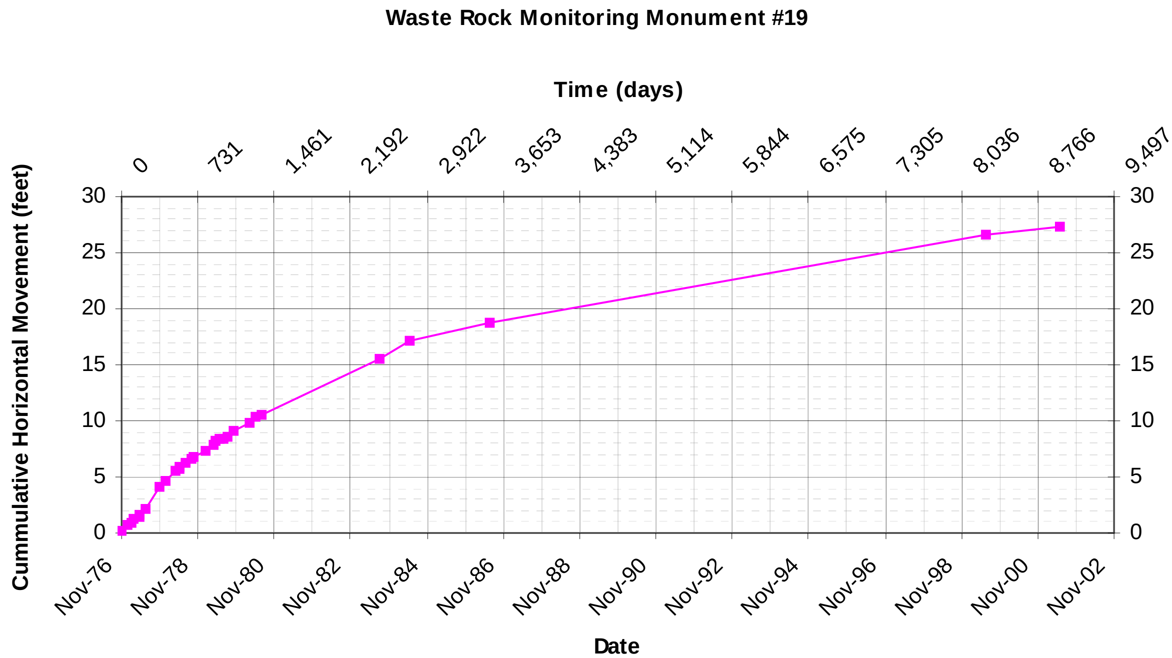

Figure 3-1) Waste Rock Dump Movements

Providing that channel stabilization measures can be constructed to accommodate anticipated creep movements, waste rock stabilization is not considered necessary in the short term, in particular given the possibility of further reductions in these movements. With this in mind, the need for waste rock stabilization could be evaluated once the channel stabilization measures are constructed and additional monitoring data is available. Because the channel stabilization works involve partial infilling of the existing channel, it is possible that the observed horizontal creep movements may be reduced or possibly halted as a result of the channel stabilization work.

It is believed that the most immediate concern with respect to the potential for a catastrophic breach of the waste rock is the integrity (stability) of the existing creek channel at the Hudgeon Lake outlet. Comparing the creek channel profiles from 1986, 1999 and 2001, it is clear that continued channel erosion is deepening (down-cutting) the channel from a point just downstream of the lake outlet to about 500m downstream of the outlet (Drawing 02). As down-cutting continues, the toe of the waste rock pile is undercut and localized slope instabilities develop (Figure 3-2). The unstable waste rock slumps into the channel and can temporarily block creek flow. In most instances, this material is quickly overtopped and transported downstream and deposited in the Clinton Creek channel downstream of the mine. As the down-cutting gradually retrogresses towards the outlet however, conditions may quickly develop where normal flow and/or an overtopping event (i.e. breach of a waste rock slump) could trigger a full scale breach of the waste rock at the lake outlet. The consequences of a breach and rapid draining of Hudgeon Lake are discussed in UMA's Risk Assessment report (UMA 2000).

Figure 3-2) Waste Rock Slumping Into Creek Channel (View Downstream)

Given the possibility of conditions worsening at the outlet before the channel can be stabilized, the stabilization work could be staged to allow the most immediate concern (the condition of the outlet) to be addressed before the overall stabilization works are completed. This strategy would allow mitigation of the catastrophic breach potential but would not significantly reduce the chronic erosion that would occur downstream of the stabilized section. The stabilization work at the outlet can likely be designed as a component of the overall channel stabilization measures.

3.1 CONDITION OF EXISTING CHANNEL

The existing channel through the waste rock dump is approximately 800m long and up to 18m below the existing mine access road on the south side of the creek channel. Side slopes of the waste rock forming the south creek bank are generally at, or steeper than, 1 horizontal to 1 vertical (1H:1V). For the first 350m downstream of the Hudgeon Lake outlet, the creek channel is flanked on the north and south sides by colluvium and waste rock material, respectively. The channel bed contains boulders and cobbles of various sizes. Downstream from this point, the channel has cut into the argillite bedrock underlying the colluvium. As a result, the north and south banks consist of bedrock and waste rock material, respectively. The channel bed consists of bedrock and numerous boulders. Although most of the exposed bedrock within the channel has some degree of weathering, the transition between the heavily weathered bedrock and underlying more intact bedrock can be visually identified. For the purposes of this report, the upper heavily fractured unit is referred to as weathered bedrock and the material below as intact bedrock, although it too is fractured. As shown in Figure 3-3, the weathered material dips at approximately the same inclination of the natural valley slope, or about 1.5H:1V and the intact rock is nearly vertical. Although the elevation of the contact between the weathered and intact rock has not been surveyed, it can be estimated from photographs and field notes. An estimated profile of the intact bedrock surface is shown on Drawing 03.

A detailed survey of the Hudgeon Lake outlet and the first 150m to 200m of channel downstream of the outlet was carried out in July 2001 to provide the necessary information for design of stabilization measures for this area of the channel. The features surveyed include general topography, channel profile (full length), channel cross sections and the location of tension cracks and springs. Drawing 04 shows the results of the survey superimposed on a 1999 aerial photo of the outlet.

Figure 3-3) Clinton Creek Channel (View Downstream)

3.2 CHANNEL STABILIZATION

3.2.1 Hydrology

Based on a regional hydrology study (UMA 2000), the 100- and 200-year maximum instantaneous unit discharges can be estimated using Equations 3-1 and 3-2:

Equation 3-1: $q_{100} = 1.4701 \times A^{-0.3117}$

Equation 3-2: $q_{200} = 1.7494 \times A^{-0.3202}$

Where: q_n = instantaneous unit discharge [m^3/s per km^2] for n-year return period and

A = drainage area [km^2]

The 100- and 200-year frequency floods for Clinton Creek and Wolverine Creek were estimated from the regional unit discharges. The 100- and 200-year floods were plotted in a log-normal graph from which the 50- and 25-year floods were then estimated by interpolation. The drainage areas and estimated discharges are summarized in Table 3-1.

Table 3-1) Drainage Areas and Discharges

Parameter	Clinton Creek	Wolverine Creek
Drainage area [km^2]	117	29
25-year flood [m^3/s]	28.9	10.0
50-year flood [m^3/s]	33.8	12.2
100-year flood [m^3/s]	39.0	14.9
200-year flood [m^3/s]	44.5	17.3

3.2.2 Constraints

There are a number of constraints that must be recognized in the selection and design of channel stabilization measures. These include:

- Continued creep movements of the waste rock may distort or shift any structures constructed in the creek channel;
- Floating debris from Hudgeon Lake such as logs and ice that may impede flow, or cause damage to structures in the creek channel;
- The requirement to direct some flow from Hudgeon lake around the construction area in the creek channel to maintain fish habitat downstream of the construction area.
- Remoteness of the mine site with respect to availability and delivery of construction materials and equipment.

3.2.3 Gabion Drop Structures

The channel stabilization work involves flattening the channel profile using grade control structures such as, gabion drop structures. Gabion structures are preferred over rigid structures (e.g. concrete) because of their flexibility that allows them to undergo deformation while remaining structurally sound, an important consideration given the observed creep movements of the waste rock. In addition, gabions are permeable and don't have the potential uplift problems associated with rigid structures. Gabion structures are also robust enough that they should withstand most problems associated with ice and logs. These structures are simple to construct using granular fill material available at the mine site and conventional construction equipment. The only materials requiring transportation over a long distance are the gabion baskets and geotextile.

Erosion of the creek channel will be reduced by flattening the channel grade and lowering the velocity of the water such that it can no longer scour or erode the bed of the creek channel. Channel grade flattening can be achieved by partially infilling the channel and constructing a series of gabion drop structures. A typical drop structure is shown on Drawing 05. Between drop structures, the channel will be lined with granular material of sufficient size and gradation to resist the anticipated flow velocities. For example, the permissible channel velocity for cobble lining is 2.5m/s compared with 1.6m/s for unprocessed material consisting of gravel and cobble sized material. Channel velocities can be maintained within this range by controlling the channel grade and cross-section. The grade between the drop structures can be maintained by adjusting the number, height and location of the drop structures and the channel section can be set to the required width and depth to pass the expected creek flows.

Gabions are placed as steps, ranging in height from 0.3m to 1.0m, which provide energy dissipation between each step as the water travels through and over the structure, and also serves as a grade control point in the channel. A draw-down reduction weir at the top of each structure creates a constriction that reduces the water surface draw-down immediately upstream of the structure to control the channel flow velocity along that length of channel. An end sill prevents a floor jet from extending downstream of the structure during high discharges. A separating layer of non-woven geotextile is used between the foundation material and the gabions to prevent migration or loss of fines due to seepage or erosion. The geotextile is anchored below the structure at the upstream and downstream ends to further confine the foundation material below the gabions. Some sand and gravel will be transported along the channel bed during spring runoff and other high flow events. The finer material will become trapped between the cobbles in the gabion baskets further stabilizing the structure. As a consequence of stabilizing the channel and reducing the sediment load, some increased erosion of the Clinton Creek channel downstream of the stabilized portion of the channel may occur as a result of the reduced bed load.

3.2.4 Channel Design

The waste rock is generally a well graded material consisting of silt, sand, gravel, cobbles and occasional boulders. The mean particle diameter (d_{50}) ranges from 0.5 mm to 10.0 mm, depending on where the material is sampled (Golder 1978). Based on the bed and bank material visible in the existing channel and considering the conditions after construction, a Manning's n-value of 0.035 was used for design. Because the sediment load in the stabilized channel will be considerably less than existing, the permissible channel velocity will be reduced to minimize erosion between the drop structures. Based on the relatively coarse channel bed material, a design flow velocity of 1.1m/s was selected to design the stabilized channel.

The estimated 25-year flood (Q=28.9 m^3/s) was used for the design of the channel stabilization works for the waste rock pile. However, the discharge of a 25-year flood at the Hudgeon Lake outlet will be smaller due to the flood attenuation caused by Hudgeon Lake, resulting in a higher level of protection than indicated by the 25-year return period. Based on the design discharge of 28.9 m^3/s, 3H:1V side slopes and a grade (between drop structures) of 0.1%, the new channel geometry will require a bed width of 7m and a flow depth of 2m. With the dimensions of the individual gabion baskets used for the drop structures (3.0m long, 1.0m wide, 0.5m high), the freeboard at the control structures will be approximately 0.2m which is sufficient to confine the 50-year flood within the new channel cross-section.

A 0.5m thick layer of riprap (D_{50} = 150 mm) is required 3m upstream and downstream of the drop structures for channel revetment. To provide a higher level of channel erosion protection, in the first 150m of the channel, it is also recommended that the channel be armoured with rip rap between the drop structures and also between the Hudgeon Lake outlet and the first drop structure.

A total hydraulic drop of 35m will be required between Hudgeon lake outlet and the natural creek bottom at the downstream end of the waste rock dump. To maintain the channel along it's existing alignment, reduce the amount of excavation for slope flattening, and maintain road access, the design profile for the stabilized channel through the middle portion of the waste rock dump will be governed by the contact elevation of the intact bedrock shown on Drawing 03. Thirteen drop structures ranging in height from 1.5m to 3m are necessary to achieve the 35m hydraulic drop (Drawing 06). The sides of the gabion structures will be tied into the valley slope (colluvium material) on the north side and the waste rock on the south side of the creek channel to confine the flow within the stabilized channel. Above the armoured portions of the channel, the waste rock side slopes will be flattened to a more stable geometry (1H:1V minimum). Partial in-filling of the channel will provide additional toe support for the waste rock pile, possibly helping reduce future creep movements of the waste rock pile and the potential for instabilities of the valley slope.

The flow from Hudgeon Lake is recognized as being highly variable and sensitive to precipitation events within the drainage basin. Dewatering of the channel will be required during construction of the stabilization measures. To maintain a base flow in the channel for fish habitat it will be necessary to divert some flow from Hudgeon lake around the work area. Additional water storage volume can be achieved by initially drawing down the elevation of Hudgeon Lake to a level about 0.2m above the lake outlet (+/- elevation 411.2m). To minimize the potential for piping, under no circumstances should the elevation of the lake be raised above the high water mark on the lake shoreline (+/- elevation 412.0m).

Granular material is available on site. Waste rock can be used for channel fill. Hard and durable rock for filling the gabions and use as channel revetment can be obtained from the weathered outcrop located at the northeast corner of the confluence of Wolverine and Clinton creeks. A sufficient quantity of boulders can be harvested from the existing vicinity of the outlet or from the west end of the waste rock dump. Locations of potential borrow sources are shown on Drawing 07.

3.2.5 Staged Construction

To provide adequate protection against a breach, in the short term, the required length of channel stabilization downstream of the outlet is in the order of 150m. The stabilization work could be completed in stages, beginning with the section of the channel immediately downstream of the Hudgeon Lake outlet. Channel erosion will continue downstream of the stabilized portion of the channel, possibly at a greater rate due to the reduced sediment load from the stabilized portion of the channel. Channel degradation of the non-stabilized portion of the channel can therefore be expected to continue in the upstream direction until the non-stabilized channel degrades up to the last (furthest downstream) gabion drop structure. A longer gabion mat should therefore be placed just downstream of this drop structure to provide adequate protection to the constructed works and provide sufficient time to either complete maintenance work or initiate stabilization of the remainder of the channel. The configuration of the drop structures for the first 150m of channel stabilization is illustrated on Drawing 08.

First Gabion Drop Structure: The control section of the first gabion drop structure is located approximately 10m downstream of the culvert at the Hudgeon Lake outlet. As the stream channel turns to the right at this location, the structure centreline will have a radius of 12.6m over a deflection angle of 50 degrees. This is necessary to re-direct the flow into the general direction of the existing stream channel along the north hillside. The upstream floor of the structure is at Elevation 410.87m (0.01m below culvert invert) and the downstream floor will be set at Elevation 408.87m, creating a 2m drop over four 0.5m high steps.

Second Gabion Drop Structure: This second structure is located approximately 18m downstream of the first drop structure and perpendicular to the existing channel. The upstream floor of the structure is set at Elevation 408.85m (0.02m lower than the downstream floor of the first drop structure) and the downstream floor is at Elevation 407.35m, resulting in a 1.50m drop over three 0.5m high steps. To infill the existing thalweg, about 0.7m of fill will be required below the gabion baskets farthest upstream. At the downstream end, the bottom of the gabion baskets will be set at Elevation 406.85m or 0.25m above the existing thalweg.

Third Gabion Drop Structure: This third structure is located approximately 50m downstream of the second drop structure, perpendicular to the channel. Five, 0.5m high steps will separate the floor of the upstream structure (Elevation 407.30m) and the floor of the downstream structure (Elevation 404.80m). Fill depths below the gabion baskets are approximately 1.5m at the upstream end of the drop structure. A shallow cut will be required at the downstream end of the structure.

Fourth Gabion Drop Structure: This fourth structure is located approximately 25m downstream of the third drop structure, perpendicular to the channel. Five, 0.5m high steps will separate the floor of the upstream structure (Elevation 404.77m) and the floor of the downstream structure (Elevation 402.27m). Fill depths below the gabion baskets are approximately 0.5m at the upstream and 0.1m at the downstream end. To protect the downstream end of the structure from eventual undercutting by channel degradation progressing in the upstream direction, one extra row of sacrificial gabions should be constructed at the downstream end of this drop structure. This extra row should only be joined to the regular structure along the base of the baskets on the floor and the lowest three baskets on the side slopes. This will allow this row of baskets to move or rotate down into the channel as head-cutting occurs, helping armour the channel and protect the integrity of the fourth gabion structure.

3.2.6 Cost Estimate

Assuming stabilization of the waste rock pile is not required, the capital construction cost to stabilize the entire creek channel through the waste rock dump is estimated to be in the range of $2,500,000 to $3,000,000. If waste rock stabilization is necessary, the estimated capital cost is in the order of $6,000,000 (including channel stabilization). Based on discussions with two Contractors from Whitehorse, the estimated construction costs to stabilize a 150m section at the Hudgeon Lake is in the order of $500,000.

4.0 WOLVERINE CREEK TAILINGS PILE

The environmental and physical hazards associated with continued down slope movements of the tailings pile and degradation of the Wolverine Creek channel include the chronic redistribution of asbestos laden tailings and flooding from failures of channel blockages (UMA 2000). A review of the historical information regarding tailings pile movements and the conceptual design of remedial measures to mitigate these hazards are discussed in the following sections.

4.1 HISTORICAL SUMMARY

Between 1968 and 1974, tailings were deposited on the upper portion of the west slope of the Wolverine Creek valley (referred to as the south lobe). In 1974, a failure of the south lobe blocked natural flow in Wolverine Creek backing up approximately 9m of water behind the landslide material (Figure 4-1). In the spring of 1974, a sudden breach of the tailings occurred resulting in flash flooding of the Wolverine creek valley to the confluence with Clinton Creek where the flooding is believed to have quickly attenuated. The eroded tailings were deposited several metres deep in the creek valley directly downstream of the south lobe (Figure 4-2). Although the majority of the tailings are believed to have been deposited upstream of the mine access road shown on Drawings 01 and 09, some of the finer material including asbestos fibres entered the Clinton Creek channel where it was deposited possibly as far downstream as the Forty Mile River.

Following the failure of the south lobe, a 9m deep channel was excavated at the toe of the tailings to facilitate creek flow and a new tailings pile was established north of the failed mass, now referred to as the north lobe (Figure 4-1). By 1977, the north lobe was showing signs of instability (Figure 4-2) and during the last months of mine operation, the tailings were placed in the northwest corner of the north lobe. Partial re-grading of the north and south lobes was undertaken in 1978 and 1979 in an unsuccessful attempt at stabilizing the tailings. In 1978, channel stabilization measures were constructed in Wolverine Creek across the tailings immediately downstream of the south lobe. These measures consisted of a rock-lined channel with a series of rock weirs (Figure 4-2). To date, these measures have performed well although some deterioration was noted during the 2001 site visit.

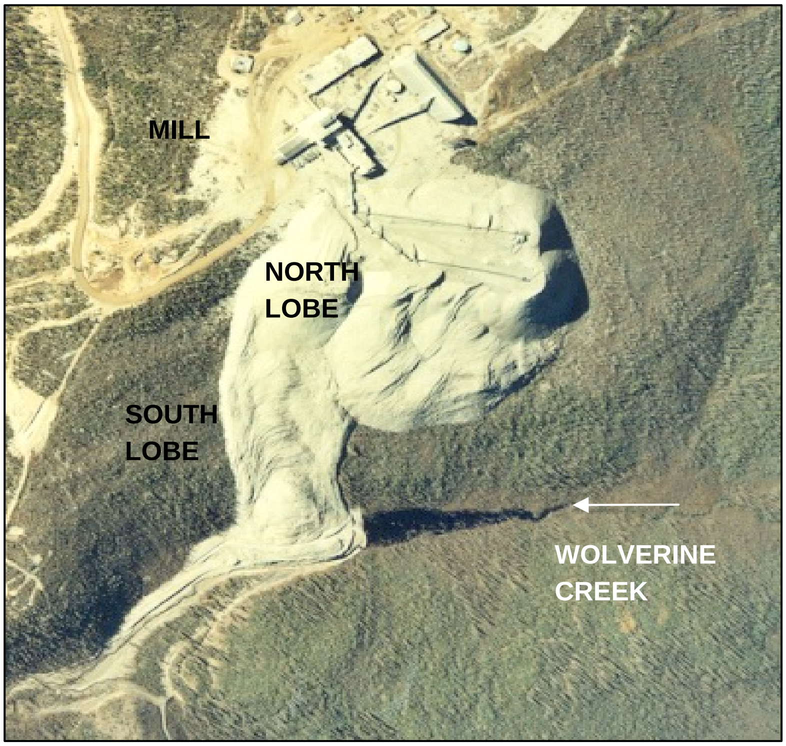

Figure 4-1) Failed South Lobe (1976)

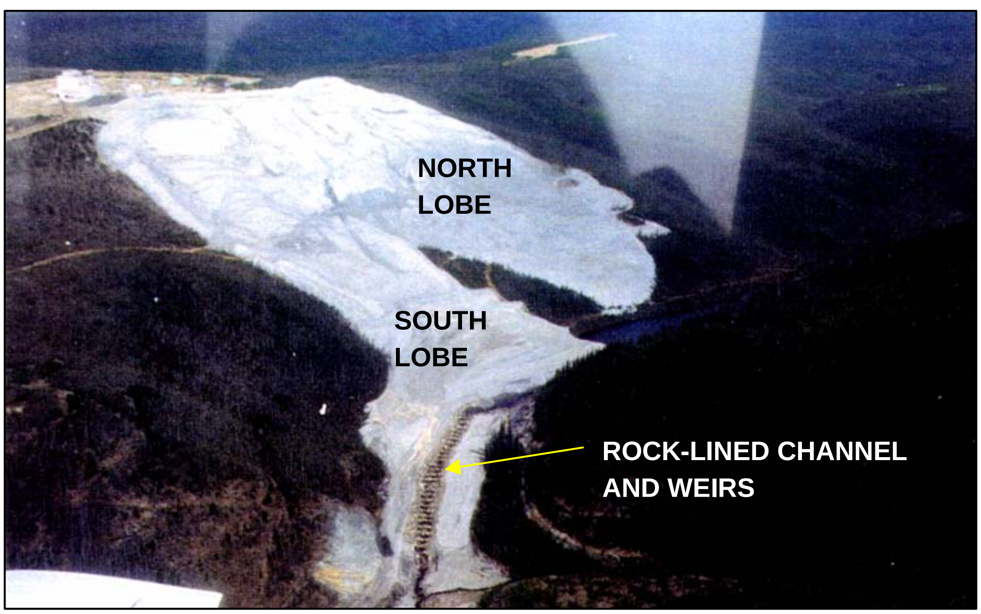

Figure 4-2) Rock-Lined Channel and Weirs (1982)

Monitoring carried out from 1976 to 1986 confirmed that displacement rates were much larger for the north lobe as compared to the south lobe. The lower displacement rates of the south lobe are attributed to the toe support provided by the tailings at the bottom of the valley. In general, the displacements varied along the length each lobe with the largest movements occurring at the toe and small displacements occurring near the top of the slope. The north lobe reached the edge of Wolverine Creek in 1984 and by 1988 reached the opposite side of the valley. The resulting blockage was followed by a small breach (Geo-Eng 1988).

4.2 BACKGROUND INFORMATION

A considerable amount of information regarding the tailings pile is contained in reports and drawings filed at INAC's Whitehorse office. Information was extracted related to geotechnical issues, previous remedial strategies and any additional information regarding the nature of the tailings pile instabilities. In chronological order, relevant information from these reports is summarized in the following sections. Anecdotal comments by the writer are provided in Italics.

- In the spring 1974, 9m of water backed up behind and breached tailings pile (south lobe) blocking Wolverine Creek. The breach resulted in a flash flood down Wolverine Creek to the mouth of Clinton Creek. The author of the report implied that the flash flooding event was predicted in previous reports when he stated "The tailings from the mill are not stable and will undoubtedly continue to slump and block the valley." (Bowie, 1974)
- To reduce the potential for another breach of the tailings pile (south lobe) a channel was bulldozed across the toe region of the slide. Water impounded upstream of the tailings was observed to be seeping through the tailings and/or the native foundation soils, and was emerging in the form of springs at a location slightly downstream of the downstream limit of the tailings. The water was clear and did not appear to be carrying suspended solids. Clinton Creek Mines indicated that tailings deposition would be shifted northward away from the area where the failure occurred where the ground is much flatter (approximately 8 degrees) above the 495.3m contour. (*This feature is evident on the airphotos and the slope below this level is about 17 degrees*). (Golder Brawner 1974)
- Routine surveying of monitoring points on the surface of the tailings began in the fall of 1976. Large displacements of the north lobe were measured after 28 days. Much smaller movements were observed in the failed portion of the south lobe. A 2m high pile of native material was pushed up in front of the advancing north lobe and is still visible today. Exposed native soil at the base of the pile slope (*i.e. the leading edge of the north lobe*) suggested that the failure surface is confined to unfrozen soil near the original ground surface. A recommendation was made to

stop placement of tailings on the northeast side of the tailings pile and to continue monitoring the tailings pile monitors.

- A Site Rehabilitation and Abandonment Plan for the Yukon Territory Water Board was prepared in 1977 (Hardy, 1977). The main points for this report were:
    - Both the north and south lobes of the tailings pile show signs of instability. Wolverine Creek shows signs of very active bank erosion and down cutting.
    - Aerial photography indicated that the north lobe is also moving in a downstream direction (laterally towards the south lobe). In turn, additional toe support is provided by the south lobe to the south edge of the north lobe.
    - Surface characteristics of the unstable tailings pile suggest that the failure mode in this area could be the result of a failure within the active layer.
    - The report indicates Cassiar Asbestos Corporation was planning to re-contour the tailings pile to reduce the rate of down slope movement. The intention was to unload the top of the pile by moving some of the tailings to the north and re-depositing them on flatter slopes. It is not certain whether this work was carried out or not. (*Re-contouring was completed in 1978 and was not successful at reducing down slope movements over the long term*)
    - Recommendations provided include the installation of additional monitoring points and a geotechnical investigation to determine the permafrost conditions beneath and within the tailings pile. (*Geotechnical investigation completed by Golder, 1978)*
- Golder Associates carried out geotechnical investigations in 1978. Report highlights are summarized as follows:
    - The failure of the south lobe occurred at the location of a small draw in the hillside. Aerial photographs indicate surficial earth materials in this draw were wetter than those in the surrounding area.
    - Following the failure of the south lobe, tailings were deposited to the north of the failed mass. This portion of the tailings pile (north lobe) subsequently started moving downslope towards Wolverine Creek as the pile developed. As of mid-June 1978, the north lobe of the tailings pile was within 107m of Wolverine Creek.

- A crust formed on the surface of the tailings pile has significantly reduced the potential for wind-blown fibres. However, movement of the tailings pile is exposing fresh tailings that are more susceptible to wind erosion.
- Substantial amounts of surficial foundation *(overburden)* soils have been displaced and pushed ahead of the north lobe as it moves downslope.
- No mass movements or creep movements have been observed in the northwest area of the tailings pile. The tailings in this area had been placed on relatively flat ground.
- A geotechnical investigation was carried out in May 1978 to identify the subsoils in the vicinity of the tailings pile, to recover samples for testing in the laboratory and to install thermistors in the tailings pile and foundation soils and to assess the ground temperature regimes.
- The mechanism of failure for the south lobe is thought to have involved the build-up of excess pore pressures within the active layer of the foundation soils beneath the tailings in the draw. As indicated by the continuous slow downslope movement of the south lobe, a point of incipient failure was reached before the 1974 failure.
- Continued downslope movement of the tailings along the length of the south lobe is slow and becoming slower with time. The movements were attributed to the thaw-consolidation process taking place in the permafrost beneath the tails as observed at the thermistor installation in this segment of the tailings pile, and to the continuous removal of support from the toe of the failure by erosion in the Wolverine Creek channel. Deceleration of these movements suggests that temperature equilibrium between the tailings and the foundation soils is approaching, and that the excess pore pressures induced as a result of the thaw-consolidation process were dissipating with time.
- The failure mechanism for the south lobe is attributed to excess porewater pressures developed as the active layer thawed and when freeze back began. During freeze back, the frost front proceeds downward from ground surface forming an impermeable layer above the unfrozen saturated soil sandwiched between this layer and the permafrost. Additional loads, such as tailings, applied over the frost front resulted in the

development of excess pore water pressures and reduced shear strengths. It is believed that the presence of a weak layer of soil confined at some depth below original ground surface would allow the bulldozing of material up in front of the tailings pile.

- Remedial work recommended and undertaken includes:
  - Trimming and re-contouring portions of the south lobe.
  - Re-grading of Wolverine Creek across the south lobe and provision of erosion protection.
  - Immediately downstream of the south lobe a rock-lined channel and rock weir system was constructed with an overall gradient of 8 percent.
  - Trimming about 6m of tailings off of the north lobe in the area where greatest movements have been measured.

- Hardy Associates completed a review of the tailings pile behaviour in 1980, after remedial work recommended by Golder was completed. Report highlights are summarized as follows:
  - The overall dump configuration apparently produces an arching effect and allows some degree of independent behaviour of the north and south lobes.
  - Horizontal movement rates indicated a favourable effect of the re-contouring works (*completed in 1978)* but the entire tailings pile was still unstable. (*The reduced horizontal movement rates only lasted for about 1 year and then increased).*
  - Seasonal changes of thermal and groundwater conditions appear to be the main factors causing the seasonal variation (*in horizontal movement rates*).
  - More detailed information on slide geometry, subgrade, groundwater and thermal conditions may modify the above conclusions that are based solely on available monitoring data.
- Comments from June 1981 site inspection report (Hardy, 1981)
  - Fresh scarps and cracks in the south lobe indicate continuing and possibly accelerating movements of key components of the south lobe.

  - Numerous wide open cracks and almost vertical relatively high scarps exist throughout the north lobe of the pile. The toe area of the north lobe is bulged and the material apparently overrides the natural ground.
  - Differences in movement rates of the monitoring points indicate that the north lobe is not moving as a single mass but that individual lobe segments move somewhat independently while interacting and influencing each other. (*This behaviour is indicative of a retrogressive failure).*
  - Visual inspection of the Wolverine Creek spillway system showed that most of the weirs and embankment armouring are performing satisfactorily. However, the outfall immediately downstream of the last weir is unprotected, retrogressive erosion is occurring and the structural integrity of the last three weirs is poor. Recommendation made to install a rip-rap apron downstream of the last weir and to rehabilitate any damaged weirs.
- Comments from Review of Rehabilitation Measures Report (Hardy, 1984)
  - Monitoring results from the north and south lobes indicate that the horizontal rate of slope movement is about 1.5 to 1.9 times greater in the summer as compared to the winter season.
  - Five alternate reclamation schemes have been considered including:
    - Use of a coarse rock drain to channel water flow through spoil dumps.
    - Conveyance of Wolverine Creek around the tailings via a 1.8m diameter hydraulic tunnel.
    - Conveyance of Wolverine Creek through the tailings via one or more large corrugated steel pipes until tailings stabilize and then construct a permanent channel over the tailings.
    - Continued monitoring and maintenance program, recommended by Klohn Leonoff (1984) as the most practical approach, would be to continue the monitoring and maintenance that has been ongoing for the past several years.

  - Dam at the toe of the south lobe to stabilize this portion of the tailings and a rock-lined channel over the tailings to control the path of water. It was recommended to construct a spillway and to install one or two corrugated steel pipes extending through the dam to the armoured channel.

- Observations from June 1984 site inspection report (Klohn-Leonoff, 1984)
  - The toe of the south lobe is considerably more cracked and upthrust than in 1983 and the creek channel between the toe of the tailings pile and the east slope of Wolverine Creek valley is being squeezed by the tailings pile movements.
  - The toe of the north lobe has entered the lake in the valley bottom and extends an estimated 6m beyond the 1983 shoreline.
  - The rock-lined channel constructed over the failed mass of the south tailings lobe has continued to perform well. (*Rock-lined channel was constructed in 1978*).
  - Measurements taken during the 1984 site visit showed that the rock-lined channel has a minimum bottom width of about 9m and an affective lined depth of about 1.2m. The riprap forming the energy-dissipating weirs and the channel lining appears to have a mean diameter of about 0.9m.
- Comments from the Clinton Creek Mine Review Report on Waste Dump and Tailings Pile Conditions (Hardy, 1985)
  - The north and south lobes of the tailings pile continue to move downslope. Blockage of the channel by either tailings pile lobe would result in a breach that may form a new channel outside of the present rock-lined spillway. The unlined channel could erode easily through the tailings and accelerate the instability, particularly of the south lobe.
  - The current program of inspection and unspecified maintenance will not resolve existing problems. However, the monitoring data are extremely useful for the evaluation of possible courses of action. They confirm that large downhill displacements are possible under present conditions with a low static factor of safety. (*Monitoring of the tailings pile was discontinued in 1986).*

- Comments from Clinton Creek Asbestos Mine Abandonment Plan (Klohn Leonoff, 1986)
  - The toe of the north lobe reached the valley bottom in 1985 and, as further movement occurs, it will begin to be buttressed against the opposite valley wall. (*Continual erosion of the tailings forming the west bank of the creek has reduced the potential for buttressing).*
  - The channel conveying the stream past the toe of the south lobe has been squeezed against the east valley wall by the advancing tailings pile. Some erosion at the toe was evident in 1985. The stream appears to have the capacity to remove the tailings at a sufficient rate to maintain the channel without major blockage.

4.3 CONDITION OF EXISTING CHANNEL

A plan view of the existing tailings piles and Wolverine Creek profile are shown on Drawing 09. Representative creek channel cross-sections are illustrated on Drawing 10. As a result of the channel blockages, the alignment and elevation of Wolverine Creek is now about 25m further to the east and about 13m higher than it was naturally. This new alignment has resulted in erosion of the east valley slope and the tailings that form the west bank of the creek (Figure 4-3).

Beaver dams located along the toe of the north and south tailings lobes are believed to have reduced channel velocities and erosion as evidenced by the relatively flat channel gradient through this stretch. Immediately downstream of the last beaver dam however, velocities increase significantly as the channel narrows and the gradient increases. Between the downstream beaver dam and the rock-lined channel, the channel has down-cut into the underlying weathered argillite bedrock resulting in undercutting and slumping of the valley slope. Downstream of the south lobe, flow appears to be contained within the original rock lined channel with no significant erosion or down-cutting observed.

Figure 4-3) Erosion at Toe of South Lobe, View Downstream (1998)

4.4 GEOTECHNICAL PROPERTIES

The geotechnical properties of the tailings and foundation soils and the permafrost and groundwater (piezometric) conditions are required to develop a slope stability model to assess various remedial options for the tailings pile. Geotechnical properties of the tailings, overburden and weathered bedrock have been previously researched, providing some information with respect to shear strength (friction angle) and unit weight of the material. A drilling program undertaken by Golder Associated in 1978 provided limited information on permafrost although the conditions reported might not be representative of the current thermal regime. There is no information on groundwater (piezometric) levels. The geotechnical properties of the major stratigraphic units are discussed separately as follows.

4.4.1 Tailings

The tailings are generally made up of serpentine bedrock particles and asbestos fibres. They are primarily sand and gravel sized particles with trace silt and clay sized particles (Golder 1978 and R.M. Hardy 1977). A saturated unit weight of 21.2 kN/m^3 was used in a slope stability analysis conducted by R.M. Hardy (1978). Peak friction angles of 45 degrees for an effective stress range of 0 to 140 kPa and 35 degrees for effective stresses greater than 140 kPa were measured in direct shear testing (Golder, 1978). A peak friction angle of 46 degrees and a residual friction angle of 30 degree were also reported (R.M. Hardy, 1977). These values generally agree with the measured angle of repose of 39 degrees measured at the crest of the tailings pile (Golder Brawner 1974).

4.4.2 Overburden

The overburden soils within the Wolverine Creek valley are reported to be colluvium comprised primarily of sand and silt with trace clay sized particles (Golder 1978). Of five samples tested, three contained gravel sized particles (12%, 20% and 30% gravel). Moisture contents of samples taken in undisturbed areas (adjacent to the tailings) ranged from 28.2 to 40.6 percent with corresponding saturated unit weights of 17.8 kN/m^3. Below the tailings pile however, moisture contents ranged from 13.5 to 19.5 percent with saturated unit weights of 21.7 kN/m^3, suggesting that consolidation of the active layer and/or thaw-consolidation of the colluvium beneath the tailings pile has taken place. A range of peak shear strengths of 27.5 to 32 degrees and a residual shear strength of 23 degrees were measured in direct shear testing (Golder, 1978 and R.M. Hardy, 1977).

4.4.3 Bedrock

The mine site is located within the unglaciated Yukon-Tanana Upland Region. Bedrock in the area consists of black argillite that was exposed to periglacial weathering and near-surface material is heavily fractured and weathered. It is also possible that thin bedding planes of graphitic material may exist in the bedrock (personal communication, Dr. N. Morgenstern). Results from laboratory testing completed by Golder, 1978 indicate the weathered argillite has a specific gravity of 2.72, unit weights ranging from 22.8 to 24.5 kN/m^3 and moisture contents ranging from 5.2 to 11.4 percent. Direct shear tests were also performed on two samples of weathered argillite comprised of gravel and sand sized particles. The peak friction angles measured were 26 and 27 degrees (Golder, 1978).

4.4.4 Permafrost

The mean annual temperature in the area of the mine is –2.5 degrees C, ranging on average from 15 degrees C in the summer to –32 degrees C during the winter. The area consists of wide spread permafrost distribution up to 60m thick (Golder, 1978). The thickness of the active layer on the slope (below the tailings pile) is not known with certainty but it is unlikely to exceed about 1m (Golder, 1978).

Eight test holes (BH-12 to BH-19) were drilled in May 1978 (Golder, 1978) in and around the tailings pile at the locations shown on Drawing 11. Logs for these test holes are attached in Appendix B. Five test holes (BH-13, 15, 17, 18 & 19) were drilled at locations away from the tailings pile to depths of 12 to 18m. Frozen foundation (colluvium) soils overlying frozen bedrock were encountered in these test holes. Of the three remaining test holes, two were drilled through the south lobe (BH-12 & 16) and one through the north lobe (BH-14). It does not appear that the tailings were frozen (the test hole logs are not clear in this regard). The foundation soils at the south lobe were unfrozen to a depth of at least 6m below the tailings (which had been in place for an estimated 4 to 5 years). In contrast, the foundation (colluvium) soils below the north lobe were frozen. These tailings had been in place for an estimated 1 to 2 years at the time of drilling.

Three thermistor strings (T-5, 7 & 8) were installed in three test holes (BH-12, 14 & 16) on the tailings (two on the south lobe and one on the north lobe) and one thermistor string (T-6) was installed about 60m northwest of the tailings pile in test hole BH-13 (Drawing 11). Each thermistor string consisted of 9 thermistors spaced at 1.5m intervals. For each thermistor string, a temperature profile for the last monitoring date (19-June-1978) is shown in Figures 4-4, 4-5, 4-6 and 4-7. The short duration of thermistor monitoring (May and June, 1978) precludes interpreting any seasonal effects or long-term trends. The monitoring results from each installation are summarized as follows:

<u>Thermistor String T6 (adjacent to tailings pile)</u>

Thermistor string T6 is located about 60m northwest of the tailings pile. The monitoring results on Figure 4-4 indicate that the overburden soils are frozen to depths of at least 12m. The relatively warm temperatures below ground surface indicate that the active layer could extend to 1.5m in this area although additional monitoring would have been necessary to measure the active layer thickness. In general, the ground temperatures decrease with depth to about -1.6 degrees C at 3m below ground surface and then increase slightly to about –1.3 degrees C below the 3m depth. This data supports the observation that permafrost exists in the area of the mine site.

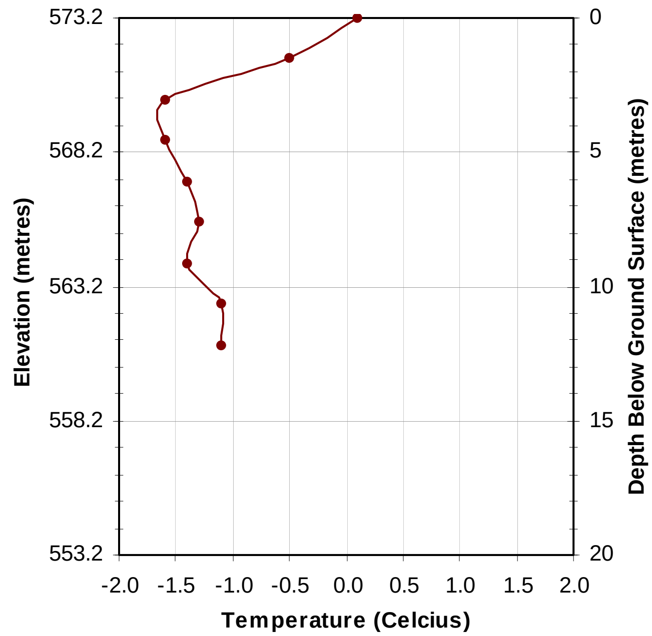

Figure 4-4) Temperature Profile for T6

Thermistor Strings T5 and T8 (South Lobe)

Thermistor strings T5 (Figure 4-5) and T8 (Figure 4-6) are located on the south lobe of the tailings pile at the crest and at mid-slope, respectively. The tailings forming the south lobe had been in place for approximately 4 to 5 years when the thermistor strings were installed. As shown in Figures 4-5 and 4-6, the midpoints of the thermistor strings were located near the interface between the tailings and overburden soils. Both the tailings and the overburden were observed to be unfrozen. The thermistor plot for T5 shows the uppermost point was about 0.5 degrees C cooler than the three points just below, possibly indicating a cool or frost front is advancing downward from the tailings surface. The points in the overburden were typically between 0.2 and 0.5 degrees C suggesting that

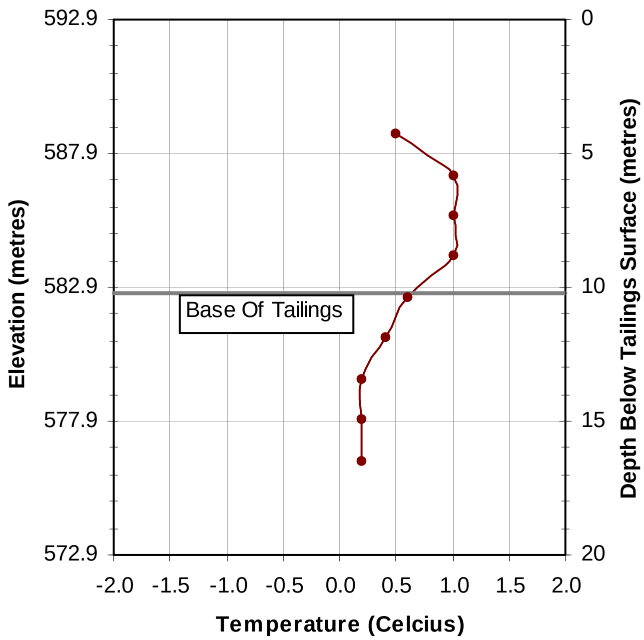

Figure 4-5) Temperature Profile for T5

degradation of the permafrost had occurred. The temperature profile from T8 decreases gradually from 0.6 to –0.1 degrees C with depth. The temperature of the overburden material appears to be very close to 0 degrees C. The slightly cooler temperatures in the overburden may be related to the thicker depth of tailings at this location (18m), which is about twice the thickness found at thermistor string T5.

