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Chilliwack Flood Protection Dykes Upgrade
Geotechnical Report
Chilliwack, B.C.

FPAF 02-02-07

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Submitted to:

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West Vancouver, B.C.

Submitted by:

AMEC Earth & Environmental Limited
Abbotsford, BC

15 March 2002

KA20481

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1.0 INTRODUCTION

AMEC Earth & Environmental Ltd. (AMEC) has carried out a geotechnical assessment associated with detailed engineering design for dyke upgrading along the Fraser River protecting Chilliwack east of Cartmell Road to Rosedale-Agassiz Bridge (Highway No. 9). The length of the section of dyke covered under the scope of this project is approximately 15 km. Dayton & Knight Engineering Ltd. is the designer for the project. They have established control stationing for the length of the project, approximately Highway No. 9 (Sta. 0+000 m) to Cartmell Road (Sta.15+216 m).

The dyke section under study lies between Brinx Road to Rosedale-Agassiz Bridge, and was most recently upgraded in 1973 under the Fraser River Flood Control Program. The reconstruction consisted of raising the dyke were necessary and installing underseepage control measures such as relief wells at slough crossings and relief pits for other sections of dyke. The design flood event adopted for the dyke design is the flood of record (the flood of 1894).

In January 2001, UMA Engineering Ltd. completed a report titled "Fraser and Harrison Rivers Hydrologic and Hydraulic Investigation" for the City of Chilliwack. As part of this investigation, UMA completed new hydrodynamic modeling for the Fraser, to update the flood design profile of the Fraser River in the Chilliwack area. The new flood design profile dictates that portions of the dyke will require raising, up to a maximum of 1 m. Figures 1.1 and 1.2 represent profiles of the dykes, showing the old and new design flood levels, and the existing and required new dyke crest elevations.

The Terms of Reference for AMEC's work in conjunction with the current assignment includes a requirement to "*conduct a geotechnical investigation and make recommendations with respect to materials to be used for dyke construction, compaction of dyke fill materials, new seepage control measures if necessary, filter layers and roadway structure where roads are present on the dyke*".

The scope of work carried out by AMEC to complete this task is as summarized below:

- All available reports and drawings pertaining to the dykes were obtained and reviewed.
- AMEC personnel carried out a detailed walk-over inspection of the dykes.
- Preliminary seepage assessments of the dykes were carried out. Based on these assessments, and on the available information, requirements for additional geotechnical investigations were determined.
- Three additional drill holes, and 30 test pits, were carried out within the dykes and the dyke foundation soils.
- Based on the existing and new geotechnical information, and on the raised design flood level, finite element seepage analyses were carried out for dyke sections considered to be the most critical in terms of seepage conditions.
- Recommendations in terms of the dyke raise design were developed.
- This report was prepared to document AMEC's work on the project.

AMEC's work scope did not include an assessment of the seismic stability of the dykes and the liquefaction susceptibility of the dyke fills and/or the foundation soils. It is understood that this issue has been addressed previously.

2.0 PREVIOUS INVESTIGATIONS AND REPORTS

The following documents and drawings were reviewed as part of this assessment:

1. Ripley, Klohn & Leonoff International Ltd. (RKL) "Chilliwack Dykes – Investigation & Remedial Treatment, Chilliwack, British Columbia" dated January 16, 1970.
2. RKL " Chilliwack Dykes – Underseepage" dated August 10, 1970.
3. Cook, Pickering & Doyle Ltd (CPD) "East Chilliwack Dykes – Report on Soil Conditions" dated August 14, 1970.
4. CPD "East Chilliwack Dykes – Amended Report on Soil Conditions" dated September 3, 1970.
5. Willis, Cunliffe, Tait, as-constructed drawings dated October 1971.
6. CPD "Camp Slough Intake Culvert & Channel – Ferry Road @ Fraser River" dated March 21, 1983.

The RKL August 10, 1970 report titled "Chilliwack Dykes – Underseepage" and the earlier RKL report dated January 16, 1970 titled "Chilliwack Dykes – Investigations and Remedial Treatment, Chilliwack, British Columbia" are key documents that set out the dyke remediation issues and geotechnical details for the dyke upgrading recommended at that time. These documents also provide detailed discussions on the nature of the seepage issue specific to dyke fill and foundation conditions for the Chilliwack dykes.

An additional relevant report concerning the East Chilliwack Dykes prepared by CPD August 14, 1970. This report presents the results of further analyses of seepage issues in the subject area, including the results of drilling several test holes in the dyke (primarily at slough locations) and field hydraulic conductivity tests, augering numerous hand test holes in the region of the dyke toe on both land and river sides, and of grain size analyses on samples recovered form the exploration. The CPD reports provide a substantial database of geotechnical information that was invaluable for the work carried out by AMEC.

Figure 2.1 shows a plan of the project area. Figures 2.2 through 2.12 show the locations of the RKL and the CPD boreholes in plan along the dyke alignment.

The as-constructed drawings prepared by Willis, Cunliffe, Tait in October 1971 show 43 cross sections reflecting the new construction, and show geotechnical details of seepage management features and where they where instituted.

3.0 DYKE DESCRIPTIONS

In general, there are three types on dykes along the project area. These types consist of dykes of low height with no seepage control measures, dykes with relief pits at the land side toe for seepage control, and dykes at slough crossings with relief wells and slough blankets for drainage control. There are seven wet and dry slough crossings along the project dyke with varying numbers of relief wells at each such crossing. At the wet slough crossings, there is water ponded against the dyke on both the land and river sides. On the land sides, an above-water slough blanket separates ponded water from the land side dyke toe.

The existing dykes range in height from approximately 1.2 to 3.7 m, except at some of the slough crossings where the dyke height is approximately 6 m. The dykes were constructed with 2.5H:1V landside slopes and 2H:1V riverside slopes, except at slough crossings where the landside and riverside slopes were constructed at 3H:1V. The existing dyke right-of-way varies over the entire length of the dykes. Various types of fences have been constructed along most of the length of the dykes, though some areas do not have fencing. The slopes of the dyke are typically covered with vegetation consisting of wild grass, blackberry bushes, and sapling trees. Selected photographs of the dykes are appended to this report.

The dykes were constructed with sands and gravels readily available in the Chilliwack area. The as-constructed drawings make reference to a "dirty sand and gravel" being placed on the riverside of the dyke. It is also mentioned on the as-constructed drawings that a "clean 3-inch minus sand and gravel" was placed on the landside slope and crest of the dyke. The presence of the clean sand and gravel was identified to extend from approximately Sta. 9+000 m to Sta. 13+500 m. The CPD reports indicate that the sand and gravel used for construction of the dykes was typically deficient in the #4 mesh to #20 mesh sizes, giving the grain size distribution curve a somewhat concave shape (i.e. the dyke fills are somewhat gap-graded). CPD (1970) noted that, to the east of about Sta. 70+00 ft (current Sta. 2+010 m), the river side slope of the dykes were covered by a 1 m thickness of sandy silt. CPD (1970) also noted "generally, even where the silt layer is not present, the soil on the land side face is slightly coarser than the river side material".

CPD (1970) carried out a detailed geotechnical investigation of the foundation soils of the dykes. This included hand auger holes drilled at 30-m spacings, 23 m upstream of the river side toe of the dykes, and at 60-m spacings, 1 m to 3 m beyond the land side toe of the dykes. The hand auger hole depths varied from 1 m to 1.5 m, depending on soil conditions. Based on these test holes, CPD (1970) noted, "the original ground surface on either side of the dyke is generally covered by a foot or more of silt or sandy silt. Table 3.1, adapted from Table 1 from CPD (1970), summarizes the length of dykes where this natural "silty blanket" is or is not present, based on the "silty blanket" satisfying either of the following criteria:

- minimum 0.3-m thickness and estimated hydraulic conductivity of 10^{-8} m/sec; or
- minimum 1-m thickness and estimated hydraulic conductivity of 10^{-7} m/sec.

Table 3.1 Estimated Lengths of Dyke With/Without Silty Blanket to River Side

Dyke Height	Length along dykes without river side silty blanket (m)	Length along dykes with river side silty blanket (m)
Up to 1.2 m	366	732
1.5 m to 2.1 m	792	5,425
2.4 m to 3.4 m	792	6,249

Based on AMEC's observations during our own site investigation, we consider that the hydraulic conductivities estimated above by CPD for the natural silty blanket are probably low by an order of magnitude. Still, as discussed in Section 6.2, the presence/absence of the natural silty blanket has a significant role in seepage conditions that would exist through and under the dykes under design flood conditions, as discussed in more detail in Section 6.2.

CPD (1970) noted that while the total depth of the silty blanket was not determined, it was generally thin as indicated at the slough crossings. Below the silty blanket, soils of higher hydraulic conductivity, including well-graded sand and gravel with minimal fines content, were encountered.

At the slough crossings, CPD (1970) found that the natural silty blanket was typically very thin to absent.

4.0 2001 & 2002 GEOTECHNICAL SITE INVESTIGATIONS

The initial stage of the investigation was a review of all existing studies (the as-built drawings, and the RKL and CPD reports), followed by a detailed walkover of the dykes. The next step was a preliminary seepage assessment (see Section 6.4.1). Once this was done, areas where AMEC judged more geotechnical information to be required were identified. AMEC then compiled a proposed program of test pit excavations and auger drill holes, which was presented to the City of Chilliwack for their approval to proceed.

The geotechnical investigation was undertaken in January 2002 and consisted of excavating twenty test pits (TP-1 to TP-20) at the crest of the dyke between Sta. 13+500 m and approximately Sta. 9+000 m to assess the extent and nature of the "clean sand and gravel" placed during the upgrade of the dyke in 1973. Another ten test pits (TP-21 to TP-30) were excavated at the toe of the dyke on both the landside and riverside to assess the presence of a silt blanket, the condition of relief pits (landside), and to also assess the foundation material of the dyke in areas where additional geotechnical information was required.

Three drill holes (DH-1 to DH-3) at Sta. 11+895 m, 4+500 m, and 4+000 m were drilled to assess the dyke embankment fill and dyke foundation conditions. At the drill hole locations, falling head tests were performed in sealed hollow stem augers at depths of 5 ft. (1.52 m), 10 ft. (3.05 m), & 15 ft. (4.57 m) to assess the permeability of the soils encountered. The location of the test pits and drill holes advanced during the second stage of the investigation are shown on the attached Figures 2.2 through 2.12 and Figures 4.1 through 4.5.

Three drill holes (AH-1 to AH-3) and three Dynamic Cone Penetration Tests (DCPT-1 to DCPT-3) were advanced at Nelson Slough (at Carey Road) during a separate dyke assignment for the City of Chilliwack in October 2001. At AH-1 and AH-2, falling head tests were performed in sealed hollow stem augers at depths of 5 ft. (1.52 m), 10 ft. (3.05 m), 15 ft. (4.57 m), and 20 ft. (6.1 m) to assess the permeability of the soils encountered. The soils data gathered during this assessment were also used during the analysis of the dyke. Recommendations with regard to the assignment at Carey Road will be presented in a separate report.

Representative samples were collected from the test pits and drill holes for visual classification and laboratory gradations. The subsurface explorations were continuously monitored by an AMEC representative who located the test hole locations, classified the soils encountered, maintained a log of each test hole and observed pertinent site features.

The borehole and test pit logs are provided in Appendices A and B respectively. Grain size distribution curves for the samples tested are provided in Appendix C.

5.0 GEOTECHNICAL CONDITIONS NOTED IN AMEC INVESTIGATIONS

The test pits and boreholes conducted by AMEC were targeted to address several questions. Those questions, and the findings of AMEC's site investigations, are as outlined below.

What is the nature of the dyke fill on the river side of the dyke crests, where it appears coarse, highly pervious materials might have been placed when the dykes were last raised?

This question is of significance because such coarse materials at the crest would represent a highly conductive conduit allowing concentrated seepage inflow to bypass the more silty material lower down on the river side slope.

The dyke crest material observed in TP-1 to TP-20 was variable in gradation, consisting of a sand & gravel with a trace to some silt to a gravely sand with a trace of silt. A "trace" means 0% to 10% by dry weight finer than 0.075 mm, while "some" means 10% to 20% by weight finer than 0.075 mm. The fill was judged to be compact in relative density. The grain size tests completed on selected samples from these 20 test pits indicated that the amount of silt present in the dyke crest material ranged from 1.0% to 14% (material by dry weight finer than 0.075 mm).

It therefore appears that at some locations along the dyke crest, raising would require minor excavation on the river side crest of the dykes to "cut off" high hydraulic conductivity (1% silt) material and tie into the more silty material lower on the river side dyke face. In other sections, the existing crest fill is sufficiently fine (10% silt content or greater), such that no such "cut off" would be required.

What is the nature of the natural thin "silt blanket" that typically underlies the dyke and extends to both the river and land sides of the dyke?

Drill holes DH-1 through DH-3, at Sta. 11+895 m, 4+500 m, and 4+000 m indicated the dyke to be underlain by a thin seam of sandy silt to a silt and sand. Below the sandy silt was a fine sand with a trace to some silt to a fine to medium grained gravely sand with a trace of silt. The silt content of samples collected from the sandy silt to the silt and sand stratum ranged from 15% to 48%. The granular soil encountered below this "silty blanket" in DH-1 to DH-3 was judged to be loose to compact in relative density.

Auger holes AH-1 through AH-3, drilled at Nelson Slough where it meets Carey Road (Sta. 8+850 m) found the dyke fill to be underlain by sand and gravel with a trace to some silt and cobbles, to a fine to medium grained sand with some silt. The gradation sieves completed on the auger hole samples at Carey Road indicate the foundation soils to have silt contents in the range of 2% to 10%.

General descriptions of the soil conditions encountered in TP-21 to TP-30 excavated at the toe of the land and river sides are shown on Figures 4.1 to 4.5. The soil conditions on the riverside and landside toes mainly consisted of silty sand and gravel (likely dyke fill) underlain by a sandy silt to a silt (the "silty blanket"). The sandy silt generally graded to a fine grained sand with a trace to some silt with depth.

The typical foundation profile, on both the land and river side of the dykes, therefore generally consists of a thin layer of sandy silt to silty sand overlying more pervious materials grading from medium to coarse sands to gravely sand with trace to no silt. This is consistent with the previous investigations of CPD and RKL.

What is the nature of the dyke fill on the river side face, where the Willis Cunliffe Tait as-constructed drawings indicate a facing of "dirty 3-inch minus sand and gravel"?

In general, a sand and gravel with trace to some silt was observed in the test pits on the riverside face of the dyke. The gradation results of soil samples tested indicated a silt content ranging from 10% to 15% was present in the sand and gravel. With this amount of silt, the river side facing would serve to reduce seepage inflows into the dyke and, in concert with the grading to more coarse and pervious soils (i.e. lower silt content) towards the land side of the dyke fill, would help maintain lower phreatic levels through the dyke during design flood conditions.

What is the nature of the "granular filter material" used as relief pits backfill?

The relief pits were exposed at Sta. 7+950 m (TP-24) and Sta. 11+000 m (TP-22). The relief pits were constructed with a filter gravel consisting of 25 mm minus sand and gravel with less than 2% silt content, indicating a highly free-draining material. The relief pits were capped with drain rock that has become covered with some silt and organics. The two relief pit sections observed showed no indication of the lower filter gravel silting up due to water seepage.

What results were obtained from insitu hydraulic conductivity test?

The results of the falling head test conducted in DH-1 to DH-3 and AH-1 & AH-2 indicated that the hydraulic conductivity of the dyke fills is relatively consistent in a narrow range, while that of the dyke foundation soils varies considerably due to the more varied soil conditions. Table 5.1 summarizes the hydraulic conductivity values estimated based on the falling head tests conducted by AMEC.

Table 5.1 Summary of Falling Head Test Results

Dyke Embankment	**Hydraulic Conductivity "K" (m/sec)**
Sand to Sand & Gravel	3.2×10^{-4} to 2.0×10^{-5}
Dyke Foundation	**Permeability "K" (m/sec)**
Sand & Silt to Silty Sand	8.2×10^{-7} to 1.54×10^{-7}
Gravely Sand to Sand & Gravel	2.6×10^{-3} to 1.34×10^{-6}

These estimated hydraulic conductivities are generally consistent with the estimated provided in Table 2 of CPD (1970).

6.0 SEEPAGE ANALYSIS OF DYKES

6.1 GENERAL

This section describes the seepage analyses carried out to assess the effects of the raised dyke height and design flood level on seepage conditions in the dykes and the foundation soils. The objectives of the analyses were:

- to check the degree to which the existing seepage control measures would affect the seepage conditions under the old design flood level conditions;
- to estimate the increased stress on the seepage control measures under the raised dyke and new design flood level condition; and
- to determine what upgrading of the seepage control systems is required for the raised dykes and the new design flood level.

Two approaches were taken for the seepage analyses:

1. A simplified, semi-qualitative assessment was made to assess the added stress to which the seepage control measures would be subjected due to the raised flood level. This change could be calculated based on the increase in the flood level as a ratio to height of the dyke. This approach allowed identification of the critical (worst case) dyke sections in terms of added stress on the seepage control measures.
2. For the dyke sections judged the most critical, finite element seepage analyses were conducted for the old and the new design flood levels. These analyses yielded estimates of seepage velocities and hydraulic gradients. Of particular interest in terms of the safety of the dykes under design flood conditions are exit gradients (where seepage daylights at ground surface) at the downstream face and toe of the dykes, and in the foundation soils to some distance downstream of the dykes. The significance of the hydraulic exit gradient is described later in this section.