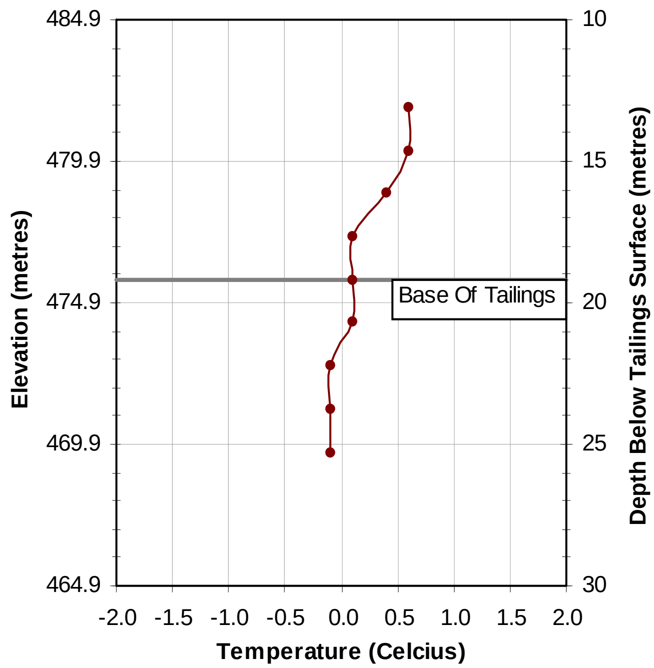

Figure 4-6) Temperature Profile for T8

Thermistor String T7 (north lobe)

Thermistor T7 is located near the northern edge of the tailings pile. The tailings at this location had been in place for approximately 1 to 2 years when the thermistor was installed. The data shown in Figure 4-7 indicates that the tailings and foundation soils were frozen with the coldest temperatures (-1.6 degrees C) just below the interface of the tailings and foundation soil. The temperatures of the foundation soils appear to gradually increase from –1.5 degrees C at the interface to nearly 0 degrees approximately 6m below the tailings. It is possible that these tailings were placed during the winter season, which would help to insulate the frozen overburden soils. This also might explain the cooler temperatures of the tailings just above the overburden soils. The warming trend with depth in the overburden may indicate where the bottom of permafrost layer is at this location.

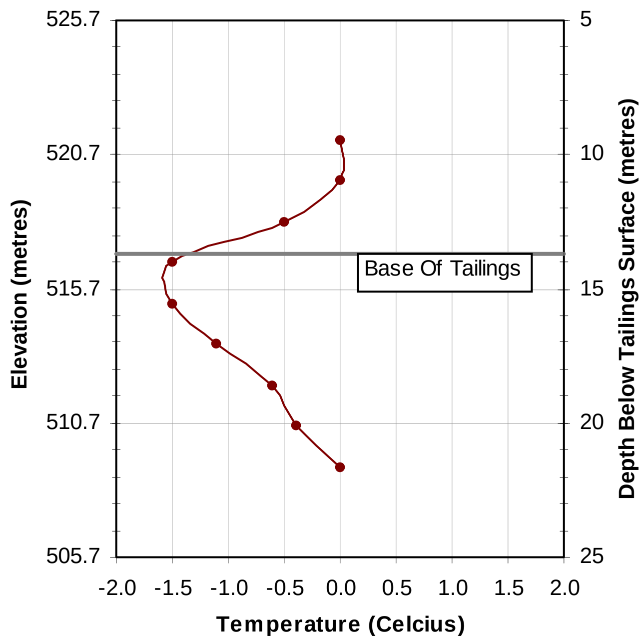

Figure 4-7) Temperature Profile for T7

4.5 TAILINGS PILE MOVEMENTS

Historical performance monitoring results have been reviewed to evaluate historical movement trends and failure surface geometries. In 1976, two monitoring points were installed in each lobe of the tailings pile. Approximately 19 additional monitoring points were added to the upper, lower and mid-slope zones on each lobe between 1977 and 1980. Tailings pile movements were monitored from December 1976 until June 1986 after which no surveys have been undertaken. Information on tailings pile movements, typically summarized as horizontal movement rates between successive monitoring events, has been discussed in a number of reports dating back to 1977.

The minimum and maximum horizontal movement rates measured at the upper, lower and mid-slope monitoring zones are shown in Table 4-1. As reported in earlier studies, the movement rates for the south lobe are about an order of magnitude less than those for the north lobe. This can be attributed to the fact that the south lobe had already failed and reached the bottom of the valley when the monitoring program was initiated (1976) that is, the tailings in the valley bottom provide toe support for the south lobe. In contrast, the toe of the north lobe had not reached the valley bottom at this point in time. The north lobe did not reach the valley bottom until 1986, at which time deceleration of the tailings was noted (Klohn Leonoff 1987) as toe support developed. The movement rates in Table 4-1 also indicate that movement rates along the north lobe decrease in the upslope direction, a behaviour indicative of a retrogressive failure pattern. A similar pattern is evident on the south lobe except the lower and mid-slope appear to be moving at nearly the same rate. It is also worth noting that the minimum movement rates for the mid and lower slope of the north lobe and the mid-slope of the south lobe occurred over the first winter season after re-grading work (terracing) was undertaken in these areas in 1978.

Table 4-1) Summary of Horizontal Movement Rates (1978 to 1986)

Monitor Location	Maximum Rate		Minimum Rate	
	metres/yr	Year Reported	metres/yr	Year Reported
South Lobe				
Upper slope	0.76	Aug 1981	0	Summer 1980
Mid-slope	6.6	June 1986	1.1	Summer 1978
Lower Slope	4.9	June 1986	0.65	June 1982
North Lobe				
Upper slope	3.6	Sept 1983	0.02	June 1986
Mid-slope	24.2	Sept 1983	3.5	Winter 78/79
Lower Slope	33.5	June 1983	3.5	Winter 78/79

Although there has not been any monitoring of the tailings since 1986, annual inspections and reconnaissance trips confirm that downslope movements of the north and south tailings lobes continue to occur, possibly at rates in the order of 5m per year. These movements are due, at least in part, to the continued erosion of tailings from the toe of the north and south lobes by Wolverine Creek. The tailings are eroded and transported downstream by Wolverine Creek almost as quickly as the tailings lobes advance into the valley bottom. Although a comparison of recent and historical aerial photography suggests there is little to no lateral spreading of the tailings within the failed area, it is likely that some mounding of the tailings is occurring (personal communication, M. Stepanek).

4.6 TAILINGS PILE STABILITY

4.6.1 Initial Tailings Pile Failure

The tailings placed in the south lobe were reported to have been moving down slope soon after placement started (Golder, 1978) and that the failure occurred in a small draw on the valley side slope. This draw is visible in 1951 aerial photography taken prior to mine development (Drawing 12). Based on the results of the geotechnical investigation completed by Golder Associates in 1978, permafrost likely existed in the valley slope prior to placement of the tailings. The failure of both the north and south lobes is likely related to a combination of factors, including a steep foundation (valley) slope and a build-up of pore water pressures within the active layer.

Cross sections through the north and south lobes are shown on Drawing 13. These sections are based on available test hole and historical survey data. The upper half of the valley slope beneath the north lobe is sloped at 13 degrees and the lower half is sloped at 18 degrees. The valley slope beneath the south lobe is sloped at 18 degrees.

In general terms, the tailings pile failures can be characterized as translational slides showing signs of retrogressive behaviour. The position of the composite failure surface is not precisely known but it is likely located within the overburden and/or weathered argillite layer, as evidenced by overburden material pushed up in front of the advancing north and south lobes. It is unlikely that a weak layer exists within the tailings based on the comparatively higher shear strengths of this material. Slope movements may very well continue until there is sufficient resistance at the toe of the slides, the development of which is impeded by continual toe erosion along Wolverine Creek.

The failure mechanism associated with the initial slides may be unique to that event, that is the mechanism may be different than that associated with the existing movements. The difference could be associated with the thermal regime early in the development of the tailings pile compared with the long-term equilibrium (steady state) condition that may have been reached after termination of mining activities. There is no physical evidence or monitoring data to indicate whether or not a steady state thermal condition has been reached. It is reasonable to assume that the most critical time period would have been the first few years of development when tailings were being actively placed over the valley slope and the initial disturbance to the thermal regime occurred. This is the time period when the rate of thaw might have been the fastest if ice-rich foundation soils were present.

4.6.2 Existing Stability

To investigate the feasibility of remedial options, a slope stability model was developed for the north and south lobes using the slope stability software package SLOPE/w by Geo-Slope. Representative cross-sections of the north and south lobes, based on historical information contained in previous reports and the 1999 aerial photography and digital mapping, were used for the slope stability model (Drawing 13). A back analysis was then carried out to determine the operating strengths and piezometric conditions necessary to achieve a factor of safety (FS) equal to unity (FS=1), a value representative of conditions where movements are about to, or are, occurring.

The movement trends presented in Table 4-1 for the monitoring points located on the upper slope indicate that relatively little movement of the tailings behind the main head scarps is occurring. Hence, it was assumed that the failure surfaces do not extend any farther back into the tailings than the obvious head scarps that have developed. The existing failure surface was assumed to be approximately parallel with the original ground surface and within a weak layer at a shallow depth in the foundation soil. Residual friction angles of shearing resistance were used for the tailings, overburden and weathered argillite, based on the direct shearing results (Section 4.4). The piezometric level within the overburden and weathered argillite bedrock was modeled using the pore water coefficient Ru, which is the ratio of pore water pressure to overburden pressure. A 3m deep, water filled tension crack located on the surface of the tailings was assumed in the model. Sensitivity analyses were subsequently carried out to determine the Ru value necessary to provide a factor of safety of 1.0. The resulting Ru values for the north and south lobes are 0.22 and 0.33, respectively. The resulting failure surface geometry and associated modeling parameters for the north and south lobes are shown in Figures 4-8 and 4-9, respectively.

The analysis indicates that a combination of residual shear strengths and high pore-water pressures in the overburden material are required to achieve a FS of unity. This observation provides further evidence that unique geological conditions, in particular a shallow weak layer within the overburden, are responsible for continued movement of the tailings pile. Almost certainly, disturbance of the thermal regime, in particular thawing of the permafrost resulting from the placement of tailings over the valley slope has been a contributing factor. Although detailed knowledge of the changes to the thermal regime that occurred during and following active placement of tailings is not known, it is possible that the thermal regime has still not reached a state of equilibrium.

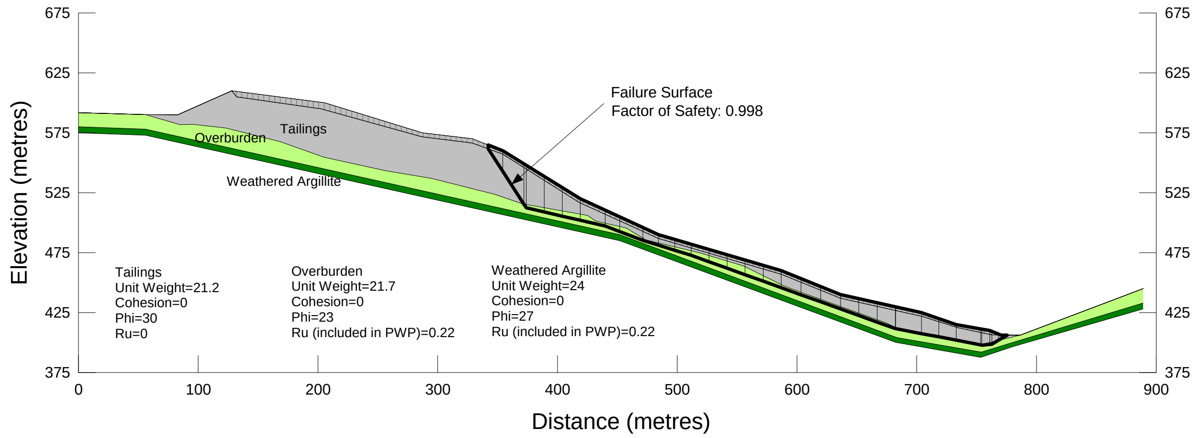

Figure 4-8) North Lobe – Existing Geometry

Given the limited site-specific geological and geotechnical information, there is considerable uncertainty in the absolute values of the factors of safety calculated from the back analysis. The slope stability model is however, considered sufficient to comment on and assess the relative improvement available through remedial options for the purposes of comparing remediation alternatives (stabilization) and selecting a preferred long term strategy for the same.

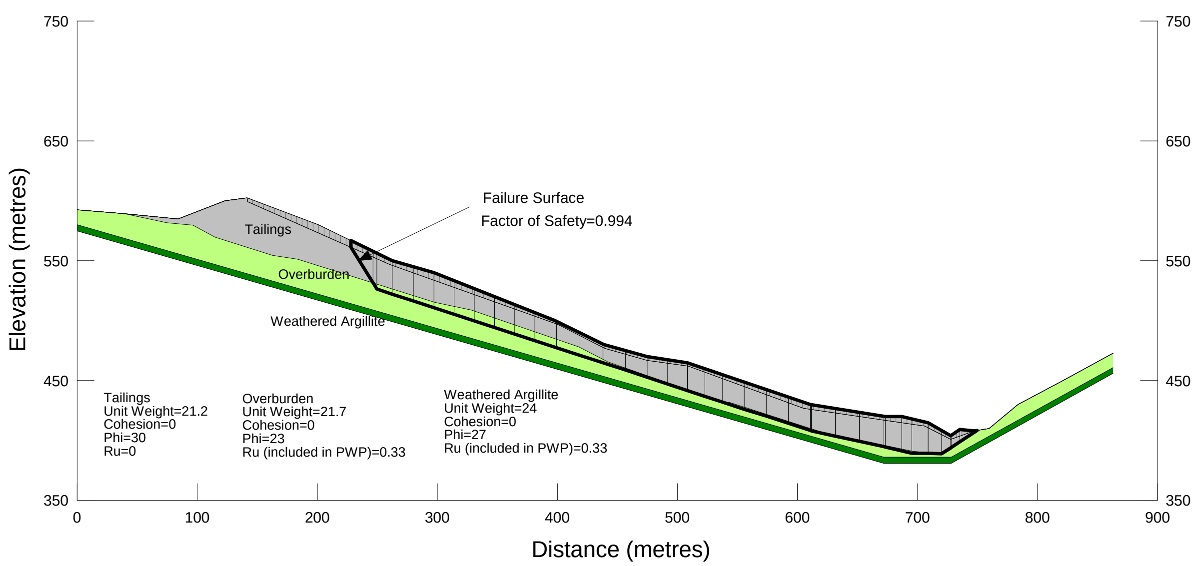

Figure 4-9) South Lobe – Existing Geometry

4.7 REMEDIATION ALTERNATIVES

Remediation alternatives must accommodate the on-going down slope movements of the tailings lobes or include measures to stabilize the tailings. Remedial strategies broadly fall into one of three categories:

i) Remove a sufficient volume of tailings from the valley and valley side slopes to completely drain the water impounded by the tailings and restore natural creek drainage,

ii) Stabilize the tailings piles and convey water over the tailings in a stabilized channel or,

iii) Convey water around the tailings via a tunnel.

Each of these alternatives requires that the tailings be handled with earth moving equipment resulting in air-borne (fugitive) asbestos dust particles. These effects are expected to be confined to the construction period and for a short time after. A protective crust layer has formed over the tailings pile since mine closure that minimizes the potential for wind and run-off erosion of asbestos particles. It is expected that this crust would re-develop following construction of remedial measures. Each remediation alternative is discussed in the following sections.

4.7.1 Valley Restoration

Of the options considered, removal of the tailings blocking the valley is the only alternative that restores natural creek drainage through the Wolverine Creek valley. Restoring the valley and the associated natural drainage has the benefit of significantly reducing or eliminating the risk associated with a breach of the tailings by the impounded water and the concern of chronic downstream sedimentation of tailings. To facilitate construction, drainage of the impounded water could be accomplished using pumps and/or siphons before removing tailings from the valley bottom. Removal of tailings from the side slopes of the valley would have to start at the upper slope and proceed in a downslope direction to prevent the development of slope instabilities. Based on previous monitoring and results from the slope stability analysis, it is anticipated that a portion of the tailings at the top of the valley could be stabilized by re-grading.

Based on existing cross sections, approximately 4,000,000 m^3 of tailings would have to be excavated to achieve a stable geometry. The excavated material could be disposed of in the open pits on the south side of Clinton creek and/or along the top of the ridge at the former mill area. An additional 1,000,000 m^3 of re-grading would be necessary to achieve stability of the tailings in the upper slope area. The area of excavated tailings and the final geometry of the tailings pile and Wolverine Valley are shown on Drawing 14. The estimated capital cost for valley restoration is in the order of $30,000,000. A detailed cost breakdown is summarized in Table 4-2.

Table 4-2) Valley Restoration - Cost Estimate

Description	Unit	Approximate Quantity	Unit Price	Amount
Mobilization & Demobilization	Lump Sum	1	$500,000	$500,000
Dewatering	Lump Sum	1	$250,000	$250,000
Restore & maintain haul road	Lump Sum	1	$200,000	$200,000
Excavation & hauling	Cubic Metre	4,000,000	$5	$20,000,000
Re-grading upper slope	Cubic Metre	1,000,000	$1	$1,000,000
Upgrade outlet under main road	Lump Sum	1	$100,000	$100,000
Subtotal				$22,050,000
30% Contingency				$6,615,000
Total Estimated Cost				**$28,665,000**

4.7.2 Convey Water Over Tailings Pile

The long term success of conveying water over the tailings in the bottom of Wolverine Creek valley is contingent on stabilizing the north and south lobes of the tailings pile. Once the tailings have been stabilized, water could be conveyed over the tailings in a channel that has been stabilized to minimize erosion. Conveyance of water through culverts buried in the tailings is not considered practical given the potential settlement and horizontal creep movements of the tailings and the potential for failure and/or blockages of the culvert.

4.7.2.1 Stabilization of Tailings Piles

Design Objective

Stabilization measures are typically designed with an objective to achieve a factor of safety (FS) that reflects the level of confidence in the interpretation of site and geological conditions and the consequences of continued movement or a slope failure. Although a high degree of uncertainty exists with respect to the site and geological conditions, a minimum factor of safety of 1.25 has been used as the design objective for the conceptual design and cost estimating of remedial measures. The use of this FS for final design is contingent on conducting more detailed site investigations to collect additional information on soil properties, permafrost and piezometric levels. If this additional information is not obtained, a FS of 1.50 should be used for final design of remedial measures. It should be recognized however, that the incremental increase in capital costs associated with higher factors of safety could easily offset the cost of any additional site investigations that is, construction costs could conceivably double if a FS of 1.50 is required.

Stability Analysis

Slope stability analyses were carried out to determine a revised geometry for the tailings pile that would achieve a minimum overall FS of 1.25. In general, this would be accomplished by re-grading the tailings to off-load material (reduce driving forces) from the upper portion of the slope and adding material at the toe (increase resisting forces). Two cases were analyzed; the first case assumes that the entire tailings pile, including the portion upslope of the head scarps, is unstable and has to be re-graded. The second case assumes that the tailings upslope of the head scarps can be stabilized independently of the active slide material. In both cases, the creek channel would be routed across the toe of the tailings. Without the benefit of more detailed subsurface information it is not possible to determine with confidence which case is most appropriate. The final re-grading plan would be completed during detailed design.

For both cases, the slope of the tailings was progressively flattened and the elevation of the tailings at the toe increased until the design objective was met. To achieve an FS of 1.25, the tailings at the toe must be increased to elevation 422.0m and 415.0m for Cases A and B respectively. These elevations are approximately 12m and 5m higher than the existing tailings surface at the toe. Cross sections through the north and south lobes for each option are shown in plan and section on Drawings 15 (Case A) and 16 (Case B).

In both cases, the re-grading plan includes in-filling the ponded areas between the north and south lobes (to the same final elevation as the north and south lobes) and upstream of the north lobe (to approximately the same elevation as the existing water surface). Filling these areas will reduce the volume of water impounded after re-grading the tailings and decease the potential for a breach by increasing the length of the channel across the tailings.

Construction Considerations

The cut volume for Case A is approximately 2,700,000m^3, of which approximately 2,250,000m^3 would be used to re-grade the lower slope of the tailings lobes and fill the existing Wolverine Creek channel. The remainder (450,000 m^3) could be placed in the vicinity of the former mill site. The cut volume for Case B is approximately 2,600,000m^3, of which approximately 860,000 m^3 would be used to re-grade the lower slope of the tailings lobes and fill the existing Wolverine Creek channel. The remainder 1,740,000 m^3 could be placed in the vicinity of the former mill site.

Depending on the time of year when construction is undertaken, it may be necessary to control discharge from Wolverine Creek. This could be accomplished by drawing down the impounded water level prior to construction and/or filling in the existing channel through the tailings and allowing water levels to rise upstream of the tailings for the construction period. Draw down of the impounded water between the north and south lobes and upstream of the north lobe may result in some localized instabilities at the toe of the natural valley side slopes. Assuming an average creek flow of about 0.25 m^3/sec during the summer, an impounded water volume up to 660,000 m^3 and a pumping capacity of 75 m^3/min, approximately one week to ten days would be required to pump the impounded water past the tailings. After pumping at this rate is discontinued, it would take about one month for water levels to recover to the pre-pumping level. Alternatively, the average creek flow and impounded water level could probably be maintained by pumping or diverting the water flow around the construction site.

4.7.2.2 Channel Stabilization

For both cases, a new channel would have to be constructed across the toe of the re-graded tailings. The new channel will require erosion protection and should be compatible with any creep movements of the tailings pile that may occur following re-grading. In this regard, channel stabilization across the re-graded north and south lobes can be achieved using a relatively flat grade and lining the channel with a non-woven geotexile covered with rock filled gabion mats or granular material large enough to resist erosion. Rigid structures (e.g. concrete linings) should be avoided due to the risk of cracking and subsequent failure. The proposed channel cross-section and profile for Case B is shown on Drawing 17. A transition between the new channel across the tailings and the existing rock-lined channel will be required to achieve the hydraulic drop of 17m or 10m for Case A and Case B, respectively. This transition could be constructed using gabion drop structures and appropriately sized granular materials.

The tailings on either side of the channel should be graded smoothly towards the rock-lined channel and any existing gullies should be filled with tailings. Run-off water could be slowed down by placing small check dams made of rocks across the tailings in a 'V' pattern pointing down slope. The presence of vegetation along the rock lined portion of the channel suggests that it may be possible to establish vegetation in the tailings adjacent to the rock-lined channel if moisture can be preserved to support plant growth (Figure 4-10). Since the establishment of vegetation would help to provide long-term stabilization of the tailings, growth could be encouraged by spreading organic matter such as wood chips, mulch or coniferous trees over the tailings in this area. Rehabilitation of the channel outlet at the confluence with Clinton Creek should also be considered to prevent future overtopping and potential washout of the mine access road and erosion at the downstream end of the culverts.

The estimated capital costs for stabilization of the tailings piles and creek channel are in the order of $6,000,000 and $5,500,000 for Case A and Case B, respectively. Detailed cost breakdowns are summarized in Table 4-3 (Case A) and Table 4-4 (Case B).

Figure 4-10) Vegetation Along Rock-Lined Channel

Table 4-3) Cost Estimate For Case A

Description	Unit	Approximate Quantity	Unit Price	Amount
Mobilization & Demobilization	Lump Sum	1	$500,000	$500,000
Dewatering	Lump Sum	1	$250,000	$250,000
Re-grading	Cubic Metre	2,700,000	$1	$2,700,000
Channel Excavation	Cubic Metre	45,000	$3	$135,000
Channel Erosion Protection	Lump Sum	1	$225,000	$225,000
Gabion Drop Structures	Lump Sum	1	$500,000	$500,000
Rehabilitate Rock-Lined Channel	Lump Sum	1	$100,000	$100,000
Upgrade outfall at main road	Lump Sum	1	$100,000	$100,000
Subtotal				$4,510,000
30% Contingency				$1,353,000
Total Estimated Cost				**$5,863,000**

Table 4-4) Cost Estimate For Case B

Description	Unit	Approximate Quantity	Unit Price	Amount
Mobilization & Demobilization	Lump Sum	1	$500,000	$500,000
Dewatering	Lump Sum	1	$250,000	$250,000
Re-grading	Cubic Metre	2,600,000	$1	$2,600,000
Channel Excavation	Cubic Metre	45,000	$3	$135,000
Channel Erosion Protection	Lump Sum	1	$225,000	$225,000
Gabion Drop Structures	Lump Sum	1	$250,000	$250,000
Rehabilitate Rock-Lined Channel	Lump Sum	1	$100,000	$100,000
Upgrade outfall at main road	Lump Sum	1	$100,000	$100,000
Subtotal				$4,160,000
30% Contingency				$1,248,000
Total Estimated Cost				**$5,408,000**

4.7.3 Convey Water Around Tailings

Conveyance of flow from Wolverine Creek around the unstable tailings through a concrete lined tunnel or directionally drilled, steel or PVC lined tunnel was also considered. The inlet structure for the tunnel would likely be located upstream of the north lobe on the east side of the valley. The outlet would be located in the Wolverine creek valley near the down stream limit of the rock-lined channel (Drawing 18). The total length for the tunnel alignment shown on Drawing 18 is approximately 700m. It would be necessary to partially infill the valley at the toe of the tailings pile to approximately Elevation 412m to construct an emergency overflow channel in the event the tunnel entrance is blocked.

A tunnel diameter on the order of 2.0m would be required to convey the estimated 200-year flood (17 m^3/s) (UMA 2000). The full supply level (FSL) would be set around elevation 405m (approximately the current impounded water elevation) and the crown of the tunnel would be placed at the same level. The proposed FSL will provide a live storage of 5m between the overflow crest at the tunnel inlet and the outflow level of the overflow channel across the tailings to generate sufficient head for the tunnel flow. To allow isolation of the tunnel for inspection and maintenance, a low-head sluice gate would be installed at the inlet. Permanent access to the inlet of the tunnel for future cleaning and maintenance could be achieved by constructing a road across the toe of the re-graded tailings. A short section of the road may have to be constructed along the east side of the valley in order to reach the outlet.

The estimated capital cost for tunnelling (based on a conventional concrete lined tunnel) is in the order of $10,000,000. A detailed cost breakdown is summarized in Table 4-5.

Table 4-5) Conveyance of Water Around Tailings – Cost Estimate

Description	Unit	Approximate Quantity	Unit Price	Amount
Mobilization & Demobilization	Lump Sum	1	$500,000	$500,000
Dewatering	Lump Sum	1	$250,000	$250,000
Permanent Access Road to Inlet	Lump Sum	1	$250,000	$250,000
Tunneling	Per Metre	700	$6,000	$4,200,000
Inlet and Outlet Structures	Lump Sum	1	$2,000,000	$2,000,000
Fill In Existing Channel	Cubic Metre	50,000	$3	$150,000
Overflow Channel Excavation	Cubic Metre	45,000	$3	$135,000
Channel Stabilization	Lump Sum	1	$225,000	$225,000
Gabion Drop Structures	Lump Sum	1	$100,000	$100,000
Upgrade Outfall at Main Road	Lump Sum	1	$100,000	$100,000
Subtotal				$7,910,000
30% Contingency				$2,373,000
Total Estimated Cost				**$10,283,000**

5.0 CONCLUSIONS AND RECOMMENDATIONS

Gabion drop structures are recommended to stabilize the Clinton Creek channel in order to mitigate the potential of a catastrophic breach of the landslide blockage at the outlet of Hudgeon Lake and reduce the chronic downstream erosion and re-distribution of the waste rock material. The results from the waste rock monitoring completed in June 2001 confirms previous (1999) observations by UMA that the waste rock pile movements are small (approximately 7 cm/year) and likely decreasing with time. Gabion drop structures are preferred because they are flexible enough to accommodate lateral creep movements of the waste rock pile. It is possible that the partial channel infilling required to construct the gabions and the prevention of further toe erosion will provide an improvement in slope stability to further reduce waste rock dump movements and possibly eliminate the requirement for waste rock re-grading. The estimated cost to complete channel stabilization is in the range of $2,500,000 to $3,000,000. If waste rock stabilization is required, the estimated capital cost would be in the order of $6,000,000 (including channel stabilization).

Given the possibility of conditions worsening at the outlet before long-term remedial measures are implemented, stabilization of the Clinton Creek waste rock channel could be completed in stages, which would allow the most immediate concern (the condition of the outlet) to be addressed before stabilizing the entire length of the channel. This strategy would address the most immediate concern regarding the potential of a catastrophic breach of the waste rock plug at the lake outlet. Erosion of the waste rock channel will however, continue to occur downstream of the stabilized section. The outlet stabilization works can be incorporated into the overall channel stabilization measures. The estimated construction costs to stabilize a 150m long section of the channel immediately downstream of the Hudgeon lake outlet is in the order of $500,000.

Stability analysis of the tailings pile indicates a shallow weak layer within the overburden and continued toe erosion are likely responsible for continued movement of the tailings pile. The loss of strength in this layer may be related to a number of geological conditions unique to the site including ice content of the permafrost, soil type and the relationship between the rate of thawing and dissipation of excess pore-water pressures. It is likely that disturbance to the thermal regime, including thawing of permafrost beneath the tailings, has resulted from placement of tailings over the valley slope.

Several remediation alternatives were considered to mitigate the existing hazards associated with on-going erosion of the tailings and a breach of the tailings should Wolverine Creek become completely blocked. Remedial strategies broadly fall into one of three categories:

i) Remove a sufficient volume of tailings from the valley and valley side slopes to completely drain the water impounded by the tailings and restore natural creek drainage,

ii) Stabilize the tailings piles and convey water over the tailings in a stabilized channel or,

iii) Convey water around the tailings via a tunnel.

Significant capital costs are associated with these options, ranging from about $5,500,000 to stabilize the tailings and construct a stabilized creek channel to about $30,000,000 to remove a sufficient amount of the tailings to restore natural creek drainage. These options have been evaluated in concept only. Should the implementation of remedial measures be considered, the work completed to date using the available information is only considered sufficient to select a preferred alternative. Upon the selection of a preferred remedial repair alternative, a feasibility study including detailed field investigations is recommended to examine the technical feasibility of the preferred option and to provide required information for final design and construction cost estimates. The level of detailed field investigations required will depend on the selected alternative.

It is recommended that the waste rock monitoring program established in 1999 be extended to confirm if there are on-going waste rock movements and to determine if stabilization of the waste rock dump is necessary. Consideration should also be given to include monitoring of the tailings piles to establish current movement rates. If we can be of further assistance, please contact the undersigned.

Respectfully Submitted,

UMA Engineering

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Senior Geotechnical Engineer
Earth and Environmental

Gil Robinson, M.Sc.
Geotechnical Engineer
Earth and Environmental

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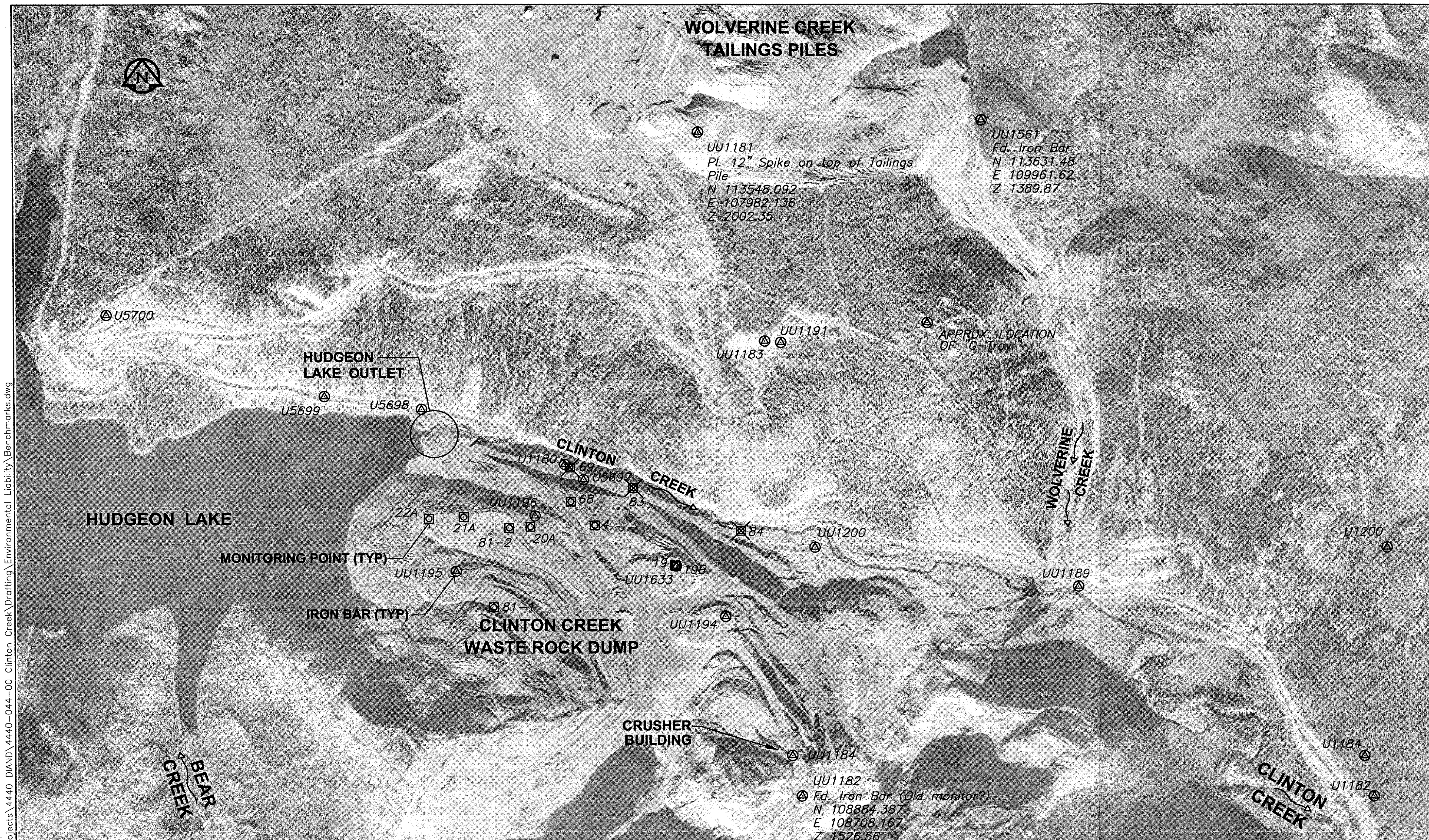
Appendix A

Waste Rock Monitoring Results



UU1196	BENCHMARK
20A	MONITOR LOCATION (ACTIVE)
83	MONITOR LOCATION (IN-ACTIVE)

REV.	DESCRIPTION	DWN.	APP.	DATE

SEAL

UMA Engineering Ltd.
• Consulting • Engineering • Construction • Management Services

APPROVED BY	DATE: JUNE, 2003
DRAWN BY: LJV	DESIGNED BY: –
CHECKED BY: GR	CHECKED BY: –
SCALE: 1:7,500	JOB No. 4440-044-00

INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE

LOCATIONS OF WASTE ROCK MONITORING MONUMENTS AND BENCHMARKS

DWG. No. REV.

Plot Scale: 1=1 mm
L:\Earth & Water\Projects\4440 DIAND\4440-044-00 Clinton Creek\Drafting\Environmental Liability\Benchmarks.dwg

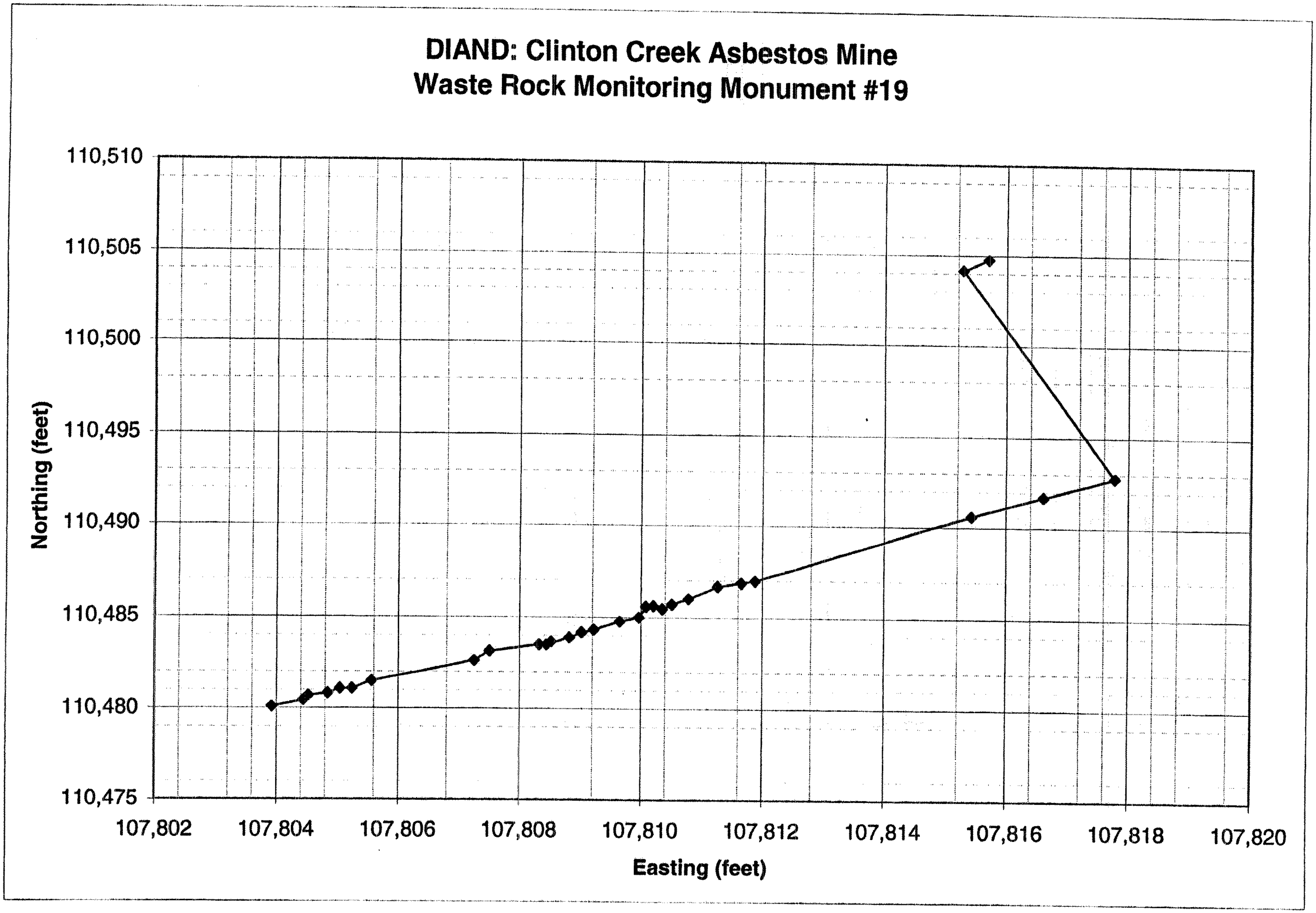
Abandoned Clinton Creek Asbestos Mine
Waste Rock Monitoring Program
Benchmarks and Monitor Points

Benchmark	Northing (feet)	Easting (feet)	Elevation (feet)
U1200*	110,632.27	108,799.62	1,231.10
U1182*	108,884.39	108,708.17	1,526.56
U1184*	109,166.80	108,640.99	1,518.80
U1189*	110,356.82	110,639.63	1,204.92
U5698*	111,604.30	106,054.56	1,360.96
U5699*	111,695.03	105,376.12	1,395.37
U5700*	112,267.10	103,861.29	1,578.49
U1180**	111215.34	107049.71	1358.37
U1194**	110,144.34	108,171.91	1,420.67
U1195**	110,464.31	106,292.95	1,497.42
U1196**	110,853.26	106,841.76	1,456.57
U5697**	111,108.38	107,182.30	1,362.63

*Control used and set by Underhill Surveys in 1999.
**Control used and set by Underhill Surveys in 2001.

Monitor Point	Northing (feet)	Easting (feet)	Elevation* (feet)
19	110,504.74	107,815.70	1,410.19
20-A	110,776.44	106,811.57	1,466.50
21-A	110,845.83	106,345.81	1,468.55
22-A	110,833.78	106,104.01	1,463.49
68	110,953.15	107,092.80	1,424.95
81-1	110,209.88	106,552.11	1,497.75
81-2	110,769.03	106,662.33	1,459.58

*Elevation at top of rebar, not ground surface.



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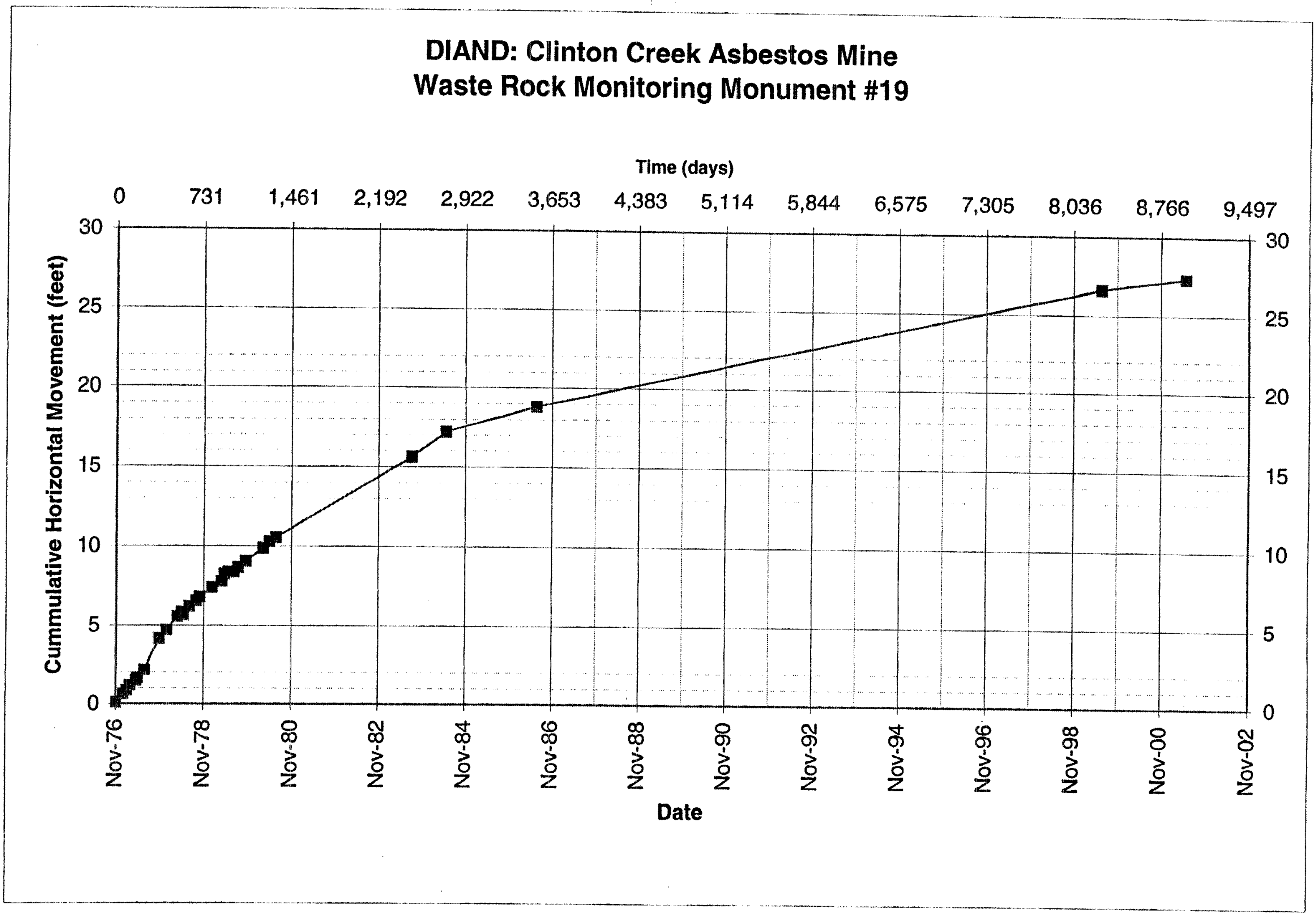
Date: 5/30/2002



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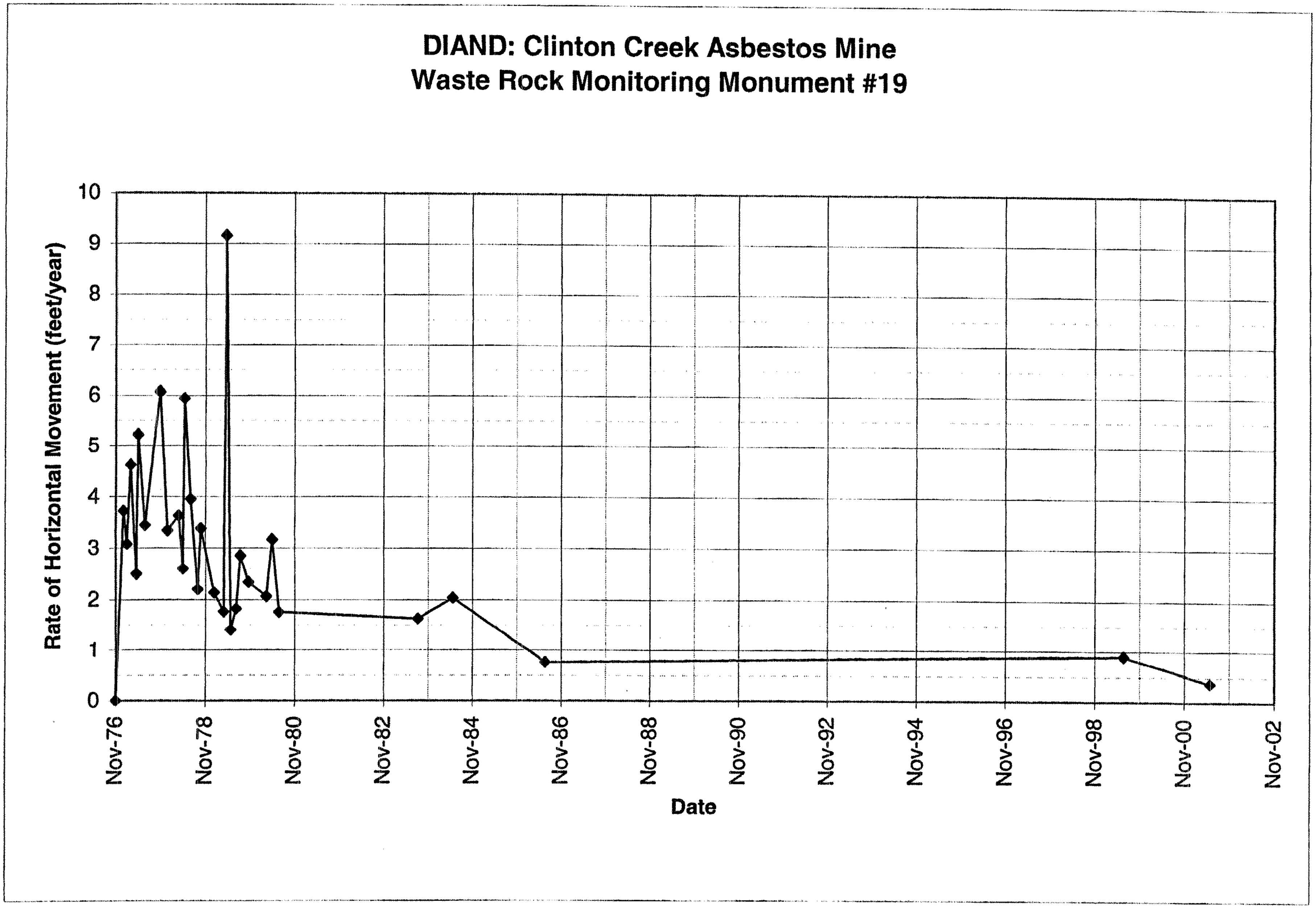

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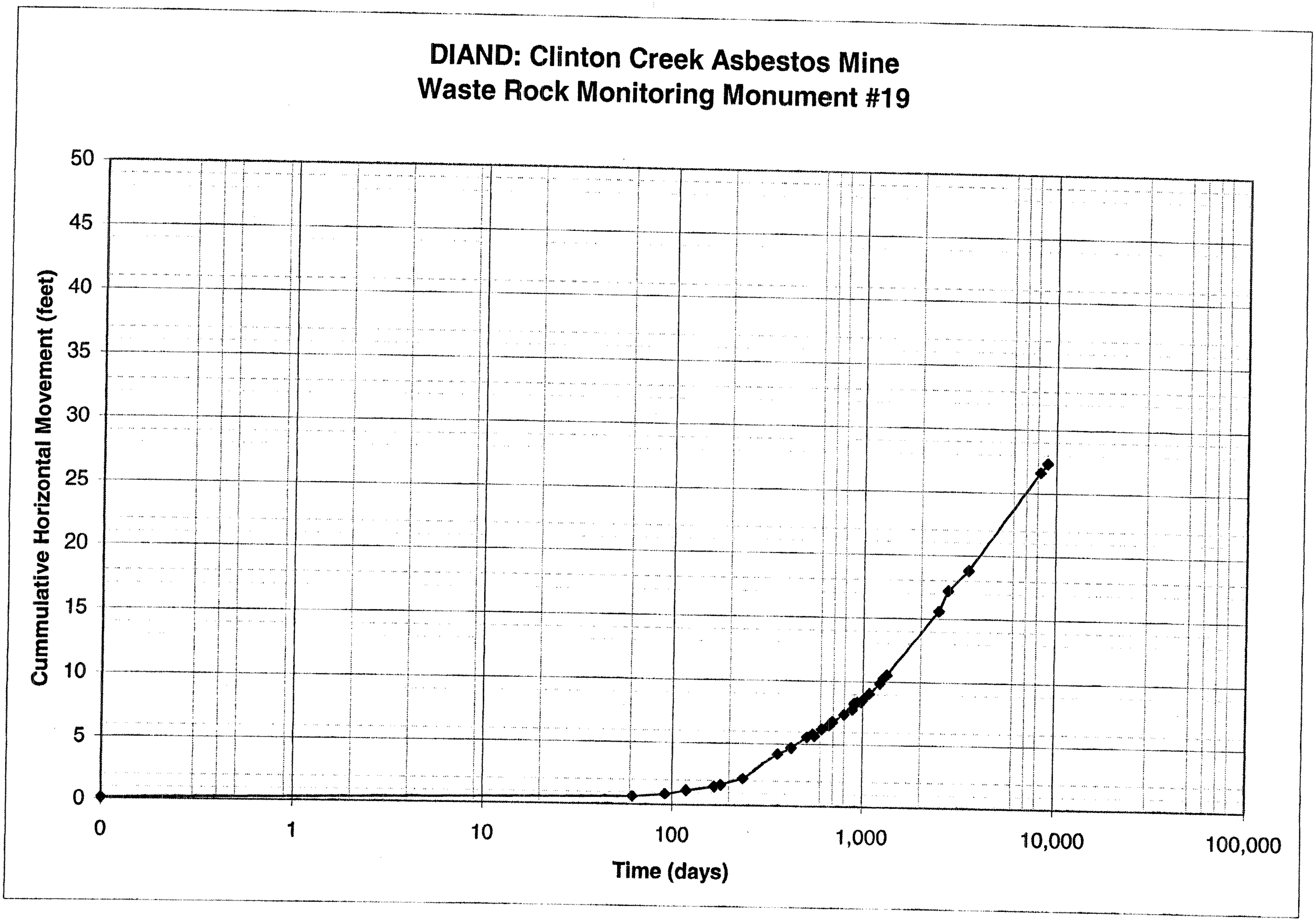

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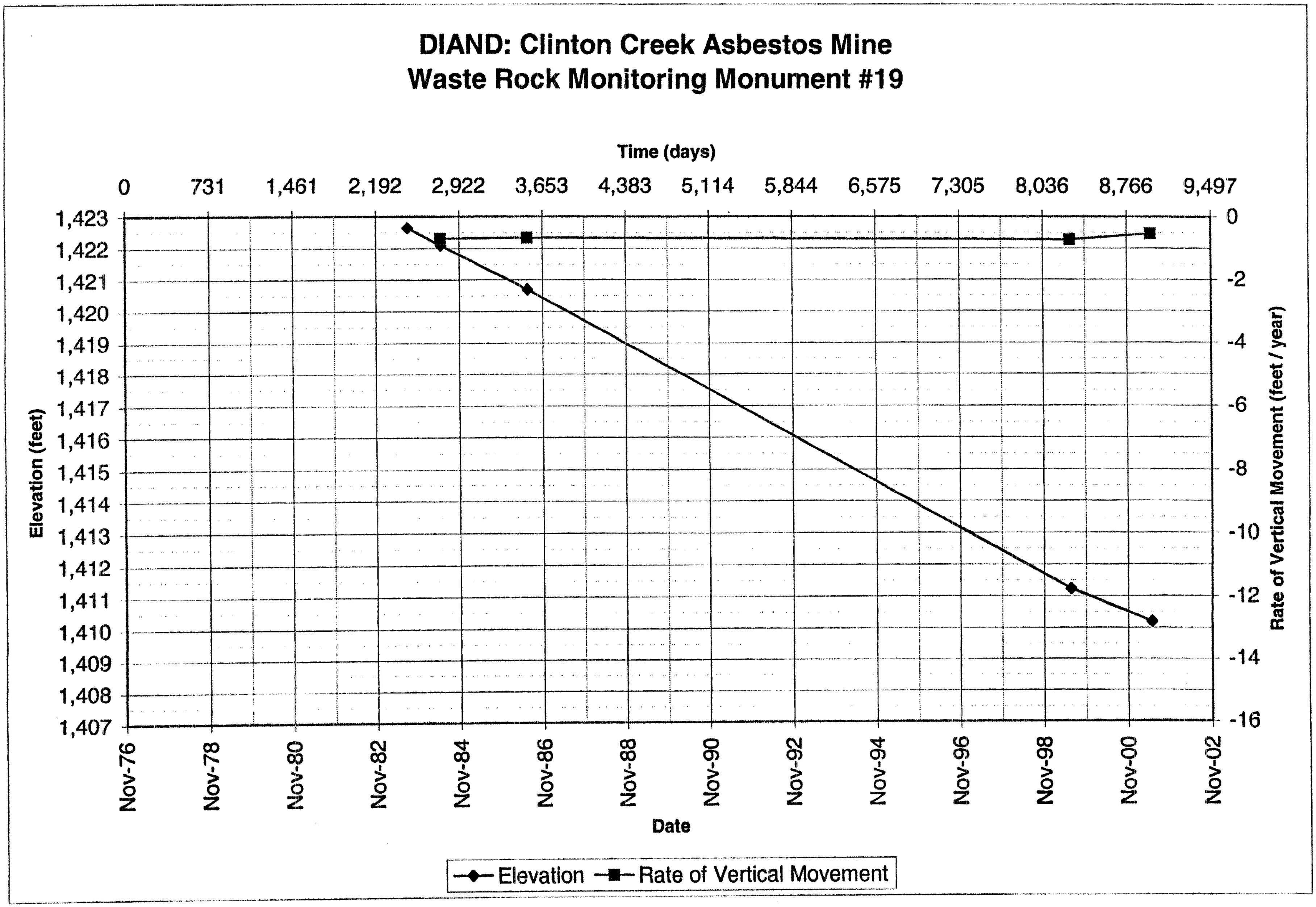
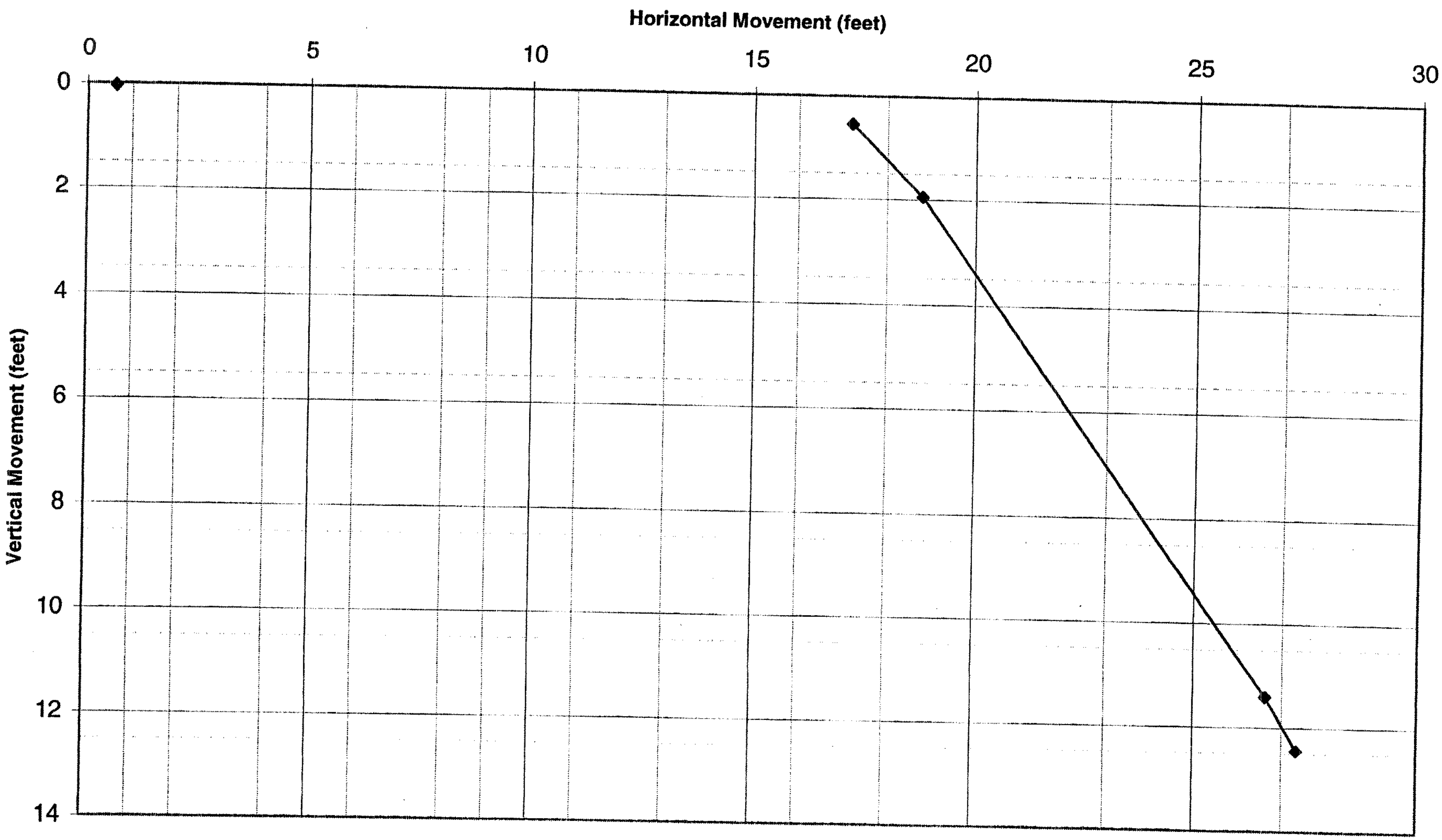
Date: 5/30/2002



File: monitoring data #19.xls Tab: #19-vert mvmnt

Date: 5/30/2002

DIAND: Clinton Creek Asbestos Mine
Waste Rock Monitoring Monument #19



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Date: 5/30/2002

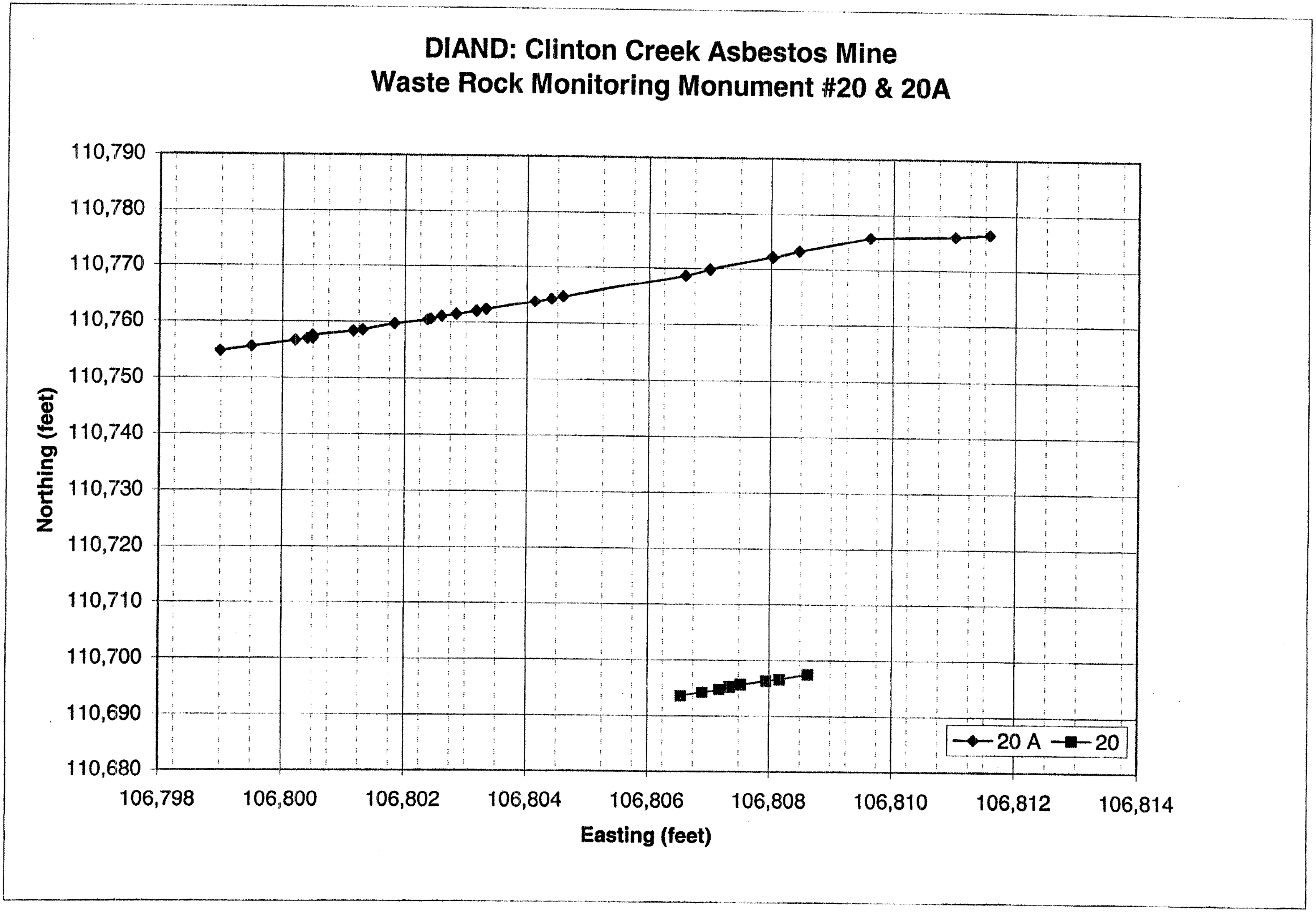
Client: DIAND
Project: Clinton Creek Asbestos Mine
Job No.: 4440-038-02-02
Date: 9-Jul-01

Waste Dump Stability - Monitoring Point #19

Notes: Assume all elevations represent top of monitoring point, not ground surface.
June 2001 survey) monitor point elevation = ground elev + monitor rod ht.

Interpolated values

Monitoring	Northing	Easting	Elevation	Time		Horizontal Movement			Vertical Movement		
Date				Total	Incremental	total	incremental	rate	total	incremental	rate
	(feet)	(feet)	(feet)	(days)	(days)	(feet)	(feet)	(feet/year)	(feet)	(feet)	(feet / year)
24-Nov-76	110,480.08	107,803.92		0	0	0	0	0			
25-Jan-77	110,480.44	107,804.44		62.0	62.0	0.63	0.63	3.723	0.04	0.04	0.24
24-Feb-77	110,480.68	107,804.52		92.0	30.0	0.85	0.25	3.078			
23-Mar-77	110,480.80	107,804.84		119.0	27.0	1.17	0.34	4.620			
10-May-77	110,481.06	107,805.04		167.0	48.0	1.49	0.33	2.494			
24-May-77	110,481.06	107,805.24		181.0	14.0	1.64	0.20	5.214			
19-Jul-77	110,481.48	107,805.56		237.0	56.0	2.16	0.53	3.442			
18-Nov-77	110,482.60	107,807.25		359.0	122.0	4.18	2.03	6.066			
20-Jan-78	110,483.12	107,807.50		422.0	63.0	4.70	0.58	3.343			
20-Apr-78	110,483.48	107,808.32		512.0	90.0	5.56	0.90	3.632			
26-May-78	110,483.64	107,808.52		548.0	36.0	5.82	0.26	2.597			
6-Jun-78	110,483.48	107,808.44		559.0	11.0	5.66	0.18	5.936			
27-Jul-78	110,483.88	107,808.82		610.0	51.0	6.20	0.55	3.949			
22-Sep-78	110,484.16	107,809.02		667.0	57.0	6.53	0.34	2.203			
19-Oct-78	110,484.31	107,809.22		694.0	27.0	6.78	0.25	3.380			
1-Feb-79	110,484.75	107,809.65		799.0	105.0	7.39	0.62	2.139			
22-Apr-79	110,484.98	107,809.96		879.0	80.0	7.78	0.39	1.761			
16-May-79	110,485.57	107,810.08		903.0	24.0	8.25	0.60	9.157			
18-Jun-79	110,485.61	107,810.20		936.0	33.0	8.37	0.13	1.399			
1-Aug-79	110,485.45	107,810.35		980.0	44.0	8.38	0.22	1.819			
7-Sep-79	110,485.69	107,810.51		1017.0	37.0	8.65	0.29	2.845			
10-Nov-79	110,486.00	107,810.78		1081.0	64.0	9.06	0.41	2.345			
4-Apr-80	110,486.67	107,811.26		1227.0	146.0	9.86	0.82	2.060			
24-May-80	110,486.86	107,811.65		1277.0	50.0	10.28	0.43	3.167			
17-Jul-80	110,486.98	107,811.88		1331.0	54.0	10.53	0.26	1.754			
1-Sep-83	110,490.61	107,815.43	1422.64	2472.0	1141.0	15.60	5.08	1.624			
14-Jun-84	110,491.69	107,816.61	1422.09	2759.0	287.0	17.20	1.60	2.034	0.55	-0.55	-0.70
15-Jul-86	110,492.77	107,817.78	1420.69	3520.0	761.0	18.79	1.59	0.764	1.95	-1.40	-0.67
16-Jul-99	110,504.16	107,815.28	1411.23	8269.0	4749.0	26.63	11.66	0.896	11.41	-9.46	-0.73
19-Jun-01	110,504.74	107,815.70	1410.19	8973.0	704.0	27.33	0.72	0.371	12.45	-1.04	-0.54


DIAND: Clinton Creek Asbestos Mine
Waste Rock Monitoring Monument #20 & 20A
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110,780
110,770
110,760
110,750
110,740
110,730
110,720
110,710
110,700
110,690
110,680
Northing (feet)
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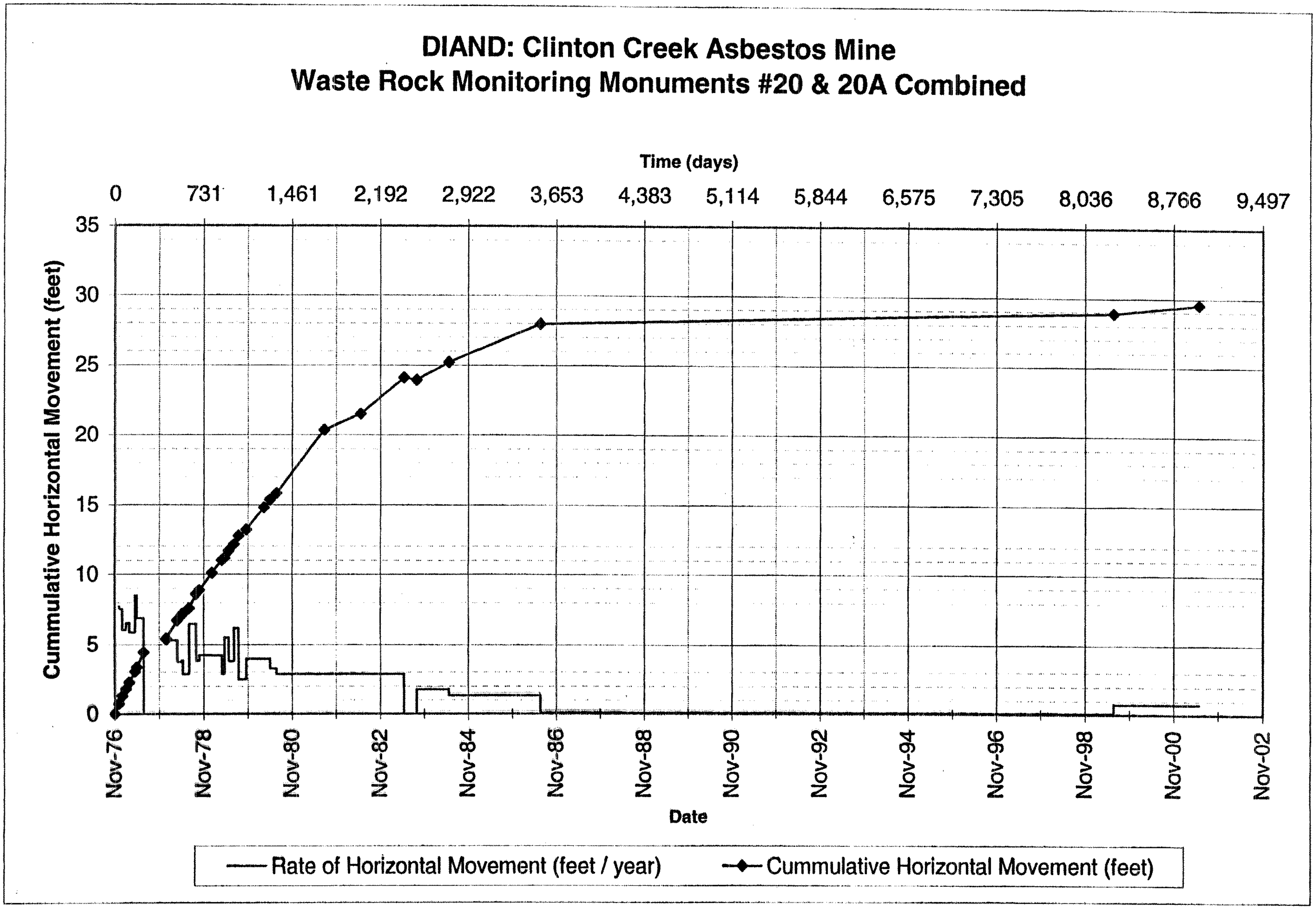
106,800
106,802
106,804
106,806
106,808
106,810
106,812
106,814
Easting (feet)
20 A
20



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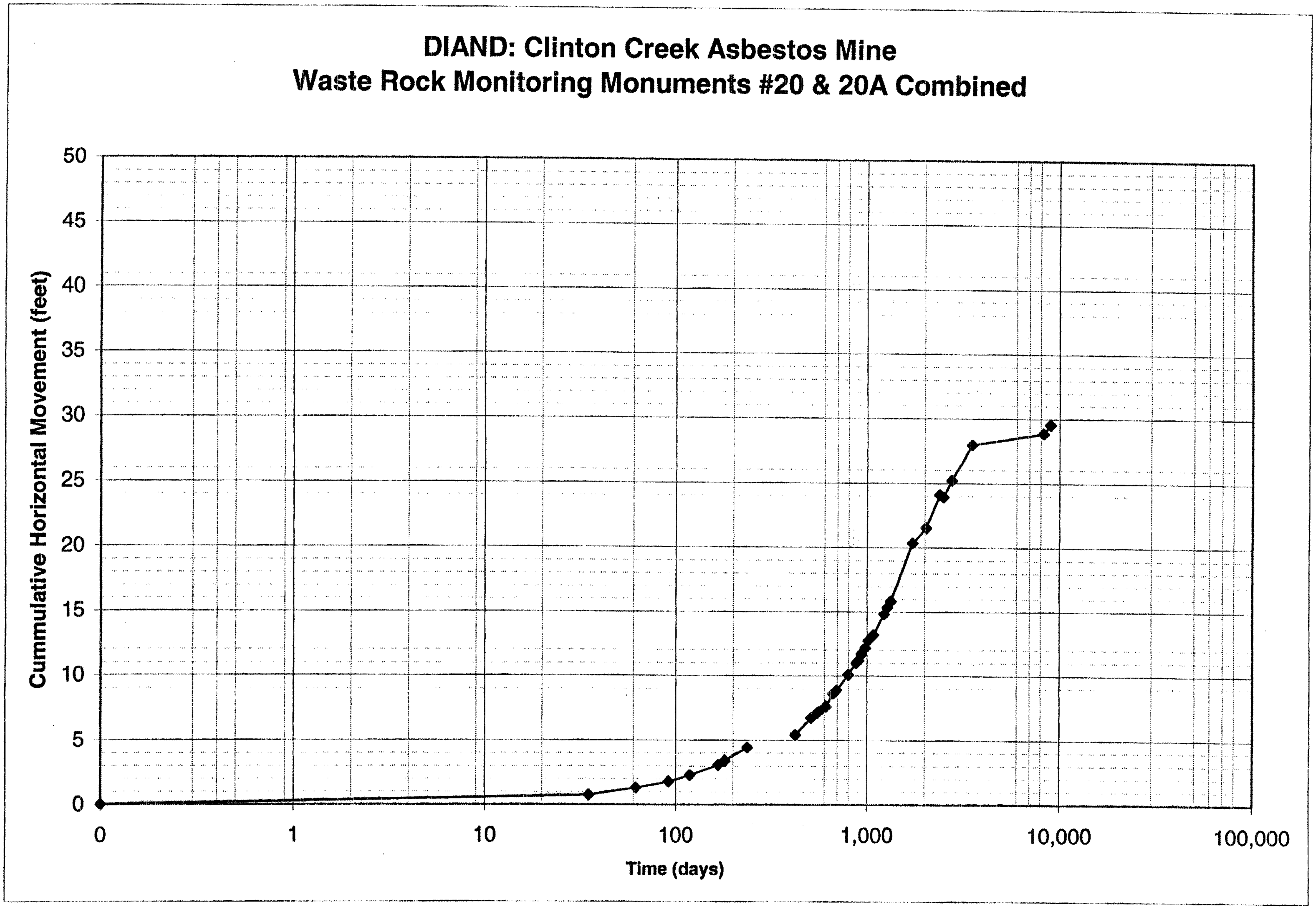

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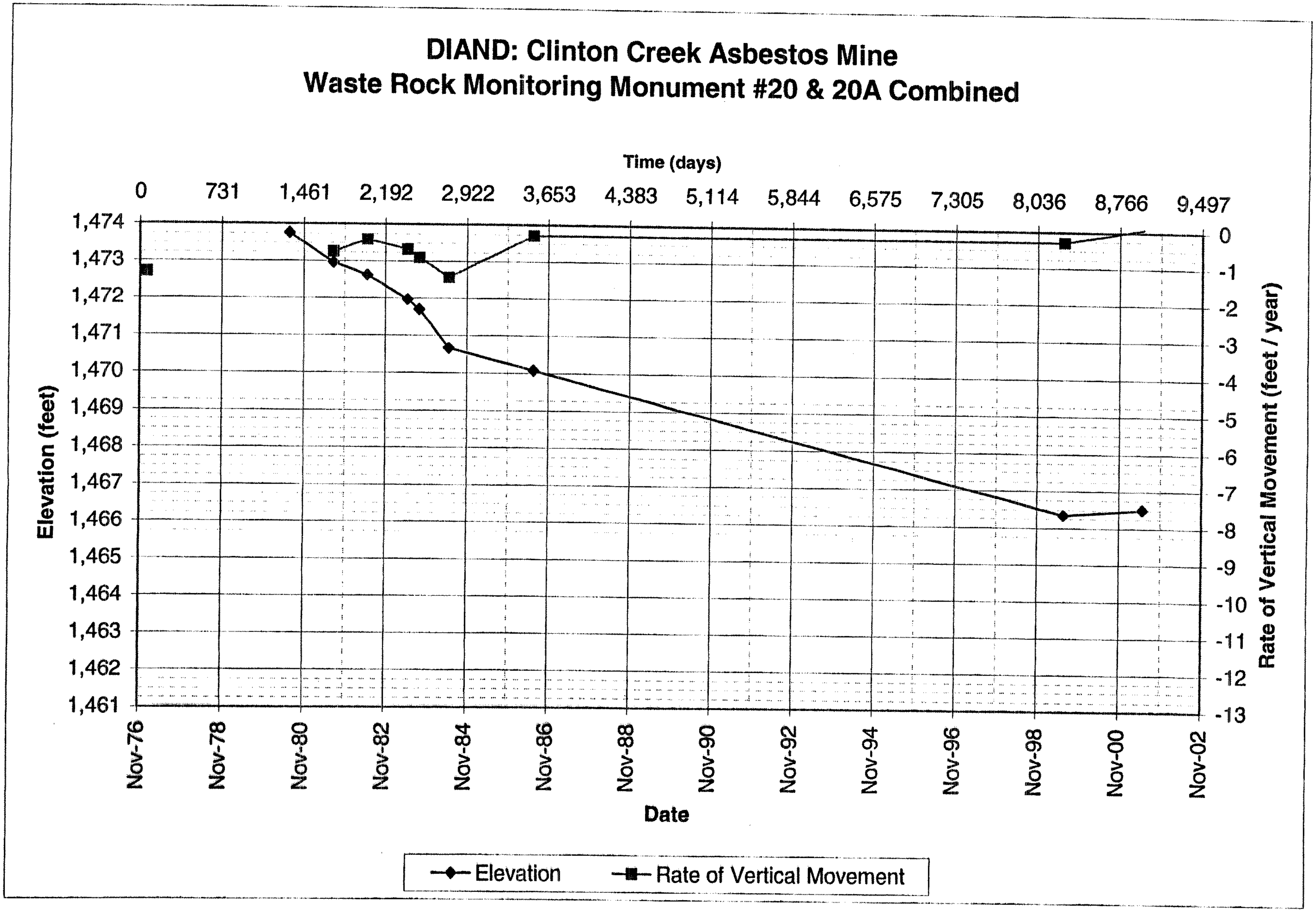
Date: 5/30/2002



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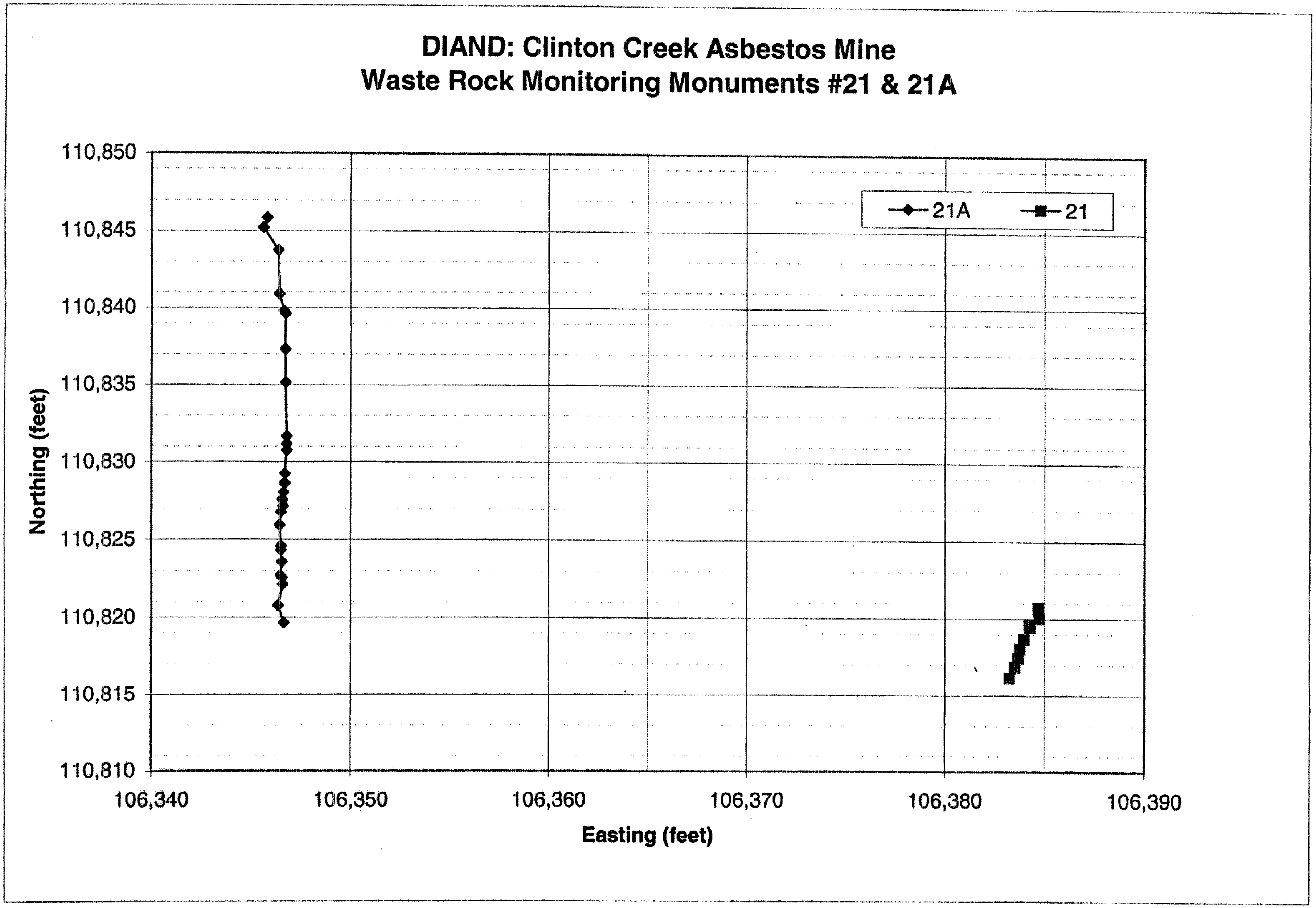
Date: 5/30/2002