The seepage analyses were carried out for the two categories of seepage protection measures in place along the dykes:

- relief pits at the downstream dyke toe (along most of the dykes alignment); and
- relief wells and downstream drainage blankets where the dyke alignment crosses sloughs.

6.2 SIGNIFICANCE OF THE NATURAL "SILTY BLANKET"

As described in Section 3, the generalized foundation profile to both the land and river sides of the dykes, for about 86% of the dyke alignment (see Table 3.1), consists of a thin blanket of silty soils overlying a thick sequence of more hydraulically conductive

sand and gravel soils. Along the remaining 14% of the dyke alignment, CPD's hand auger program indicated the silty blanket to be absent.

Where the silty blanket is present on the river side of the dykes, it would have the effect of reducing seepage into the underlying more pervious foundation soils to a lower rate than would be the case were the blanket not present. This is of benefit in terms of the stability of the dykes under design flood seepage conditions. Where the dyke fill is more hydraulically conductive than the silty blanket, then seepage would be more concentrated through the dyke fill, that being the path of least resistance to flow. Where more silty material is present on the upstream face of the dyke (as appears generally to the be the case), seepage through the dyke would be reduced. With more coarse dyke fill toward the land side slope, the result would be a low phreatic surface through the dyke, and low hydraulic exit gradients where seepage discharges from the land side toe of the dyke, which is favorable.

The presence of the silty blanket on the land side of the dykes, where it overlies much more hydraulically conductive sand and gravels, is less favorable. This situation, without appropriate seepage control measures, could result in upward hydraulic gradients through the silty blanket sufficiently high as to induce piping (see discussion in Section 6.3 below).

The least favorable condition is one where the silty blanket is absent on the river side of the dyke, but is present on the land side. In this circumstance, seepage into the highly conductive foundation soils to the river side of the dyke would occur at a high rate. The presence of a low hydraulic conductivity silty blanket to the land side would result in high seepage pressures and upward hydraulic gradients, that would increase the likelihood of piping and potential seepage-induced failure of the dykes. It would also result in a raised phreatic level through the dyke fill, and seepage discharge from higher elevations on the land side face of the dyke.

Where the silty blanket is absent on both the river and land sides of the dyke, with hydraulically-conductive sand and gravel soils directly exposed, seepage through the foundation soils will be increased and, depending on the height and width of the dyke, the potential remains for unsafe hydraulic exit gradients, and seepage-induced sloughing, at the land side toe of the dykes.

6.3 EXIT HYDRAULIC GRADIENTS AND SAFETY AGAINST INTERNAL EROSION

As groundwater flows through soil, it exerts a seepage force. This seepage force is directly proportional to the hydraulic gradient. The hydraulic gradient is the ratio of hydraulic head loss to the flow path length over which that loss occurs. If the hydraulic gradient where seepage flow daylights (for example, from foundation soils immediately downstream of a flood protection dyke) exceeds a certain critical value, then a boiling condition, sometimes referred to as a quick condition (e.g. quicksand) will develop. Once this happens, seepage discharging from a soil surface will begin to erode the soil, a process referred to as internal erosion. Internal erosion, which begins at the point of seepage exit (e.g. land side of a flood control dyke, where the hydraulic head is low), can rapidly retrogress towards where the hydraulic head is high (e.g. the water impounded upstream of the dyke), a process referred to as piping. As this occurs, and

the flowpath length reduces, hydraulic gradients increase, and therefore the rate of piping also increases. Once the pipe reaches the impounded reservoir, flow and erosion become so severe that the dyke becomes breached, and general failure ensues. With the exception of over-topping, the piping process is the most frequent failure mode for earthfill dams and flood protection dykes.

The critical exit gradient at which the piping process can be initiated by upward flow (e.g. through a low permeability silt layer overlying a confined aquifer of permeable sand and gravel) can be estimated as follows:

$$i_c = (G-1)/(1+e)$$

where:

i_c	=	the critical exit gradient
G	=	specific gravity of the solids of the soil (typically 2.65)
e	=	void ratio of the soil (ratio of volume of void spaces to the volume occupied by the soil grains)

The critical exit gradient for soils ranges from 0.8 to 1.2, with 1 usually considered the typical value. Good design practice applies a factor of safety to the critical gradient. For example, for a factor of safety of 2 against piping by upward groundwater flow, design would allow for an exit gradient no higher than 0.5. A factor of safety of 3 or 4 is often considered appropriate, meaning that the maximum allowable upward gradient would be between 0.25 and 0.33.

Hydraulic gradients where seepage discharges laterally from the downstream face of a water retaining embankment, if too high, can lead to a retrogressive erosive failure of the embankment fill. The seepage force, due to frictional drag exerted by groundwater movement through the soil, applied per unit volume of soil is computed as the product of the hydraulic gradient and the unit weight of water. Where this seepage force is sufficient to mobilize soil particles, the danger of a retrogressive internal erosion failure is present. The seepage force required to initiate such erosion on the downstream face of an embankment depends on the slope of the embankment (the flatter the slope, the better) and the nature of the soil making up the slope. For example, a uniform, cohesionless silt would be much more vulnerable to seepage erosion than would a well graded gravel and sand, as silt particles could be eroded by seepage velocities lower than those required to mobilize sand and gravel sizes. A further benefit of a gravel and sand on the downstream face of a water retaining embankment is that, while sand particles might be initially eroded by seepage discharge from the embankment face, the remaining gravel sizes would tend to form a filter to prevent additional loss of sand sizes. With a uniformly graded material, no such potential self-healing filter could form.

6.4 EXISTING SEEPAGE CONTROL MEASURES

6.4.1 Relief Pits

Relief pits are in place along most of the alignment of the dykes, as shown on the as-constructed drawings prepared by Willis Cunliffe Tait (1975). Where present, the

as-built construction drawings show the typical section for relief pits are 15 ft in length, with 25-ft gaps between the pits, meaning the relief pits are spaced at 40 ft center to center. The relief pits consist of trenches excavated at the downstream toe of the dykes, backfilled with a clean, free-draining sand and gravel (filter material), and capped by a 1 ft thickness of drain rock (no sand sizes). The relief pits were constructed where the effective head (defined as the elevation difference between the old design flood level and the land side ground elevation) was greater than 4 ft. The depths of the relief pits are, for any section along the dykes, equal to the effective head, or 1 ft into the permeable soil layer, whichever is less, with a minimum depth of 4 ft.

For those relief pit sections where the effective head exceeds 8 ft, a berm was constructed at the downstream dyke toe, with the relief pit being located at the downstream toe of the berm rather than the downstream toe of the dyke. The berms were constructed of clean, 3-inch minus gravely sand. The intent of the berms was to provide a filter against the downstream toe of the dyke in the event of seepage breakout on the downstream dyke face. These berms are typically 9 ft in width (measured in the upstream-downstream direction) and 2 ft in height.

The objective of the relief pits is to provide groundwater pressure relief near the downstream toe of the dykes. Such pressure relief is required because, along much of the dykes alignment, subsoil conditions consist of a layer of relatively low permeability silt overlying a more pervious, and thick, zone of sand and gravel. The more hydraulically conductive sand and gravel would transmit high groundwater pressures to below and downstream of the dykes, resulting in high uplift pressures against the less conductive silt layer. Such a condition would lead to high hydraulic gradients upwards through the silt layer. Sufficiently high gradients could induce internal erosion (described above in Section 6.3) that could in turn lead to failure of the dykes. Groundwater flow follows the path of least resistance. The relief pits, by penetrating the silt layer and connecting with the underlying more conductive sand and gravel soils, would provide the path of least resistance and thereby reduce seepage gradients through the silt layer.

The following comments pertaining to the existing relief pits were observed during an assessment of their conditions, 28 February 2002 to 6 March 2002.

- Relief pits are not constructed as shown on the as-constructed drawings;
- The relief pits length varies from approximately 15 ft to 21 ft, and the gaps spacing also varies from approximately 18 ft to 25ft;
- The depth of the drain rock (capping material) varied from 1 ft to 2 ft thick;
- Some of the relief pit sections have had farm equipment access roads constructed over them since being constructed during the dyke upgrade project in 1975.

6.4.2 Relief Wells and Slough Blankets

The height of the dykes, and therefore the effective head as defined above, is greatest at slough crossings. As such, the slough crossings are the most critical dyke sections in

terms of seepage control requirements. At these sections, seepage control measures on the land side of the dykes consist of relief wells together with a slough blanket comprised of a 2-ft thickness of clean 3-inch minus gravely sand. The relief well and slough blanket details are shown on the Willis Cunliffe Tait as-constructed drawings.

The relief wells extend to typical depths of 50 ft below the land side ground elevation, and consist of slotted 4-inch diameter plastic pipe in a pea gravel-backfilled borehole of approximately 12 inches in diameter. The wells are spaced at 50 ft center to center. Well flow discharges into pea gravel and drain rock backfill at the well head, which is protected by 24-inch diameter concrete casing with a concrete lid. The well heads terminate above the typical water levels in the sloughs. These wells are maintained by the City of Chilliwack on a regular basis.

The function of the relief wells is the same as that of the relief pits as described above: to provide a highly conductive path for groundwater flow and so relief seepage pressures in the subsoils on the land side of the dykes. Such pressure relief is of particular importance where land side subsoil conditions consist of relatively low hydraulic conductivity material (silt) overlies highly conductive material (clean sand and gravel). The gravelly sand blanket (termed "slough blanket" on the as-constructed drawings) allows the well head to be constructed and accessed above the slough water level, and also provides filter protection for the foundation soils downstream of the dyke. At wet slough crossings, the blanket extends to 100 ft downstream of the dyke toe, except at Nelson Slough at Carey Road where a portion of the blanket, and the foundation sand and gravel to some depth, has been removed in a quarrying operation. At dry slough crossings, the blanket extends from the dyke toe to the location of the relief wells.

6.4.3 Areas With No Seepage Control Measures

Where the effective head was less than 4 ft, no seepage control measures were constructed.

6.5 ANALYSIS METHODOLOGY

6.5.1 Simplified Assessment

A simplified analytical approach was the first step taken in assessment of the effect of a raised design flood level on the dykes. This assessment consisted of the following steps:

1. All existing data pertaining to the design, construction, and performance of the dykes were reviewed. The data available are as listed in Section 2.0. The geotechnical site investigation data provided by these reports were used in the seepage analyses, and to identify any gaps in geotechnical information requiring additional field work by AMEC.

2. A detailed walk-over of the dykes was carried out. Selected photographs taken during this walk-over are appended to this report. The walk-over was carried out to assess the condition of the dykes, the seepage control measures, and to identify any areas requiring additional geotechnical investigation work.

3. Simplified, semi-qualitative seepage analyses for those portions of the dykes with relief pits, and for the slough crossings (with relief wells and drainage blankets). These analyses were carried out to identify the most critical (worst case) sections that were in turn analyzed in greater detail using finite element seepage modeling.

6.5.2 Finite Element Seepage Modeling

Finite element seepage modeling was carried out for the two dyke sections considered the most critical in terms of the raised design flood level:

- Dyke section with relief pits for seepage control – Sta. 11+900 m
- Slough crossing section with relief wells and slough blanket for seepage control – Sta. 11+940 m (Nelson Slough).

These sections were considered to be the most critical, and therefore the most appropriate for seepage analysis, primarily because:

- the existing dyke heights at these sections are higher than other sections along the dykes; and
- the increase in the design flood elevation is highest at these sections.

Therefore, these sections were selected for analysis because they represent the most critical sections in terms of geometry. However, they do not necessarily represent the critical sections in terms of the hydraulic conductivity values of the foundation soils and the dyke fills. As such, analyses for these sections were carried out for a range of combinations of dyke and subsoil hydraulic conductivity values. If the analyses indicated that the existing seepage control measures for the two dyke sections selected were adequate for the worst case geometry conditions (i.e. combination of dyke height and increased design flood level), then they would be so for other dyke sections less critical in terms of geometry.

6.6 ANALYTICAL RESULTS – RELIEF PIT SECTIONS

6.6.1 Simplified Assessment

For the relief pit sections, the key question in terms of the raised flood level is whether or not the relief pits are sufficiently conductive to water flow to accommodate the increased groundwater flows that will occur as a result of the raised design flood elevation. To assess this issue, the flow capacity of the relief pit was first estimated. Based on an assumed hydraulic conductivity of 0.1 m/sec (appropriate for the well graded, clean sand and gravel in the relief pit trenches), the width of the relief pits (0.6 m measured in the upstream-downstream direction), and an assumed hydraulic gradient of 0.5, the flow capacity of the relief pits was estimated at 0.03 m^3/sec (30 litres/sec) per meter length of dyke. A hydraulic gradient of 0.5 was considered conservative (low).

With an estimate of the flow capacity of the relief pits, the next step was to estimate, for both the old and new design flood levels, the maximum amount of flow to which the relief pits could be subjected. A flow net analysis would be required to determine the actual flow through the relief pits. For simplicity, however, it was assumed that all flowlines from the phreatic surface through the dyke to a point below the natural ground surface equivalent to the dyke height would report to the relief pit. In other words, for a dyke height of 5 m, all flow between the dyke crest to a point 10 m below the dyke crest was assumed to report to the relief pit. This is an overly conservative assumption, as borne out the finite element analyses described in subsequent sections. To estimate the seepage flow, Darcy's Law was used:

$$Q = K \times i \times A$$

where:

Q	=	seepage flow (m^3/sec per meter length of dyke)
K	=	average hydraulic conductivity of dyke fill and foundation soils
I	=	hydraulic gradient = elevation difference between design flood level and ground elevation on land side dyke toe
A	=	area of flow, estimated as twice the height of the dyke

The average hydraulic conductivity of the dyke fill and foundation soils was assumed at 0.005 m/sec. This value is higher than the average values noted for the dyke fill and foundation soils noted by CPD (1970). As such, in terms of seepage flow reporting to the relief pits, it would yield a high (conservative) estimate.

The ratio of the relief pit flow capacity to Q, termed the Capacity Ratio (CR) was then determined, for both the old (CR_{old}) and new (CR_{new})design flood levels.

Table 6.1 summarizes the simplified calculations carried out for a number of relief pit dyke sections.

Table 6.1 Simplified Analysis Results – Relief Pit Sections

Station	Dyke Section	Dyke Height (m)	CRold	delta h (m)	CRnew/CRold	Comments
13+470	SECTION 1 - STA#442+00:RKL 443+25	3.5	7.2	0.5	0.83	May consider this section for analysis. Analysis of Sta. 390 to be completed first, however.
13+137	SECTION 2 - STA#431+20:RKL 430+00 (SEC 1009)	2.8	7.1	0.6	0.79	Analysis not required for this section due to low dyke height.
11+890	SECTION 3- STA#390+00:CPD 394+00	5.9	2.6	0.8	0.87	This section may represent the worst case combination of dyke height, increase in design flood level, and low CRnew/CRold. To be assessed using SEEP/W.
11+280	SECTION 4- STA#370+00:RKL 370+00 (SEC 1008)	2.2	9.9	0.7	0.68	Analysis of this section is not required to determine if remedial measures needed. CRnew is still > 10 (since dyke height is minimal), so likely not a problem. If relief pits suffice for greater dyke heights, they will for this section, where new dyke height only about 2.5 m.
9+875	SECTION 5- STA#324+00:RKL 320+00 (SEC 1007)	2.3	8.7	0.3	0.88	Analyses not required for these sections. Dyke heights are relatively low, and there is a relatively minor increase in design flood level.
8+290	SECTION 6- STA#272+00:RKL 270+00 (SEC 1006)	3.6	3.7	0.4	0.90	
7+800	SECTION 7- STA#256+00:RKL 260+00 (SEC 1020)	2.5	6.8	0.3	0.84	
6+830	SECTION 8- STA#224+00:NEAR CPD 228+50	4.2	4.9	0.5	0.87	These sections are less critical than 11+890 m.
6+550	SECTION 9- STA#215+00:RKL 216+00 (SEC 1005)	2.4	8.9	0.4	0.84	
3+170	SECTION 10- STA#104+00:RKL 106+30 (SEC DH 3)	5.1	2.3	0.1	0.97	Less than 10% reduction in capacity ratio - no remedial measures needed. CRold for Station 26 section is very high because low hydraulic conductivity values are assigned for the dyke fill and the foundation soils. Increasing hydraulic conductivity of both for this section by a factor of 10 reduces CRold to about 25.
3+050	SECTION 11- STA#104+00:RKL 100+00 (SEC 1017)	5.0	2.2	0.2	0.96	
1+100	SECTION 12- STA#36+00:NEAR RKL 40+00 (SEC 1016 & 1012)	4.8	3.4	0.4	0.91	
0+792	SECTION 13- STA#26+00:NEAR RKL 28+40 (SEC DH1)	6.1	2.0	0.3	0.95	

The stations selected for this simplified analysis correspond to sections where there is existing geotechnical information (subsoil stratigraphy and estimated hydraulic conductivities) from the previous RKL and CPD investigations. The following are noted for the results as summarized in the table:

1. In all cases, with $CR_{old} > 1$, the implication is that the flow capacity of the relief pit exceeds the flow rate to which the relief pits would be subjected (based on the simplified methodology described above). The CR value represents, in a semi-quantitative sense, a "factor of safety" in terms of flow capacity available in the relief pit versus flow capacity required.

2. The column "delta h" represents the increase in the design flood elevation for each given dyke section. Note that the maximum increase in design flood elevation (0.8 m) occurs at Sta. 11+980 m.