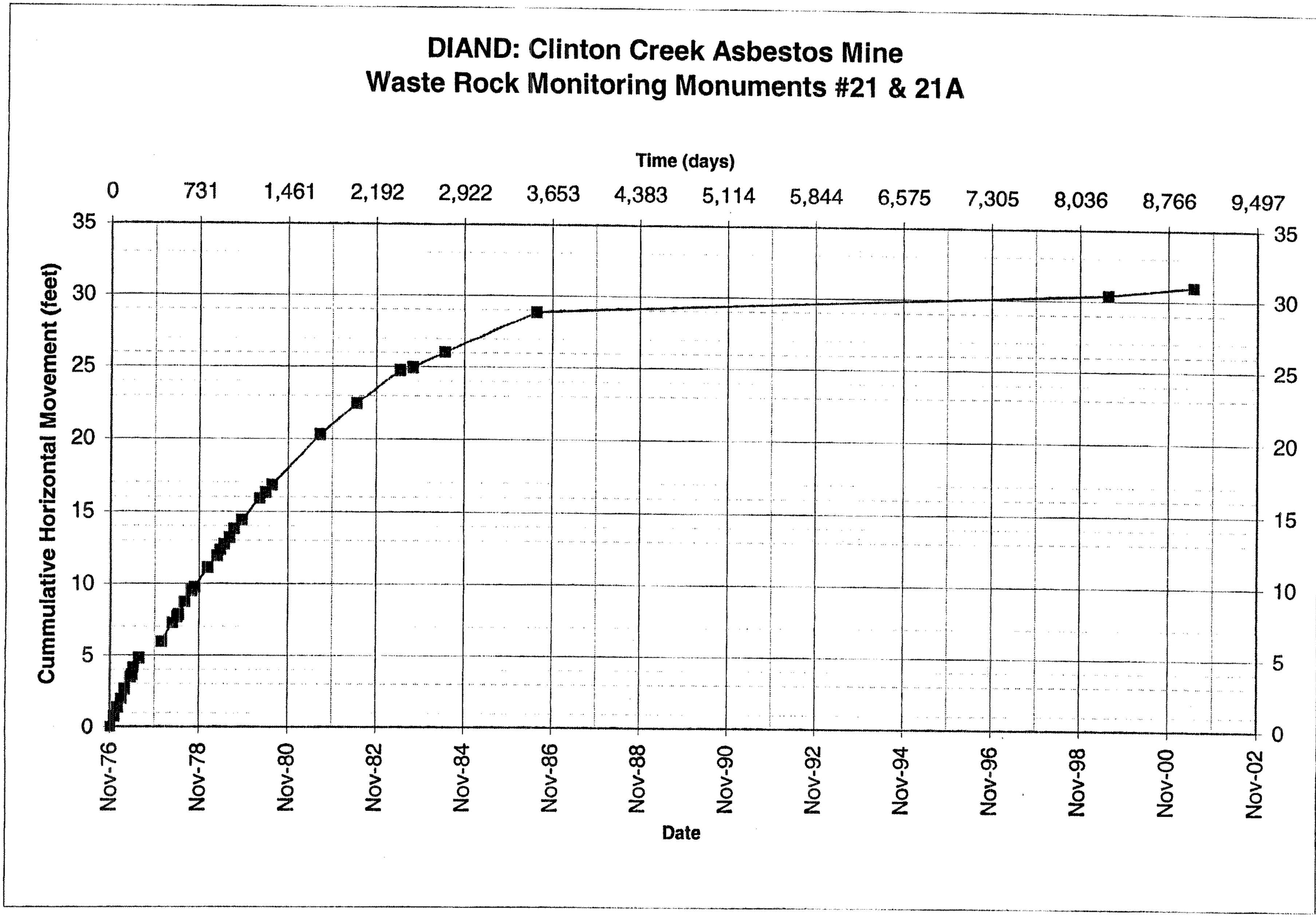
Client: DIAND
Project: Clinton Creek Asbestos Mine
Job No.: 4440-038-02-02
Date: 9-Jul-01

Waste Dump Stability - Monitoring Point #20A

Notes: Assume all elevations represent top of monitoring point, not ground surface.
June 2001 survey) monitor point elevation = ground elev + monitor rod ht.
Interpolated values

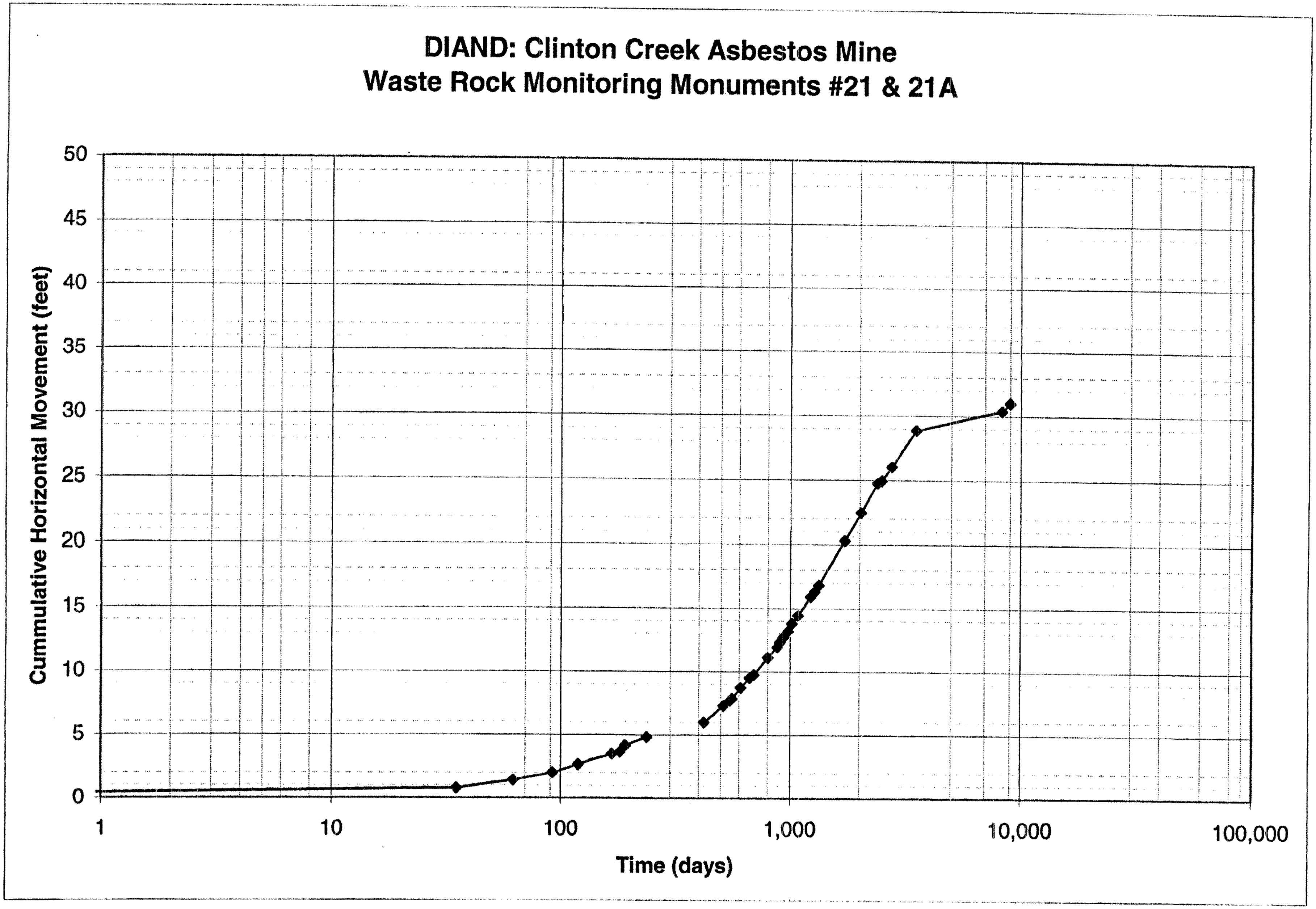
Monitoring Date	Northing (feet)	Easting (feet)	Elevation (feet)	Time: Total (days)	Time: Incremental (days)	Horizontal Movement: total (feet)	Horizontal Movement: incremental (feet)	Horizontal Movement: rate (feet/year)	Vertical Movement: total (feet)	Vertical Movement: incremental (feet)	Vertical Movement: rate (feet / year)
#20											
24-Nov-76	110,693.60	106,806.55		0	0	0	0	0			
29-Dec-76	110,694.25	106,806.90		35.0	35.0	0.74	0.74	7.699			
25-Jan-77	110,694.73	106,807.18		62.0	27.0	1.29	0.56	7.512	0.22		
24-Feb-77	110,695.20	106,807.35		92.0	30.0	1.79	0.50	6.081			
23-Mar-77	110,695.65	106,807.53		119.0	27.0	2.27	0.48	6.552			
10-May-77	110,696.30	106,807.95		167.0	48.0	3.04	0.77	5.885			
24-May-77	110,696.55	106,808.17		181.0	14.0	3.37	0.33	8.682			
19-Jul-77	110,697.50	106,808.63		237.0	56.0	4.42	1.06	6.880			
#20A											
18-Nov-77	110,754.71	106,798.99		0	0	0	0	0			
20-Jan-78	110,755.53	106,799.50		63.0	63.0	0.97	0.97	5.595			
20-Apr-78	110,756.63	106,800.21		153.0	90.0	2.27	1.31	5.310			
26-May-78	110,756.94	106,800.41		189.0	36.0	2.64	0.37	3.740			
6-Jun-78	110,757.02	106,800.50		200.0	11.0	2.76	0.12	3.996			
27-Jul-78	110,757.49	106,800.49		251.0	51.0	3.16	0.47	3.364			
22-Sep-78	110,758.27	106,801.16		308.0	57.0	4.17	1.03	6.584			
19-Oct-78	110,758.51	106,801.31		335.0	27.0	4.45	0.28	3.826			
1-Feb-79	110,759.61	106,801.83		440.0	105.0	5.66	1.22	4.230			
22-Apr-79	110,760.36	106,802.37		520.0	80.0	6.58	0.92	4.217			
16-May-79	110,760.55	106,802.42		544.0	24.0	6.77	0.20	2.988			
18-Jun-79	110,761.02	106,802.60		577.0	33.0	7.27	0.50	5.567			
1-Aug-79	110,761.41	106,802.84		621.0	44.0	7.73	0.46	3.799			
7-Sep-79	110,761.94	106,803.17		658.0	37.0	8.35	0.62	6.159			
10-Nov-79	110,762.35	106,803.33		722.0	64.0	8.79	0.44	2.510			
4-Apr-80	110,763.73	106,804.13		868.0	146.0	10.38	1.60	3.988			
24-May-80	110,764.20	106,804.40		918.0	50.0	10.92	0.54	3.957			
17-Jul-80	110,764.65	106,804.59	1473.76	972.0	54.0	11.41	0.49	3.302			
15-Aug-81	110,768.70	106,806.60	1472.96	1366.0	394.0	15.93	4.52	4.189		-0.8	
15-Jun-82	110,769.80	106,807.00	1472.62	1670.0	304.0	17.08	1.17	1.405		-0.34	
9-Jun-83	110,772.19	106,808.02	1471.96	2029.0	359.0	19.67	2.60	2.642		-0.66	
23-Sep-83	110,772.00	106,808.03	1471.7	2135.0	106.0	19.51	0.19	0.655		-0.26	
14-Jun-84	110,773.22	106,808.46	1470.66	2400.0	265.0	20.79	1.29	1.782		-1.04	
15-Jul-86	110,775.69	106,809.62	1470.07	3161.0	761.0	23.52	2.73	1.309		-0.59	
17-Jul-99	110,776.00	106,811.01	1466.34	7911.0	4750.0	24.45	1.42	0.109		-3.73	
19-Jun-01	110,776.44	106,811.57	1466.5	8614.0	703.0	25.11	0.71	0.369		0.16	
20 & 20A Combined											
24-Nov-76				0.1	0.0	0.00	0.00	0.000			
29-Dec-76				35.0	35.0	0.74	0.74	7.699			
25-Jan-77				62.0	27.0	1.29	0.56	7.510	-0.22		-1.3
24-Feb-77				92.0	30.0	1.79	0.50	6.024			
23-Mar-77				119.0	27.0	2.27	0.48	6.534			
10-May-77				167.0	48.0	3.04	0.77	5.849			
24-May-77				181.0	14.0	3.37	0.32	8.451			
19-Jul-77				237.0	56.0	4.42	1.05	6.873			
18-Nov-77				359.0							
20-Jan-78				422.0	63.0	5.39					
20-Apr-78				512.0	90.0	6.69	1.31	5.309			
26-May-78				548.0	36.0	7.06	0.37	3.740			
6-Jun-78				559.0	11.0	7.18	0.12	3.850			
27-Jul-78				610.0	51.0	7.58	0.40	2.856			
22-Sep-78				667.0	57.0	8.59	1.01	6.470			
19-Oct-78				694.0	27.0	8.87	0.28	3.826			
1-Feb-79				799.0	105.0	10.08	1.21	4.211			
22-Apr-79				879.0	80.0	11.00	0.92	4.199			
16-May-79				903.0	24.0	11.19	0.19	2.873			
18-Jun-79				936.0	33.0	11.69	0.50	5.496			
1-Aug-79				980.0	44.0	12.15	0.46	3.797			
7-Sep-79				1017.0	37.0	12.77	0.62	6.155			
10-Nov-79				1081.0	64.0	13.21	0.44	2.482			
4-Apr-80				1227.0	146.0	14.80	1.60	3.988			
24-May-80				1277.0	50.0	15.34	0.54	3.957			
17-Jul-80			1473.76	1331.0	54.0	15.83	0.49	3.279			
15-Aug-81			1472.96	1725.0	394.0	20.35	4.52	4.184		-0.8	-0.7
15-Jun-82			1472.62	2029.0	304.0	21.50	1.16	1.391		-0.34	-0.4
9-Jun-83			1471.96	2388.0	359.0	24.09	2.59	2.634		-0.66	-0.7
23-Sep-83			1471.7	2494.0	106.0	23.93	-0.16	-0.565		-0.26	-0.9
14-Jun-84			1470.66	2759.0	265.0	25.21	1.28	1.765		-1.04	-1.4
15-Jul-86			1470.07	3520.0	761.0	27.94	2.73	1.308		-0.59	-0.3
17-Jul-99			1466.34	8270.0	4750.0	28.87	0.93	0.071		-3.73	-0.3
19-Jun-01			1466.5	8973.0	703.0	29.53	0.66	0.342		0.16	0.1


DIAND: Clinton Creek Asbestos Mine
Waste Rock Monitoring Monuments #21 & 21A
Northing (feet)
110,850
110,845
110,840
110,835
110,830
110,825
110,820
110,815
110,810
Easting (feet)
106,340
106,350
106,360
106,370
106,380
106,390
21A
21



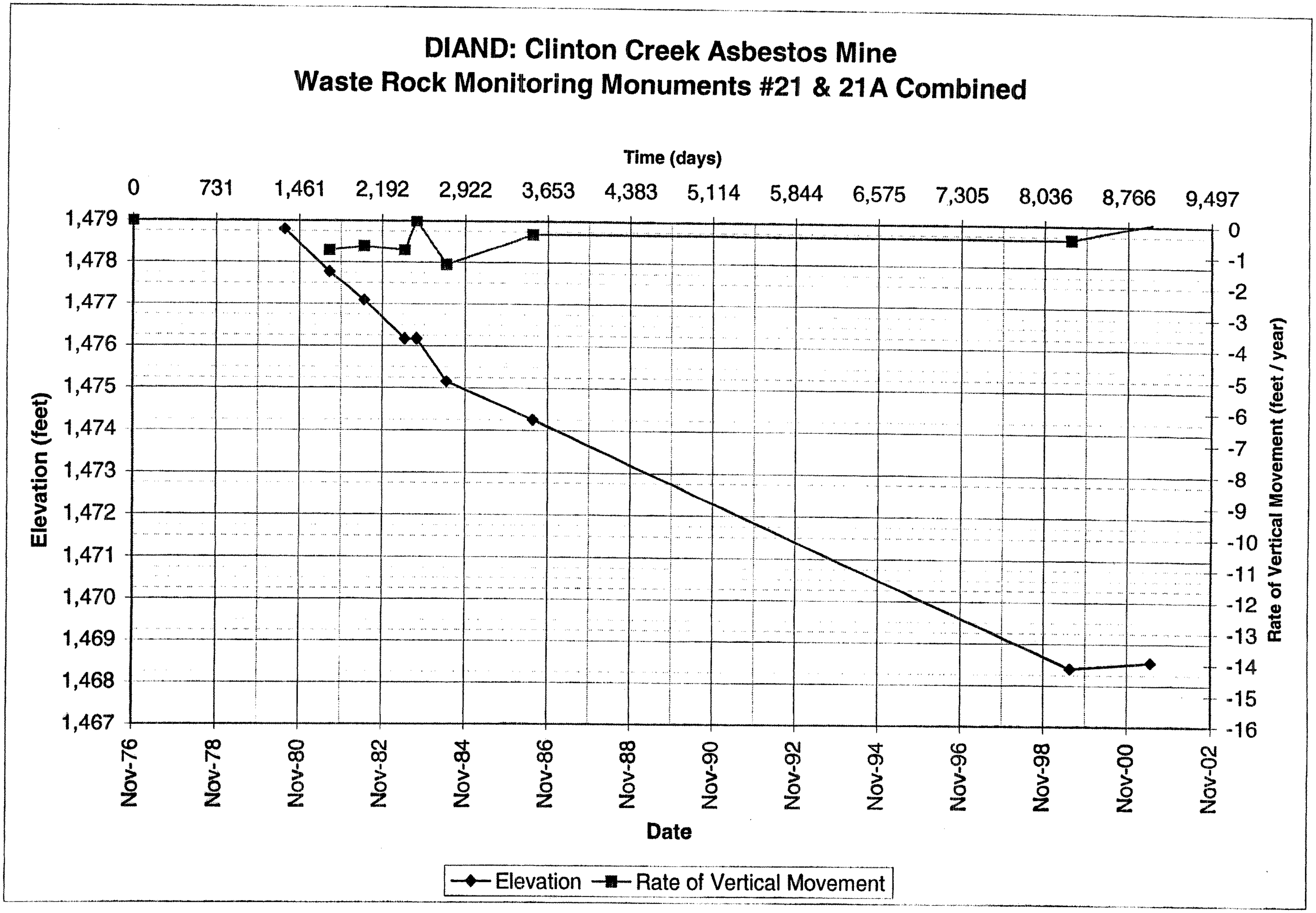
File: monitoring data #21A.xls Tab: #21A-horiz mvmnt

Date: 5/30/2002



File: monitoring data #21A.xls Tab: #21A-semilog

Date: 5/30/2002



File: monitoring data #21A.xls Tab: #21A-vert mvmnt

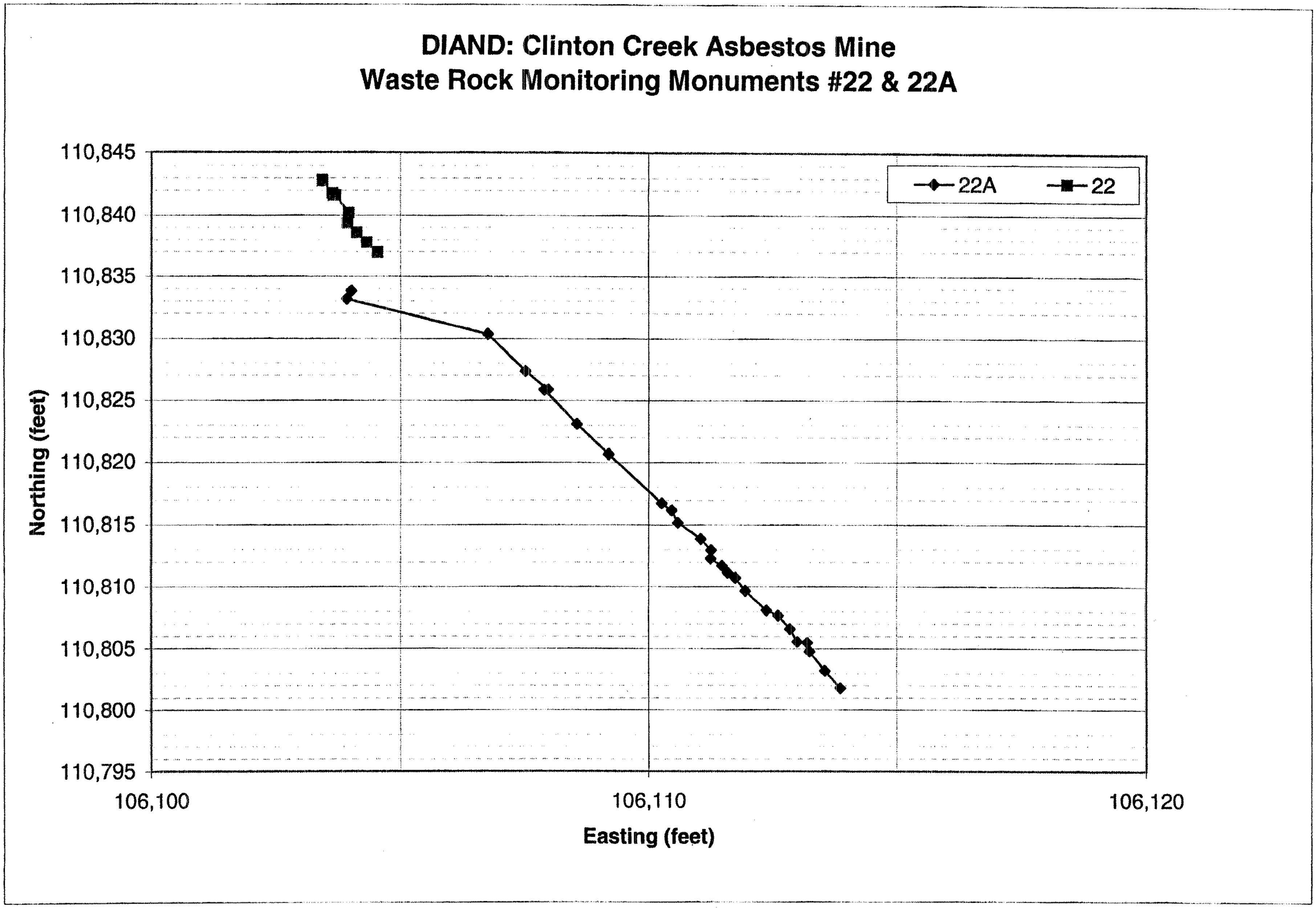
Date: 5/30/2002

Client: DIAND
Project: Clinton Creek Asbestos Mine
Job No.: 4440-038-02-02
Date: 9-Jul-01

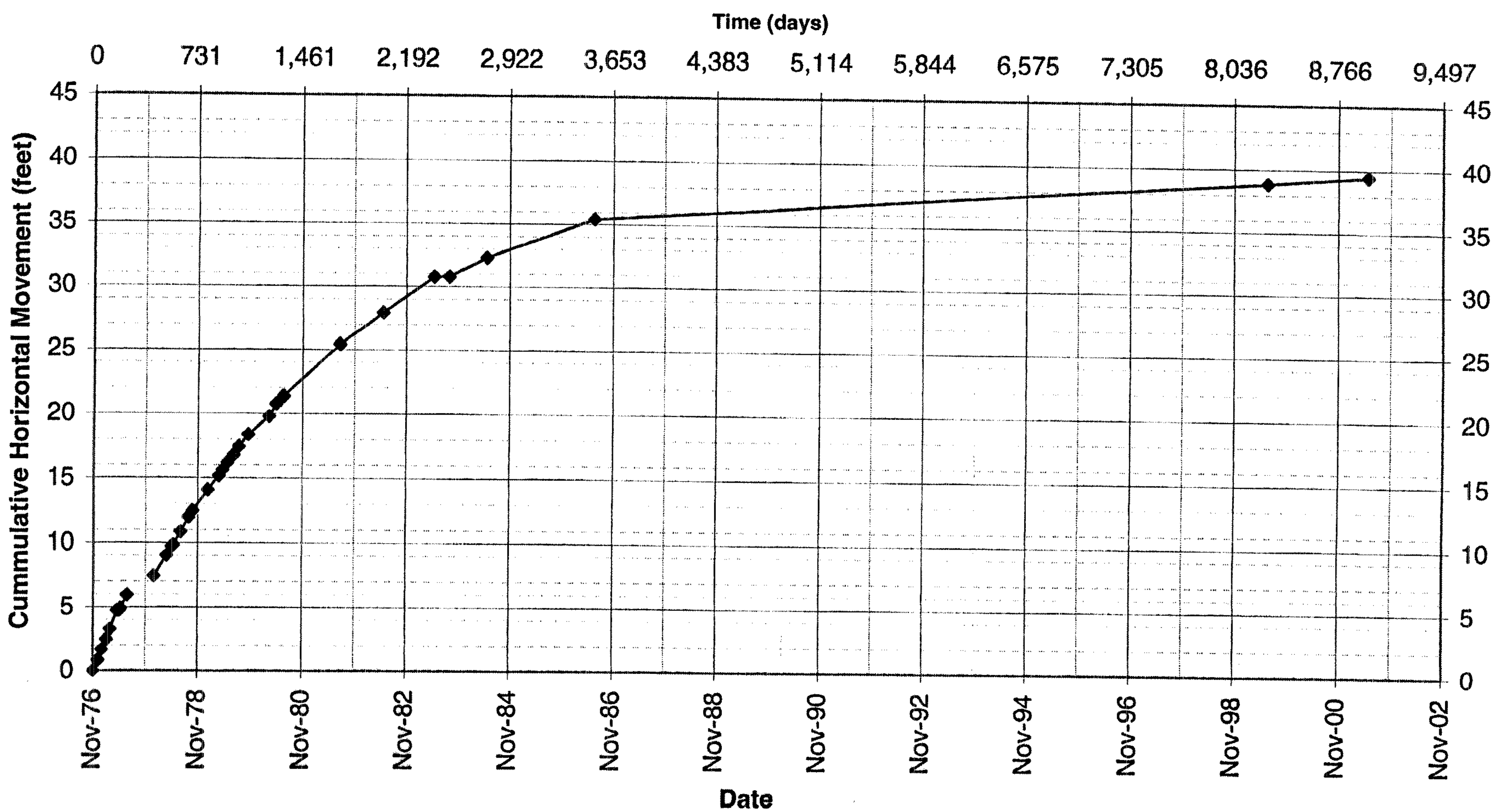
Waste Dump Stability - Monitoring Points #21 & 21A

Notes: Assume all elevations represent top of monitoring point, not ground surface.
June 2001 survey) monitor point elevation = ground elev + monitor rod ht.
interpolated using survey and rates of movement

Monitoring	Northing	Easting	Elevation	Time		Horizontal Movement			Vertical Movement		
Date				Total	Incremental	total	incremental	rate	total	incremental	rate
	(feet)	(feet)	(feet)	(days)	(days)	(feet)	(feet)	(feet / year)	(feet)	(feet)	(feet / year)
Monitor Point #20											
24-Nov-76	110,816.15	106,383.25		0	0	0	0	0			
29-Dec-76	110,816.86	106,383.51		35.0	35.0	0.76	0.76	7.885			
25-Jan-77	110,817.46	106,383.68		62.0	27.0	1.38	0.62	8.430	0.45		
24-Feb-77	110,818.06	106,383.76		92.0	30.0	1.98	0.61	7.365			
23-Mar-77	110,818.69	106,383.97		119.0	27.0	2.64	0.66	8.977			
10-May-77	110,819.48	106,384.26		167.0	48.0	3.48	0.84	6.399			
24-May-77	110,819.66	106,384.23		181.0	14.0	3.64	0.18	4.758			
3-Jun-77	110,820.02	106,384.73		191.0	10.0	4.14	0.62	22.488			
19-Jul-77	110,820.73	106,384.68		237.0	46.0	4.80	0.71	5.648			
Monitor Point #20 A											
18-Nov-77	110,819.65	106,346.63		0	0	0	0	0			
20-Jan-78	110,820.78	106,346.36		63.0	63.0	1.16	1.16	6.731			
20-Apr-78	110,822.12	106,346.59		153.0	90.0	2.47	1.36	5.514			
26-May-78	110,822.55	106,346.55		189.0	36.0	2.90	0.43	4.379			
6-Jun-78	110,822.71	106,346.47		200.0	11.0	3.06	0.18	5.936			
27-Jul-78	110,823.57	106,346.55		251.0	51.0	3.92	0.86	6.181			
22-Sep-78	110,824.35	106,346.49		308.0	57.0	4.70	0.78	5.009			
19-Oct-78	110,824.60	106,346.51		335.0	27.0	4.95	0.25	3.390			
1-Feb-79	110,825.95	106,346.43		440.0	105.0	6.30	1.35	4.701			
22-Apr-79	110,826.78	106,346.51		520.0	80.0	7.13	0.83	3.804			
16-May-79	110,827.18	106,346.60		544.0	24.0	7.53	0.41	6.235			
18-Jun-79	110,827.56	106,346.56		577.0	33.0	7.91	0.38	4.226			
1-Aug-79	110,828.04	106,346.63		621.0	44.0	8.39	0.49	4.024			
7-Sep-79	110,828.63	106,346.68		658.0	37.0	8.98	0.59	5.841			
10-Nov-79	110,829.25	106,346.69		722.0	64.0	9.60	0.62	3.536			
4-Apr-80	110,830.75	106,346.79		868.0	146.0	11.10	1.50	3.758			
24-May-80	110,831.15	106,346.79		918.0	50.0	11.50	0.40	2.920			
17-Jul-80	110,831.65	106,346.79	1478.79	972.0	54.0	12.00	0.50	3.380			
15-Aug-81	110,835.13	106,346.73	1477.77	1366.0	394.0	15.48	3.48	3.224		-1.02	-0.94
15-Jun-82	110,837.30	106,346.71	1477.09	1670.0	304.0	17.65	2.17	2.606		-0.68	-0.82
9-Jun-83	110,839.58	106,346.72	1,476.17	2029.0	359.0	19.93	2.28	2.318		-0.92	-0.94
23-Sep-83	110,839.78	106,346.64	1,476.17	2135.0	106.0	20.13	0.22	0.742		0	0.00
14-Jun-84	110,840.87	106,346.41	1,475.15	2400.0	265.0	21.22	1.11	1.534		-1.02	-1.40
15-Jul-86	110,843.70	106,346.38	1,474.25	3161.0	761.0	24.05	2.83	1.357		-0.9	-0.43
16-Jul-99	110,845.21	106,345.62	1,468.40	7910.0	4749.0	25.58	1.69	0.130		-5.85	-0.45
19-Jun-01	110,845.83	106,345.81	1,468.55	8614.0	704.0	26.19	0.65	0.336		0.15	0.08
Monitoring Points 21 & 21A combined											
24-Nov-76				0.0	0.0	0.00	0.00	0.00	0	0	0
29-Dec-76				35.0	35.0	0.76	0.76	7.885			
25-Jan-77				62.0	27.0	1.38	0.62	8.417			
24-Feb-77				92.0	30.0	1.98	0.60	7.277			
23-Mar-77				119.0	27.0	2.64	0.66	8.965			
10-May-77				167.0	48.0	3.48	0.84	6.385			
24-May-77				181.0	14.0	3.64	0.16	4.287			
3-Jun-77				191.0	10.0	4.14	0.50	18.217			
19-Jul-77				237.0	46.0	4.80	0.65	5.195			
18-Nov-77				359.0	122.0						
20-Jan-78				422.0	63.0	5.96	1.16	6.731			
20-Apr-78				512.0	90.0	7.27	1.31	5.307			
26-May-78				548.0	36.0	7.70	0.43	4.368			
6-Jun-78				559.0	11.0	7.86	0.16	5.411			
27-Jul-78				610.0	51.0	8.72	0.86	6.131			
22-Sep-78				667.0	57.0	9.50	0.78	5.003			
19-Oct-78				694.0	27.0	9.75	0.25	3.371			
1-Feb-79				799.0	105.0	11.10	1.35	4.699			
22-Apr-79				879.0	80.0	11.93	0.83	3.777			
16-May-79				903.0	24.0	12.33	0.40	6.069			
18-Jun-79				936.0	33.0	12.71	0.38	4.206			
1-Aug-79				980.0	44.0	13.19	0.48	3.979			
7-Sep-79				1017.0	37.0	13.78	0.59	5.822			
10-Nov-79				1081.0	64.0	14.40	0.62	3.536			
4-Apr-80				1227.0	146.0	15.90	1.50	3.752			
24-May-80				1277.0	50.0	16.30	0.40	2.920			
17-Jul-80			1478.79	1331.0	54.0	16.80	0.50	3.379			
15-Aug-81			1477.77	1725.0	394.0	20.28	3.48	3.223		-1.02	-0.94
15-Jun-82			1477.09	2029.0	304.0	22.45	2.17	2.605		-0.68	-0.82
9-Jun-83			1,476.17	2388.0	359.0	24.73	2.28	2.318		-0.92	-0.94
23-Sep-83			1,476.17	2494.0	106.0	24.93	0.20	0.688		0.00	0.00
14-Jun-84			1,475.15	2759.0	265.0	26.02	1.09	1.503		-1.02	-1.40
15-Jul-86			1,474.25	3520.0	761.0	28.85	2.83	1.357		-0.90	-0.43
16-Jul-99			1,468.40	8269.0	4749.0	30.38	1.53	0.117		-5.85	-0.45
19-Jun-01			1,468.55	8973.0	704.0	30.99	0.61	0.318		0.15	0.08

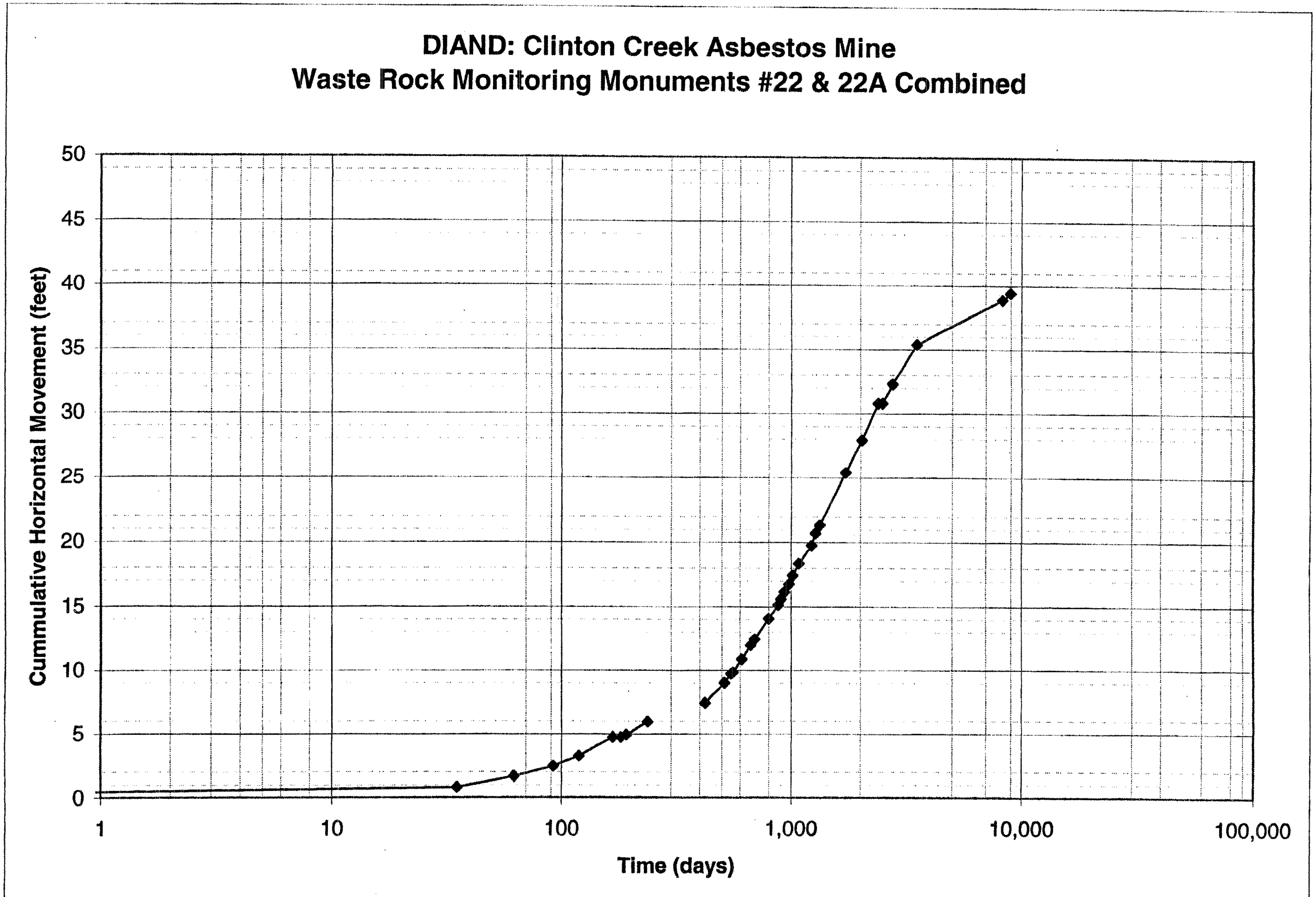

DIAND: Clinton Creek Asbestos Mine
Waste Rock Monitoring Monuments #22 & 22A
Northing (feet)
110,845
110,840
110,835
110,830
110,825
110,820
110,815
110,810
110,805
110,800
110,795
106,100
106,110
106,120
Easting (feet)
22A
22

DIAND: Clinton Creek Asbestos Mine
Waste Rock Monitoring Monuments #22 & 22A Combined



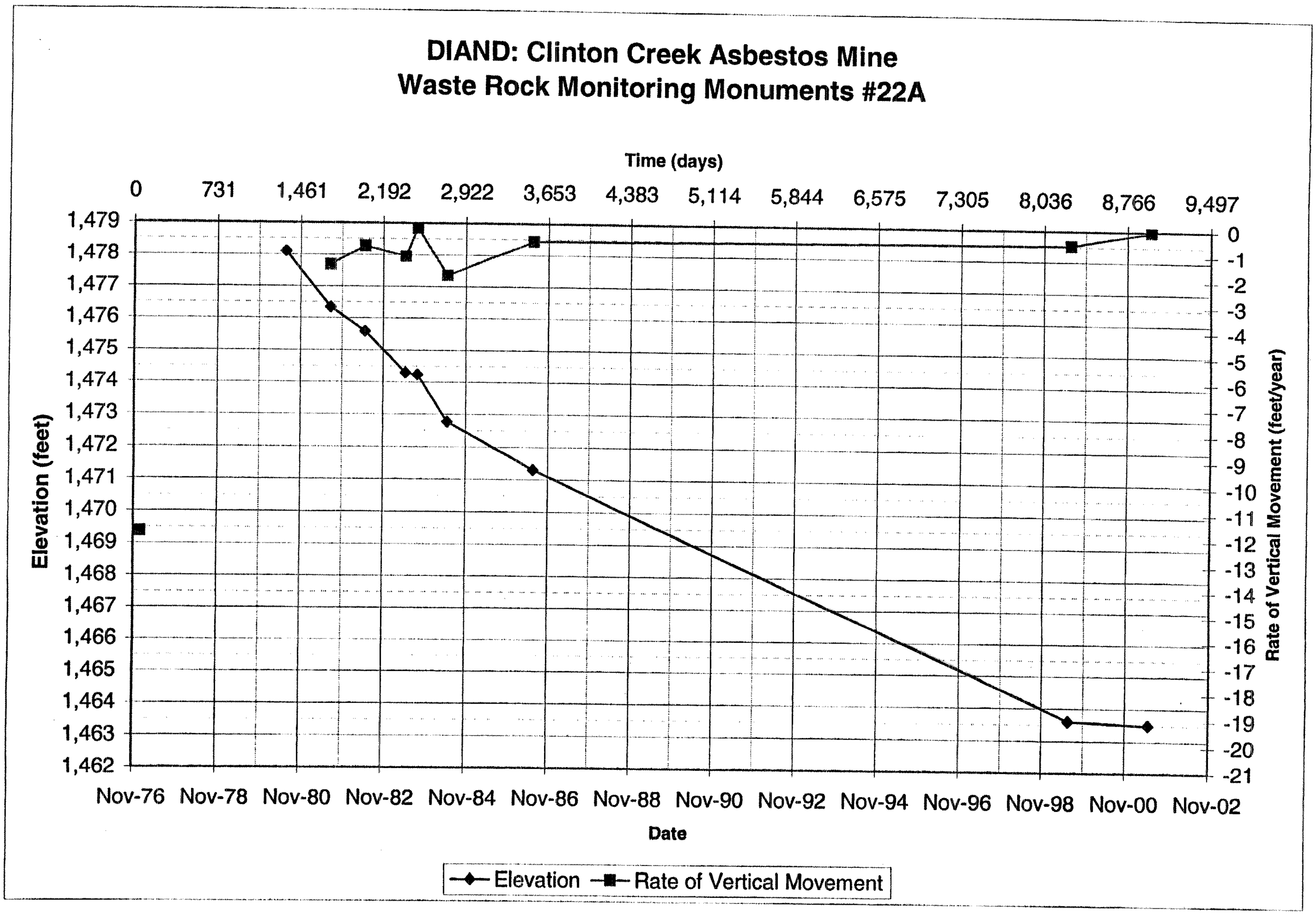
File: monitoring data #22.xls Tab: #22-horiz movmt

Date: 5/30/2002



File: monitoring data #22.xls Tab: #22-semilog

Date: 5/30/2002



File: monitoring data #22.xls Tab: #22-vert mvmnt

Date: 5/30/2002

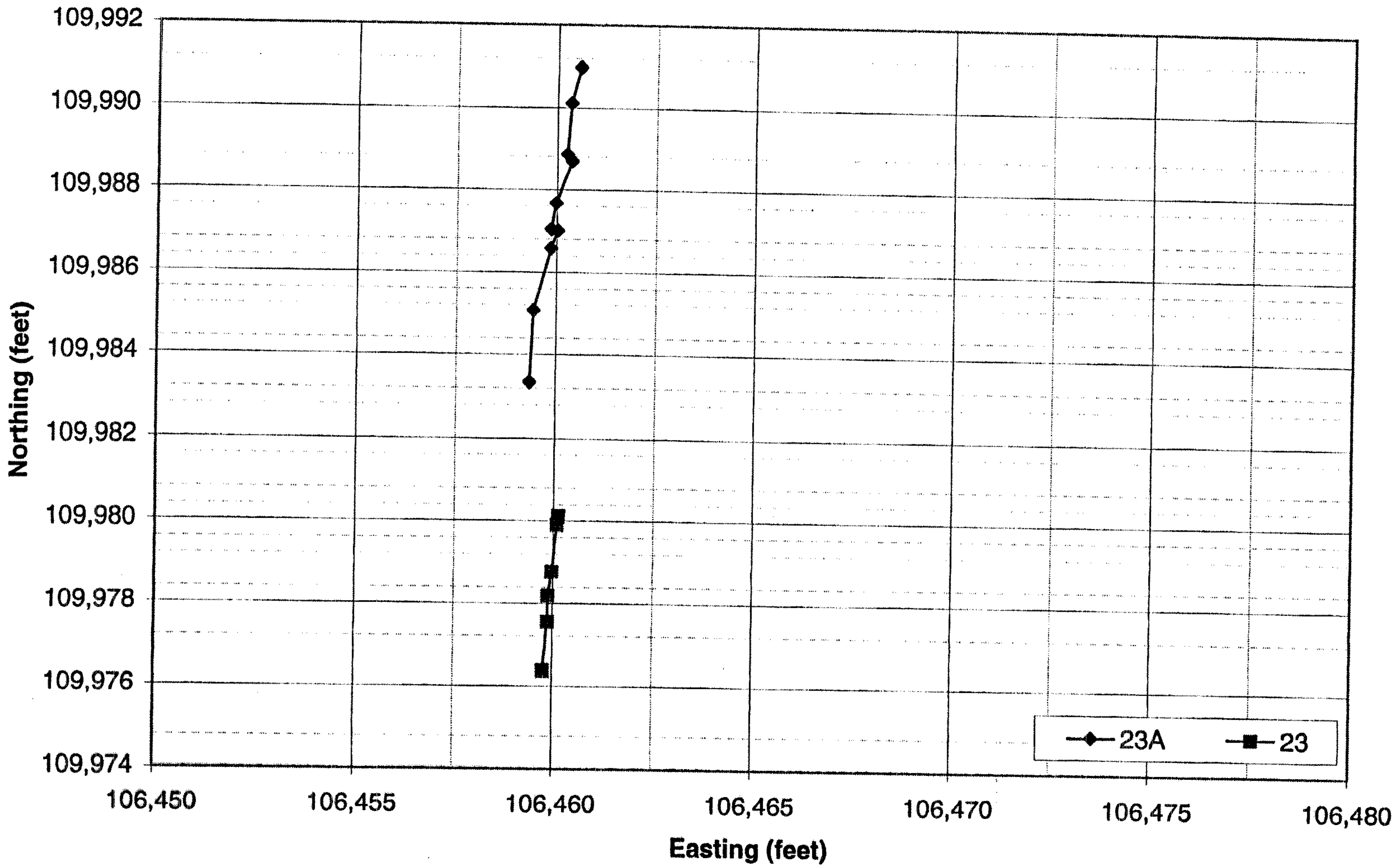
Client: DIAND
Project: Clinton Creek Asbestos Mine
Job No.: 4440-038-02-02
Date: 9-Jul-01

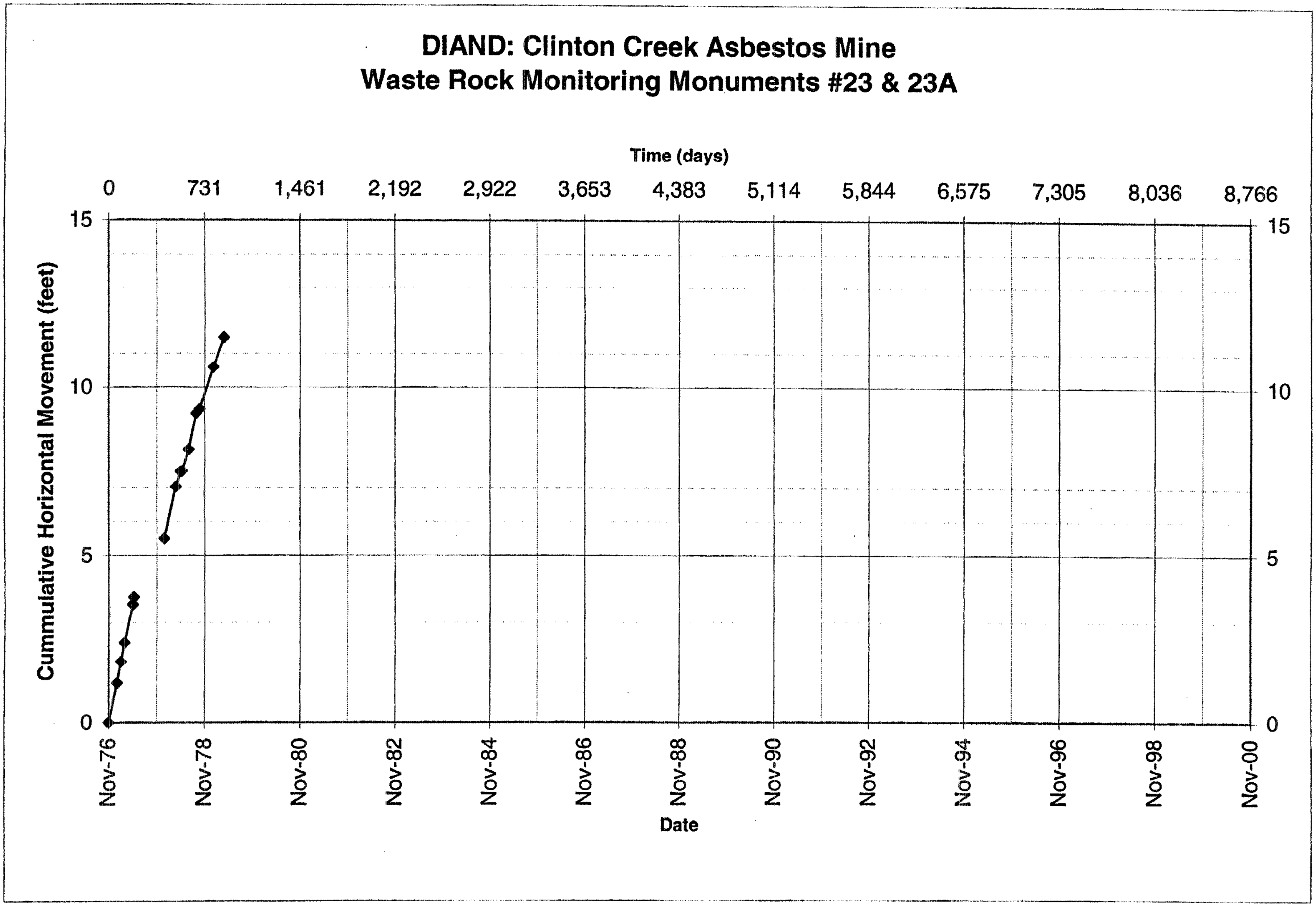
Waste Dump Stability - Monitoring Points #22 & 22A

Notes: Assume all elevations represent top of monitoring point, not ground surface.
June 2001 survey) monitor point elevation = ground elev + monitor rod ht.

Monitoring Date	Northing (feet)	Easting (feet)	Elevation (feet)	Time: Total (days)	Time: Incremental (days)	Horizontal Movement: total (feet)	Horizontal Movement: incremental (feet)	Horizontal Movement: rate (feet/year)	Vertical Movement: total (feet)	Vertical Movement: incremental (feet)	Vertical Movement: rate (feet / year)
Monitor Point #22											
24-Nov-76	110,836.97	106,104.54		0	0	0	0	0			
29-Dec-76	110,837.76	106,104.32		35.0	35.0	0.82	0.82	8.552			
25-Jan-77	110,838.57	106,104.12		62.0	27.0	1.65	0.83	11.279	-0.88	-0.88	-11.90
24-Feb-77	110,839.33	106,103.93		92.0	30.0	2.44	0.78	9.531			
23-Mar-77	110,840.16	106,103.96		119.0	27.0	3.24	0.83	11.228			
10-May-77	110,841.60	106,103.69		167.0	48.0	4.71	1.47	11.141			
24-May-77	110,841.60	106,103.65		181.0	14.0	4.71	0.04	1.043			
3-Jun-77	110,841.77	106,103.63		191.0	10.0	4.89	0.17	6.248			
19-Jul-77	110,842.80	106,103.43		237.0	46.0	5.93	1.05	8.325			
Monitor Point #22A											
18-Nov-77	110,801.76	106,113.86		0	0	0	0	0			
20-Jan-78	110,803.21	106,113.55		63.0	63.0	1.48	1.48	8.591			
20-Apr-78	110,804.77	106,113.24		153.0	90.0	3.07	1.59	6.450			
26-May-78	110,805.48	106,113.20		189.0	36.0	3.78	0.71	7.210			
6-Jun-78	110,805.56	106,113.00		200.0	11.0	3.90	0.22	7.148			
27-Jul-78	110,806.57	106,112.85		251.0	51.0	4.91	1.02	7.308			
22-Sep-78	110,807.67	106,112.61		308.0	57.0	6.04	1.13	7.210			
19-Oct-78	110,808.10	106,112.38		335.0	27.0	6.51	0.49	6.592			
1-Feb-79	110,809.66	106,111.95		440.0	105.0	8.13	1.62	5.625			
22-Apr-79	110,810.72	106,111.75		520.0	80.0	9.21	1.08	4.922			
16-May-79	110,811.15	106,111.59		544.0	24.0	9.66	0.46	6.978			
18-Jun-79	110,811.70	106,111.48		577.0	33.0	10.22	0.56	6.204			
1-Aug-79	110,812.28	106,111.25		621.0	44.0	10.84	0.62	5.176			
7-Sep-79	110,812.95	106,111.26		658.0	37.0	11.49	0.67	6.610			
10-Nov-79	110,813.85	106,111.05		722.0	64.0	12.41	0.92	5.271			
4-Apr-80	110,815.18	106,110.59		868.0	146.0	13.81	1.41	3.518			
24-May-80	110,816.16	106,110.46		918.0	50.0	14.80	0.99	7.217			
17-Jul-80	110,816.74	106,110.26	1478.09	972.0	54.0	15.41	0.61	4.147			
15-Aug-81	110,820.65	106,109.19	1476.33	1366.0	394.0	19.46	4.05	3.76		-1.76	-1.63
15-Jun-82	110,823.07	106,108.55	1475.58	1670.0	304.0	21.96	2.50	3.01		-0.75	-0.90
9-Jun-83	110,825.85	106,107.89	1474.30	2029.0	359.0	24.82	2.86	2.91		-1.28	-1.30
23-Sep-83	110,825.87	106,107.97	1474.24	2135.0	106.0	24.82	0.08	0.28		-0.06	-0.21
14-Jun-84	110,827.32	106,107.52	1472.76	2400.0	265.0	26.33	1.52	2.09		-1.48	-2.04
15-Jul-86	110,830.34	106,106.76	1471.30	3161.0	761.0	29.45	3.11	1.49		-1.46	-0.70
16-Jul-99	110,833.15	106,103.92	1463.60	7910.0	4749.0	32.93	4.00	0.31		-7.70	-0.59
19-Jun-01	110,833.78	106,104.01	1463.49	8614.0	704.0	33.50	0.64	0.33		-0.11	-0.06
Monitor Point 22 & 22A Combined											
24-Nov-76				0.0	0.0	0	0.00	0.00			
29-Dec-76				35.0	35.0	0.82	0.82	8.552			
25-Jan-77				62.0	27.0	1.65	0.83	11.276	-0.88	-0.88	-11.90
24-Feb-77				92.0	30.0	2.44	0.78	9.531			
23-Mar-77				119.0	27.0	3.24	0.80	10.879			
10-May-77				167.0	48.0	4.71	1.47	11.141			
24-May-77				181.0	14.0	4.71	0.01	0.193			
3-Jun-77				191.0	10.0	4.89	0.17	6.232			
19-Jul-77				237.0	46.0	5.93	1.05	8.325			
18-Nov-77				359.0							
20-Jan-78				422.0	63.0	7.42					
20-Apr-78				512.0	90.0	9.01	1.59	6.450			
26-May-78				548.0	36.0	9.71	0.70	7.147			
6-Jun-78				559.0	11.0	9.83	0.12	3.916			
27-Jul-78				610.0	51.0	10.85	1.02	7.291			
22-Sep-78				667.0	57.0	11.98	1.13	7.209			
19-Oct-78				694.0	27.0	12.45	0.47	6.350			
1-Feb-79				799.0	105.0	14.06	1.62	5.622			
22-Apr-79				879.0	80.0	15.14	1.08	4.916			
16-May-79				903.0	24.0	15.60	0.46	6.926			
18-Jun-79				936.0	33.0	16.16	0.56	6.199			
1-Aug-79				980.0	44.0	16.77	0.62	5.126			
7-Sep-79				1017.0	37.0	17.42	0.65	6.404			
10-Nov-79				1081.0	64.0	18.35	0.92	5.271			
4-Apr-80				1227.0	146.0	19.75	1.40	3.501			
24-May-80				1277.0	50.0	20.73	0.98	7.178			
17-Jul-80			1478.09	1331.0	54.0	21.34	0.61	4.127			
15-Aug-81			1476.33	1725.0	394.0	25.39	4.05	3.754		-1.76	-1.63
15-Jun-82			1475.58	2029.0	304.0	27.90	2.50	3.005		-0.75	-0.90
9-Jun-83			1474.30	2388.0	359.0	30.75	2.86	2.905		-1.28	-1.30
23-Sep-83			1474.24	2494.0	106.0	30.75	0.00	0.001		-0.06	-0.21
14-Jun-84			1472.76	2759.0	265.0	32.27	1.52	2.087		-1.48	-2.04
15-Jul-86			1471.30	3520.0	761.0	35.38	3.11	1.494		-1.46	-0.70
16-Jul-99			1463.60	8269.0	4749.0	38.86	3.48	0.267		-7.7	-0.59
19-Jun-01			1463.49	8973.0	704.0	39.44	0.57	0.298		-0.11	-0.06

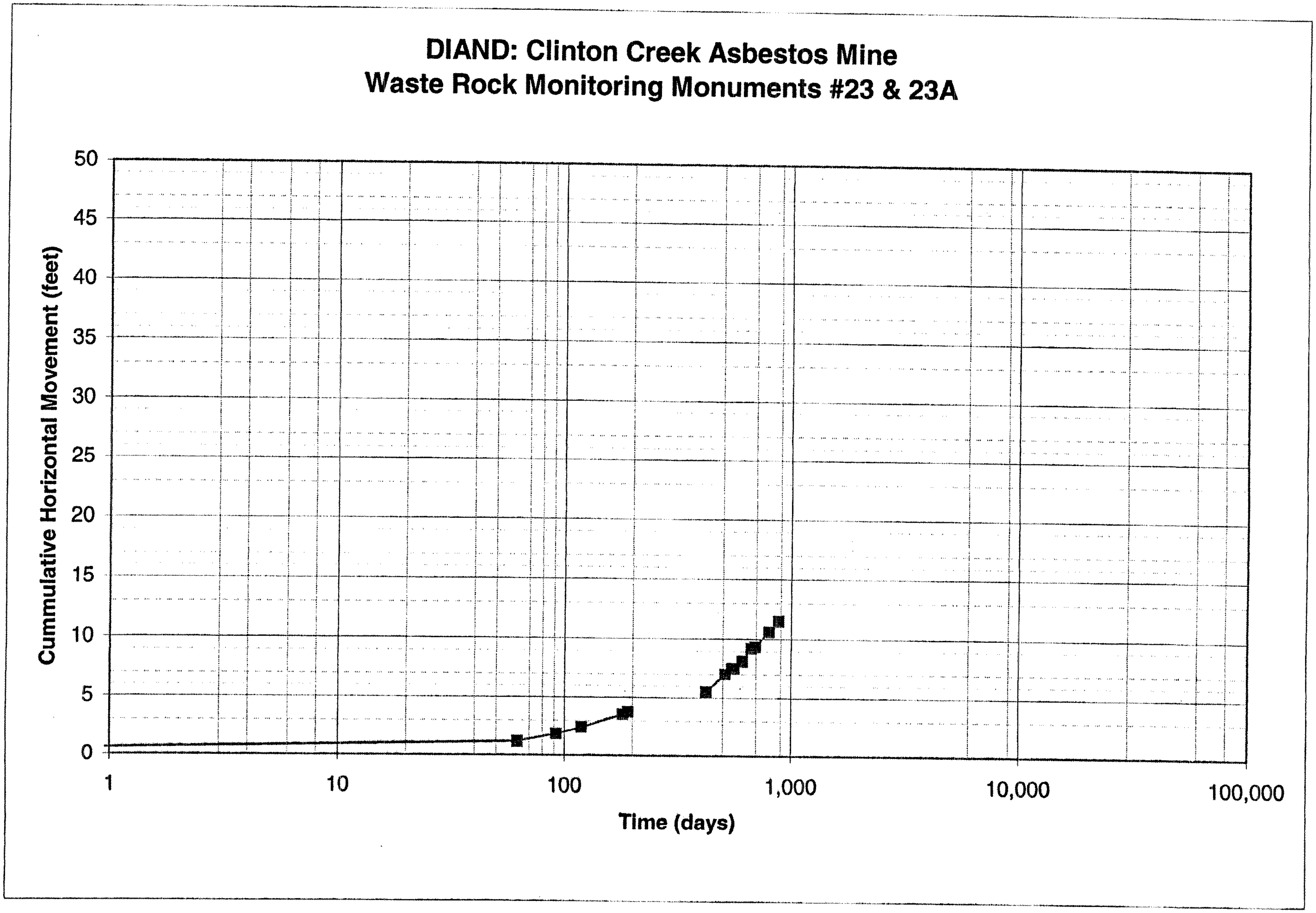
DIAND: Clinton Creek Asbestos Mine
Waste Rock Monitoring Monuments #23 & 23A





File: monitoring data #23.xls Tab: #23-horiz mvmnt

Date: 5/30/2002



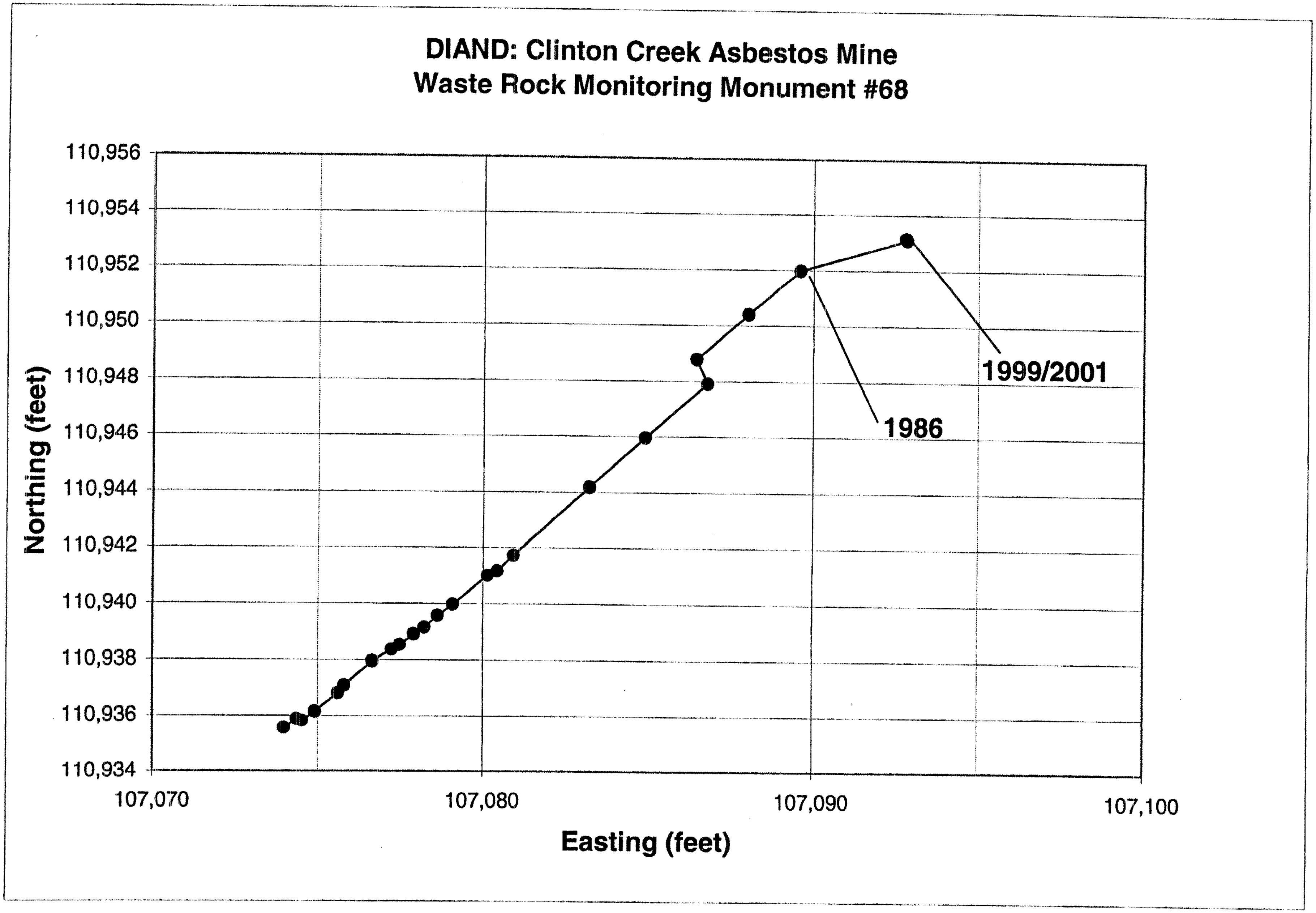
File: monitoring data #23.xls Tab: #23-semilog

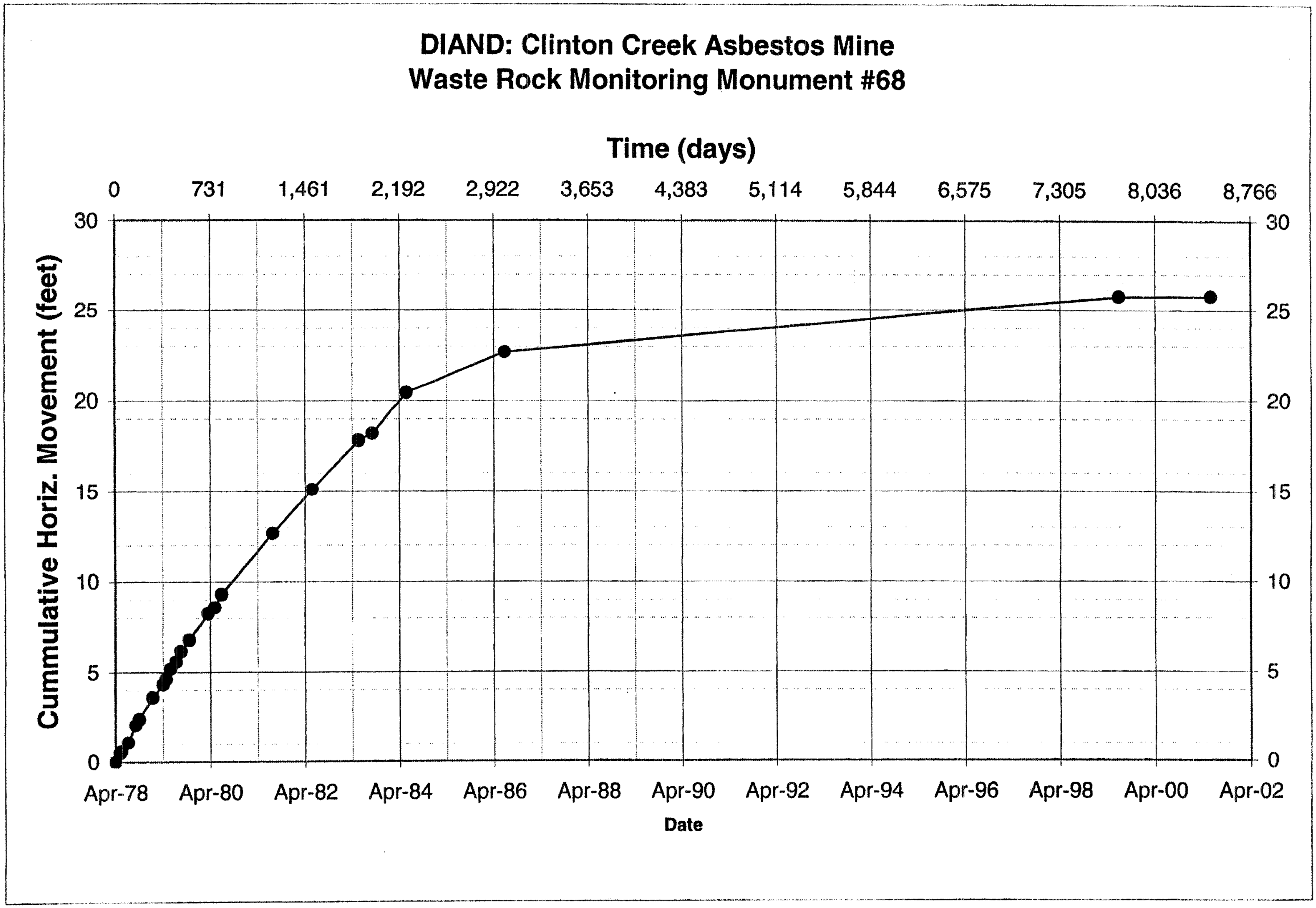
Date: 5/30/2002

Client: DIAND
Project: Clinton Creek Asbestos Mine
Job No.: 4440-038-02-02
Date: 22-Sep-00

Waste Dump Stability - Monitoring Points #23 & 23A

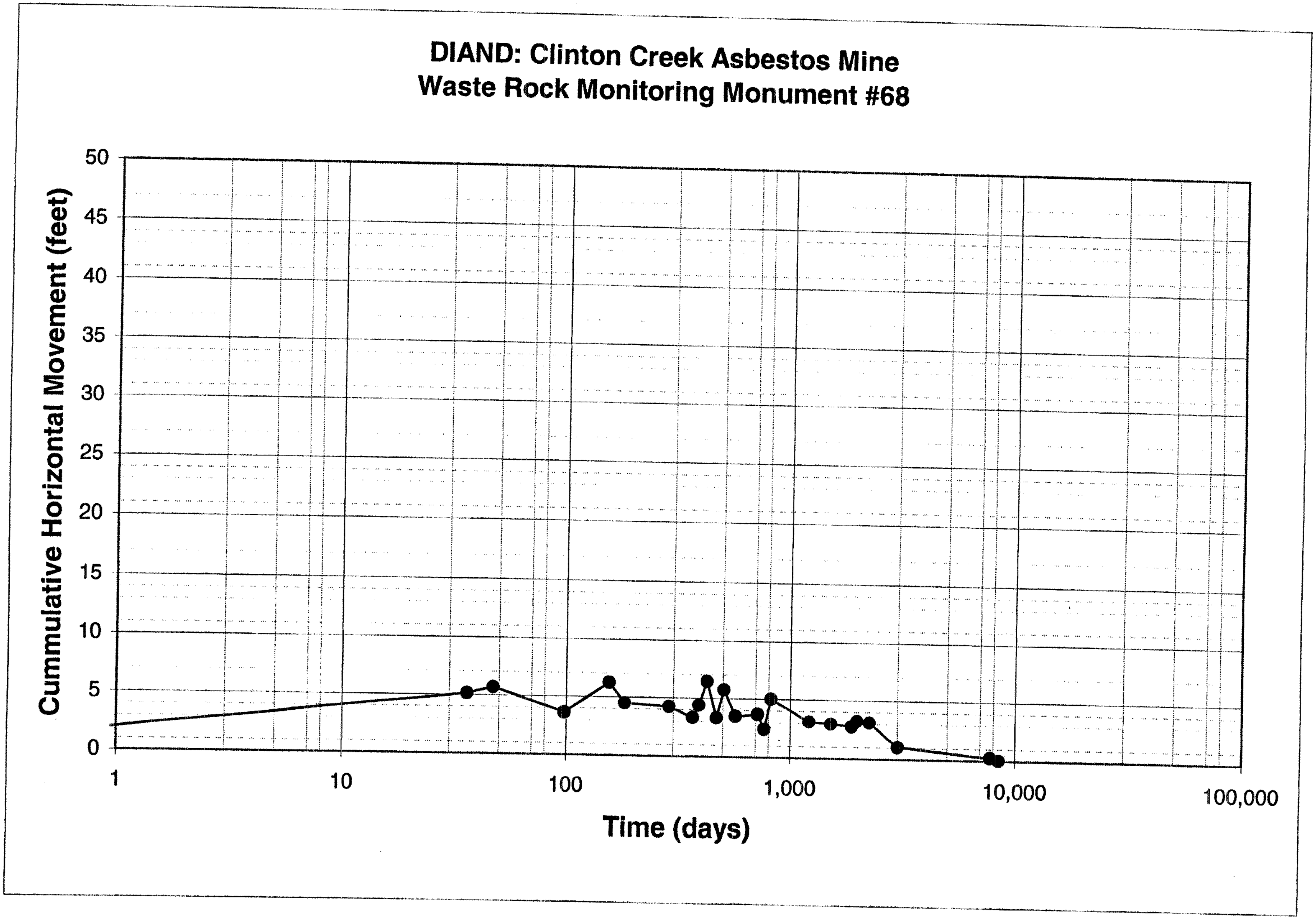
Monitoring Date	Northing (feet)	Easting (feet)	Elevation (feet)	Time: Total (days)	Time: Incremental (days)	Horizontal Movement: total (feet)	Horizontal Movement: incremental (feet)	Horizontal Movement: rate (feet/year)	Vertical Movement: total (feet)	Vertical Movement: incremental (feet)	Vertical Movement: rate (feet/year)
Monitor Point #23											
24-Nov-76	109,976.37	106,459.76		0	0	0	0	0			
25-Jan-77	109,977.55	106,459.87		62.0	62.0	1.19	1.19	0.573	0.68		
24-Feb-77	109,978.19	106,459.87		92.0	30.0	1.82	0.64	0.640			
23-Mar-77	109,978.76	106,459.97		119.0	27.0	2.40	0.58	0.643			
24-May-77	109,979.90	106,460.08		181.0	62.0	3.54	1.15	0.554			
3-Jun-77	109,980.11	106,460.12		191.0	10.0	3.76	0.21	0.641			
Monitor Point #23A											
18-Nov-77	109,983.34	106,459.35		0	0	0	0	0			
20-Jan-78	109,985.07	106,459.43		63.0	63.0	1.73	1.73	0.825			
20-Apr-78	109,986.57	106,459.86		153.0	90.0	3.27	1.56	0.520			
26-May-78	109,987.00	106,460.02		189.0	36.0	3.72	0.46	0.382			
06-Jun-78	109,987.04	106,459.86		200.0	11.0	3.73	0.16	0.450			
27-Jul-78	109,987.67	106,459.98		251.0	51.0	4.38	0.64	0.377			
22-Sep-78	109,988.69	106,460.37		308.0	57.0	5.45	1.09	0.575			
19-Oct-78	109,988.85	106,460.25		335.0	27.0	5.58	0.20	0.222			
01-Feb-79	109,990.11	106,460.35		440.0	105.0	6.84	1.26	0.361			
22-Apr-79	109,990.98	106,460.57		520.0	80.0	7.74	0.90	0.337			
Monitor Points 23 & 23A Combined											
24-Nov-76				0.0	0.0	0.00	0.00	0.00			
25-Jan-77				62.0	62.0	1.19	1.19	6.98			
24-Feb-77				92.0	30.0	1.82	0.64	7.76			
23-Mar-77				119.0	27.0	2.40	0.58	7.79			
24-May-77				181.0	62.0	3.54	1.15	6.74			
3-Jun-77				191.0	10.0	3.76	0.21	7.77			
18-Nov-77				359.0	168.0						
20-Jan-78				422.0	63.0	5.49					
20-Apr-78				512.0	90.0	7.03	1.54	6.24			
26-May-78				548.0	36.0	7.48	0.45	4.57			
06-Jun-78				559.0	11.0	7.49	0.01	0.47			
27-Jul-78				610.0	51.0	8.13	0.64	4.58			
22-Sep-78				667.0	57.0	9.20	1.07	6.86			
19-Oct-78				694.0	27.0	9.34	0.14	1.85			
01-Feb-79				799.0	105.0	10.60	1.26	4.38			
22-Apr-79				879.0	80.0	11.49	0.89	4.08			


DIAND: Clinton Creek Asbestos Mine
Waste Rock Monitoring Monument #68
Northing (feet)
110,956
110,954
110,952
110,950
110,948
110,946
110,944
110,942
110,940
110,938
110,936
110,934
107,070
107,080
107,090
107,100
Easting (feet)
1986
1999/2001



File: monitoring data #68.xls Tab: #68-horiz mvmnt

Date: 5/30/2002



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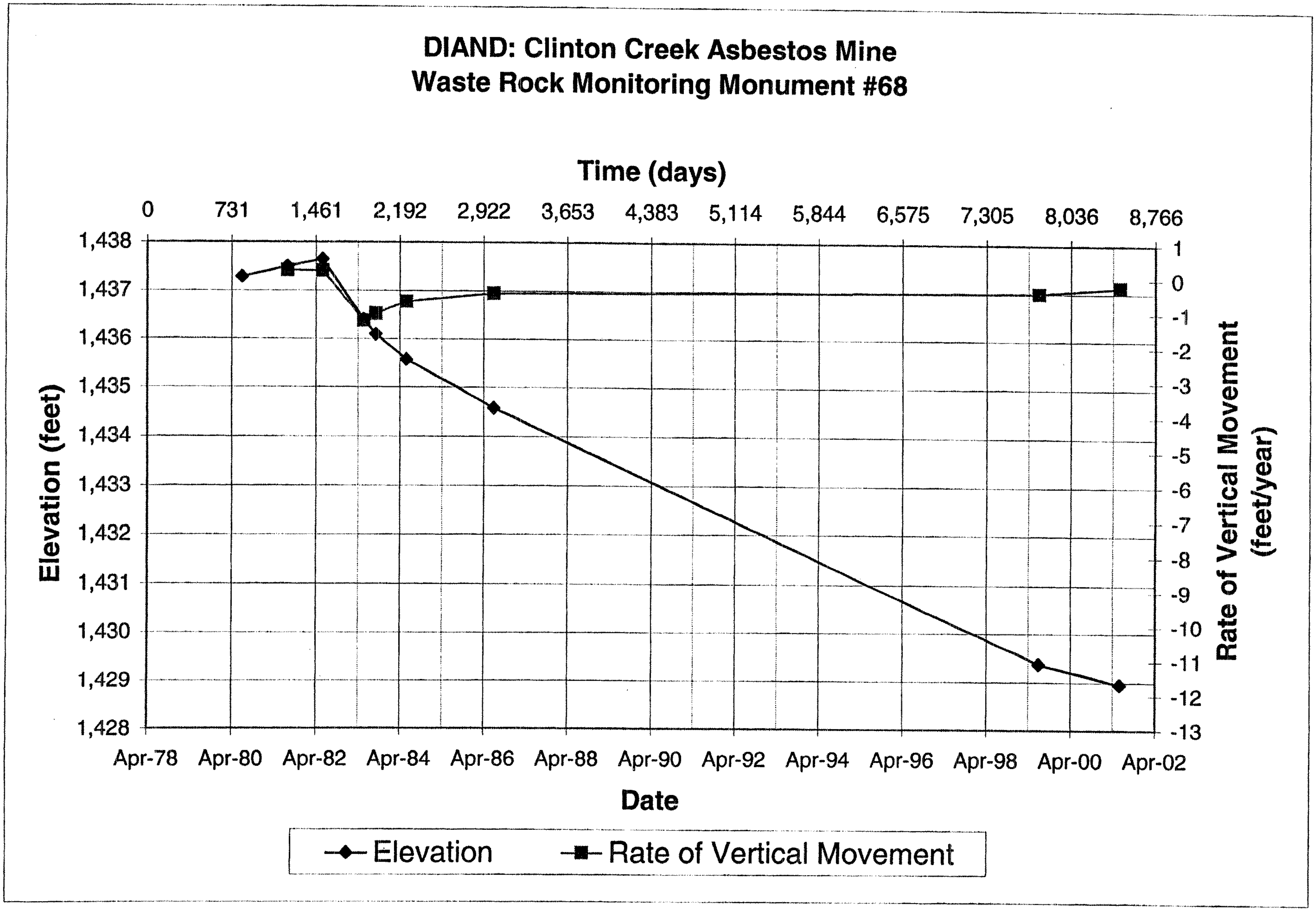
Date: 5/30/2002



File: monitoring data #68.xls Tab: #68- vert mvmnt

Date: 5/30/2002

Client: DIAND
Project: Clinton Creek Asbestos Mine
Job No.: 4440-038-02-02
Date: 9-Jul-01

Waste Dump Stability - Monitoring Point #68

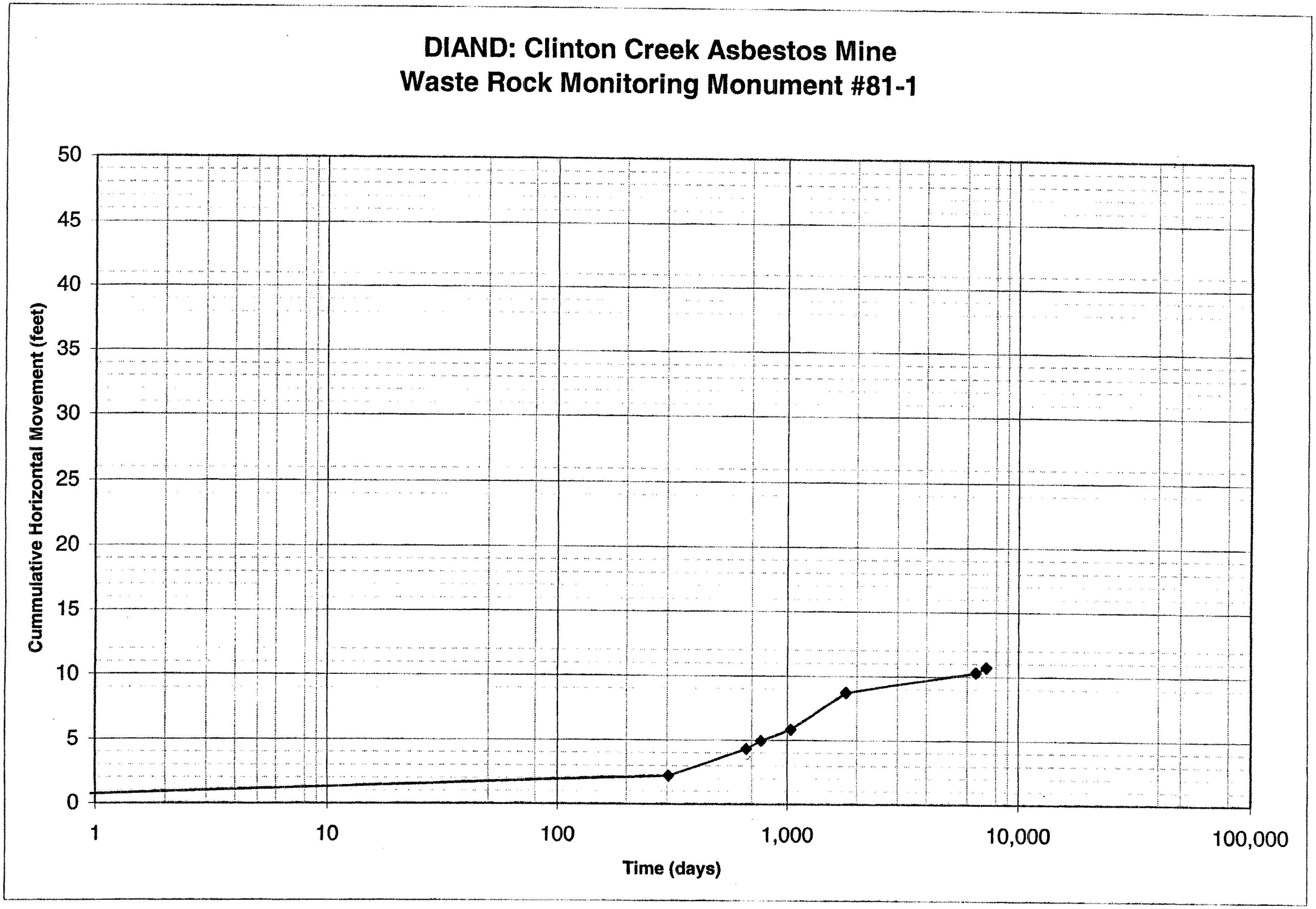
Notes: Assume all elevations represent top of monitoring point, not ground surface.
June 2001 survey) monitor point elevation = ground elev + monitor rod ht.