3. The ratio of CR_{new} to CR_{old} represents the degree to which the "factor of safety" will be reduced under the increased design flood elevation. For example, the ratio for Sta. 11+890 m is 0.87. This means that, under the new design flood elevation, the "factor of safety" would be reduced by 13%. It is this change in the CR value that is of significance, as it indicates the degree to which the "stress" on the seepage control measures (i.e. the relief pits) will increase under the raised design flood levels. The change in the CR value is determined solely by geometrical considerations: the change in design flood level, and the height of

the dykes. It is therefore independent of the various simplifying assumptions made to estimate the new and old CR values.

4. As noted in the table, the ratio of CR_{new} to CR_{old} ranges between 0.68 to 0.97. The low value, for Sta. results from a large increase in design flood level (0.8 m) being divided by a low dyke height (only 2.6 m). The higher values occur for those sections where the increase in design flood level is minimal.

On the basis of the results of this simplified approach, the dyke section at Sta. 11+900 m was selected for more detailed finite element seepage modeling. This section was considered to represent the worst case scenario in terms of the combination of dyke height and increase in design flood level.

6.6.2 Finite Element Modeling

The results of the SEEP/W analyses for the section at Sta. 11+900 m are provided in Appendix D. Analyses were carried out for both the old and new dyke configurations and flood levels. Three types of plots are provided in Appendix D:

- finite element mesh;
- plots showing contours of total hydraulic head; and
- plots showing seepage velocity vectors (larger vectors indicate areas of higher seepage velocities).

A finite element seepage analysis requires the definition of boundary conditions around the perimeter of the mesh. Boundary conditions for the seepage model were as follows:

- Bottom boundary of mesh – no flow
- Right and left vertical boundaries – no flow
- River-side top boundary from right hand side to flood level on against the upstream face of the dyke – constant head boundary
- Land-side top boundary – review boundary.

At the review boundary on the land side, an iterative process was used by SEEP/W to determine seepage conditions.

The relief pit consisted of two elements at the land side toe of the dyke. It was modeled with a width of 0.6 m in the upstream-downstream direction, and a depth of 1.2 m (4 ft, the minimum design depth for the relief pits). The assumed depth of 1.2 m was slightly conservative, as based on the effective head (defined earlier in the description of the relief pits) the actual depth at this section would be about 2 m).

For the analyses, the presence of the more silty sand and gravel blanket on the upstream face of the dykes was neglected, introducing an additional degree of conservatism to the analyses.

The results of the analyses are summarized in Table 6.2. The following are indicated by the analyses:

1. The relief pits were installed to relieve foundation seepage pressures where a relatively low permeability layer of silt overlies more pervious sand and gravel. The analyses predict that the relief pits would achieve this objective. The analyses indicate that the difference in seepage conditions that would exist for the old versus new design flood levels is sufficiently small that the existing relief pits will afford satisfactory protection against the condition they are intended to address. For cases where there is no silt blanket in the foundation, and the dyke is in direct contact with more pervious soils, the relief pit has no significant influence on seepage conditions at the land side dyke toe.

2. In all cases, seepage breakout on the downstream face of the dyke fill is predicted. Seepage breakout would occur higher up the face where the dyke fill is more pervious than the foundation soils. Seepage breakout would occur lower on the face for less pervious dyke fills. Seepage breakout on the dyke face is consistent with the lack of a toe drain and/or a drainage blanket in the dykes design. By neglecting the more silty sand and gravel on the upstream face of the dyke and the coarser materials to the land side of the dyke, the analyses are somewhat conservative, but only slightly so, in their predictions of seepage conditions on the downstream face of the dyke. Defense against seepage-induced sloughing and erosion at the land side toes of the dykes is therefore provided by:
   - the coarse nature of the well-graded, clean 3-inch minus sand and gravel placed over the downstream dyke slopes, which would offer more resistance to erosion than would a uniform sand;
   - the 2.5H:1V slope of the land side of the dyke (the flatter the slope, the less likely that seepage-induced erosion would occur); and
   - The vegetative cover over the downstream dyke slope. A vegetative cover of grasses over the downstream slope as currently exists is beneficial and should be maintained. Larger shrubs, the roots of which could effectively shorten the seepage path length through the dyke, should be avoided.

Table 6.2 Summary of SEEP/W Analyses: Sta. 11+900 m

Scenario	Dyke Configuration and Flood Level	K_{dyke} (m/sec)	$K_{foundation}$ (m/sec)	K_{relief_pit} (m/sec)	Analysis Results and Implications
1	Old	0.001	0.001	0.01	No tendency for concentration of flow into relief pit, due to low contrast in hydraulic conductivity (K) between relief pit and dyke/foundation soils. Exit gradients at dyke toe about 0.25 (acceptable).
	Old – but without relief pit				These analyses essentially identical to analyses that accounted for the relief pit, again showing that, for high hydraulic conductivity value in foundation soils, yielding low K contrast with relief pit fill, relief pit serves no significant function.
	New – dyke raised by adding fill to crest, but not adding fill to river side slope, therefore narrowing crest.				High exit gradients, and high seepage velocities, predicted on lower 1/5 of the dyke face on the land side. Note the velocity vectors extending sub-horizontally out of the lower dyke face High seepage velocities for this case considered unacceptable in terms of the safety of the dyke, raising potential for internal erosion on the lower slope.
	New – dyke crest width maintained by adding fill to river side as well as to crest				Reduced exit gradients at toe (about 0.25), and seepage breakout point lower on dyke face relative to case above where dyke crest width narrowed by raising.
2		0.001	0.0001	0.01	Seepage concentrated mainly through the dyke fill given that it is 10X to 100X more conductive than foundation soils. Seepage breakout at lower 1/4 of land side dyke slope for Scenario 2, and lower 1/3 of slope for Scenario 3. Exit gradients about 0.3 at toe. Under these conditions, relief pit does not influence seepage regime. For both scenarios, sloughing and/or erosion due to seepage discharge could occur at toe. Coarse sand/gravel on downstream slope would resist such conditions. Note that these analyses neglect more silty sand and gravel on upstream face of dyke, so somewhat conservative. Still, where very pervious dyke fill underlain by much lower permeability foundation soils, continuous monitoring during flood events, and ability to quickly buttress downstream slope with gravel/sand fill from stockpiles, will be essential.
3		0.01	0.00001	0.01	
4		0.0001	0.001	0.01	Foundation soils 10X more conductive than dyke fill. Seepage flow is now concentrated through foundation soils. Relief pit now has some influence on seepage conditions at the land side dyke toe. Exit gradients through dyke are low, and seepage breakout is very low on land side dyke toe.
5a		0.001	0.01 (lower) and 0.0001 (upper)	0.01	Assume upper silt layer on land side and below dyke, but absent to upstream of dyke, underlain by highly pervious sand and gravel. This condition is conducive to high upward gradients through the silt layer that could initiate piping, and is the condition for which the relief pits are intended. For Scenario 5a, upward gradients of near 1 are predicted in the relief pit and the silt near the relief pit, indicating that piping could develop. For Scenario 5b, with relief pit being 10X more conductive than for 5a, exit gradients are reduced by relief pit to acceptable levels. This analysis demonstrates importance of maintaining free discharge of relief pits (i.e. they should not be covered by low permeability materials). Analysis is somewhat conservative in that it assumes that the silt blanket terminates only a short distance upstream of the dyke, and neglects more silty sand and gravel on upstream dyke face.
5b				0.1	

3. For the section modeled (Sta. 11+900 m), raising of the dyke by adding fill only to the crest, thereby narrowing it, would result in a potentially unacceptable seepage condition at the downstream dyke toe. As such, it is recommended that, wherever the existing dyke height (measured as elevation difference between existing crest and ground elevation at the land side toe) exceeds 4 m, and the design flood elevation is 0.4 m or more higher than the old design flood elevation, the current width of the dyke crest be maintained by adding fill to the river side slope as well as to the dyke crest. For other relief pit sections, where dyke raising would require only addition of fill to the crest (i.e. where the dyke crest can be narrowed), narrowing of the crest is acceptable.

4. The different results yielded by Scenarios 5a and 5b clearly emphasize the importance of maintaining free-draining conditions for the relief pits. During the inspections of the dykes, in some areas it appeared that farmers had graded low permeability soils overtop of the relief pits, effectively plugging them. This condition must be rectified, and measures enacted to prevent future such occurrences. A fence to the land side of the relief pits would be beneficial in this regard.

5. Given the hydraulic exit gradients that could exist where seepage discharges from the land side face of the dyke, it is important that the land side dyke slope be no steeper than the design slope of 2.5H:1V. The upgrading of the dykes should therefore include slope flattening of the land side dyke slope wherever this design criterion is not satisfied.

6.7 ANALYTICAL RESULTS – SLOUGH SECTIONS WITH RELIEF WELLS

6.7.1 Simplified Assessment

For the slough crossing sections, the change in seepage conditions was first assessed based on the increase in the effective head elevation, defined as the elevation difference between the design flood level and ground elevation on the land side of the dyke. The ratio of the new to old effective head (H) values represents the increase in seepage flow that would result from the raised flood elevation. This ratio is provided for each slough crossing in Table 6.3 below.

Table 6.3 Comparative Analysis – Slough Crossing Sections

SLOUGH	STATION (m)	EFFECTIVE HEAD - H (m)			Number of Relief Wells
		H_{old}	H_{new}	H_{new}/H_{old}	
Old Camp	1+750	1.83	2.29	1.25	8
Un-named Slough	1+505	0.76	1.18	1.55	1
Old Dry Slough	6+950	1.8	2.1	1.17	3
Nelson Slough @ Carey Rd	8+932	4.24	4.45	1.05	8
Orchard Slough	10+185	3.73	4.075	1.09	3
Nelson Slough	12+000	2.96	3.75	1.27	7
McDonald Rd/Bell Slough	13+600	3.64	4.1	1.13	2

Relief wells at each slough crossing are 15.2 m in depth, and are spaced at 15.2 m intervals. Therefore, the number of relief wells at each slough crossing is indicative of the length of dyke at each crossing.

The highest ratio (1.55) is indicated for the slough at Sta. 1+505 m, where a 55% increase in seepage flow is predicted. However, this does not represent the critical slough section as the effective head under the new design flood level is only 1.18 m. The most critical section is Nelson Slough, where a 27% increase in seepage flow is predicted, with a new effective head of about 4 m. Accordingly, finite element analyses were conducted for this section, as presented below. This slough section was also selected for analysis as there is information on the dyke fills and foundation soils from the previous CPD and RKL investigations.

6.7.2 Finite Element Modeling

The finite element modeling results for Nelson Slough are presented in Appendix E. Due to the two-dimensional nature of the seepage model, the relief wells were modeled as a continuous zone. To account for this simplification, the relief well zone in two-dimensional space was assigned a hydraulic conductivity of 0.01 m/sec, much lower than the actual capacity of the individual wells. So long as the relief well zone was modeled with hydraulic conductivity at least one order of magnitude higher than that of the foundation soils, the effect of the relief wells, in semi-quantitative terms, could be assessed.

Given the amount of information available for this section, the presence of the silt blanket covering the foundation to the river side of the dyke, and the silty sand and gravel on the river side of the dyke, could be incorporated into the model.

The results of the seepage analyses for the Nelson Slough section are summarized in Table 6.4.

The seepage modeling carried out for Nelson Slough assumed the water level on the land side of the dyke to rise no higher than the elevation of the slough blanket. This is a conservative assumption, because in fact the water level in the slough would likely rise above this level in response to seepage under design flood conditions. A higher water level on the land side of the dyke would reduce the overall hydraulic gradients from upstream to downstream.

The scenarios modeled and summarized in Table 6.4 do not include the case where the silty blanket is continuous below the dyke and to the land side of the dyke. This scenario was not assessed because:

- The relief pit seepage modeling (Appendix D), for the case with the silty blanket to the land side of the dyke, clearly indicates the relief pit to be effective in reduction of hydraulic exit gradients through the thin silty layer. With the relief wells being much deeper (in absolute terms and relative to the dyke heights) than the relief pit, it is clear, without need for additional analysis, that the wells would be even more effective than the relief pits in this regard.

- As noted by CPD (1970), in the slough crossing sections the natural silty blanket generally appears absent on the land side of the crossings.

The following are indicated by the analyses as summarized in Table 6.4:

1. The existing relief wells and slough blankets are considered adequate seepage control measures under the raised design flood level. No upgrades of these installations are considered necessary, although continued maintenance of the relief wells is required to maintain them in proper working order.

2. Depending on dyke fill and foundation seepage conditions, there may be instances of hydraulic exit gradients somewhat higher than desirable where seepage discharges from the land side of the dykes at the slough crossing sections. However, the coarse nature of the soils on the land side face of the dykes, together with the relatively flat (3H:1V) land side dyke slope, are considered an adequate defence against this gradients. During the dyke upgrading, the 3H:1V land side slope should be re-established if existing slopes are steeper.

3. The effect of the natural silty blanket on the river side of the dyke appears to be relatively minor (if about 1 m in thickness as per the model), if its hydraulic conductivity is only ½ an order of magnitude lower than that of the underlying sand and gravel soils. However, the presence of more silty material, even if only in a thin blanket, on the river side face of the dykes makes a significant and very beneficial difference in terms of the position of the phreatic surface through the dyke section. On that basis, the more silty zone on the river side dyke face should be raised to the new design flood level to maintain its continuity. This same recommendation holds true for those sections of the dykes where relief pits are in place for seepage control.

4. The dyke raising at the slough crossings can be carried out by adding fill only to the crest where the design grade can be achieved and the dyke crest narrowed. Fill need only be added to the dyke slopes if those slopes are steeper than the design grades.

Table 6.4 Nelson Slough: Summary of Seepage Analyses

Scen-ario	Hydraulic Conductivity (m/sec)				Analysis Results
	Dyke Fill	Upstream Dyke Face	Found.	Relief Well "Zone"	
1	0.001	0.0004	0.001	0.01	With a relatively pervious foundation, and a hydraulic conductivity contrast of only one order of magnitude between the foundation soils and the relief well zone, the relief well has no effect on the seepage regime. Although the silty sand and gravel zone on the upstream dyke face is thin and is only ½ an order of magnitude less than the dyke fill, it has some effectiveness in terms of maintaining a low phreatic surface through the dyke fill.
2			0.0001		With a greater contrast in hydraulic conductivity (100X) between the foundation soils and the relief well zone, the relief well now has a marked effect on the seepage regime, reducing upward hydraulic gradients near the well. With a reduced hydraulic conductivity in the foundation soils, more seepage is concentrated through the dyke, resulting in a high phreatic level (relative to Scenario 1) and higher exit gradients at the land side dyke toe (about 0.4). The silty zone on the upstream dyke face continues to be of benefit.
3	0.01		0.00001	0.01	The relief well zone again has a marked effect on the seepage regime owing to its hydraulic conductivity contrast (1000X) with the foundation soils. However, most of the seepage flow is concentrated through the highly pervious dyke fill. The closely spaced total head contours on the upstream dyke face demonstrate the effectiveness of the silty zone in reducing seepage pressures. The effectiveness of the blanket increases with its thickness (greater at the crest than lower down the upstream slope).
4	0.0001		0.001	0.01	Low phreatic surface through the dyke, with seepage breakout only just above foundation level on the land side. Seepage is concentrated through the more pervious foundation soils. Exit gradients are very low on land side.
5a	0.001	0.001	0.0001	0.01	Seepage breakout on lower 1/3 of land side dyke face, with exit gradients of about 0.4. Note that by neglecting the silty blanket on the river side face of the dyke(compare Scenario 5a with Scenario 2), the result is a raised phreatic surface through the dyke.
5b			0.001		Lower phreatic surface through the dyke because, with foundation soils assumed 10X more pervious than for 5a, seepage is less concentrated through the dyke. Note for this scenario that the relief well zone has no significant influence on the seepage regime. In comparison with Scenario 1, the absence of the natural silty blanket to the river side modeled by Scenario 5 b makes little difference to foundation seepage conditions. This occurs because the blanket as modeled is thin (about 1 m thick), and only ½ an order of magnitude lower (for Scenario 1) than the underlying sand and gravel soils.

7.0 INTAKE CULVERT AND CHANNEL STRUCTURE AT CAMP SLOUGH

At about Sta. 1+700 m (see Figure 2.3), where the dyke crosses Camp Slough, and intake culvert and wing wall structures were constructed through the dyke in 1983-1984, the purpose of which was to augment flows into Camp Slough during dry weather periods. An approach channel between the river and the slough was also constructed. Five 0.76-m diameter steel culverts, 18.3 m in length, were extended through the dyke, with gates at the upstream end. A reinforced concrete headwall, apron, and wingwalls are present at both the river side and land side ends.

Construction specifications (CPD, 1983) called for good compaction of the silty sand backfill placed around the culverts, and called for a three types of soil backfill progressing from the river side to the land side, as follows:

- river side – silty sand or sandy silt
- below dyke crest – medium sand
- 1.5-m length downstream of medium sand zone – coarse sand
- further downstream – 0.25-inch to 0.4-inch rounded gravel, wrapped in a filter fabric.

The backfill design therefore provided a sequence of filter-compatible materials, with the hydraulic conductivity of those materials increasing towards the land side.

Field inspection reports presented in CPD (1983) indicate that an acceptable level of compaction was achieved for those field density tests that were taken. CPD concluded that, on the basis of their inspections, the backfill around the structure was completed in general accordance with their design specifications.

CPD (1983) also recommended that "after construction, the installation should be monitored in order to observe whether boils occur adjacent to the installation. If boils develop, remedial work should be undertaken."