Interpolated Values

Monitoring Date	Northing (feet)	Easting (feet)	Elevation (feet)	Time: Total (days)	Time: Incremental (days)	Horizontal Movement: total (feet)	Horizontal Movement: incremental (feet)	Horizontal Movement: rate (feet/year)	Vertical Movement: total (feet)	Vertical Movement: incremental (feet)	Vertical Movement: rate (feet/year)
20-Apr-78	110,935.55	107,073.97		0	0	0	0	0			
26-May-78	110,935.87	107,074.36		36.0	36.0	0.50	0.50	5.115			
6-Jun-78	110,935.81	107,074.52		47.0	11.0	0.61	0.17	5.670			
27-Jul-78	110,936.13	107,074.91		98.0	51.0	1.10	0.50	3.610			
22-Sep-78	110,936.80	107,075.61		155.0	57.0	2.06	0.97	6.205			
19-Oct-78	110,937.07	107,075.80		182.0	27.0	2.38	0.33	4.463			
1-Feb-79	110,937.94	107,076.65		287.0	105.0	3.59	1.22	4.228			
22-Apr-79	110,938.37	107,077.24		367.0	80.0	4.32	0.73	3.331			
16-May-79	110,938.53	107,077.48		391.0	24.0	4.60	0.29	4.387			
18-Jun-79	110,938.92	107,077.91		424.0	33.0	5.18	0.58	6.421			
1-Aug-79	110,939.16	107,078.23		468.0	44.0	5.58	0.40	3.318			
7-Sep-79	110,939.58	107,078.63		505.0	37.0	6.16	0.58	5.722			
10-Nov-79	110,939.98	107,079.09		569.0	64.0	6.77	0.61	3.477			
4-Apr-80	110,941.01	107,080.14		715.0	146.0	8.24	1.47	3.677			
24-May-80	110,941.17	107,080.43		765.0	50.0	8.56	0.33	2.418			
17-Jul-80	110,941.72	107,080.92	1437.29	819.0	54.0	9.29	0.74	4.979			
15-Aug-81	110,944.17	107,083.22	1437.50	1213.0	394.0	12.64	3.36	3.113		0.2	0.19
15-Jun-82	110,945.95	107,084.91	1437.65	1517.0	304.0	15.09	2.45	2.947		0.2	0.18
9-Jun-83	110,947.91	107,086.79	1436.40	1876.0	359.0	17.81	2.72	2.761		-1.3	-1.27
23-Sep-83	110,948.78	107,086.46	1436.09	1982.0	106.0	18.19	0.93	3.204		-0.3	-1.07
14-Jun-84	110,950.38	107,088.03	1435.57	2247.0	265.0	20.44	2.24	3.088		-0.5	-0.72
15-Jul-86	110,951.97	107,089.60	1434.58	3008.0	761.0	22.67	2.23	1.072		-1.0	-0.47
17-Jul-99	110,953.09	107,092.83	1429.37	7758.0	4750.0	25.76	3.42	0.263		-5.2	-0.40
19-Jun-01	110,953.15	107,092.80	1428.95	8461.0	703.0	25.77	0.07	0.035		-0.4	-0.22

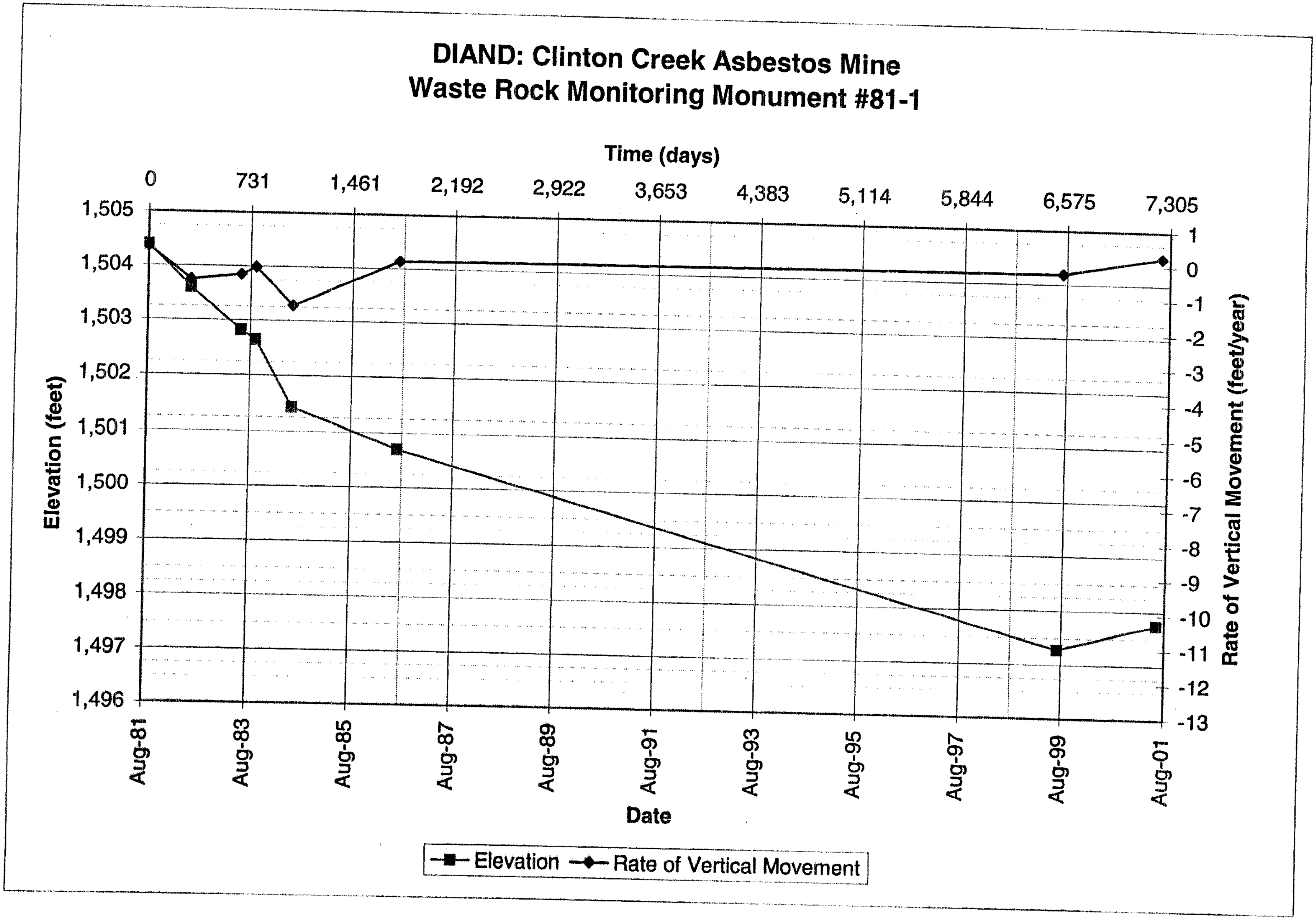
File: monitoring data #68.xls Tab: Monument #68

Date: 5/30/2002



File: monitoring data #81-1.xls Tab: #81-1-semilog

Date: 5/30/2002



File: monitoring data #81-1.xls Tab: #81-1-vert mvmnt

Date: 5/30/2002

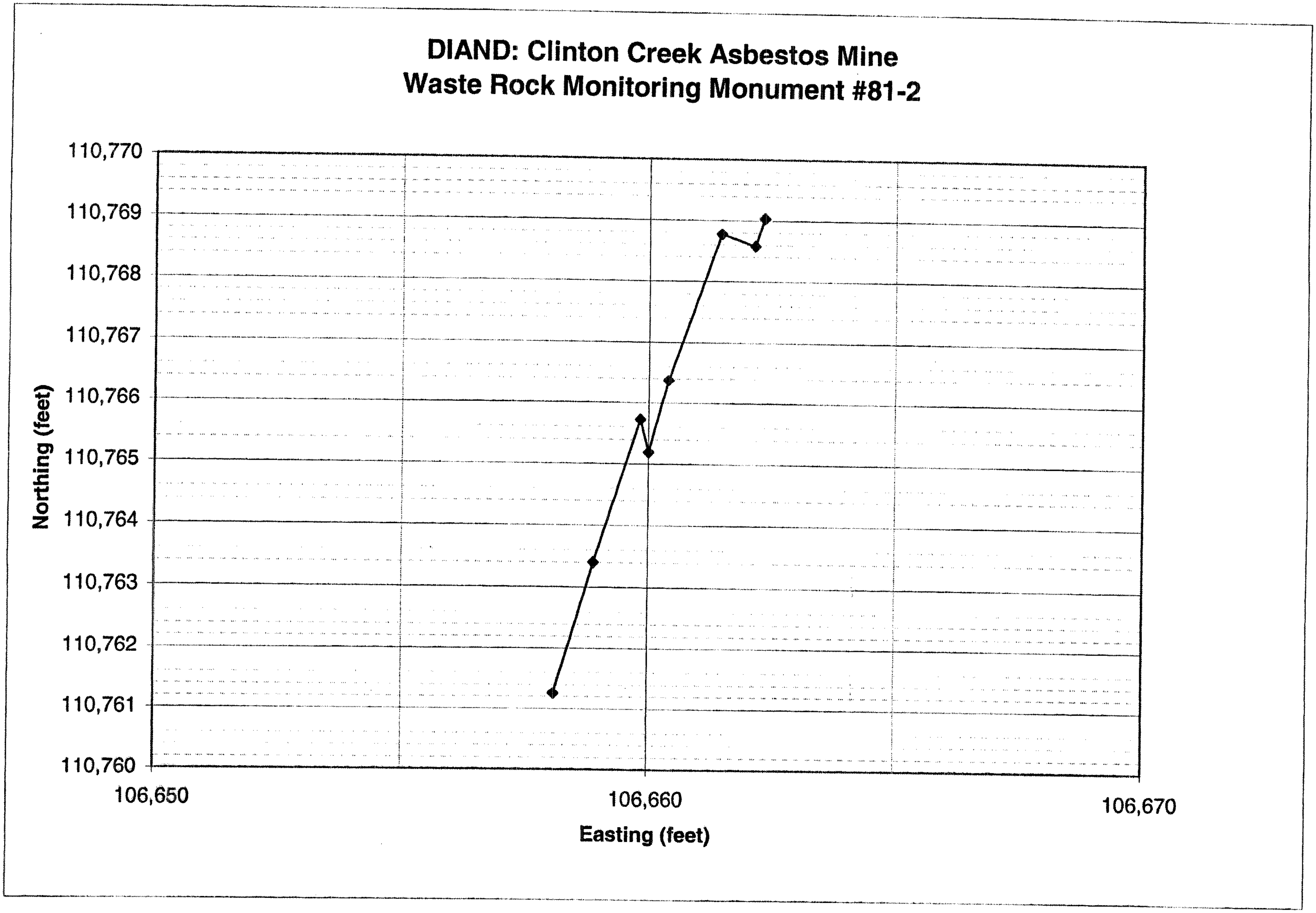
Client: DIAND
Project: Clinton Creek Asbestos Mine
Job No.: 4440-038-02-02
Date: 9-Jul-01

Waste Dump Stability - Monitoring Point #81-1

Notes: Assume all elevations represent top of monitoring point, not ground surface.
June 2001 survey) monitor point elevation = ground elev + monitor rod ht.

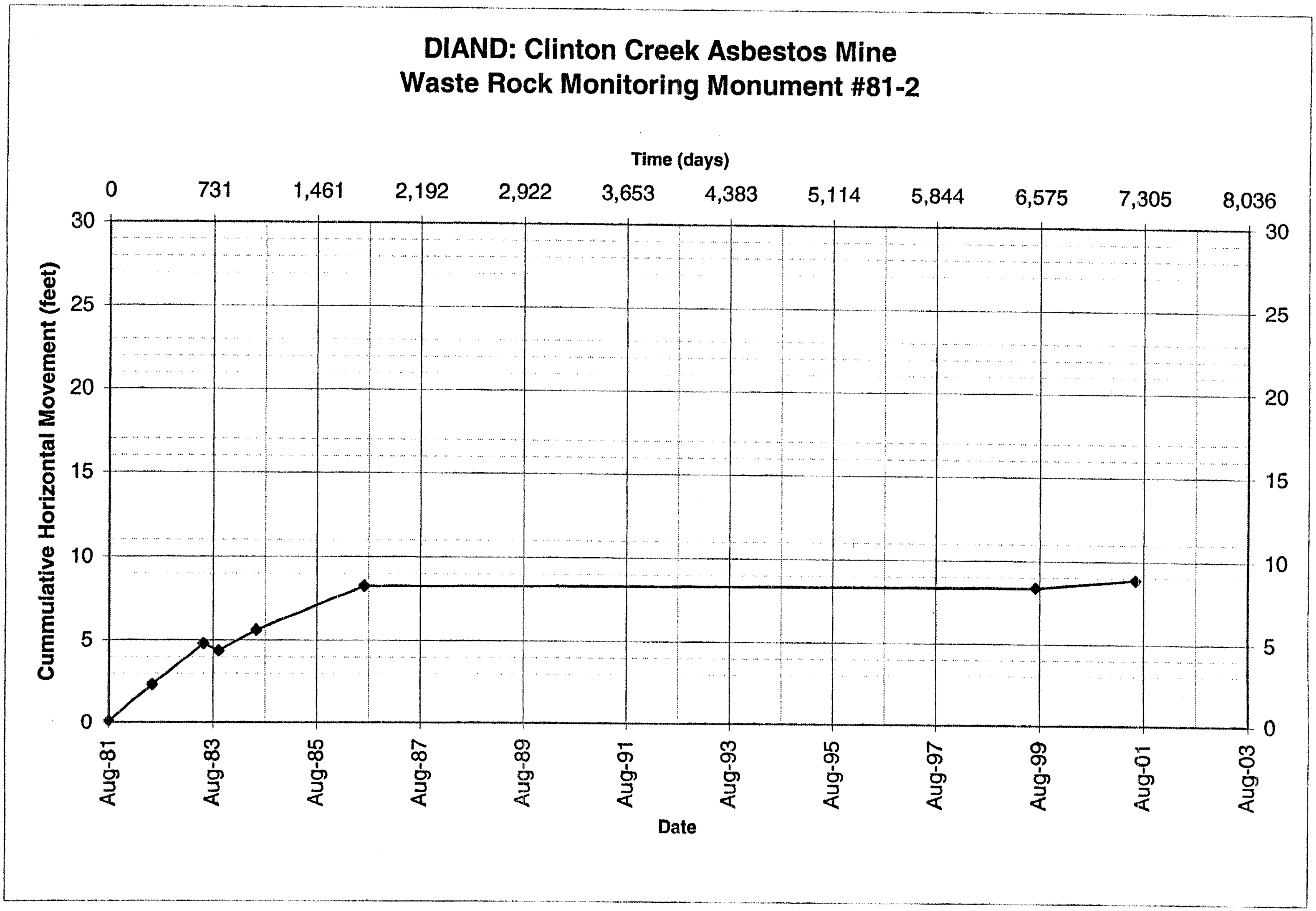
Extrapolated Values Based On Movement rates

Monitoring Date	Northing (feet)	Easting (feet)	Elevation (feet)	Time: Total (days)	Time: Increment (days)	Horizontal Movement: total (feet)	Horizontal Movement: increment (feet)	Horizontal Movement: rate (feet/year)	Vertical Movement: total (feet)	Vertical Movement: incremental (feet)	Vertical Movement: rate (feet/year)
15-Aug-81	110200.3	106547.35	1504.39	0	0	0	0	0	0	0	0
15-Jun-82	110202.4	106547.95	1503.6	304.0	304.0	2.18	2.18	2.622	-0.79	-0.79	-0.95
9-Jun-83	110,204.42	106,548.55	1502.82	663.0	359.0	4.29	2.11	2.142	-1.57	-0.78	-0.79
23-Sep-83	110,205.01	106,548.86	1502.65	769.0	106.0	4.95	0.67	2.295	-1.74	-0.17	-0.59
14-Jun-84	110,205.95	106,548.85	1501.43	1034.0	265.0	5.85	0.94	1.295	-2.96	-1.22	-1.68
15-Jul-86	110,208.65	106,549.84	1500.7	1795.0	761.0	8.71	2.88	1.379	-3.69	-0.73	-0.35
17-Jul-99	110,209.51	106,551.95	1497.28	6545.0	4750.0	10.29	2.28	0.175	-7.11	-3.42	-0.26
19-Jun-01	110,209.88	106,552.11	1497.75	7248.0	703.0	10.70	0.40	0.209	-6.64	0.47	0.24



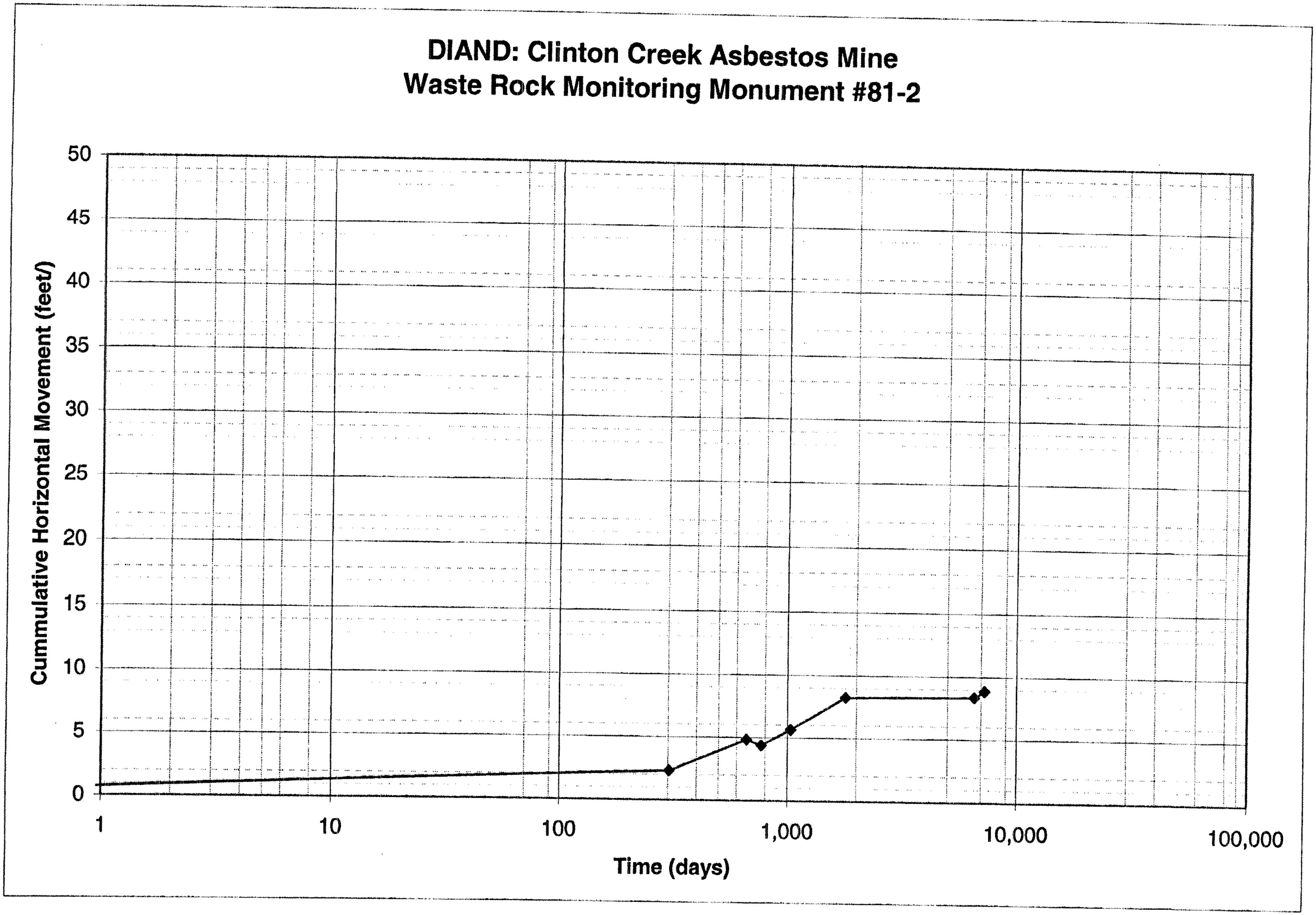
File: monitoring data #81-2.xls Tab: 81-2 - NE

Date: 5/30/2002



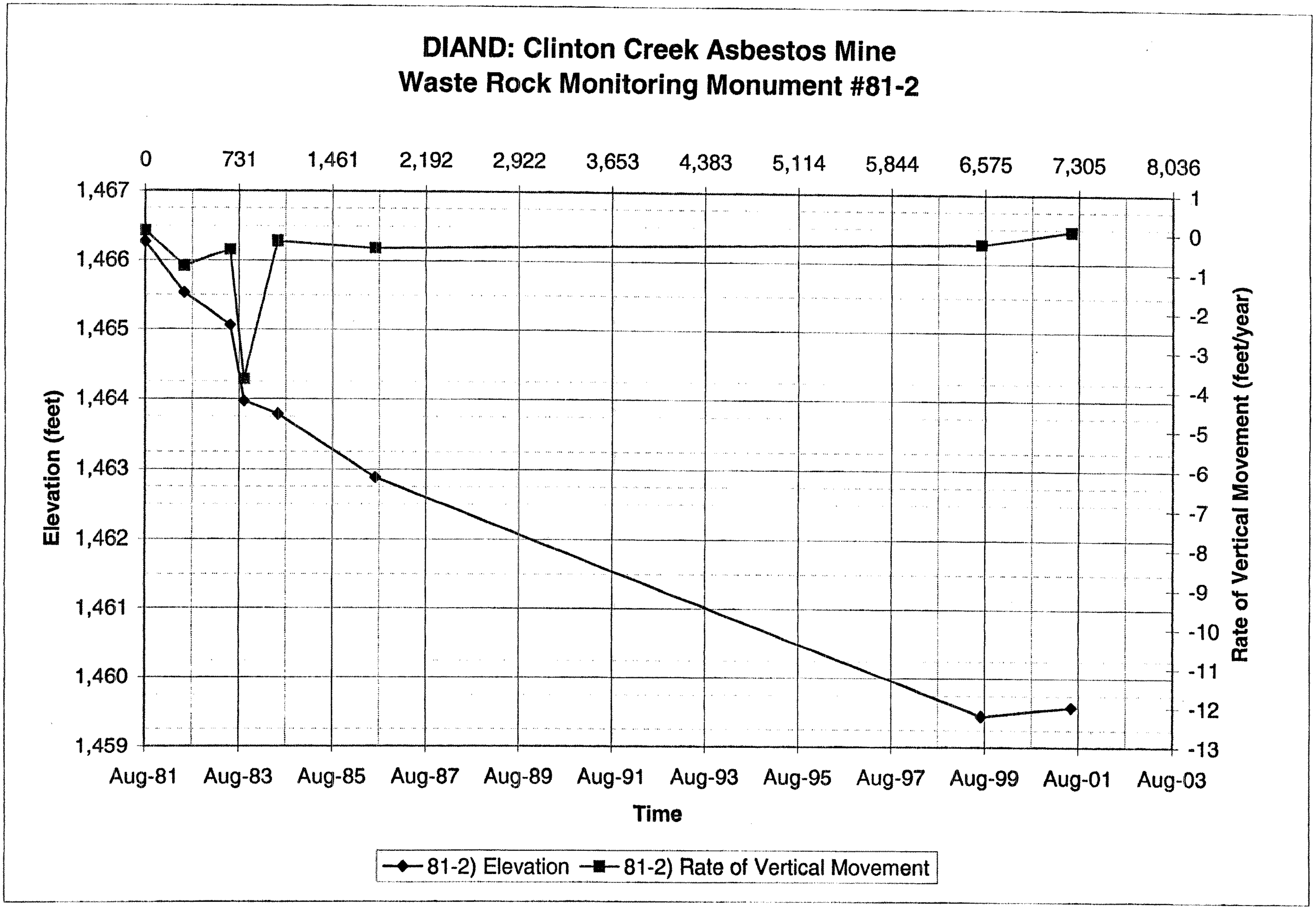
File: monitoring data #81-2.xls Tab: #81-2-horiz mvmnt

Date: 5/30/2002



File: monitoring data #81-2.xls Tab: #81-2-semilog

Date: 5/30/2002



File: monitoring data #81-2.xls Tab: #81-2-vert mvmnt

Date: 5/30/2002

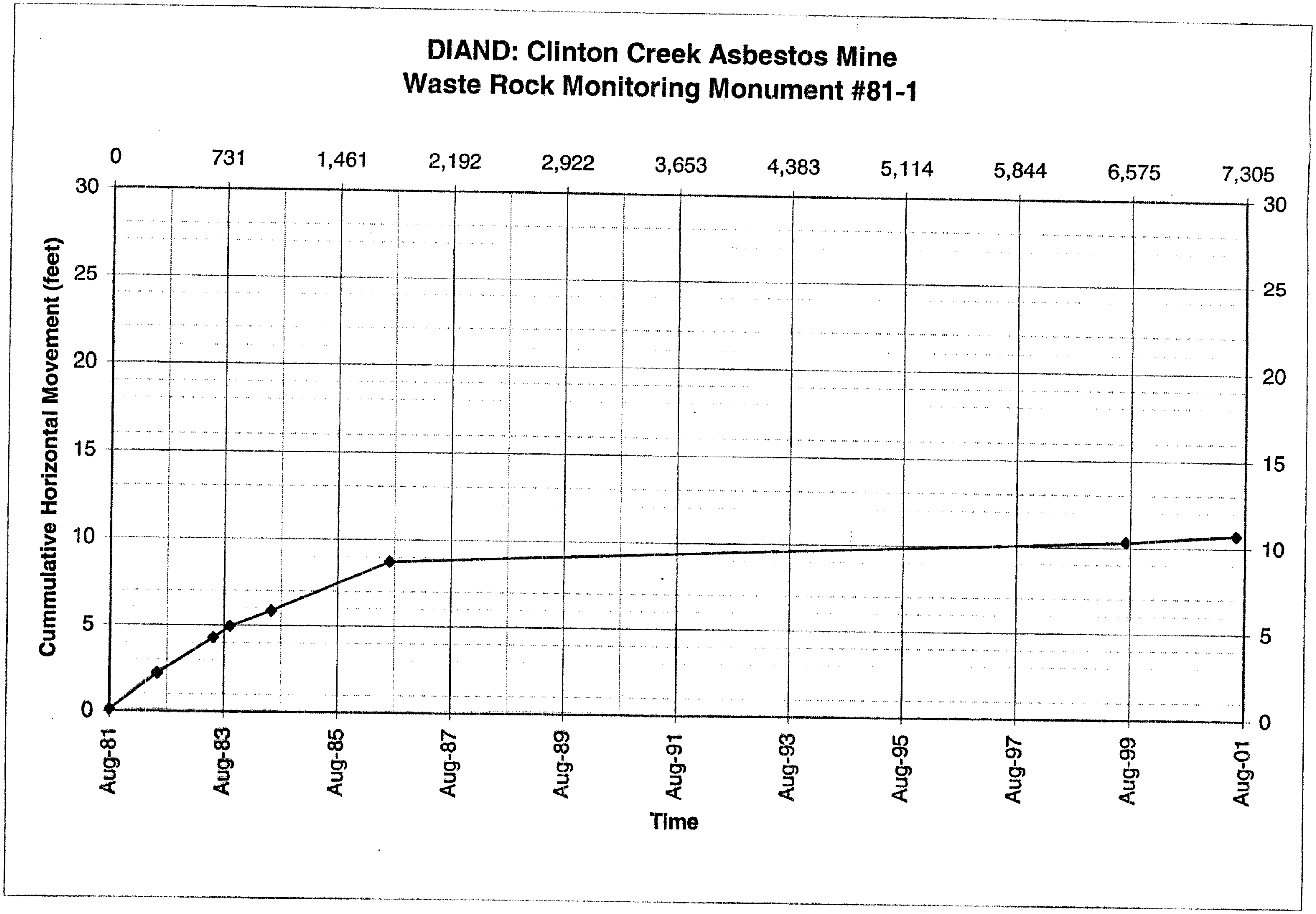
Client: DIAND
Project: Clinton Creek Asbestos Mine
Job No.: 4440-038-02-02
Date: 9-Jul-01

Waste Dump Stability - Monitoring Point #81-2

Notes: Assume all elevations represent top of monitoring point, not ground surface.
June 2001 survey) monitor point elevation = ground elev + monitor rod ht.

Values extrapolated

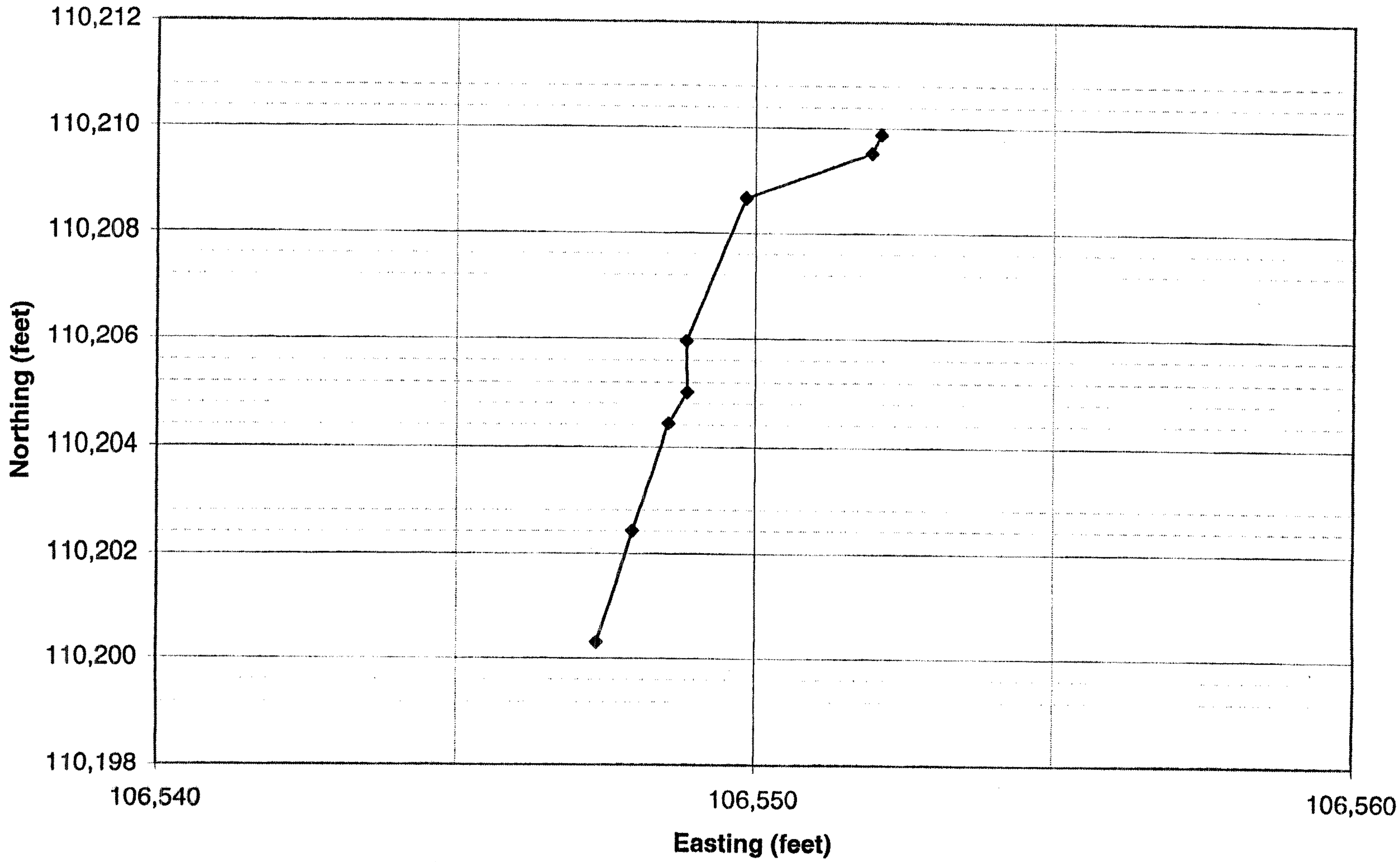
Monitoring Date	Northing	Easting	Elevation	Time		Horizontal Movement			Vertical Movement		
				Total	Increment	total	increment	rate	total	incremental	rate
	(feet)	(feet)	(feet)	(days)	(days)	(feet)	(feet)	(feet/year)	(feet)	(feet)	(feet/year)
15-Aug-81	110761.25	106658.1	1466.27	0	0	0	0	0	0	0	0
15-Jun-82	110763.4	106658.9	1465.53	304.0	304.0	2.29	2.29	2.754	-0.74	-0.74	-0.89
9-Jun-83	110,765.72	106,659.83	1465.06	663.0	359.0	4.79	2.50	2.541	-1.21	-0.47	-0.48
23-Sep-83	110,765.19	106,660.00	1463.97	769.0	106.0	4.37	0.56	1.917	-2.30	-1.09	-3.75
14-Jun-84	110,766.37	106,660.40	1463.78	1034.0	265.0	5.61	1.25	1.716	-2.49	-0.19	-0.26
15-Jul-86	110,768.77	106,661.46	1462.88	1795.0	761.0	8.24	2.62	1.258	-3.39	-0.9	-0.43
17-Jul-99	110,768.58	106,662.14	1459.45	6545.0	4750.0	8.37	0.71	0.054	-6.82	-3.43	-0.26
19-Jun-01	110,769.03	106,662.33	1459.58	7248.0	703.0	8.86	0.49	0.254	-6.69	0.13	0.07



File: monitoring data #81-1.xls Tab: #81-1 horiz mvmnt

Date: 5/30/2002

DIAND: Clinton Creek Asbestos Mine
Waste Rock Monitoring Monument #81-1



File: monitoring data #81-1.xls Tab: 81-1 - NE

Date: 5/30/2002

Appendix B

Test Hole Logs

RECORD OF BOREHOLE 12 (T-5)

LOCATION (See Figure 6)
BORING DATE May 9, 1978
BOREHOLE TYPE
BOREHOLE DIAMETER 6 in.
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.
DATUM

SOIL PROFILE ELEV. / DEPTH	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT (W_P — W — W_L; 10, 20, 30, 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1945.2'	Surface of Tailing Pile							
0.0'	Tails							
1911.7' / 33.5'	Compact, light brown, sub-rounded, fine to med. GRAVEL with clay, silt and sand - fluvial lacustrine traces of organics at bottom of tails		1 2				⊙	Thermistor cable instal… to 54 ft. (9 units at 5' intervals…
1891.2 / 54.0'	End of Hole							

VERTICAL SCALE

Golder Associates

DRAWN R.O.

Project No. Y11016

RECORD OF BOREHOLE 13 (T-6)

LOCATION (See Figure 6)

BOREHOLE TYPE

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

BORING DATE May 9, 1978

BOREHOLE DIAMETER 6 in.

DATUM

SOIL PROFILE: ELEV. / DEPTH	SOIL PROFILE: DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT (W_P — W — W_L) 10 20 30 40	PIEZOMETER OR STANDPIPE INSTALLATIO[N] / ADDITIONAL LAB. TESTIN[G]
1880.6' / 0.0'	Ground Surface in Roadway Cut							
	Frozen, light brown sub-rounded fine to med. GRAVEL with clay, silt & sand fluvial lacustrines		1					
			2					
			3					
			4					
1840.6' / 40.0'	End of Hole							Thermisto[r] cable insta[lled] to 40 ft. (9 units a[t] 5' interva[ls])

VERTICAL SCALE 1 inch to 20 feet

Golder Associates

DRAWN R.R

RECORD OF BOREHOLE 14 (T-7)

LOCATION (See Figure 6)

BOREHOLE TYPE

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

BORING DATE May 10, 1978

BOREHOLE DIAMETER 6 in.

DATUM

ELEV. / DEPTH	SOIL PROFILE – DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT (W_P – W – W_L)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1741.0	Surface of Tailing Pile							
0.0'	Tails							
1696.0' / 45.0'	-Frozen - ice crystals - light brown - sub-rounded - fine to med. GRAVEL with clay, silt & sand. - fluvial - lacustrine		1 2					Thermistor cable instal[led] to 74ft. (9 units at 5' intervals
1667.0' / 74.0'	End of Hole							

VERTICAL SCALE
1 inch to 20 feet

Golder Associates

DRAWN E.P.

RECORD OF BOREHOLE 15 (ST-8)

LOCATION (See Figure 6)

BOREHOLE TYPE

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

BORING DATE May 11, 1978

BOREHOLE DIAMETER 6 in.

DATUM

SOIL PROFILE: ELEV. / DEPTH	SOIL PROFILE: DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT (W_P — W — W_L) 10 20 30 40	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1607.2'	Ground Surface in Road Cut							
0.0' / 1607	-Frozen - light brown - sub rounded -fine to med. GRAVEL with clay silt & sand - fluvial lacustrines		1				W ≈ 6	
	-Frozen - black - ARGILLITE weathered bedrock		2				W ≈ 11	
1567.2 / 40.0'	End of Hole							

VERTICAL SCALE
1 inch to 20 feet

Golder Associates

DRAWN R.D.

RECORD OF BOREHOLE 16 (T-8)

LOCATION (See Figure 6)

BOREHOLE TYPE

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

BORING DATE May 12, 1978

BOREHOLE DIAMETER 6 in.

DATUM

ELEV. DEPTH	SOIL PROFILE – DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P — W — W_L (10, 20, 30, 40)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
1623.8	Surface of Tailing Pile							
0.0'	Tails							
1560.8 / 63.0'	- light brown - sub-rounded - fine to med. GRAVEL with clay silt & sand. fluvial lacustrine		1				⊙ (approx. 13)	Thermistor cable insta[lled] to 83.0 ft. (9 units a[t] 5' interva[ls])
1540.8 / 83.0'	End of Hole							

VERTICAL SCALE

Golder Associates

DRAWN R.[...]

RECORD OF BOREHOLE 17 (D.S.-2)

LOCATION (See Figure 6)

BOREHOLE TYPE

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

BORING DATE May 16, 1978

BOREHOLE DIAMETER 6 in.

DATUM

SOIL PROFILE		STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
ELEV. DEPTH	DESCRIPTION							
0.0'	Frozen dark brown organic silty SAND							
3.0'	Frozen dark brown PEAT							
5.0'	Frozen, light brown, sub-rounded, fine to med. GRAVEL with clay, silt & Sand (fluvial Lacustrine)							
7.0'	ARGILLITE -hard, dry unweathered							
19.0'	SERPENTINE, weathered frozen							
21.0'	ARGILLITE BEDROCK soft, weathered, frozen		1					
46.0'	ARGILLITE BEDROCK unweathered, frozen		2					
57.0'	End of Hole							

VERTICAL SCALE 1 inch to 20 feet

Golder Associates

DRAWN R.D.

RECORD OF BOREHOLE 18 (D.S-5)

Project No. 771077

LOCATION (See Figure 6)
BOREHOLE TYPE
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

BORING DATE May 17, 1978
BOREHOLE DIAMETER 6 in.
DATUM

ELEV. DEPTH	DESCRIPTION (SOIL PROFILE)	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_p — W — W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
0.0'	Frozen, dark brown, organic silty, SAND.							
8.0'	Frozen, light brown, sub-rounded, fine to med. GRAVEL with clay, silt & sand (fluvial lacustrian)		1					
19.0'	ARGILLITE frozen, weathered (ice lens approx. 3in. thick recovered with sample)		2					
37.0'	ARGILLITE -frozen, becoming harder with depth, unweathered		3					
			4					
60.0'	End of Hole							

VERTICAL SCALE
1 inch to ... feet

Golder Associates

DRAWN R.C

RECORD OF BOREHOLE 19(D-S-6)

LOCATION (See Figure 6)
BOREHOLE TYPE
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 IN.

BORING DATE May 18, 1978
BOREHOLE DIAMETER 6 in.
DATUM

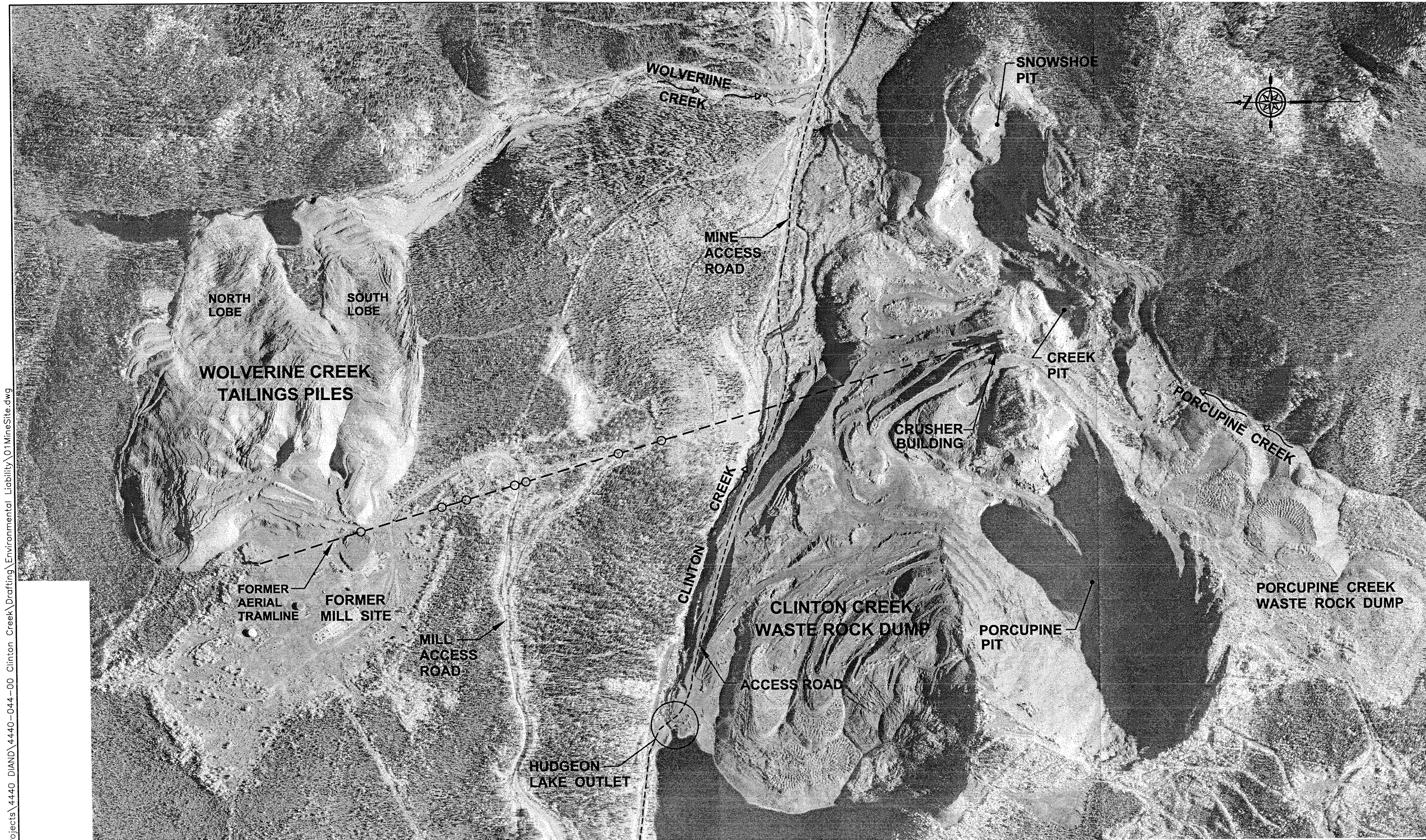
SOIL PROFILE ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_p — W — W_L	PIEZOMETER OR STANDPIPE INSTALLATION ADDITIONAL LAB. TESTING
0.0'	Frozen, light brown sub-rounded, fine to medium GRAVEL with clay, silt & sand (alluvial)							
7.0'	Frozen silt with layers of fibrous peat							
17.0'	Frozen, light brown sub-rounded, fine to medium GRAVEL with clay, silt and sand (fluvial lacustrine)		1					
20.0'	ARGILLITE frozen, weathered							
32.0'	ARGILLITE - frozen becoming harder with depth, unweathered		2, 3					
60.0'	End of Hole							

VERTICAL SCALE
1 inch to 20 feet

Golder Associates

DRAWN R.D.
CHECKED [illegible]

Drawings



REV.	DESCRIPTION	DWN.	APP.	DATE

SEAL

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SCALE: 1:7,500	JOB No. 4440-044-00

INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

MINE SITE PLAN

DWG. No. 01 REV. –

Plot Scale: 1=1
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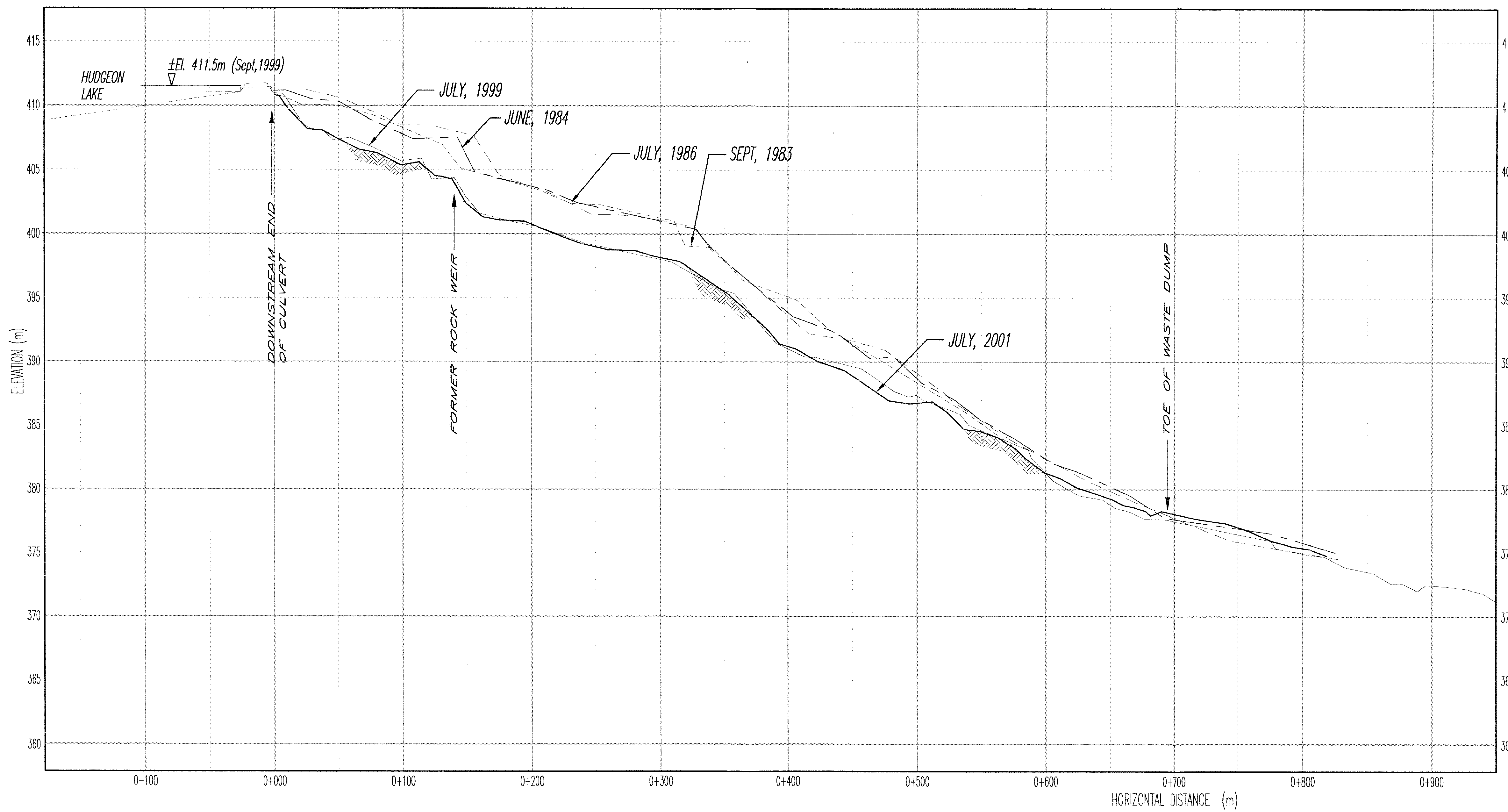

CLINTON CREEK CHANNEL PROFILE

VER. SCALE 1:300
HOR. SCALE 1:3000

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INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

CLINTON CREEK CHANNEL PROFILES

DWG. No. 02
REV. –

Plot Scale: 1=1
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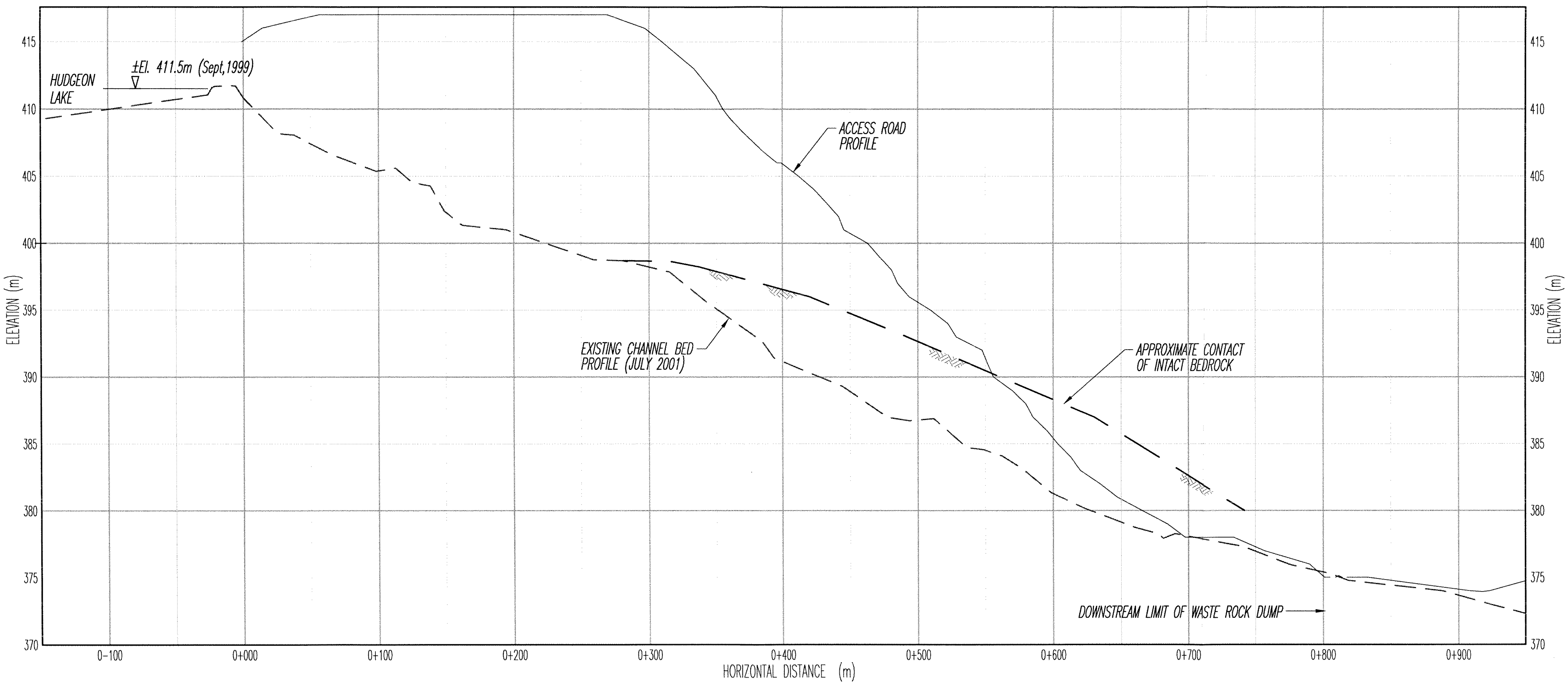

VER. SCALE 1:300
HOR. SCALE 1:3000

REV.	DESCRIPTION	DWN.	APP.	DATE

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INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

CLINTON CREEK CHANNEL
APPROXIMATE BEDROCK PROFILE

DWG. No. 03 | REV. –

Plot Scale: 1=1
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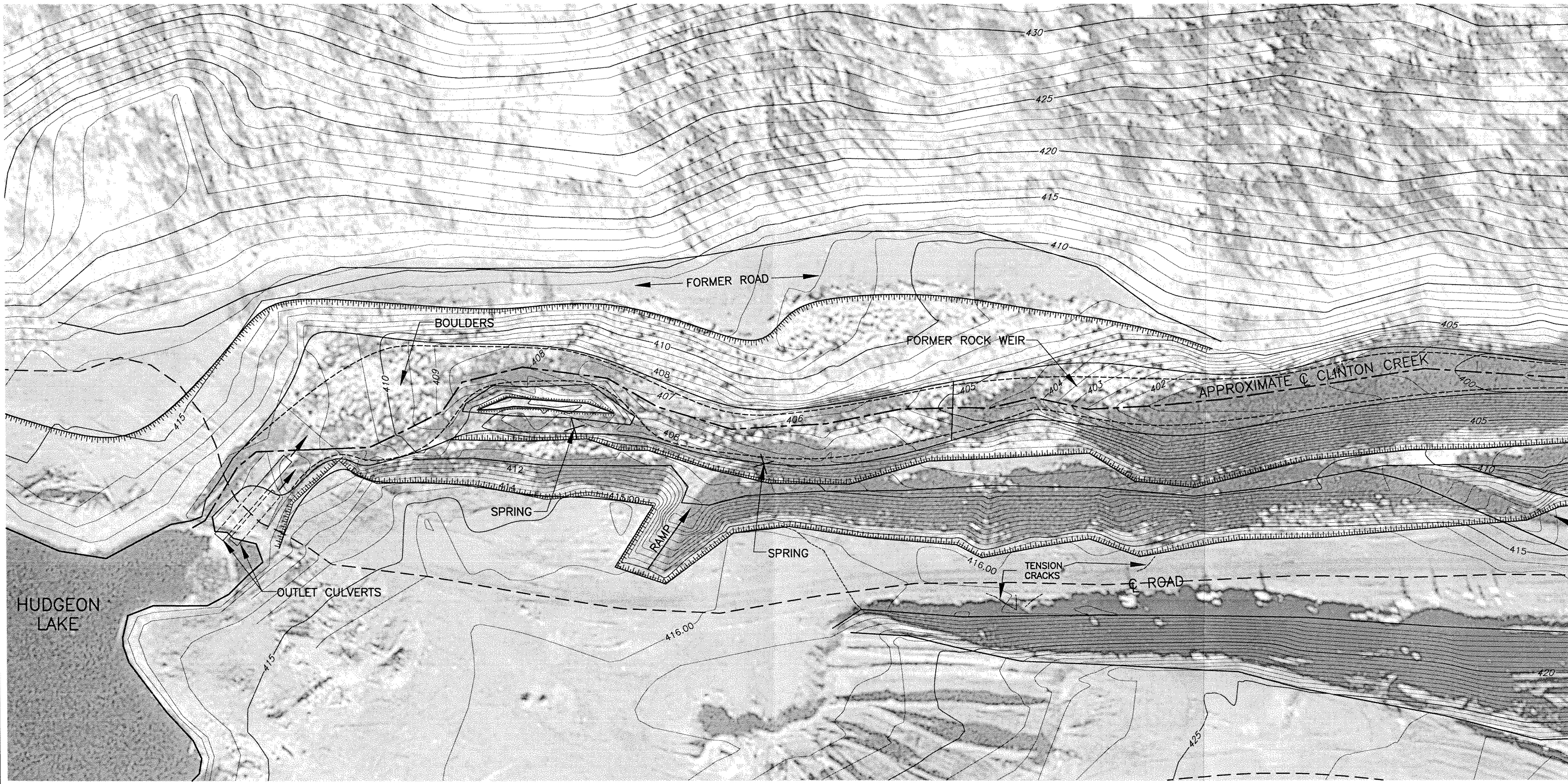

DATE OF AERIAL PHOTOGRAPHY: SEPTEMBER 1999

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INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

HUDGEON LAKE OUTLET
SITE PLAN

DWG. No. 04 — REV. –

Plot Scale: 1=1
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Plot Scale: 1=2
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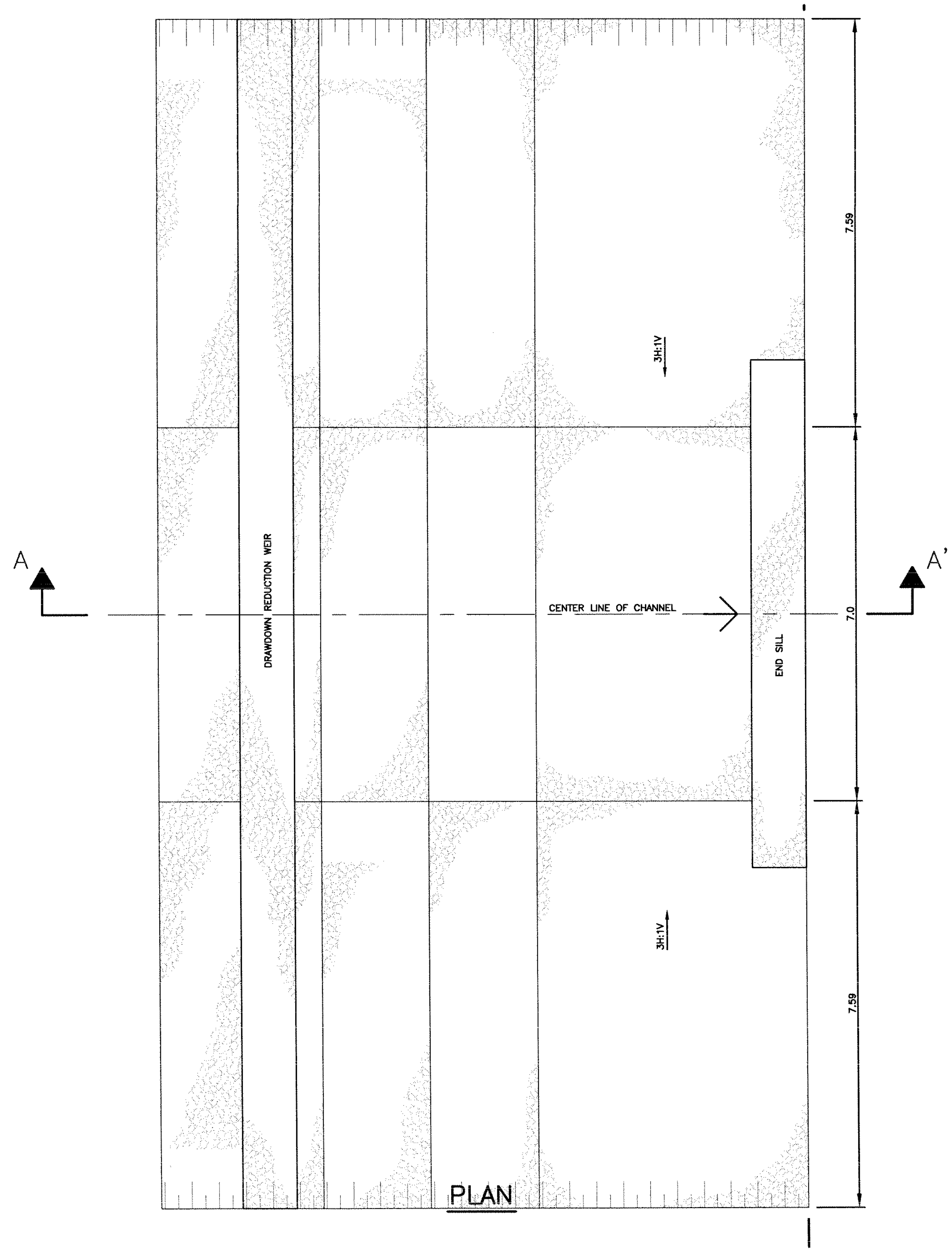
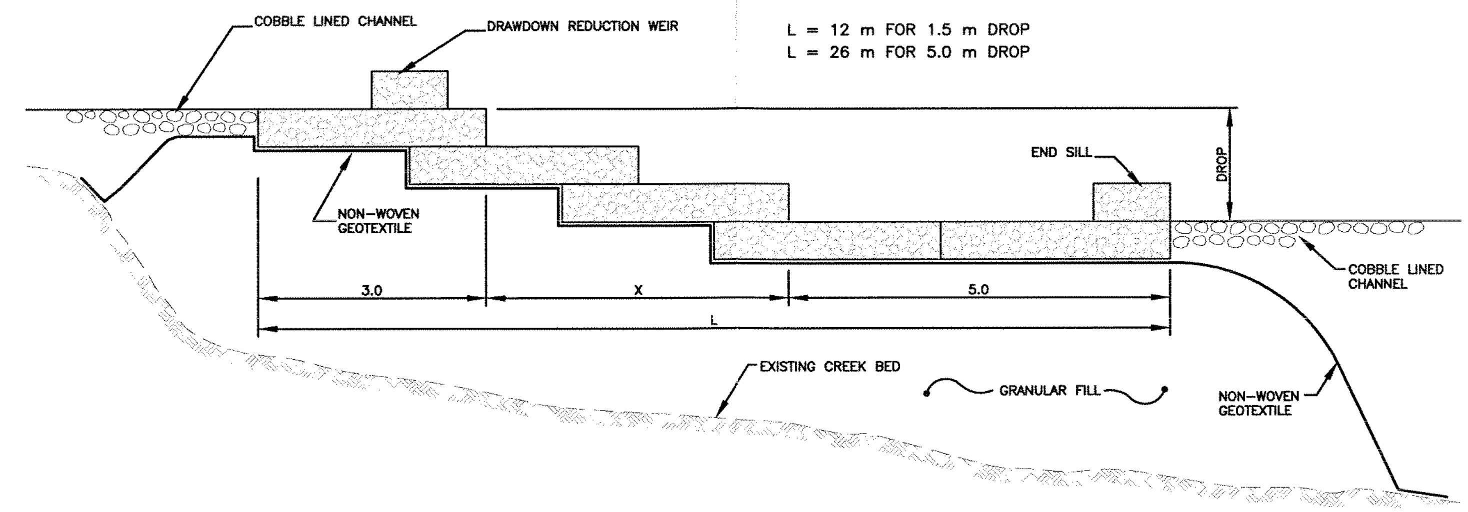


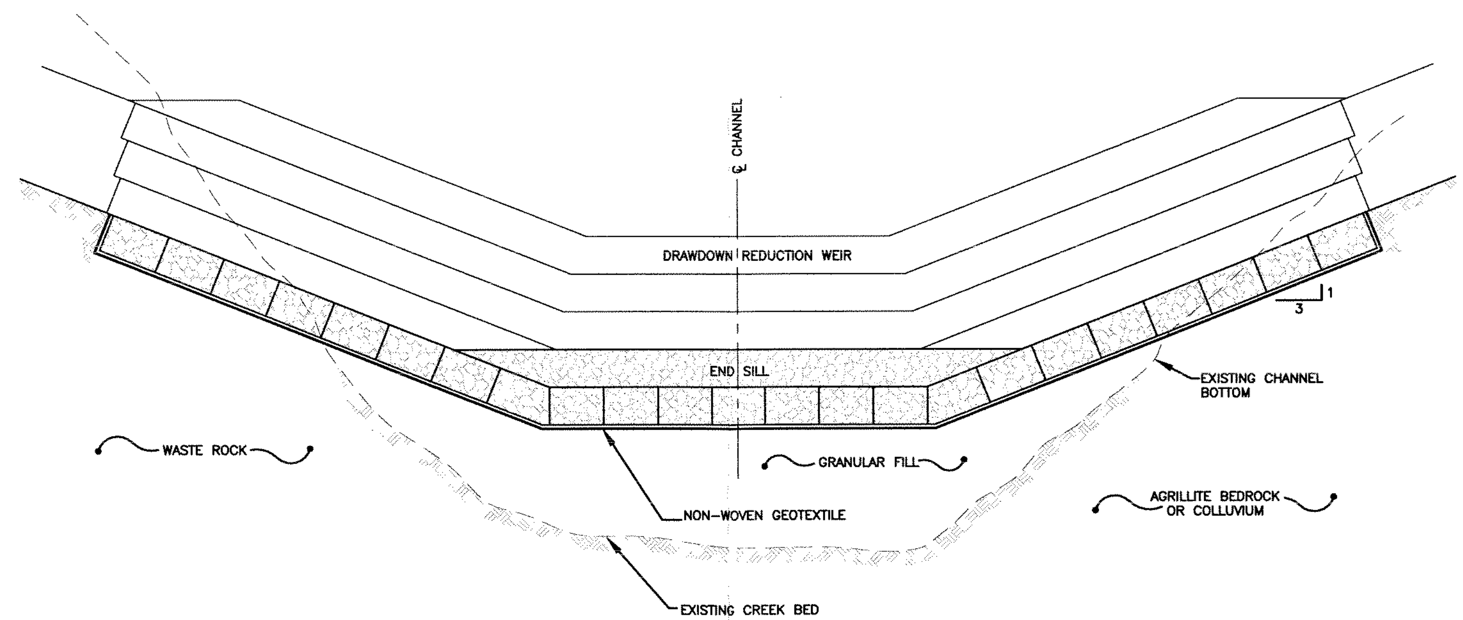
PLAN

NOTE: X = (NUMBER OF 0.5 m STEPS − 1) x 2m
ie. FOR 3 STEPS (1.5 m DROP) X = (3 −1) x 2 = 4 m
FOR 10 STEPS (5 m DROP) X = (10 − 1) x 2 = 18 m

L = 12 m FOR 1.5 m DROP
L = 26 m FOR 5.0 m DROP



CENTERLINE PROFILE A–A'



SECTION B–B'

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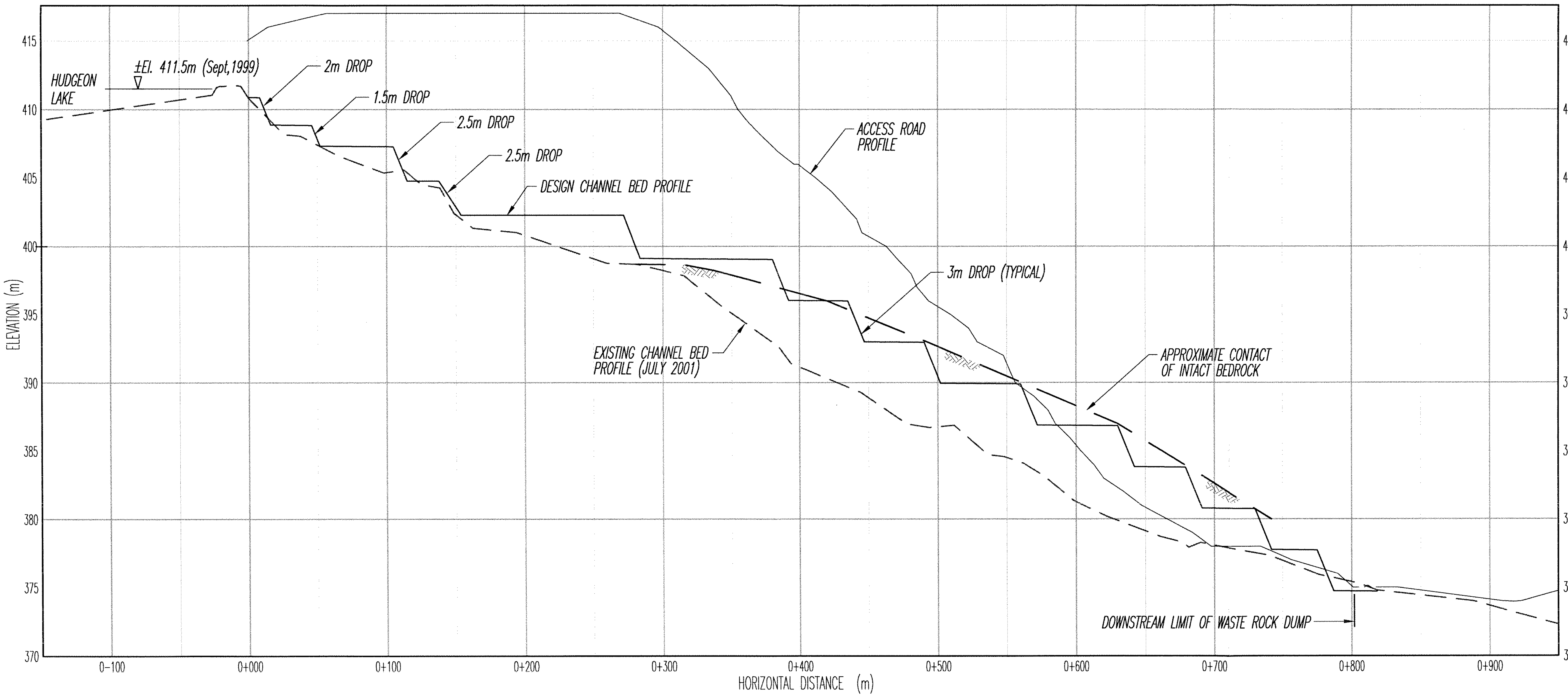
INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

TYPICAL GABION DROP STRUCTURE

DWG. No. 05 REV. –



VER. SCALE 1:300
HOR. SCALE 1:3000

REV.	DESCRIPTION	DWN.	APP.	DATE

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INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

CLINTON CREEK PROPOSED CHANNEL PROFILE

DWG. No. 06 | REV. –

Plot Scale: 1=1
L:\Earth & Water\Projects\4440 DIAND\4440-044-00 Clinton Creek\Drafting\Environmental Liability\06ChannelProfile.dwg

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INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

POTENTIAL BORROW SOURCES

DWG. No.	REV.
07	–

Plot Scale: 1=1
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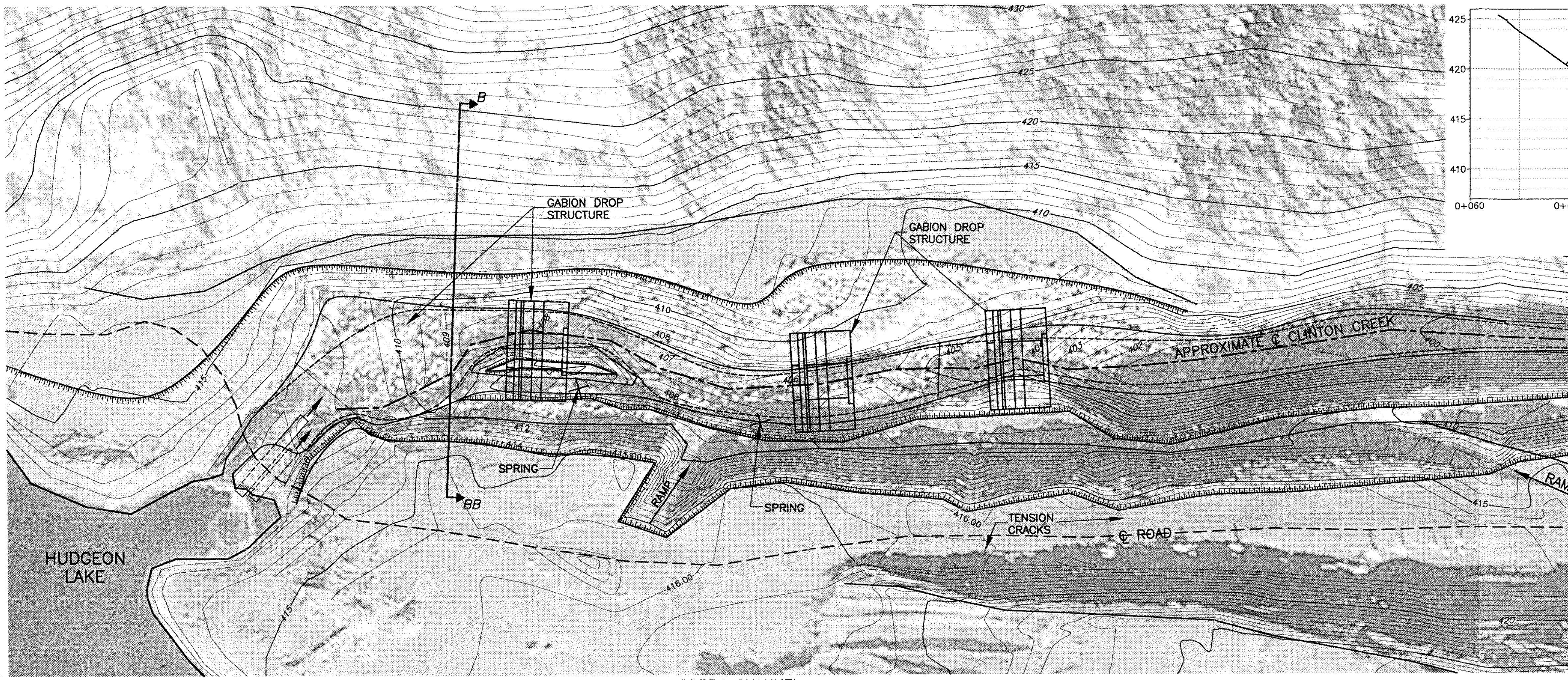
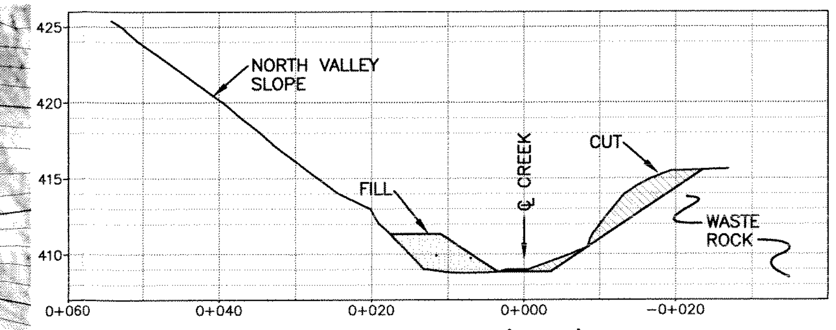
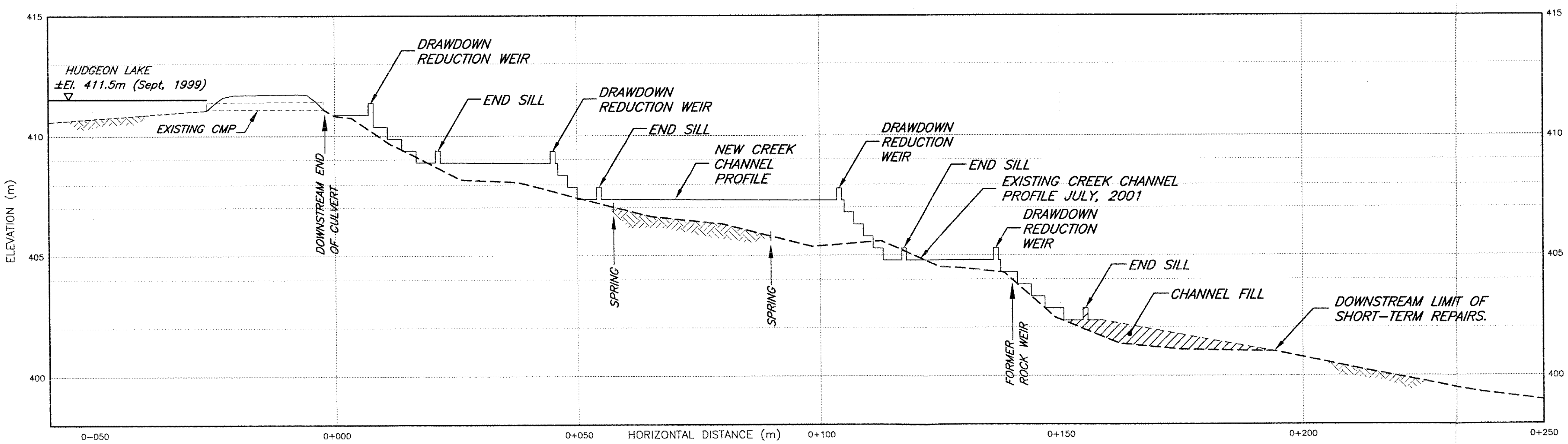


CLINTON CREEK CHANNEL
SCALE 1:1000



CROSS SECTION 'B–BB'
VER. SCALE 1:200
HOR. SCALE: 1:400



CLINTON CREEK CHANNEL PROFILE
VER. SCALE 1:200
HOR. SCALE: 1:1000

REV.	DESCRIPTION	DWN.	APP.	DATE

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INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

HUDGEON LAKE OUTLET AND CLINTON CREEK
STAGED CONSTRUCTION

DWG. No. 08 — REV. –

Plot Scale: 1=1
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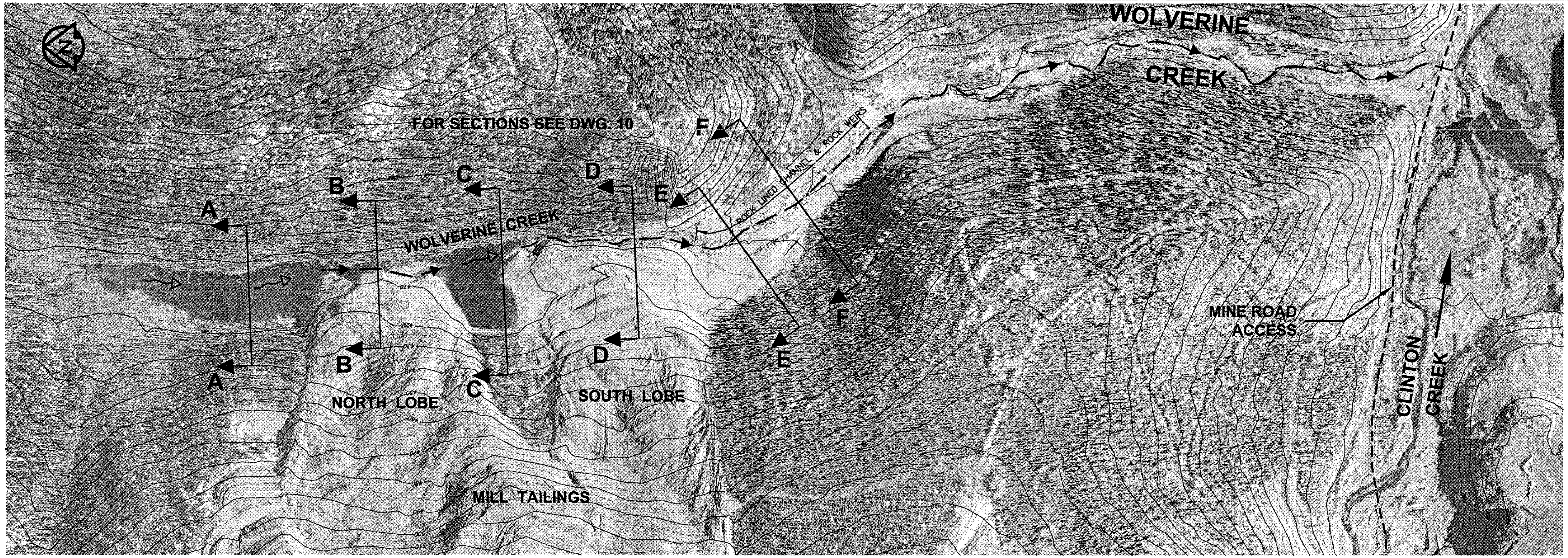
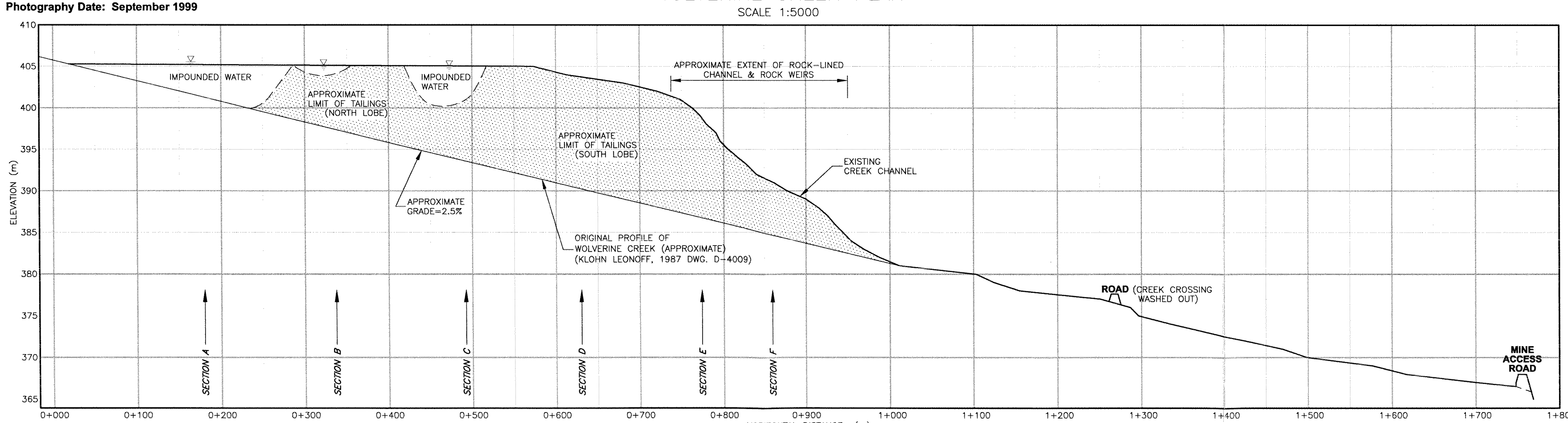


NAD 83 UTM Zone 7
Photography Date: September 1999

WOLVERINE CREEK PLAN

SCALE 1:5000



EXISTING WOLVERINE CREEK CHANNEL PROFILE

VER SCALE 1:500
HOR SCALE 1:5000

REV.	DESCRIPTION	DWN.	APP.	DATE

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INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

WOLVERINE CREEK
PLAN AND PROFILE

DWG. No. 09
REV. –

Plot Scale: 1=1
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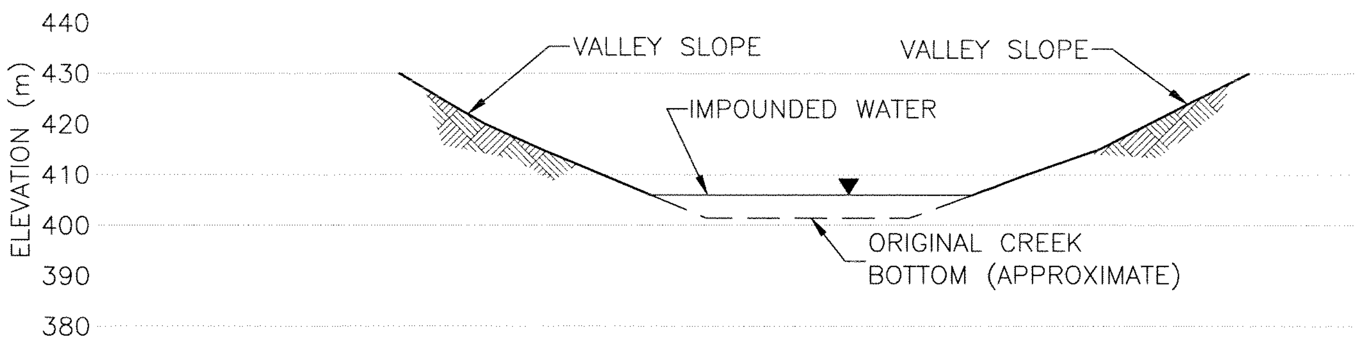
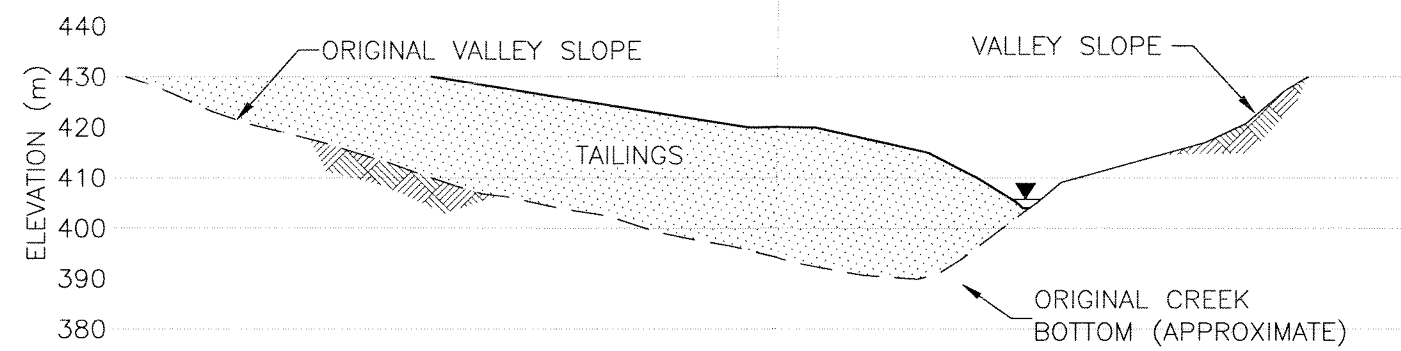
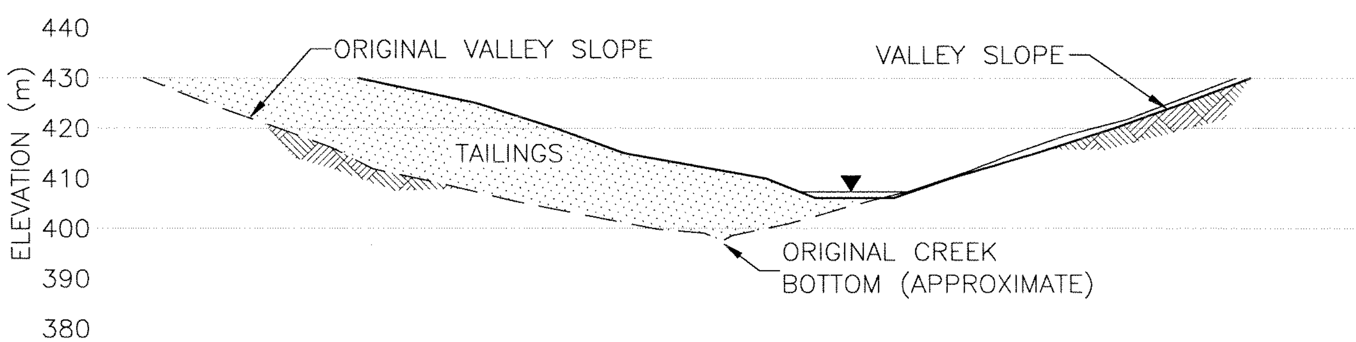