It is understood that boils have been noted immediately downstream of the land side apron, and these boils have occurred under normal operating conditions, not the much more severe seepage conditions that would exist should the dyke be subjected to the flood of record (1894) level. The boils represent a hydraulic exit gradient of about 1, which means the potential for piping exists. Such piping can eventually retrogress to the point that a failure of the dyke results.

As shown on Figure 2.3, the end of the apron on the land side is in line with the relief wells, and coincides with the termination of the slough blanket (sand and gravel), which for the Camp Slough crossing extends only 6 m to the land side of the toe of the dyke. The presence of the boils immediately to the land side of the apron suggests that the flowpath for the seepage that is causing the boils is likely along the outside of the steel culverts. Despite the construction records indicating acceptable compaction, it is unclear whether acceptable compaction was also obtained below the haunches of the culverts. It is possible that a relatively loose zone immediately around the culverts provides a preferred flow path for concentrated seepage. It is also possible, though less likely, that the boils are not the result of the construction of the culverts structure, and that the proximity of the boils to the structure is merely coincidental.

Whatever the explanation for the boils, this represents a situation that must be rectified as part of the dykes upgrading project. The most effective means of addressing this problem would be to extend the sand and gravel slough blanket further to the land side, providing a filter over the natural soils in which the boils have been observed. This will not prevent strong upward gradients from occurring, but will prevent erosion (manifested in the form of boils) of the natural soils that is caused by these gradients.

Accordingly, AMEC recommends that the land side slough blanket be extended a further 24 m beyond its current limit, extending between the east end of the slough to a point at least 40 m to the west of the concrete apron (see proposed extension as shown in plan on Figure 2.3). This would put the termination of the extended blanket at a point 30 m downstream of the land side dyke toe, consistent with the typical slough crossing sections. The slough blanket extension should have a minimum thickness of 0.6 m, and should be constructed of clean 75-mm minus gravelly sand with a maximum fines content (percent by dry weight finer than 0.075 mm) of 5%, and a sand content (sizes between 0.075 mm and 4.75 mm) of between 30% and 60%. The slough blanket need only be compacted by the trafficking of the dozers that will be used to place it.

8.0 CONCLUSIONS AND RECOMMENDATIONS

Following are the conclusions and recommendations arising from AMEC's work on the dykes upgrade project:

1. For those portions of the dykes where relief pits or relief well/slough blanket installations are in place, these existing installations represent adequate seepage control measures for the raised design flood level. However, in some areas the relief pits have been covered over by farming activities. The drain rock at the surface of the relief pits must be exposed, and kept exposed, at all locations. Measures to prevent covering over the relief pits in the future, such as fences, are recommended along the entire length of the dykes.

2. For those sections of the dykes where no seepage control measures were deemed necessary for the previous dykes upgrade, the same holds true except for the section of dyke between Sta. 11+250 m and 11+475 m. Along this 225-m section, relief pits construction is recommended. The relief pits should be 1.2 m in depth, and constructed according the same design as illustrated on Willis Cunliffe Tait as-constructed drawing #105225.

3. At the location of the Camp Slough intake culvert and channel, additional seepage control measures are recommended, given the past observations of boiling noted immediately to the land side of the concrete apron. It is recommended that the sand and gravel slough blanket be extended to the land side a further 24 m, between the east end of the slough to a point at least 40 m to the west of the concrete apron (see Figure 2.3). The slough blanket extension should consist of a minimum 0.6-m thickness of clean 75-mm minus gravelly sand with a maximum fines content (percent by dry weight finer than 0.075 mm) of 5%, and a sand content (sizes between 0.075 mm and 4.75 mm) of between 30% and 60%. The slough blanket fill need only be compacted by the trafficking of the dozers used to place it. Some form of erosion control measures will be required for that portion of the blanket immediately to the land side of the land side apron. The degree of erosion control necessary will depend on design flows through the culverts.

4. The upstream and downstream slopes of the dykes should be no steeper than the design slopes set out in the original design criteria for the dykes, which were as follows:
   - Slough crossing sections – 3H:1V river side and land side slopes
   - Outside of slough crossing sections: 2H:1V river side slope, and 2.5H:1V land side slope

   These design slopes need to be re-established in those locations where current slopes are steeper than design.

5. Prior to any fill placement on the land side slopes, the slope must be stripped of all organics to expose the clean sand and gravel soils that make up the land side slope of the dykes. Any fill added to the land side slope of the dykes to re-establish the maximum design slope should consist of a well-graded, clean 75-mm minus gravelly sand with a maximum fines content (percent by dry weight finer than 0.075 mm) of 5%, and a sand content (sizes between 0.075 mm and 4.75 mm) of between 30% and 60%. Compaction of fill placed on the land side slopes of the dykes is not required.

6. Prior to any fill placement on the river side slopes, the slope must be stripped of all organics to expose the more silty sand and gravel soils that typically make up the river side slope of the dykes. This stripping must be carefully done to prevent removal of the silty sand and gravel blanket on the river side dyke face, and must be closely monitored by inspection personnel. Fill placed on the river side slope of the dykes should consist of a well-graded, 75-mm minus well-graded material with a minimum fines content (as defined above) of 10%, and at least 55% by dry weight finer than 4.75 mm (#4 series sieve). Compaction of fill placed on the river side slopes is not required.

7. Along much of the length of the dykes, it will be possible to raise the dykes to their new design grade by adding fill only to the crest, with no need to extend either the river or land side slopes. For areas outside of slough crossings, wherever the existing dyke height (measured as elevation difference between existing crest and ground elevation at the land side toe) exceeds 4 m, and the design flood elevation is 0.4 m or more higher than the old design flood elevation, the current width of the dyke crest should be maintained by adding fill to the river side slope as well as to the dyke crest. For other relief pit sections, where dyke raising would require only addition of fill to the crest (i.e. where the dyke crest can be narrowed), such narrowing of the crest is acceptable.

8. At slough crossings, where the dyke geometry is such that the new design grade can be achieved by adding fill only to the existing dyke crest, this is acceptable.

9. Prior to fill placement on the crest of the dykes, any organic soils are to be removed. Following this, the fill at the river side crest of the existing dykes must be assessed. If this material has a silt content of less than 10% (based on 75-mm minus sample fraction), then provided there is to be no fill placement to extend the river side slope, it should be removed for a width of 1 m at the river side dyke face, to expose the underlying, more silty sand and gravel. Fill placed on the dyke crest that is at or below the new design flood level should consist of a well-graded, 75-mm minus well-graded material with a minimum fines content (as defined above) of 10%, and at least 55% by dry weight finer than 4.75 mm (#4 series sieve). Material placed above the new design flood elevation may be any material suitable as a road base. Fill placed on the dyke crests shall be in lift thicknesses no greater than 0.3 m, and compacted to a minimum 90% of Modified Proctor maximum dry density (MPD) in accordance with ASTM D-1557 (in areas where the dyke crest is not an asphalt paved surface).

10. Prior to fill placement on the crest of dykes where the asphalt pavement exists, the existing full depth of asphalt pavement is to be ground up. The existing ground asphalt may be left in place to form part of the granular sub-base layer. It is assumed that the classification of existing paved roads on the crest of the dykes is "local". Based on a local road classification, AMEC recommends that the pavement structure be constructed with a minimum section of: 75 mm of asphaltic conrete surface course (40 mm lower course & 35 mm upper course), underlain by; 100 mm of 19 mm minus crushed gravel base course compacted to a minimum 95% MPD, underlain by; 200 mm of 75 mm minus well-graded material with a minimum fines content (as defined above) of 10%, and at least 55% by dry weight finer than 4.75 mm (#4 series sieve) sub-base course compacted to a minimum 95% MPD, underlain by the existing granular dyke material. It

is recommended that the granular subgrade be recompacted prior to the placement of the pavement structure to locate soft areas.

Recommendations presented herein are based on a geotechnical evaluation of the findings of studies noted. If conditions other than those reported are noted during subsequent phases of the project, AMEC should be notified and be given the opportunity to review and revise the current recommendations, if necessary. Recommendations presented herein may not be valid if an adequate level of review or inspection is not provided during construction.

This report has been prepared for the exclusive use of Dayton and Knight Engineering Limited and the City of Chilliwack. Any use which a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. AMEC accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report. It has been prepared in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

Respectfully submitted,

AMEC Earth & Environmental Limited

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Project Engineer

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
T. E. MARTIN
28/15/02

Todd E. Martin, P.Eng., P.Geo.
Senior Geotechnical Engineer

Reviewed by:

Al Dahlman, P.Eng.
Project Manager

Peter C. Lighthall, P.Eng.
Vice President

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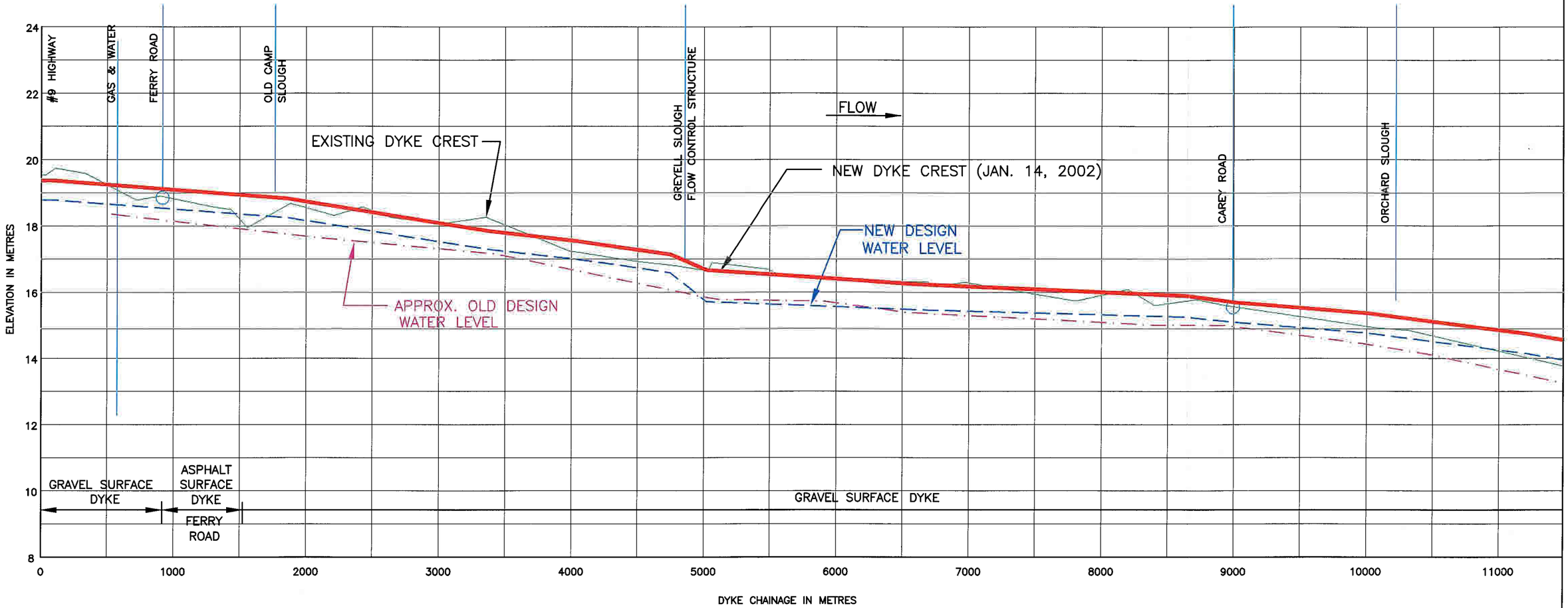
CPD "East Chilliwack Dykes – Amended Report on Soil Conditions" dated September 3, 1970.

CPD "Camp Slough Intake Culvert & Channel – Ferry Road @ Fraser River" dated March 21, 1983.

Ripley, Klohn & Leonoff International Ltd. (RKL) "Chilliwack Dykes – Investigation & Remedial Treatment, Chilliwack, British Columbia" dated January 16, 1970.

RKL " Chilliwack Dykes – Underseepage" dated August 10, 1970.
Cook, Pickering & Doyle Ltd (CPD) "East Chilliwack Dykes – Report on Soil Conditions" dated August 14, 1970.

Willis, Cunliffe, Tait, as-constructed drawings dated October 1971.



(EXISTING DYKE TAKEN FROM CITY OF CHILLIWACK, 1998)

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CHILLIWACK DYKE UPGRADE
DYKE PROFILE

DATE:	FEB. 2002
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FIGURE No.	FIG. 1.1

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14, 2002)

CAREY ROAD

ORCHARD SLOUGH

NELSON SLOUGH

FLOW

McDONALD ROAD

BELL SLOUGH

NORTH END OF YOUNG ROAD

CARTMELL ROAD

WING DYKE

NEW DYKE CREST (JAN. 14, 2002)

NEW DESIGN WATER LEVEL

APPROX. OLD DESIGN WATER LEVEL

ASPHALT SURFACE DYKE

GRAVEL SURFACE DYKE

ASPHALT SURFACE DYKE

CONCRETE PLANT

TOWER ROAD

ELEVATION IN METRES

24
22
20
18
16
14
12
10
8

8000 9000 10000 11000 12000 13000 14000 15000 16000 17000 18000 19000

DYKE CHAINAGE IN METRES

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CHILLIWACK DYKE UPGRADE
DYKE PROFILE

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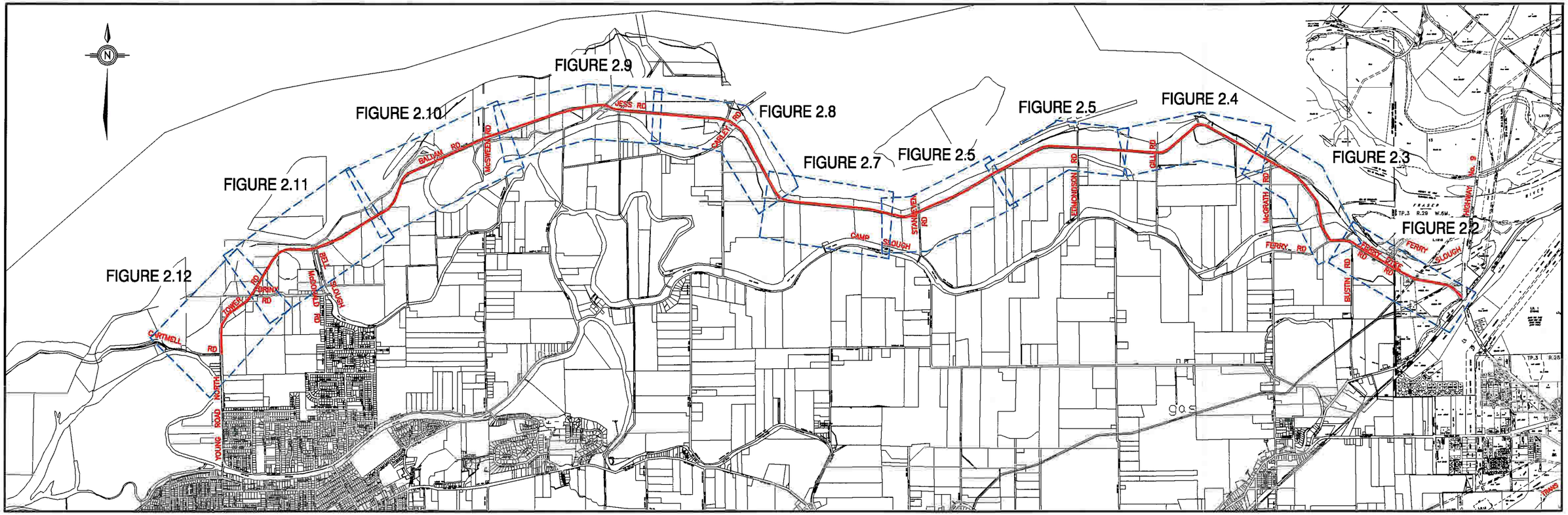
FIGURE No.	FIG. 1.2



KEY PLAN

0 200 400 800 1200 1600 2000 METRES
100 300

AREA PLAN

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KEY PLAN, AREA PLAN AND INDEX TO DRAWINGS

DATE:	FEB. 2002
PROJECT NO:	KA20481
REV. NO.:	-
FIGURE No.	FIG. 2.1

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CHEAM INDIAN RESERVE NO. 1

CHEAM INDIAN RESERVE NO. 1

BURIED
OWN,
CONFIRM LOCATION
G OF WORK

0+800
0+700
0+600
0+500
0+400
0+300
0+200
0+100
0+000

STATUTORY RIGHT OF WAY PLAN 51976

PARCEL "A"
L.T.O. PLAN 31218
C.L.S.R. PLAN 53350

WESTCOAST ENERGY INC. S/R/W PLAN 19806

WESTCOAST ENERGY INC.

STAT. R/W PLAN 15881

RKL–TP1001
RKL–TP1001A
RKL–DH1015A
RKL–DH1001
RKL–DH1015

L.T.O. PLAN 21655 C.L.S.R. PLAN 5010
ROSEDALE – AGASSIZ HIGHWAY NO. 9

DISTRICT LOT 469 GROUP 2

PLAN LMP 2280

PLAN LMP 2280

PLAN LMP 2280

PLAN 7234

DL 469 GP2
DL 468 GP2

LOCATION OF EXIST. BURIED TELUS CABLES UNKNOWN, CONTRACTOR SHOULD CONFIRM LOCATION PRIOR TO COMMENCING OF WORK

McGRATH ROAD

SLOUGH

FERRY ROAD

FERRY ROAD

RKL–AH1016C
RKL–AH1016B
RKL–DH1012
RKL–DH1016A
RKL–DH1016

BENCH MARK 89C174.

BUSTIN ROAD

REF. PLAN LMP3463

WATER RESOURCES BENCH MARK 767.

RKL–DH–1

RKL–DH–8

RKL–DH–9

STATUTORY RIGHT OF WAY PLAN 51976

PARCEL "A"
REFERENCE PLAN 7083
DISTRICT LOT 468 GROUP 2

BARN

PARCEL "A"
PLANS WITH ABSOLUTE FEE PARCEL BOOK 12/53/3878F

DISTRICT LOT 469 GROUP 2

DH2

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LEGEND

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CHILLIWACK DYKE UPGRADE
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SHEET 1 OF11

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FIGURE No.	FIG. 2.2

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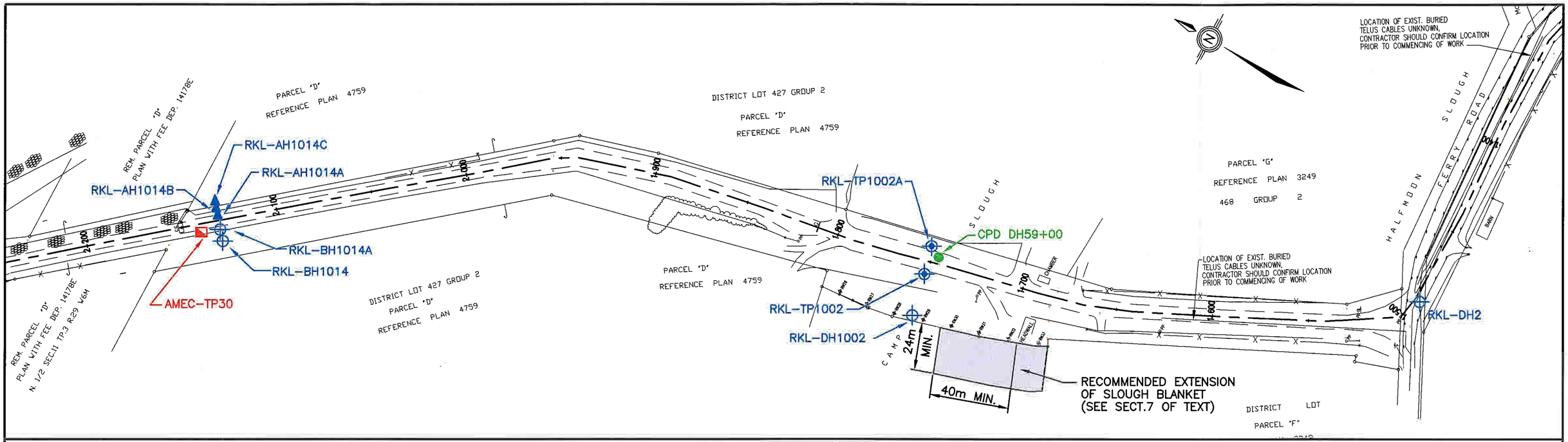

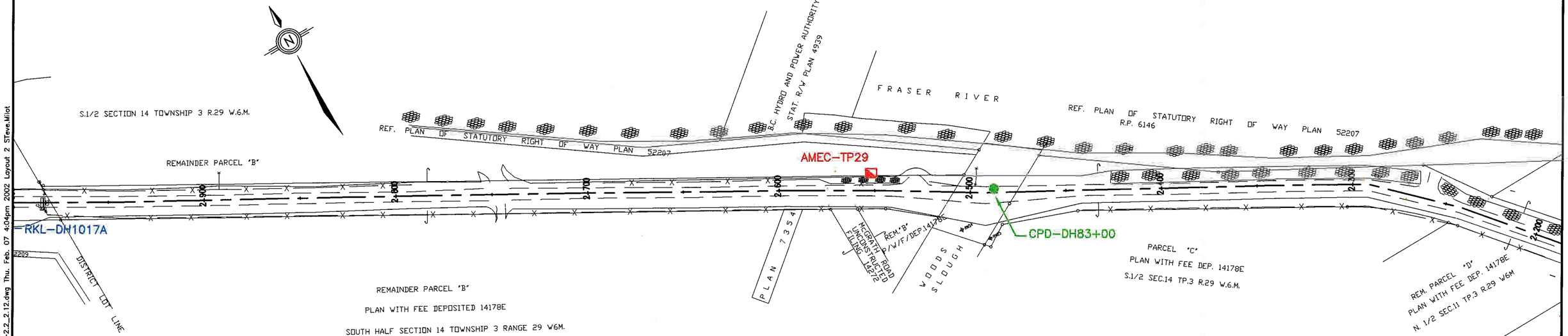

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FIGURE No.	FIG. 2.3

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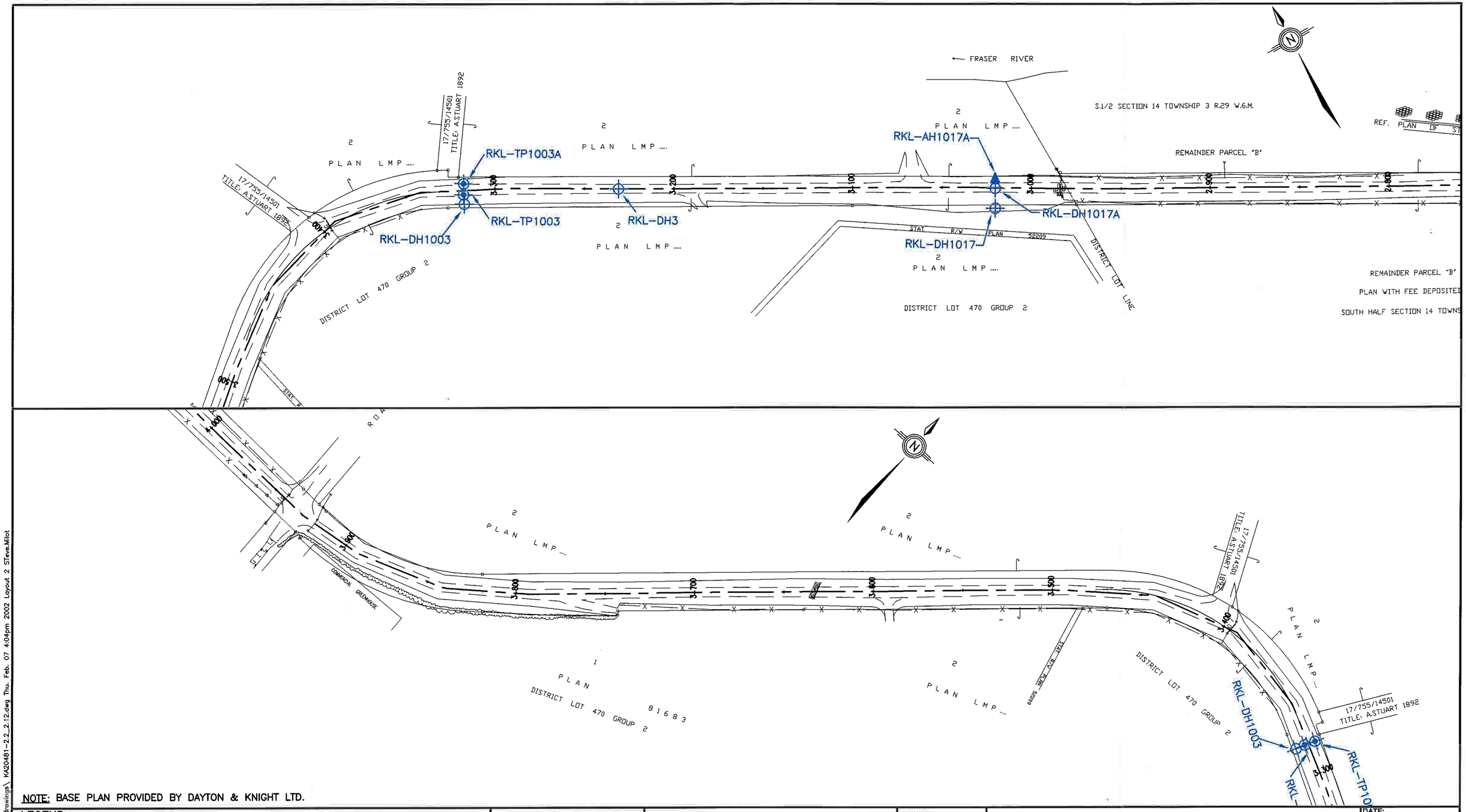

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FIGURE No.	FIG. 2.4

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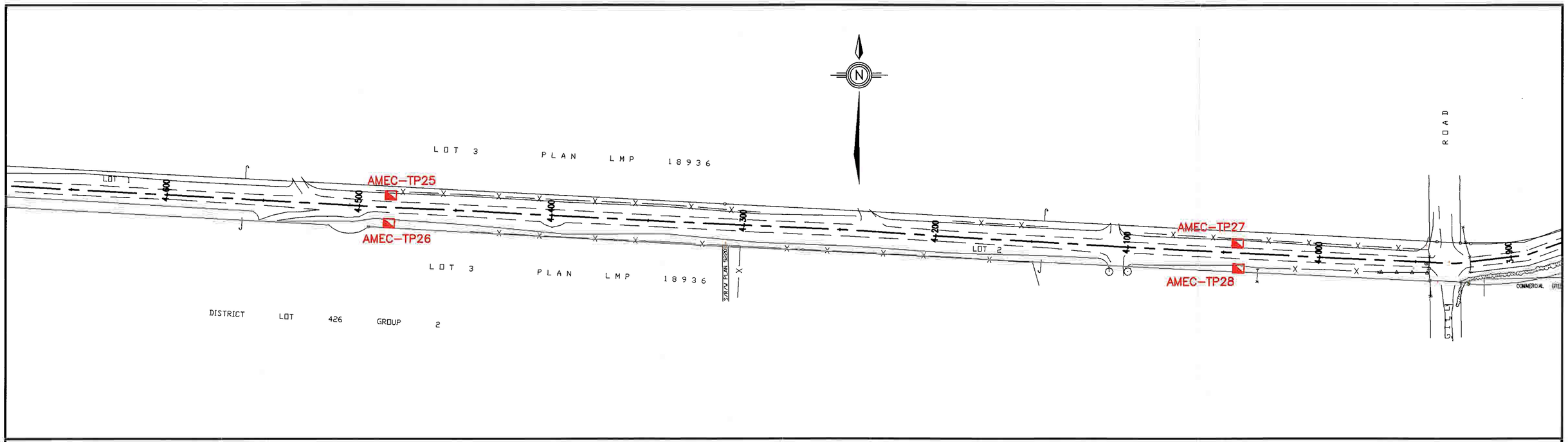

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SHEET 4 OF11

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FIGURE No.	FIG. 2.5

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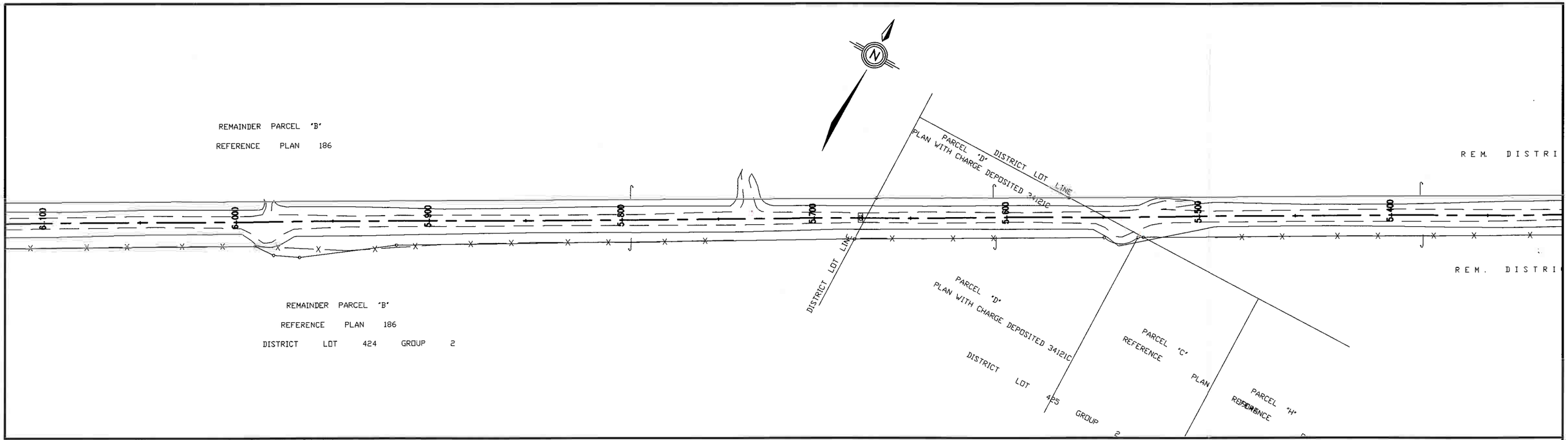

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- DRILL HOLE

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CHILLIWACK DYKE UPGRADE
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REV. NO.:	-
FIGURE No.	FIG. 2.6

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REMAINDER WEST HALF OF DISTRICT LOT 462 GROUP 2

REMAINDER PARCEL "B" EXP. PLAN 14271F

CROWN LAND

SLOUGH

CDP-230+00

7+500 7+400 7+300 7+200 7+100 7+000 6+900 6+800

REMAINDER WEST HALF OF DISTRICT LOT 462 GROUP 2

REMAINDER PARCEL "B" EXP. PLAN 14271F

DISTRICT LOT 462 GROUP 2

REMAINDER PARCEL "B" EXP. PLAN 14271F

REMAINDER PARCEL "B" EXP. PLAN 14271F

DISTRICT LOT 462 GROUP 2

REM.5 PLAN 5506

8+300 8+200 8+100 8+000 7+900 7+800

RKL-AH1006

RKL-TP1006

RKL-DH1006

DISTRICT LOT 418 GROUP 2

REM.4 PLAN 5506

REM.1 PLAN 5506

RKL-DH1020A

AMEC-TP23

AMEC-TP24

RKL-DH1020

REM. 1 PLAN 5506

DISTRICT LOT BOUNDARY

STAT. R/W PLAN 52289

REM. 1 PLAN 5506

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CHILLIWACK DYKE UPGRADE
TEST HOLE LOCATION PLAN
SHEET 6 OF11

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FIGURE No.	FIG. 2.7

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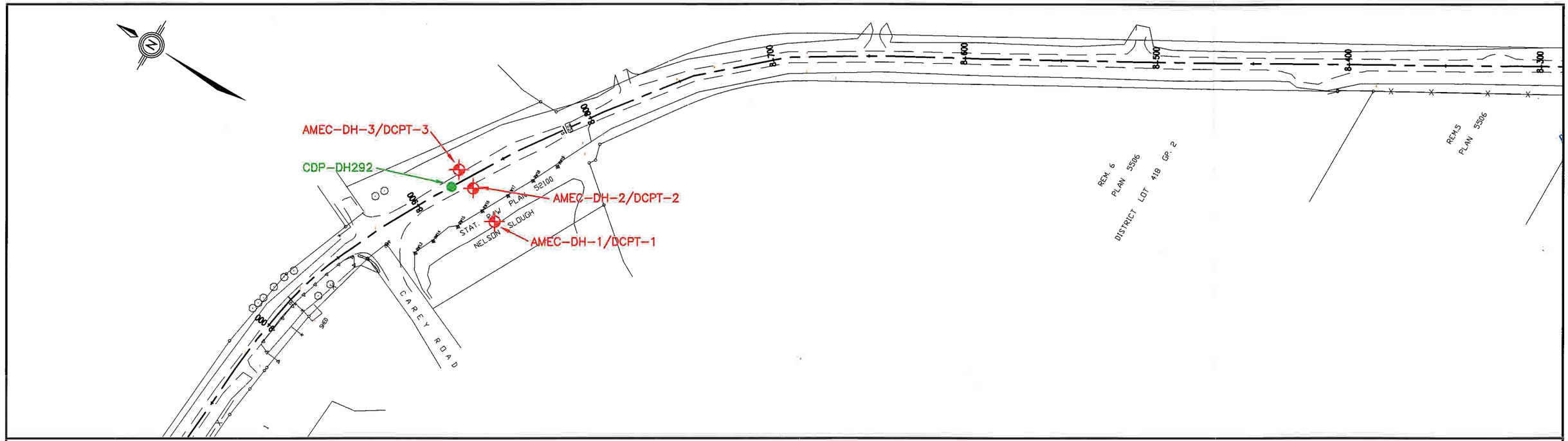

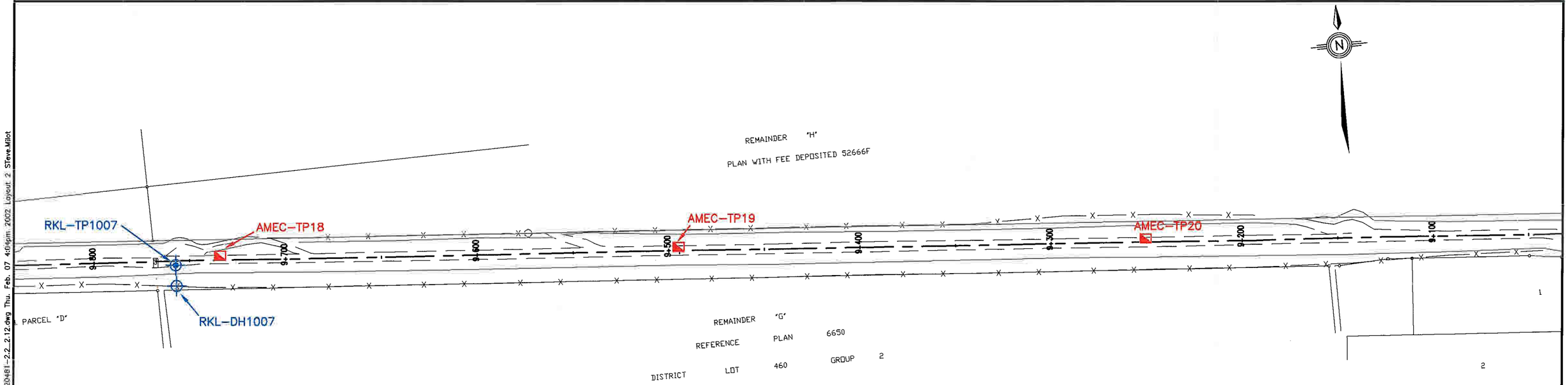

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- AUGER HOLE

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- DRILL HOLE

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- DRILL HOLE

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TEST HOLE LOCATION PLAN
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FIGURE No.	FIG. 2.8