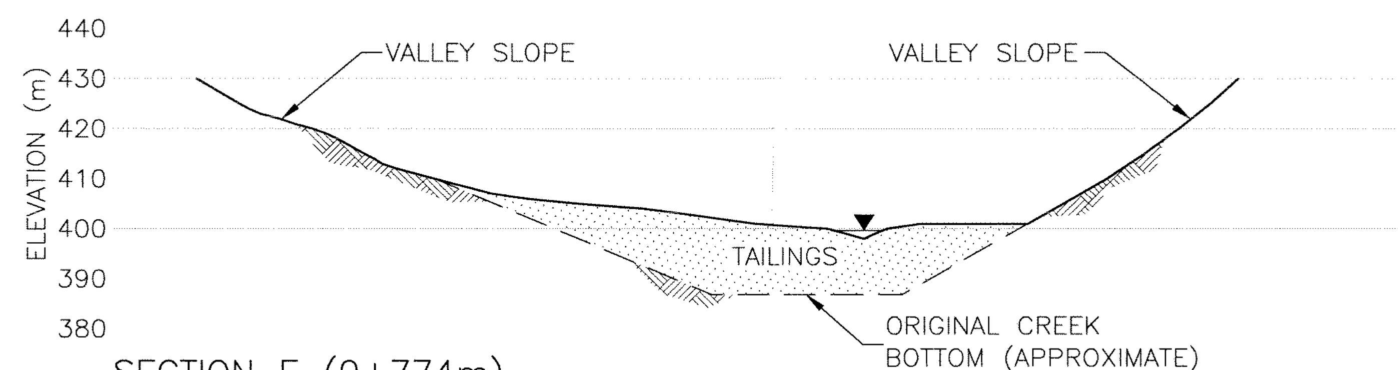
SECTION A (0+180m)



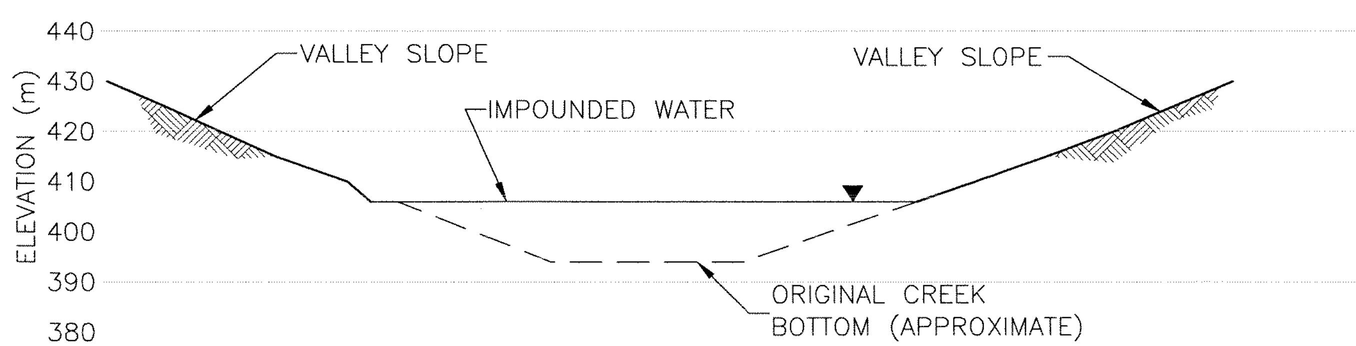
SECTION D – SOUTH TAILINGS LOBE (0+630m)



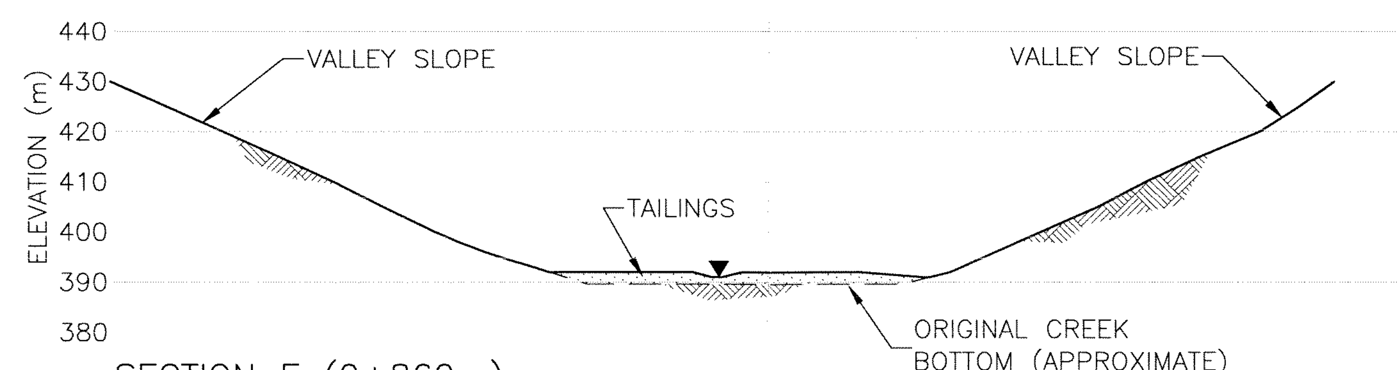
SECTION B – NORTH TAILINGS LOBE (0+337m)



SECTION E (0+774m)



SECTION C (0+492m)



SECTION F (0+860m)

REV.	DESCRIPTION	DWN.	APP.	DATE

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SCALE: 1:1,500 | JOB No. 4440-044-00

INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE ENVIRONMENTAL LIABILITY

WOLVERINE CREEK EXISTING SECTIONS

DWG. No. 10 | REV. –

Plot Scale: 1=1
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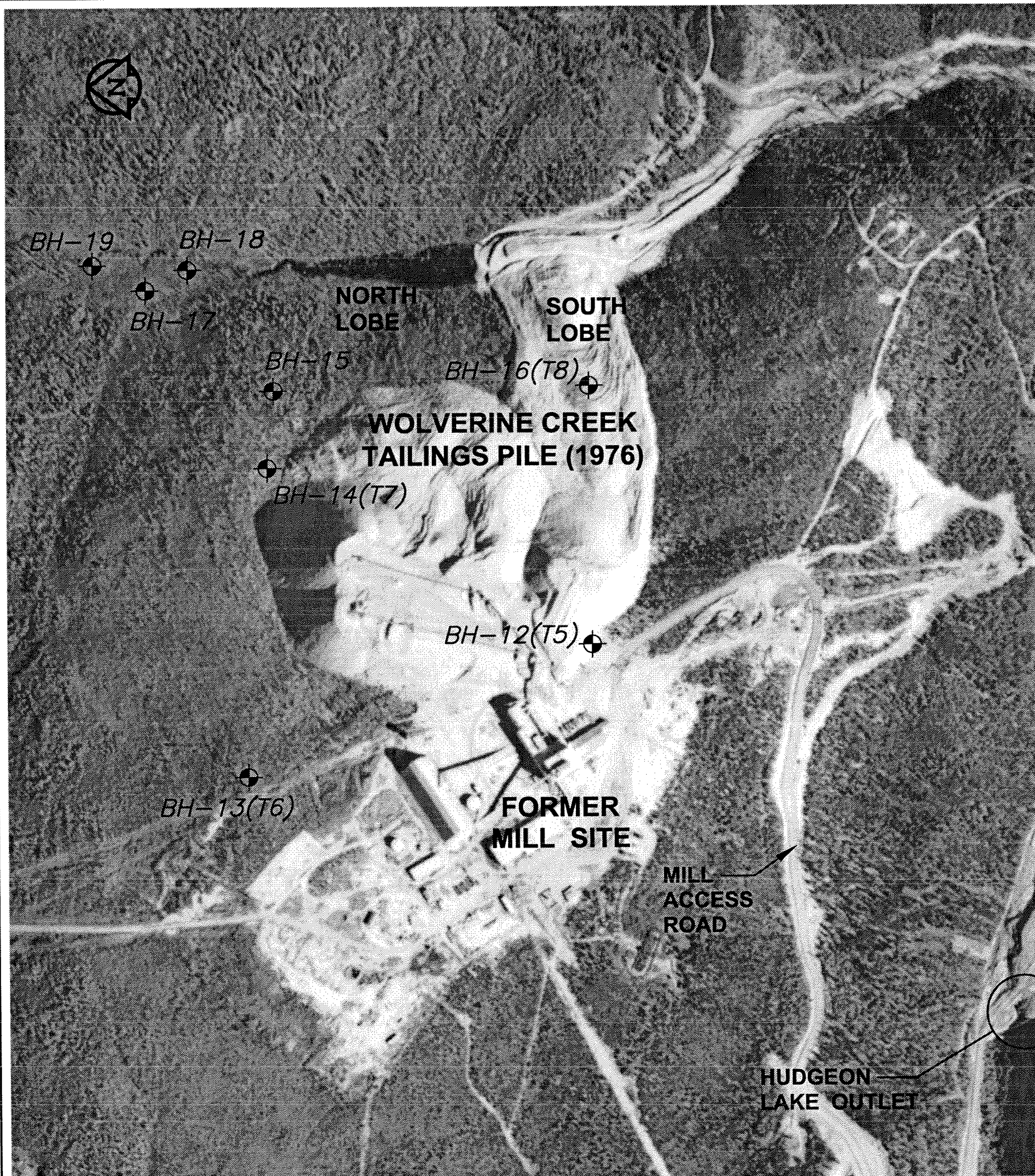

DATE OF PHOTOGRAPHY: 1976

LEGEND

(T5) ⊕ THERMISTOR (GOLDER, 1978)

BH−15 ⊕ BORE HOLE (GOLDER, 1978)

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DRAWN BY: LJV	DESIGNED BY: −	TAILINGS PILE TEST HOLES AND THERMISTORS	DWG. No. 11	REV. −
CHECKED BY: GR	CHECKED BY: −			
SCALE: 1:7500	JOB No. 4440−044−00			

Plot Scale: 1=1
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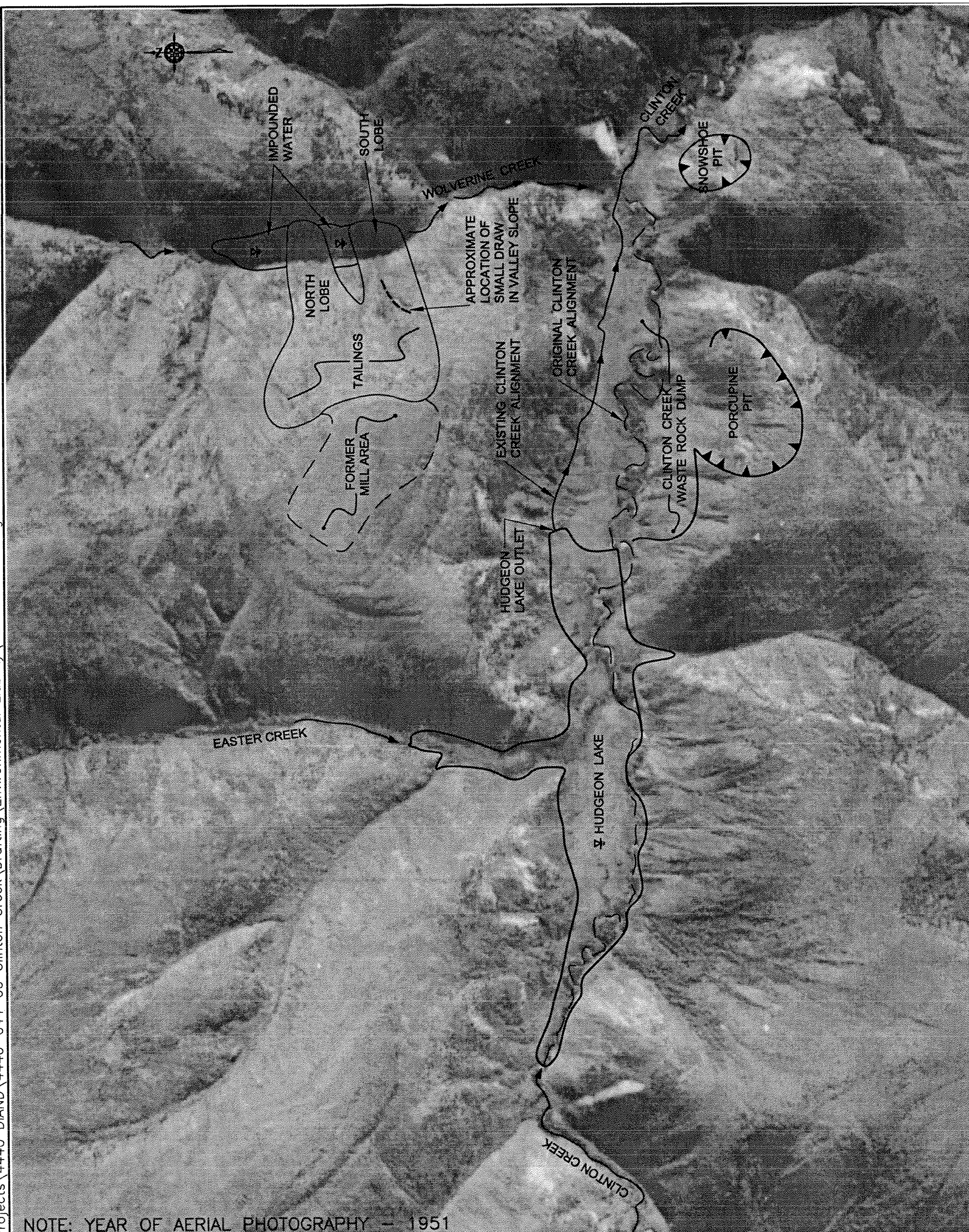

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SCALE:	1:20,000	JOB No.	4440–044–00

INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

ORIGINAL SITE CONDITIONS WITH
EXISTING MINE SITE FEATURES

DWG. No. 12
REV. –

Plot Scale: 1=1
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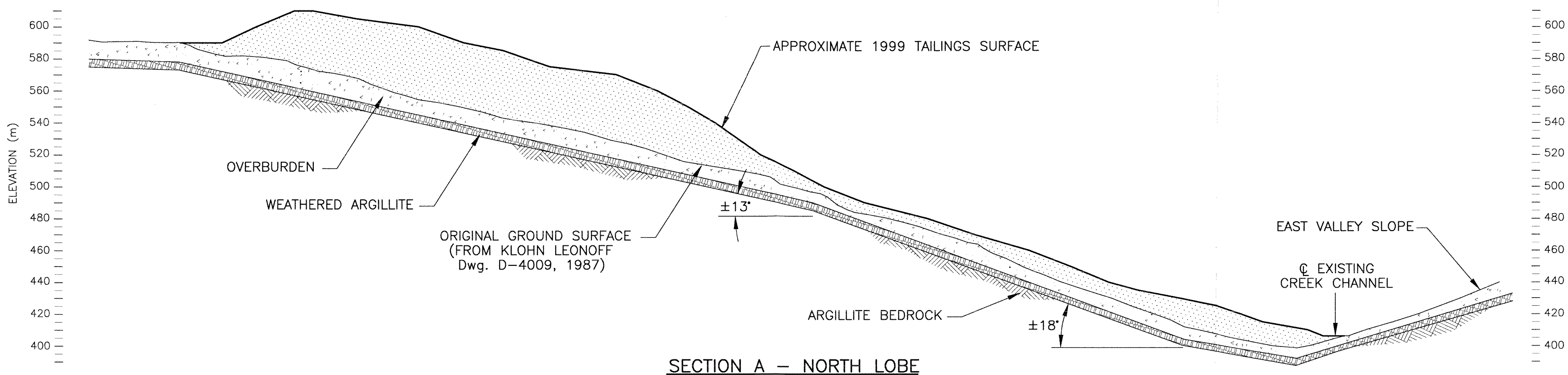
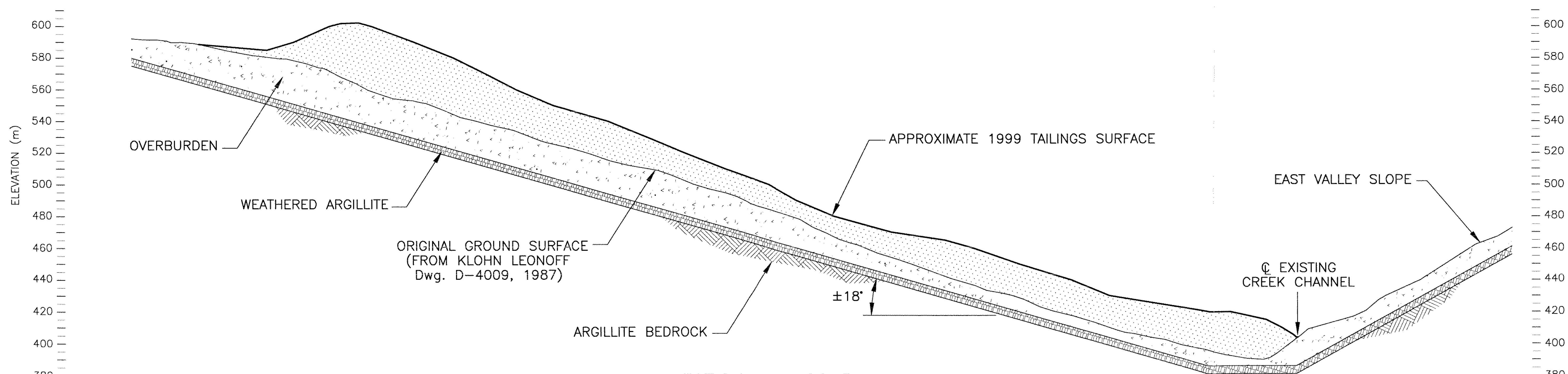


SECTION A – NORTH LOBE

SCALE 1:2500



SECTION B – SOUTH LOBE

SCALE 1:2500

REV.	DESCRIPTION	DWN.	APP.	DATE

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SCALE: AS NOTED	JOB No. 4440-044-00

INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

WOLVERINE CREEK TAILINGS
EXISTING SECTIONS

DWG. No. 13 REV. –

Plot Scale: 1=1
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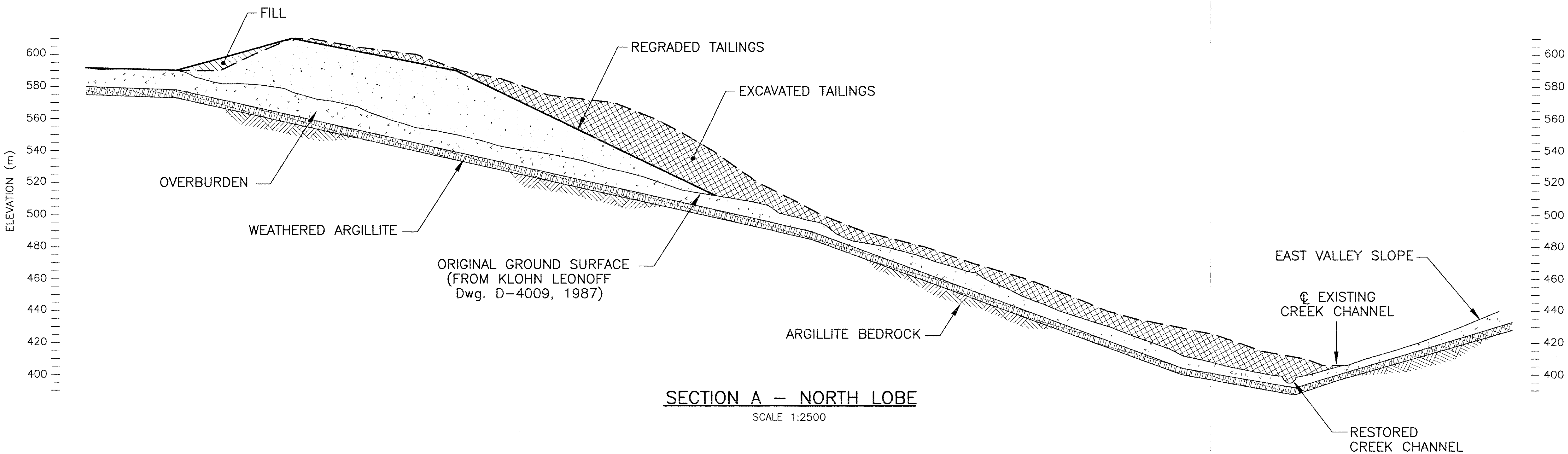
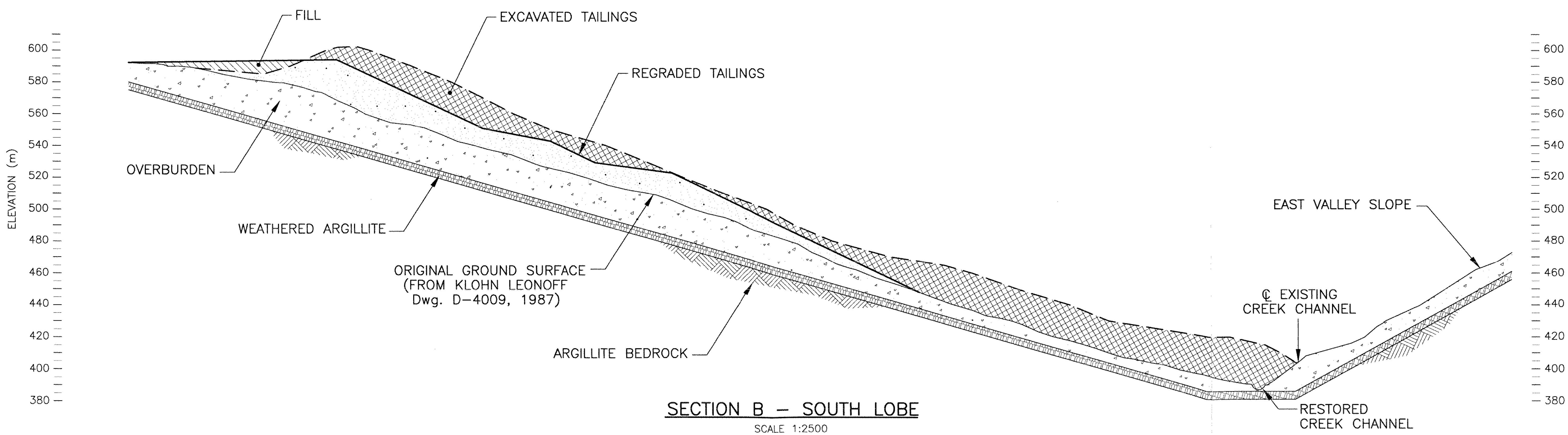


SECTION A – NORTH LOBE
SCALE 1:2500



SECTION B – SOUTH LOBE
SCALE 1:2500

REV.	DESCRIPTION	DWN.	APP.	DATE

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INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

WOLVERINE CREEK TAILINGS
VALLEY RESTORATION

DWG. No. 14 | REV. –

Plot Scale: 1=1
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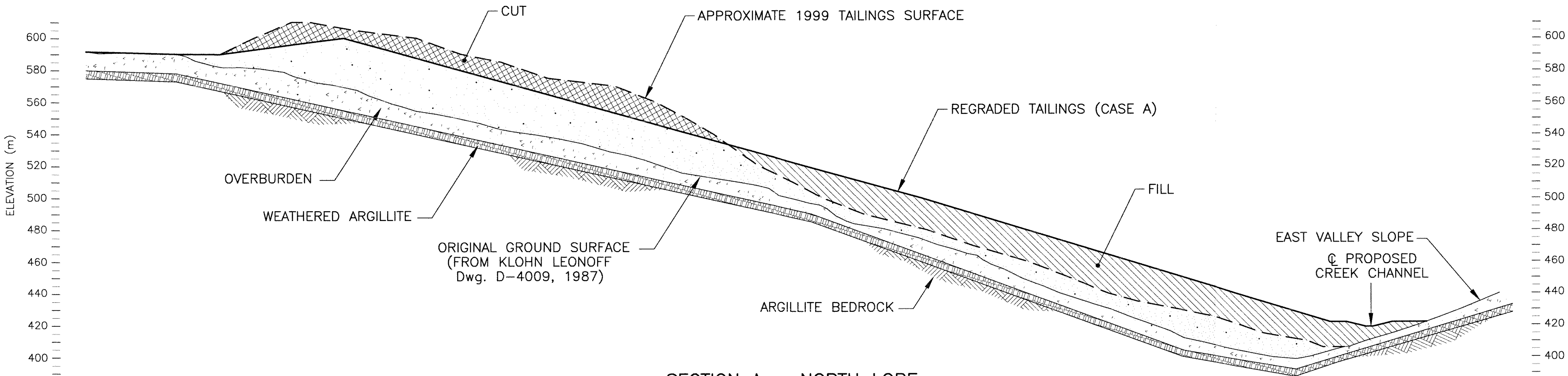
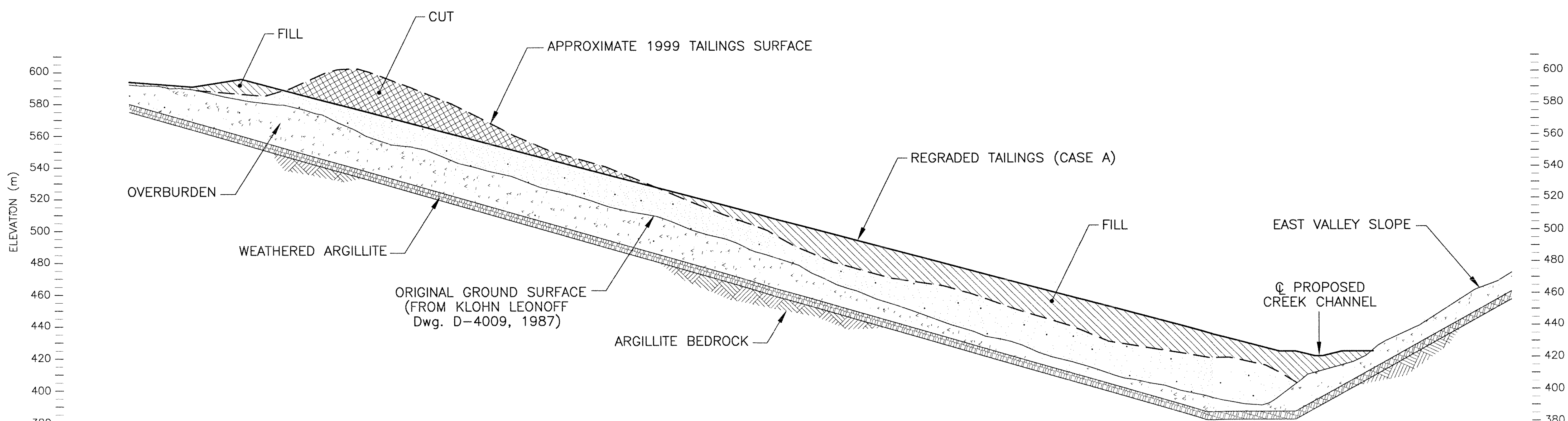


SECTION A – NORTH LOBE

SCALE 1:2500



SECTION B – SOUTH LOBE

SCALE 1:2500

REV.	DESCRIPTION	DWN.	APP.	DATE

SEAL

UMA Engineering Ltd.
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CHECKED BY: GR	CHECKED BY: –
SCALE: AS NOTED	JOB No. 4440-044-00

INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

WOLVERINE CREEK TAILINGS
REGRADING – CASE A

DWG. No. 15 REV. –

Plot Scale: 1=1
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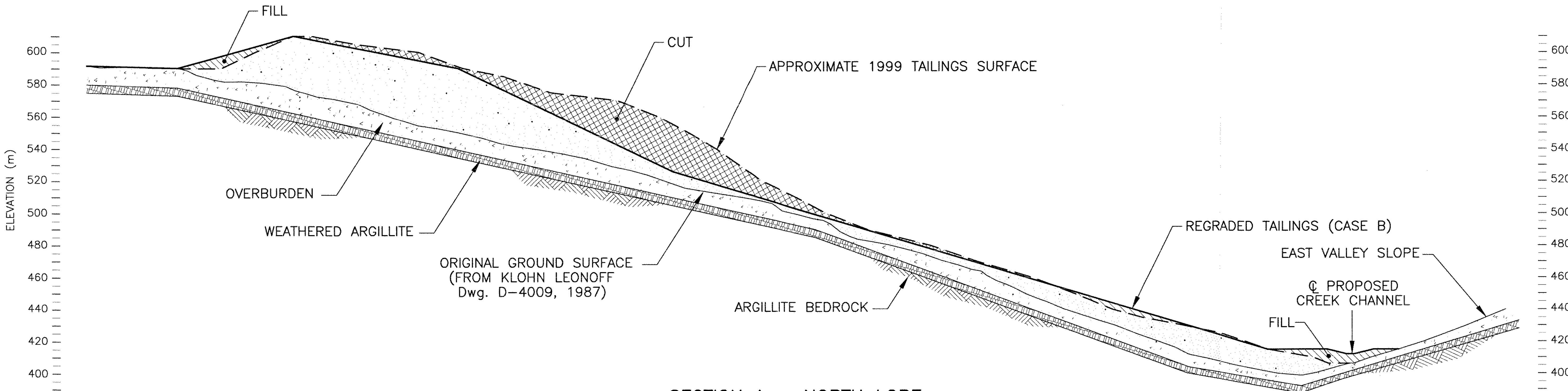

SECTION A – NORTH LOBE

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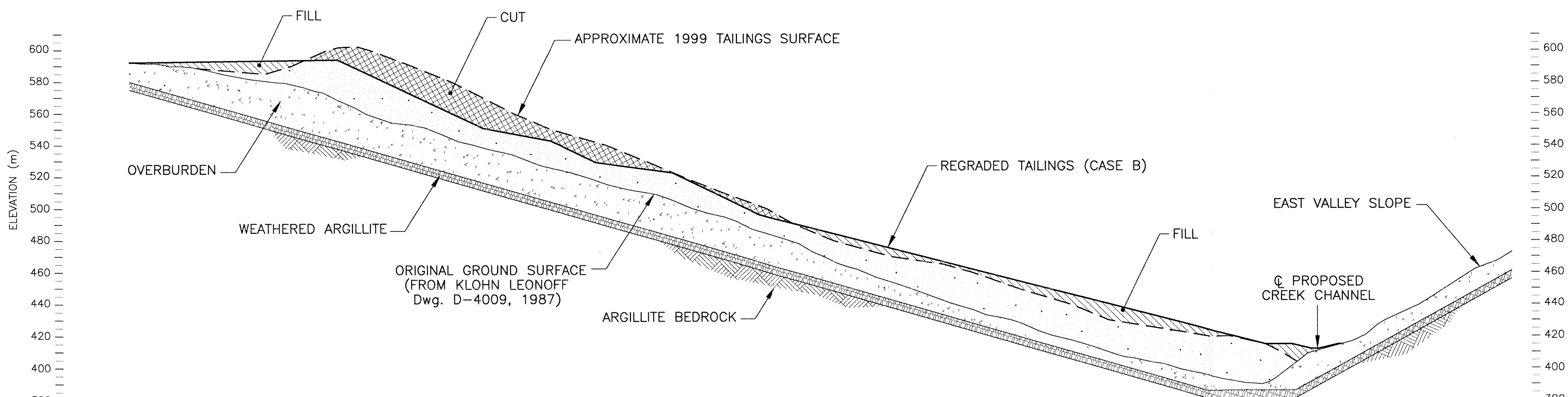

SECTION B – SOUTH LOBE

SCALE 1:2500

REV.	DESCRIPTION	DWN.	APP.	DATE

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INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

WOLVERINE CREEK TAILINGS
REGRADING – CASE B

DWG. No. 16 REV. –

Plot Scale: 1=1
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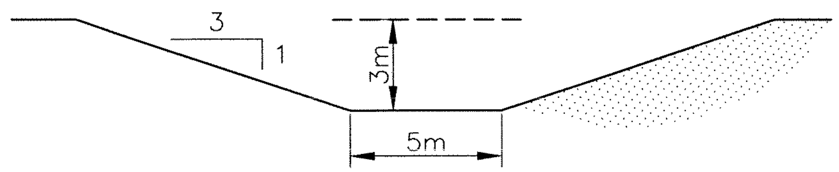
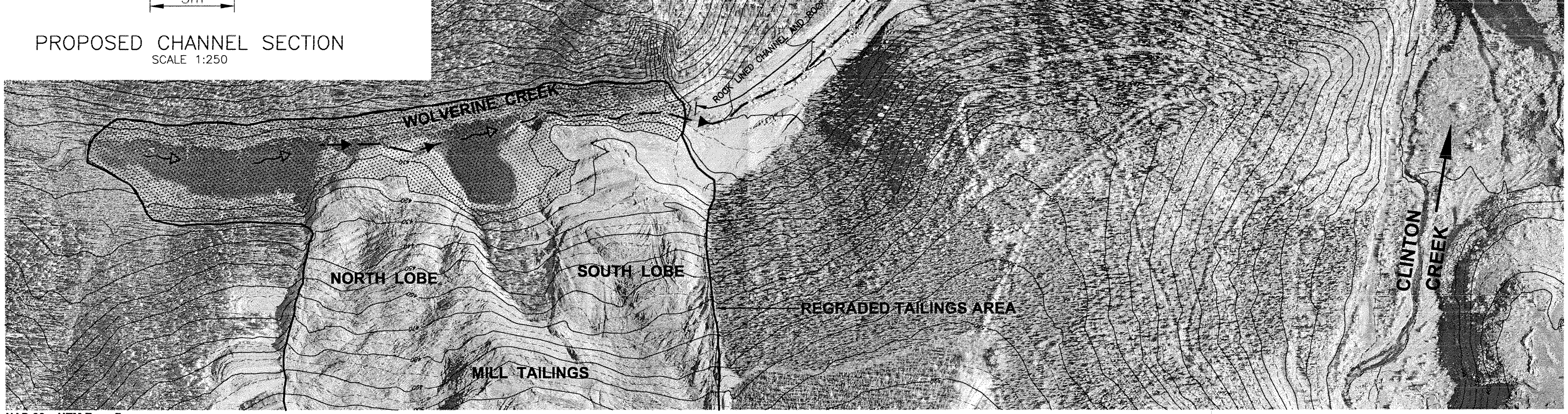
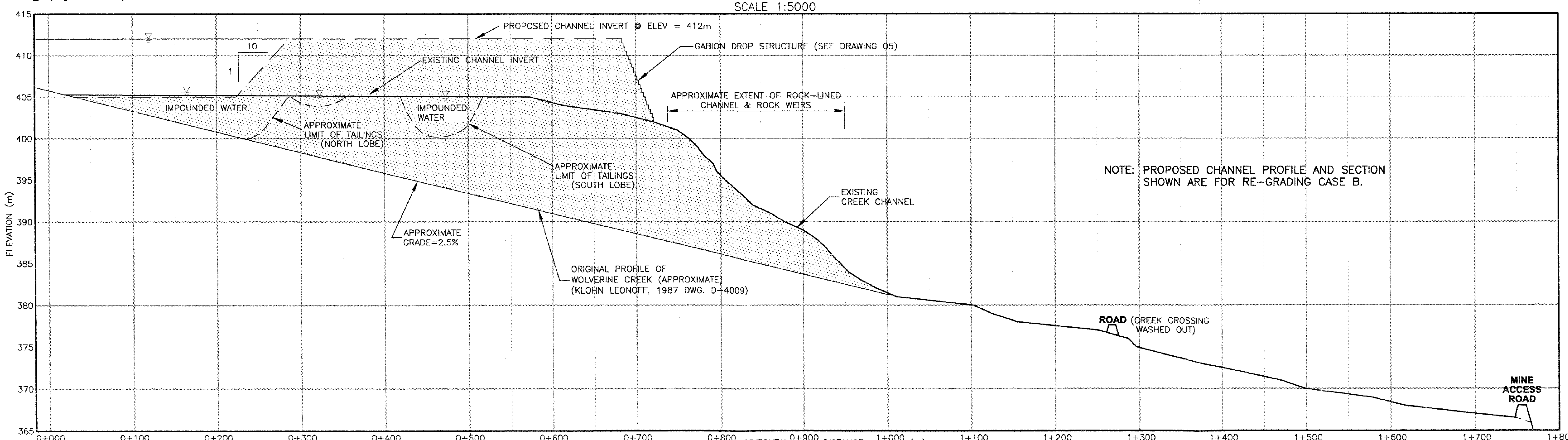


PROPOSED CHANNEL SECTION
SCALE 1:250



NAD 83 UTM Zone 7
Photography Date: September 1999

WOLVERINE CREEK PLAN
SCALE 1:5000



PROPOSED WOLVERINE CREEK CHANNEL PROFILE
VER SCALE 1:500
HOR SCALE 1:5000

REV.	DESCRIPTION	DWN.	APP.	DATE

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INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

WOLVERINE CREEK PROPOSED PLAN, PROFILE AND SECTION

DWG. No. 17 | REV. –

Plot Scale: 1=1
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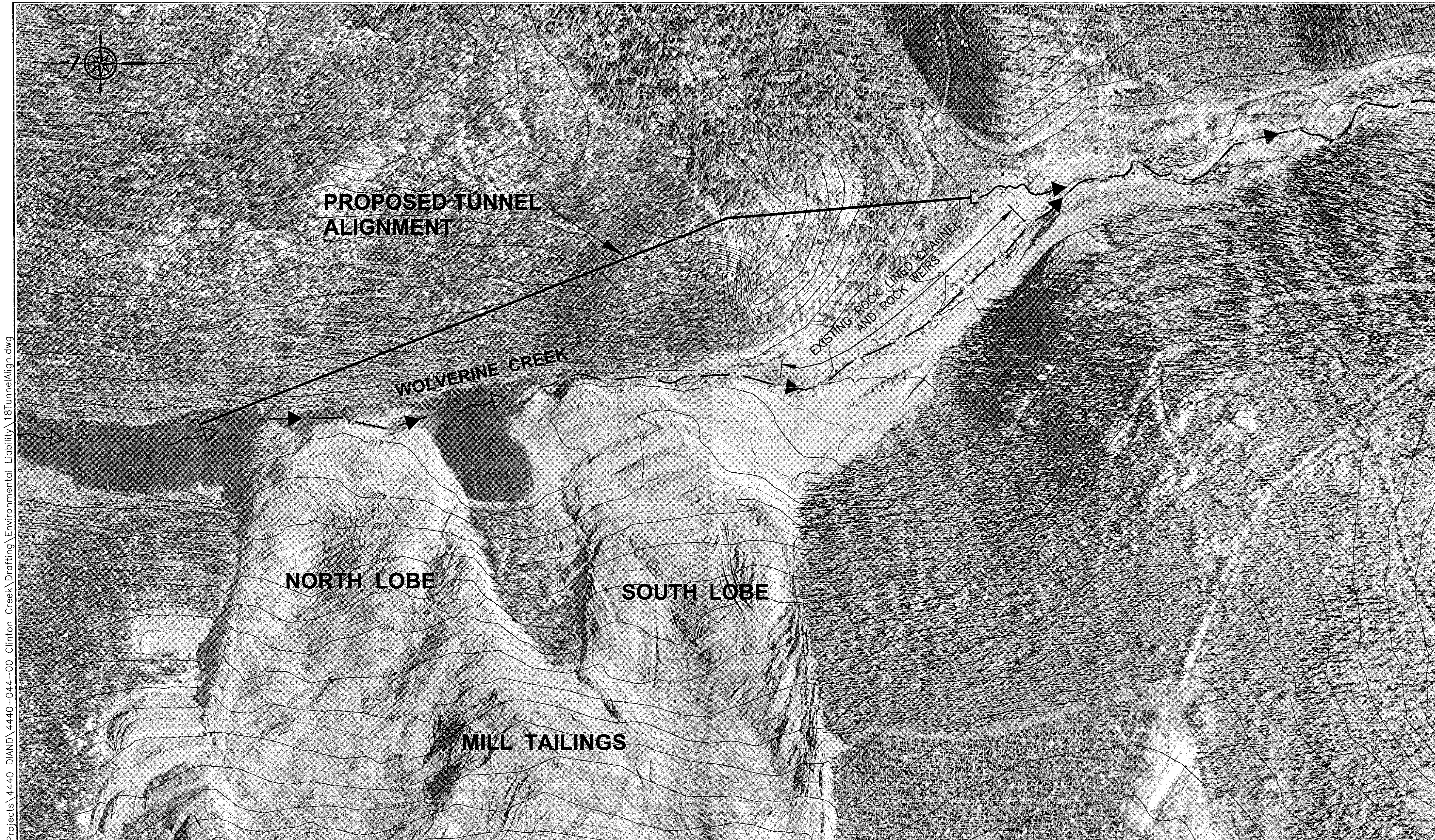

REV.	DESCRIPTION	DWN.	APP.	DATE

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APPROVED BY	DATE: JUNE, 2003
DRAWN BY: LJV	DESIGNED BY: –
CHECKED BY: GR	CHECKED BY: –
SCALE: 1:3000	JOB No. 4440-044-00

INDIAN AND NORTHERN AFFAIRS CANADA

ABANDONED CLINTON CREEK ASBESTOS MINE
ENVIRONMENTAL LIABILITY

WOLVERINE CREEK
PROPOSED TUNNEL ALIGNMENT

DWG. No. 18 REV. –

Plot Scale: 1=1
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