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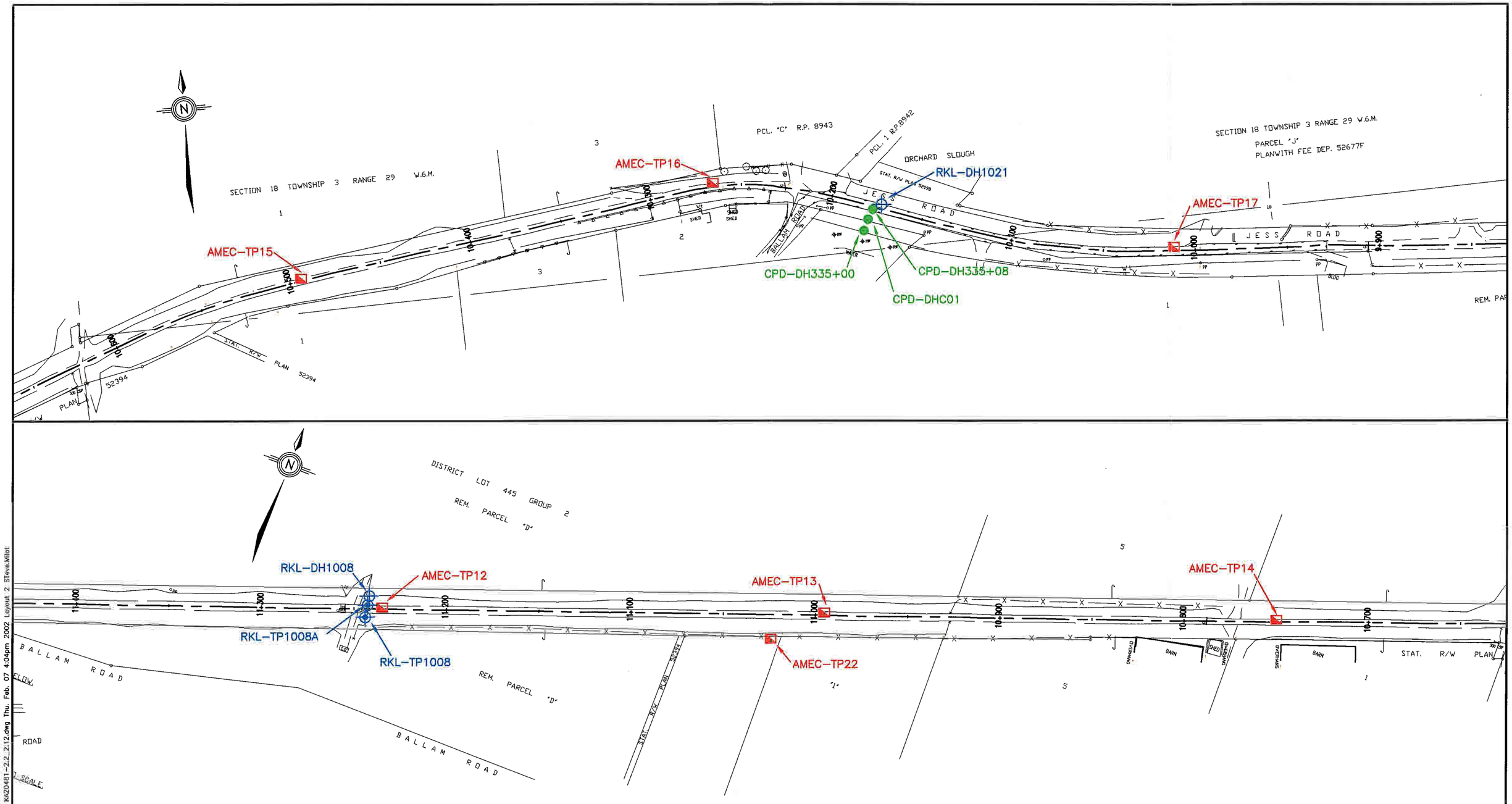

NOTE: BASE PLAN PROVIDED BY DAYTON & KNIGHT LTD.

LEGEND

RIPLEY, KLOHN & LEONOFF INTERNATIONAL LTD. (1970 INVESTIGATION

- TEST PIT
- DRILL HOLE
- AUGER HOLE

COOK, PICKERING & DOYLE (1970 INVESTIGATION)

- DRILL HOLE

AMEC HOLES (2001 & 2002 INVESTIGATION)

- TEST PIT
- DRILL HOLE

amec

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2227 Douglas Road
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V5C 5A9
Tel. 294-3811
Fax. 294-4664

Client

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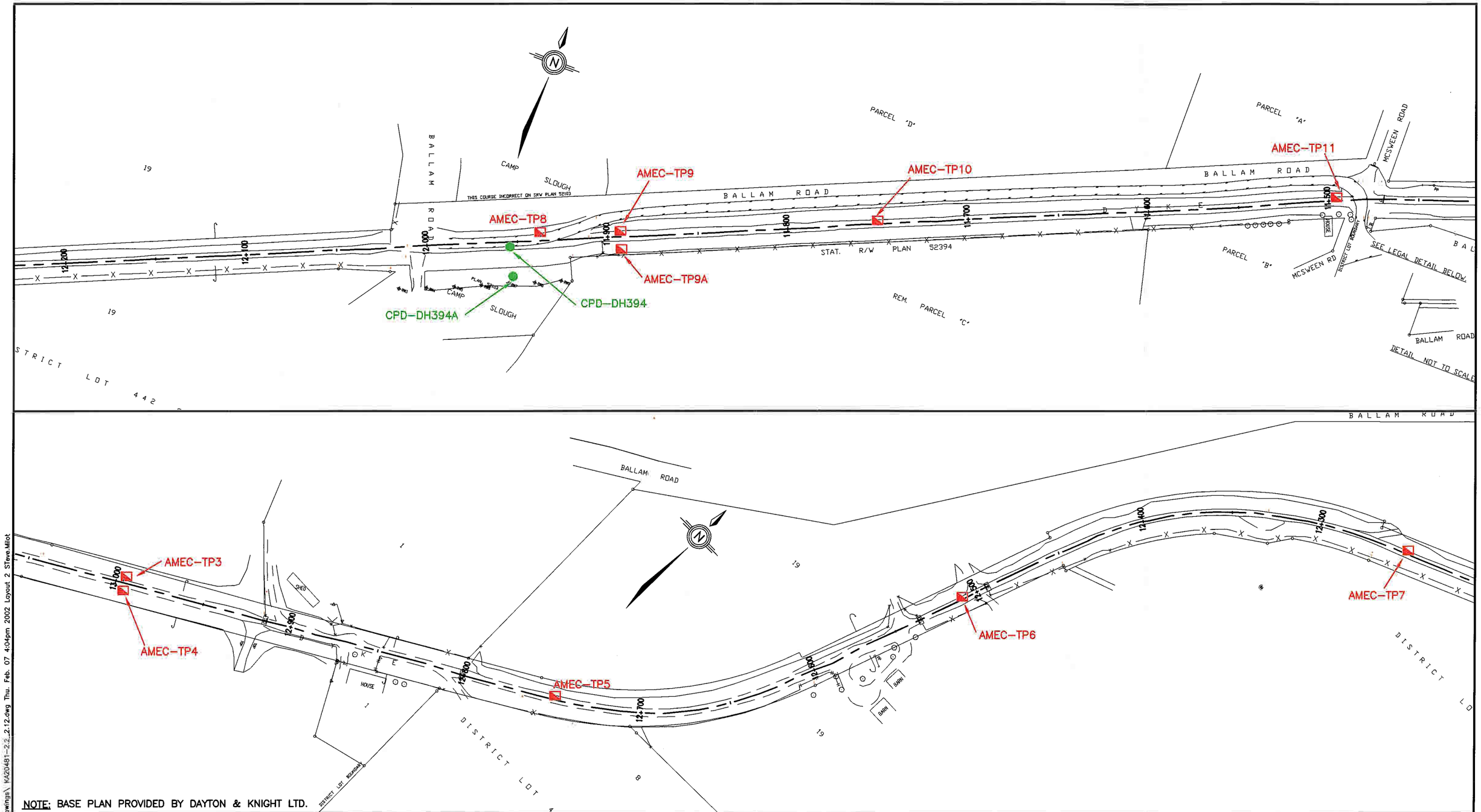
CHK'D BY:	DR
APP.	TM
SCALE	1:2000

CHILLIWACK DYKE UPGRADE
TEST HOLE LOCATION PLAN
SHEET 8 OF11

DATE:	FEB. 2002
PROJECT NO:	KA20481
REV. NO.:	-
FIGURE No.	FIG. 2.9

*PLOT 1:1=D D:\OOFFICE\KA20481\drawings\ KA20481-2.2_2.12.dwg Thu. Feb. 07 4:04pm 2002 Layout 2 STeve.Milot



NOTE: BASE PLAN PROVIDED BY DAYTON & KNIGHT LTD.

LEGEND

RIPLEY, KLOHN & LEONOFF INTERNATIONAL LTD. (1970 INVESTIGATION)
- TEST PIT
- DRILL HOLE
- AUGER HOLE

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- TEST PIT
- DRILL HOLE

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CHILLIWACK DYKE UPGRADE
TEST HOLE LOCATION PLAN
SHEET 9 OF11

DATE:	FEB. 2002
PROJECT NO:	KA20481
REV. NO.:	-
FIGURE No.	FIG. 2.10

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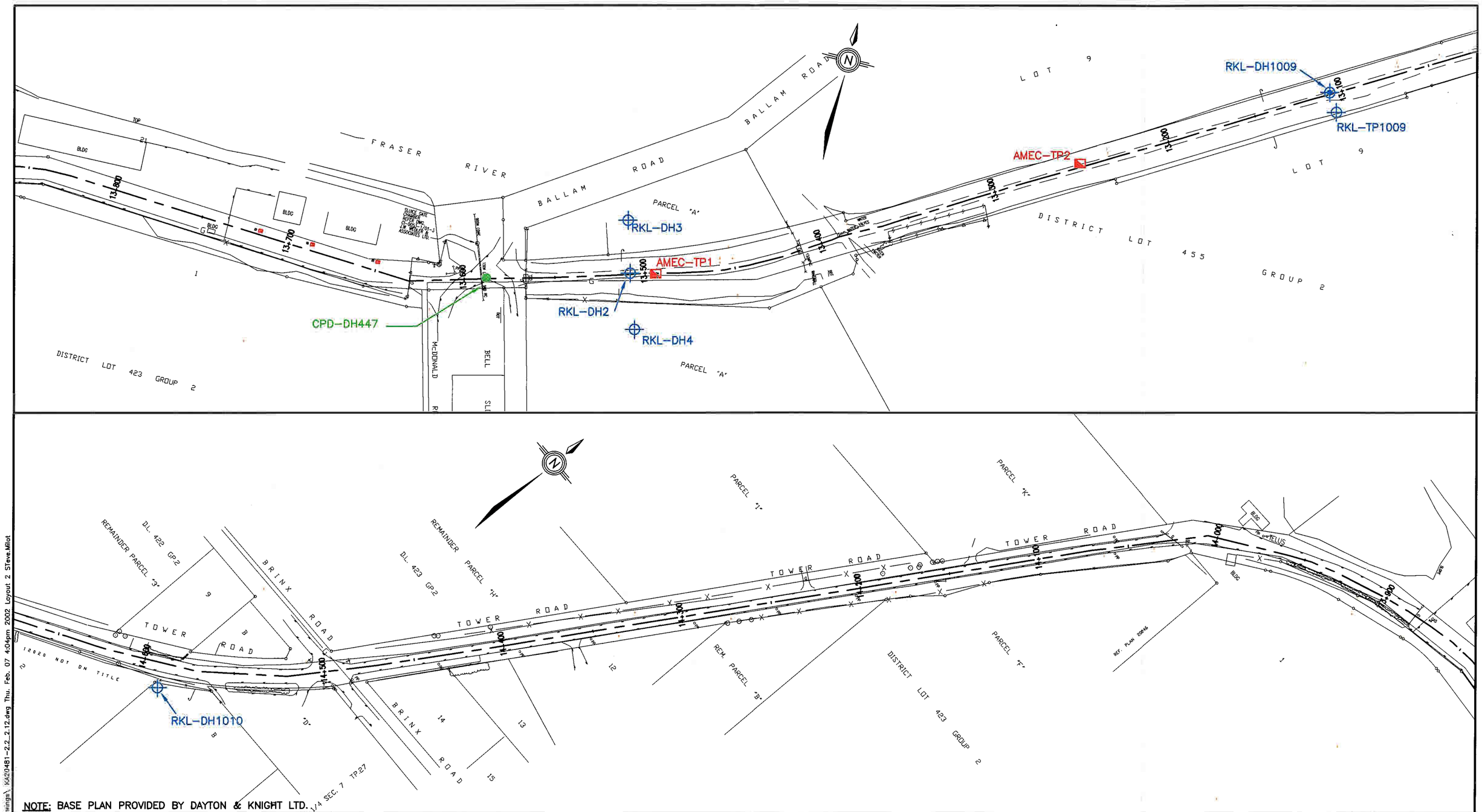

NOTE: BASE PLAN PROVIDED BY DAYTON & KNIGHT LTD.

LEGEND

RIPLEY, KLOHN & LEONOFF INTERNATIONAL LTD. (1970 INVESTIGATION)
- TEST PIT
- DRILL HOLE
- AUGER HOLE

COOK, PICKERING & DOYLE (1970 INVESTIGATION)
- DRILL HOLE

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- DRILL HOLE

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SCALE	1:2000

CHILLIWACK DYKE UPGRADE
TEST HOLE LOCATION PLAN
SHEET 10 OF11

DATE:	FEB. 2002
PROJECT NO:	KA20481
REV. NO.:	-
FIGURE No.	FIG. 2.11

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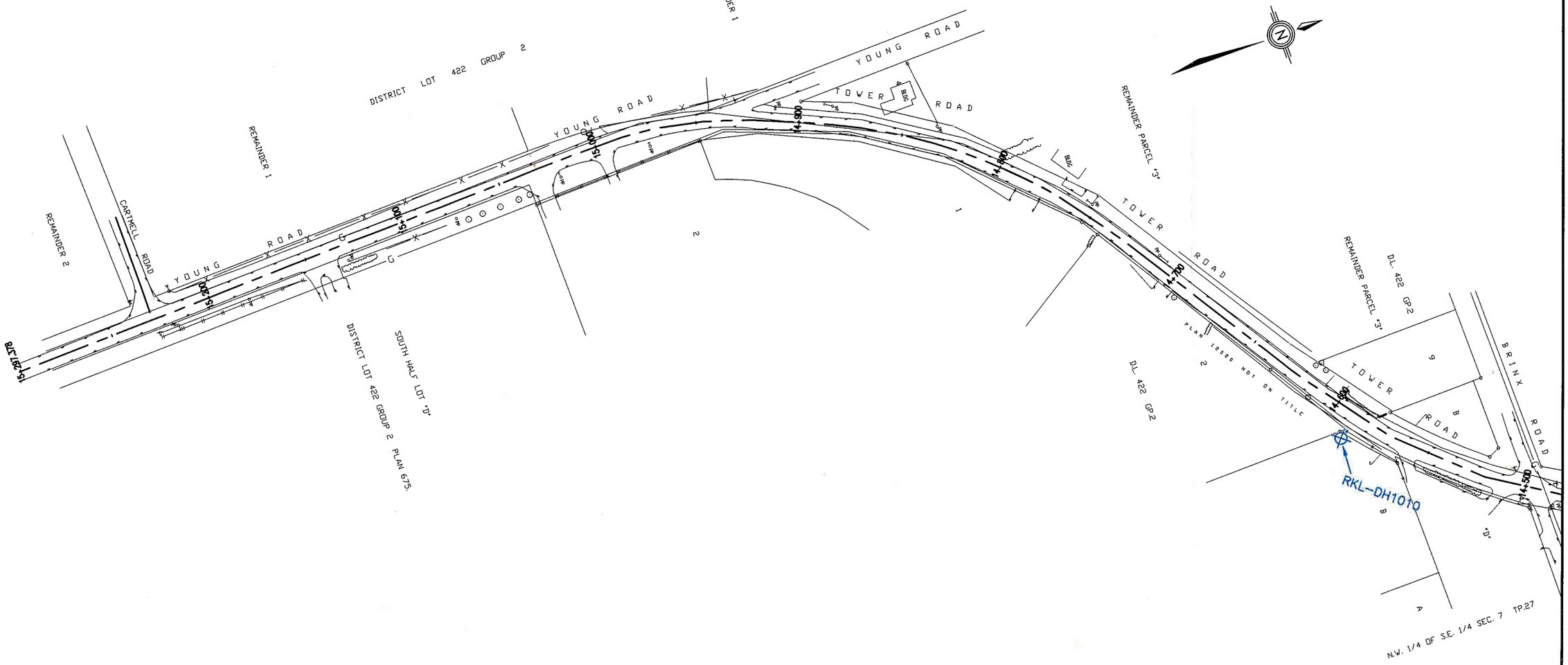

NOTE: BASE PLAN PROVIDED BY DAYTON & KNIGHT LTD.

LEGEND

RIPLEY, KLOHN & LEONOFF INTERNATIONAL LTD. (1970 INVESTIGATION
- TEST PIT
- DRILL HOLE
- AUGER HOLE

COOK, PICKERING & DOYLE (1970 INVESTIGATION)
- DRILL HOLE

AMEC HOLES (2001 & 2002 INVESTIGATION)
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- DRILL HOLE

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CHILLIWACK DYKE UPGRADE
TEST HOLE LOCATION PLAN
SHEET 11 OF11

DATE:	FEB. 2002
PROJECT NO:	KA20481
REV. NO.:	-
FIGURE No.	FIG. 2.12

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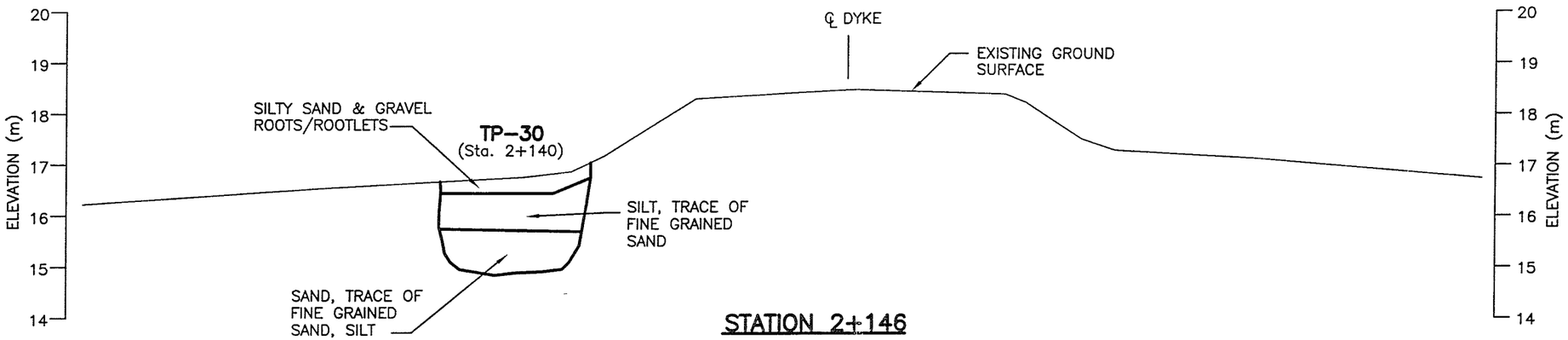
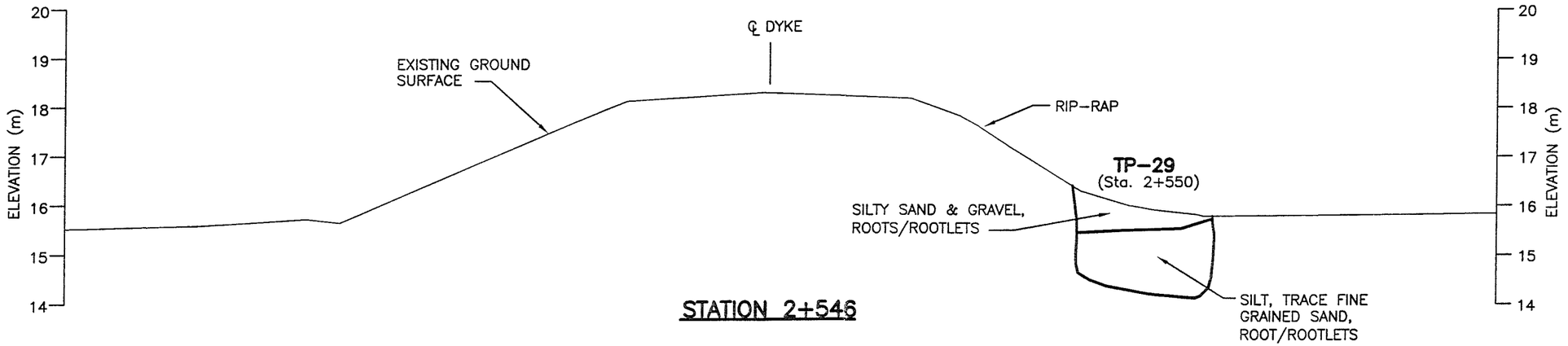


STATION 2+146



STATION 2+546

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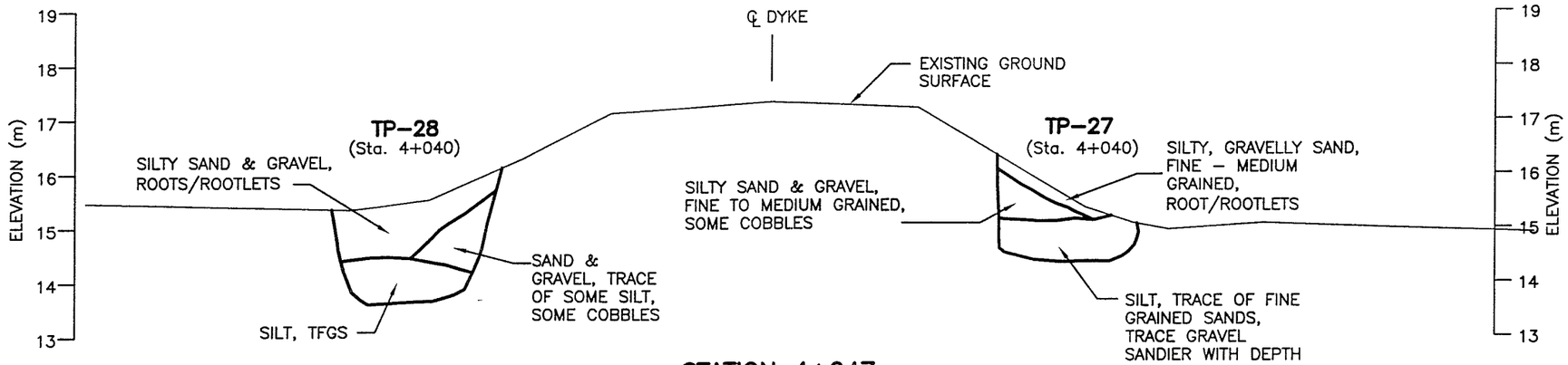
CHK'D BY:	DR
APP.	TM
SCALE	1:100

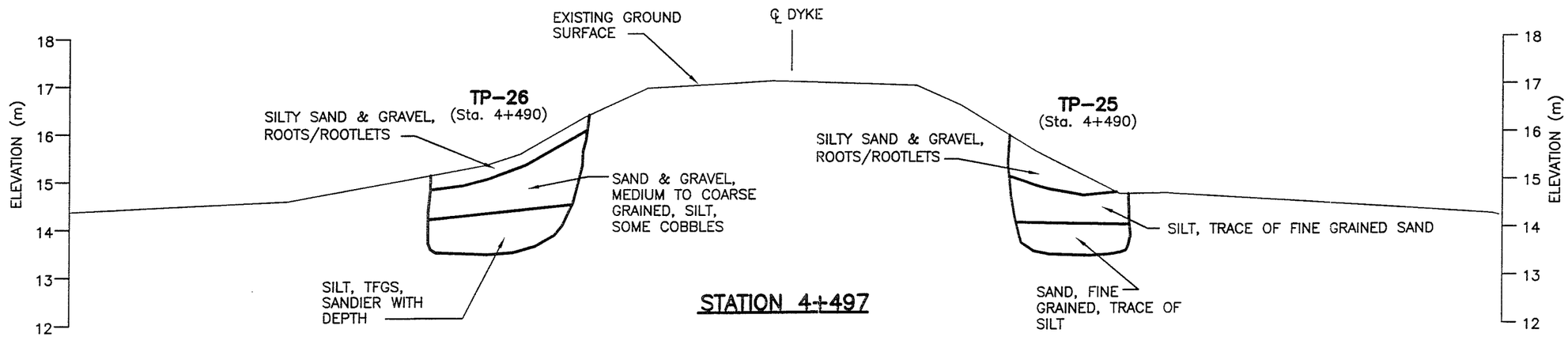
CHILLIWACK DYKE UPGRADE
TEST PIT SECTIONS
SHEET 1 OF 5

DATE:	FEB. 2002
PROJECT NO:	KA20481
REV. NO..	-
FIGURE No	FIG. 4.1

*PLOT 1:1=D D:\OFFICE\KA20481\drawings\ KA20481-4.1_4.5.dwg Thu. Feb. 07 11:29am 2002 Layout1 STeve.Milot



STATION 4+047



STATION 4+497

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APP.	TM
SCALE	1:100

CHILLIWACK DYKE UPGRADE
TEST PIT SECTIONS
SHEET 2 OF 5

DATE:	FEB. 2002
PROJECT NO:	KA20481
REV. NO.:	-
FIGURE No.	FIG. 4.2

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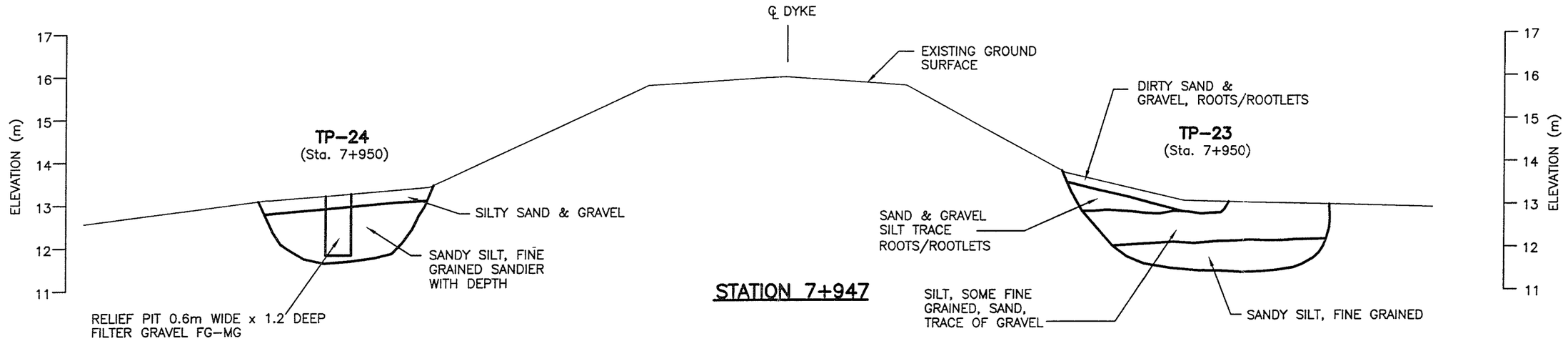

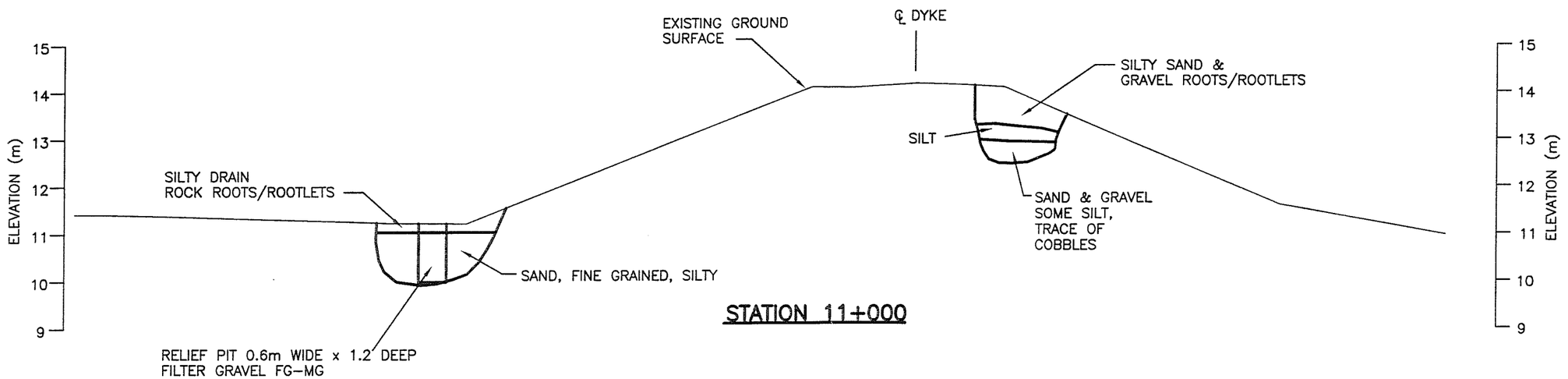

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CHILLIWACK DYKE UPGRADE
TEST PIT SECTIONS
SHEET 3 OF 5

DATE:	FEB. 2002
PROJECT NO:	KA20481
REV. NO.:	-
FIGURE No.	FIG. 4.3

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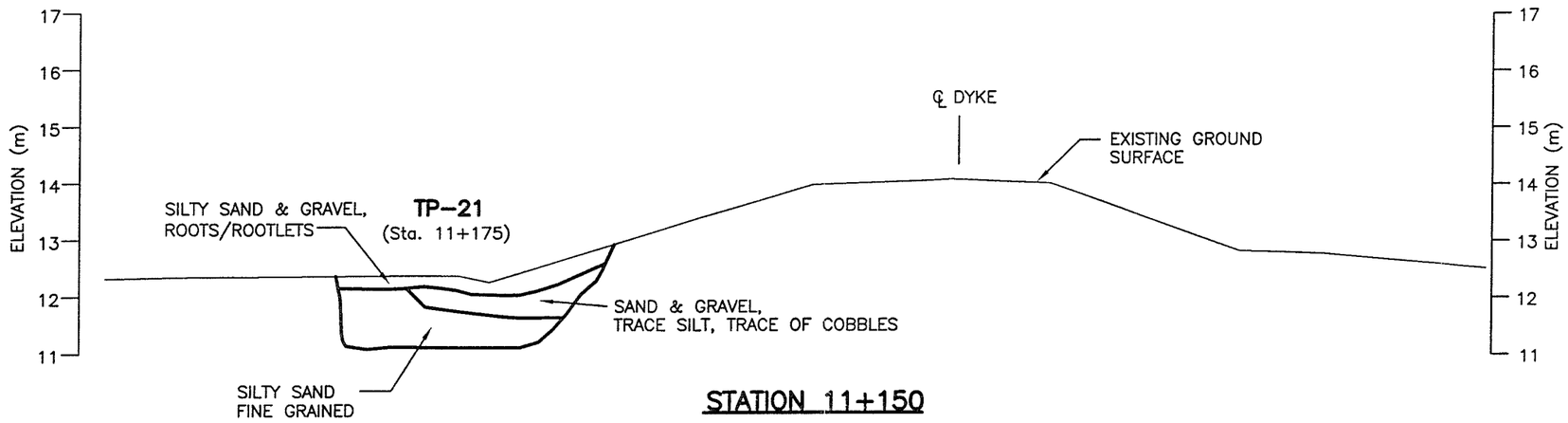
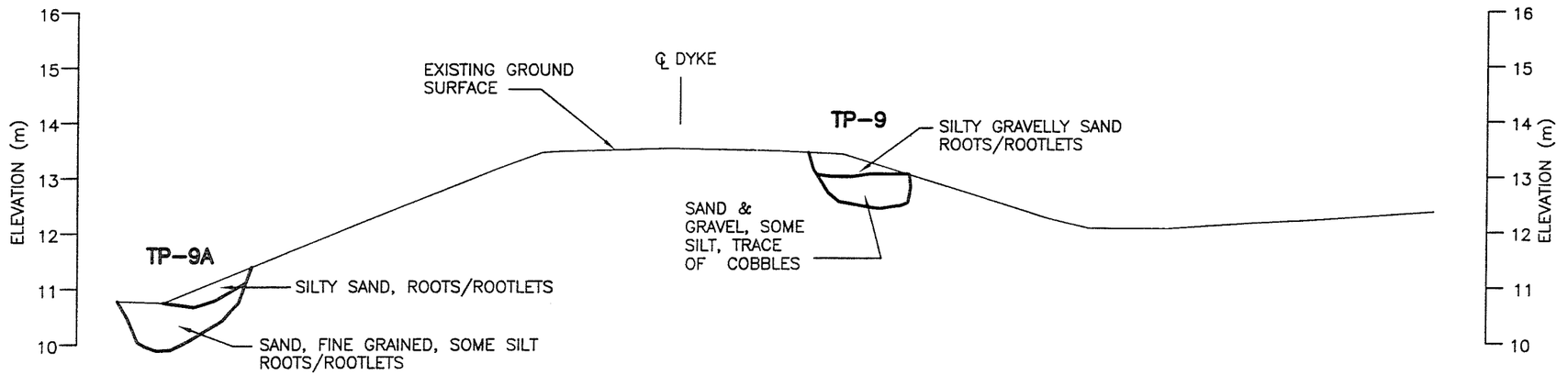


STATION 11+150



STATION 11+900

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SCALE	1:100

CHILLIWACK DYKE UPGRADE
TEST PIT SECTIONS
SHEET 4 OF 5

DATE:	FEB. 2002
PROJECT NO:	KA20481
REV. NO.:	-
FIGURE No.	FIG. 4.4

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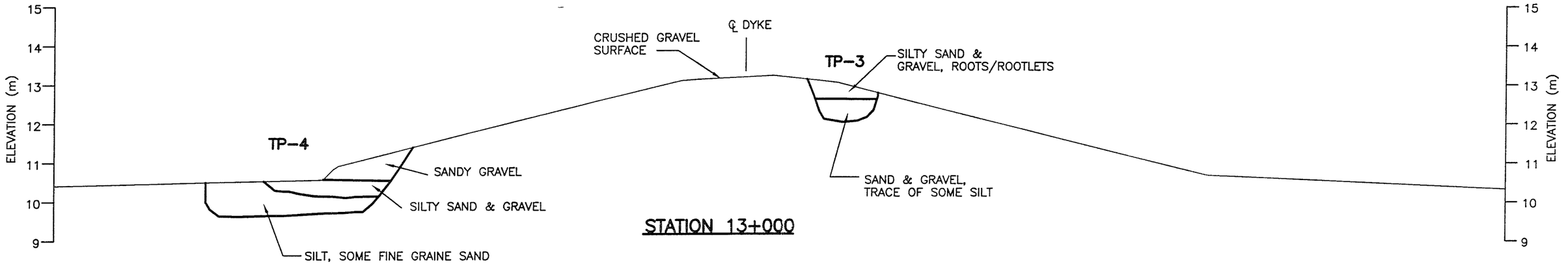

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APP.	TM
SCALE	1:100

CHILLIWACK DYKE UPGRADE
TEST PIT SECTIONS
SHEET 5 OF 5

DATE:	FEB. 2002
PROJECT NO:	KA20481
REV NO.:	-
FIGURE No.	FIG. 4.5

Land side of dyke – Bell Slough near McDonald Road.

Looking east along river side slope of dyke.

Typical slough crossing, taken from land side of dyke..

Looking east along crest of dyke near Bell Slough and tower Road. River side is to the left..

Slough crossing – land side to the left.

Land side slope of dyke – note proximity of fence line to relief pits alignment.

River side slope of dyke.

Typical slough blanket. Arrows indicate relief well locations.

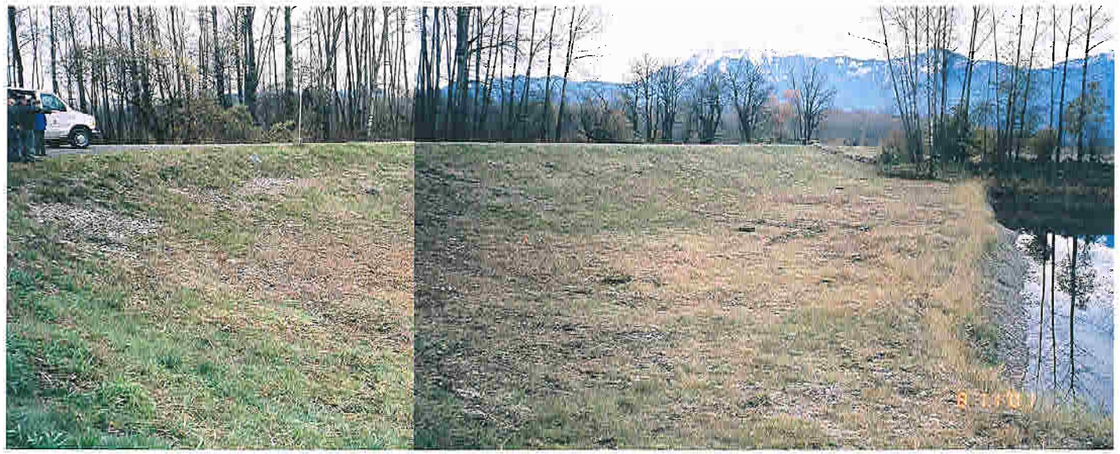
Panorama of slough crossing, looking at river side slope of the dyke.

Land side slope of dyke.

River side face of dyke at same slough crossing shown in above photograph.

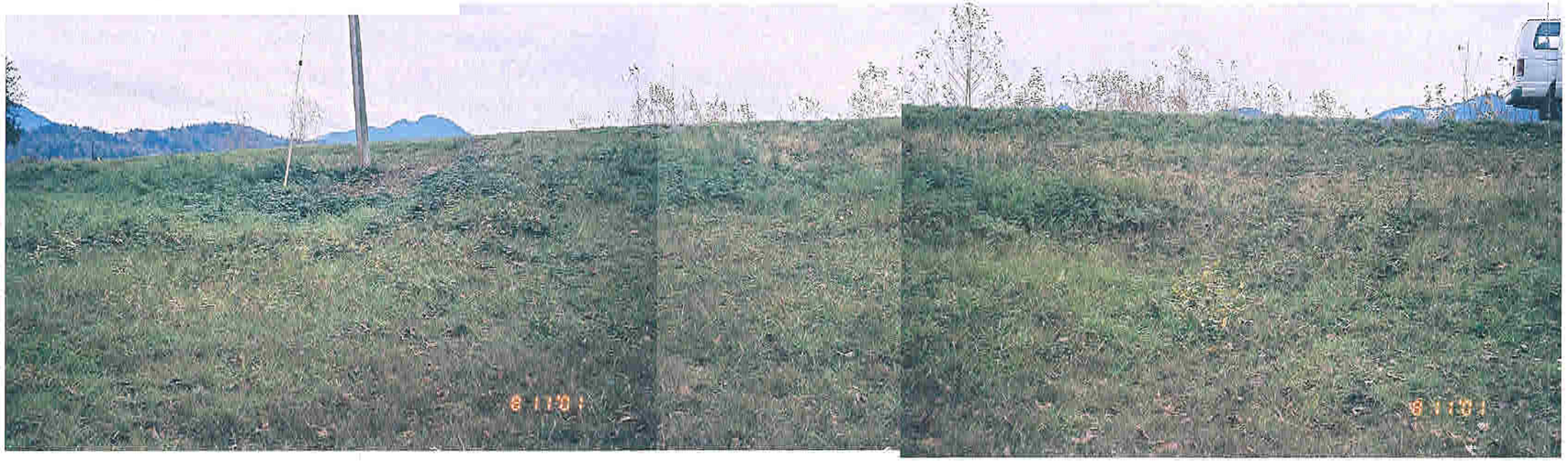
Dyke crest – river side to right,

Control structure at Old Camp Slough – intake channel on left photo, and outlet (land side of dyke) in the right photo.



Land side of dyke at Bell slough near Carey Road, where gravel mining has removed portion of the slough blanket.

River side slope of dyke at Bell slough near Carey Road.

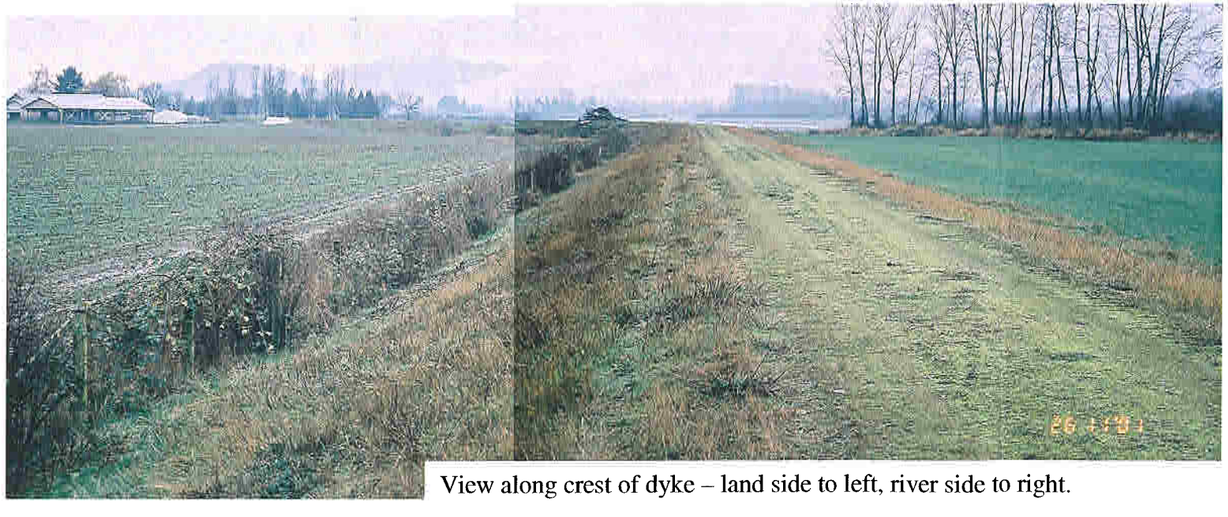


Panorama of land side slope of dyke.

Typical slough crossing – looking to land side of dyke.

Farming activity up to land side toe of dyke – in some areas, this has resulted in silty soils being placed overtop of the relief pits.

View along crest of dyke – land side to right (note proximity of homes), and river side to left.



View along crest of dyke – land side to left, river side to right.

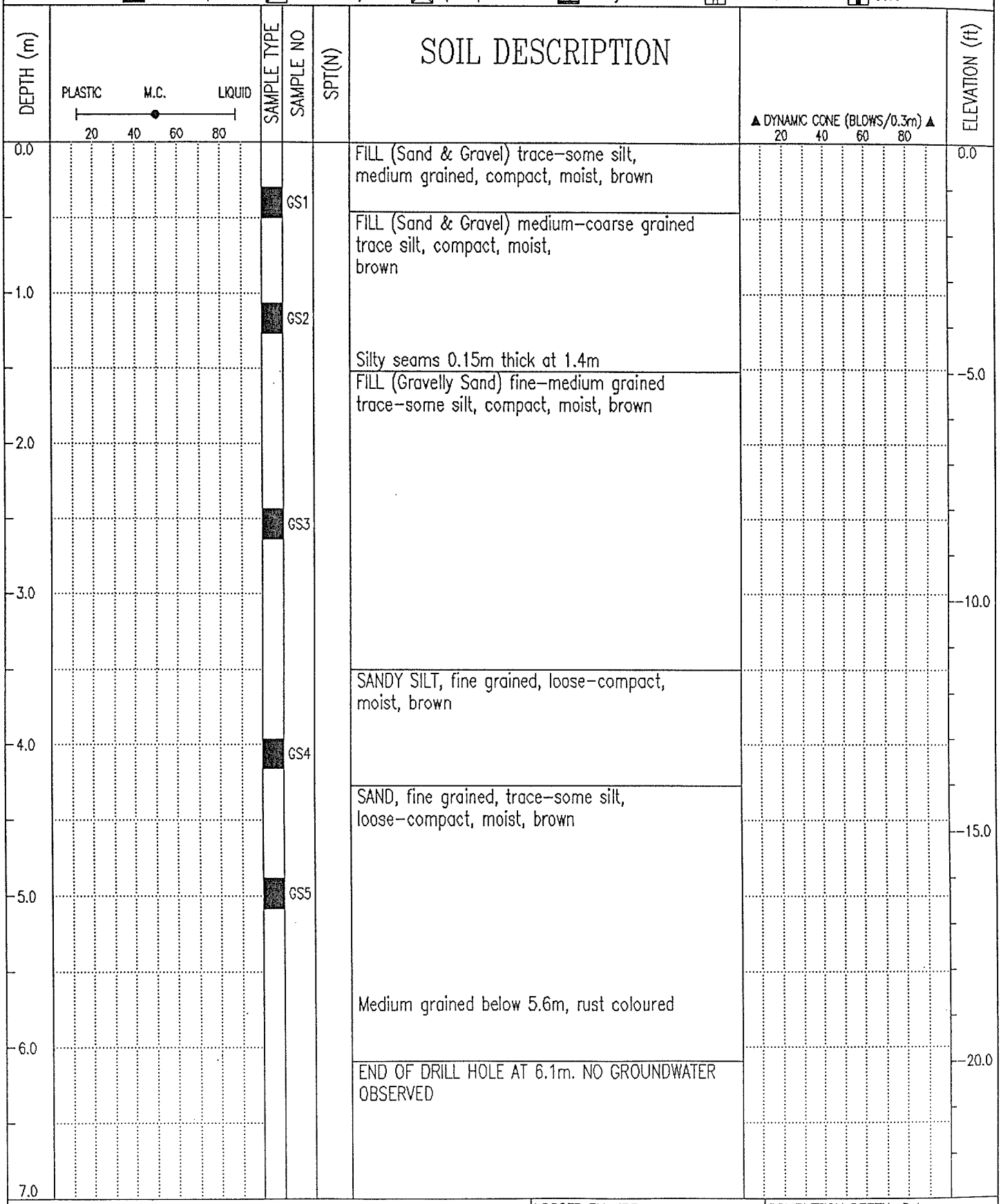
Looking to land side – another area where farming activity appears to have covered relief pits.

APPENDIX A

AMEC Drill Hole Logs

Chilliwack Dyke	STA 11+895	BOREHOLE No: DH-1
DAYTON & KNIGHT		Project No: KA20481
START DATE: 01/21/02	See Drawing for Drill Hole Locations	ELEVATION: 0.00 (m)

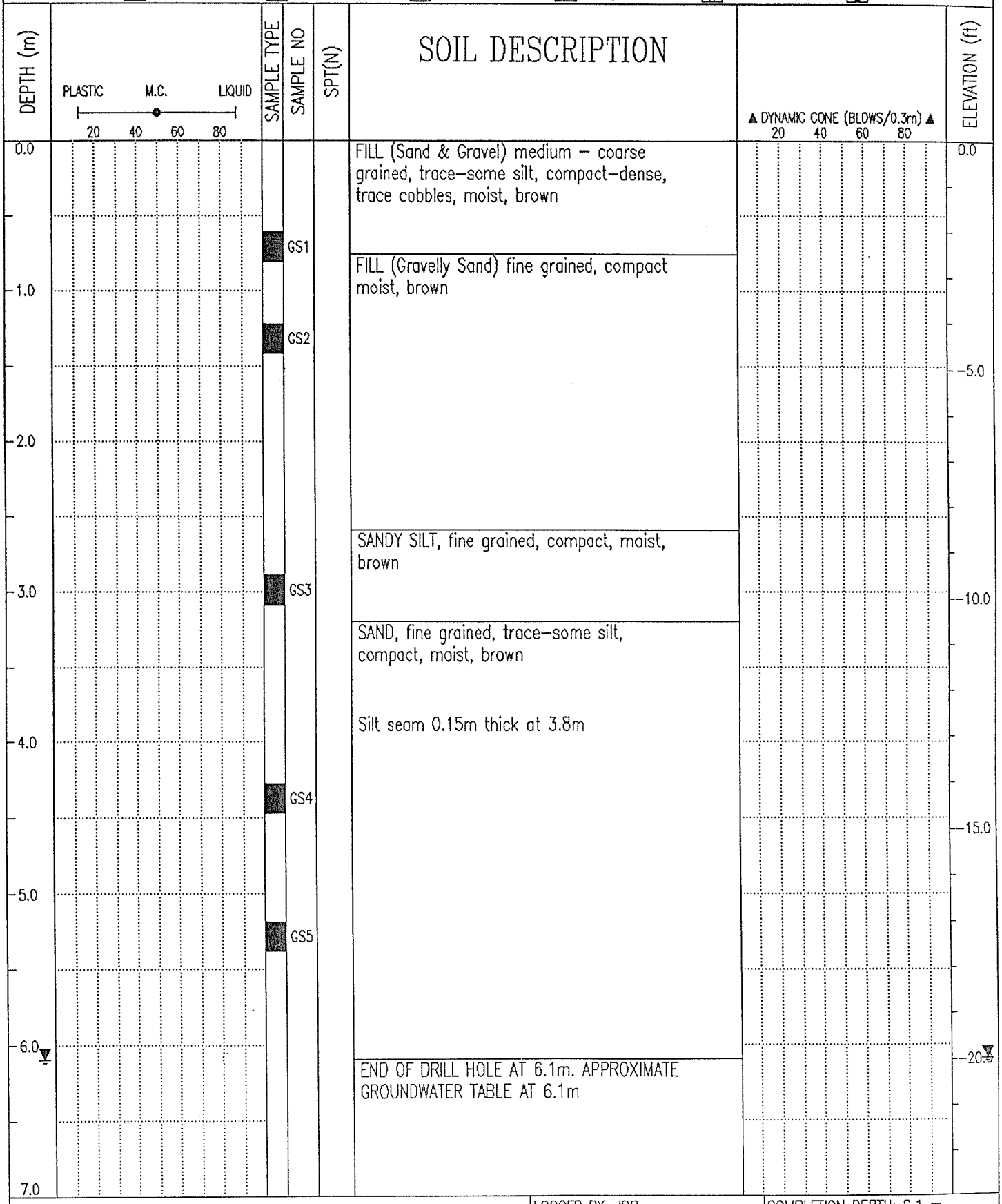
SAMPLE TYPE: Grab Sample | No Recovery | Split Spoon | Shelby Tube | Hollow Stem | Core



AMEC Earth & Environmental Limited	LOGGED BY: JDR	COMPLETION DEPTH: 6.1 m
	REVIEWED BY: AED	COMPLETE: 01/21/02
	Fig. No: 1	Page 1 of 1

Chilliwack Dyke	STA 4+500	BOREHOLE No: DH-2
DAYTON & KNIGHT		Project No: KA20481
START DATE: 01/21/02	See Drawing for Drill Hole Locations	ELEVATION: 0.00 (m)

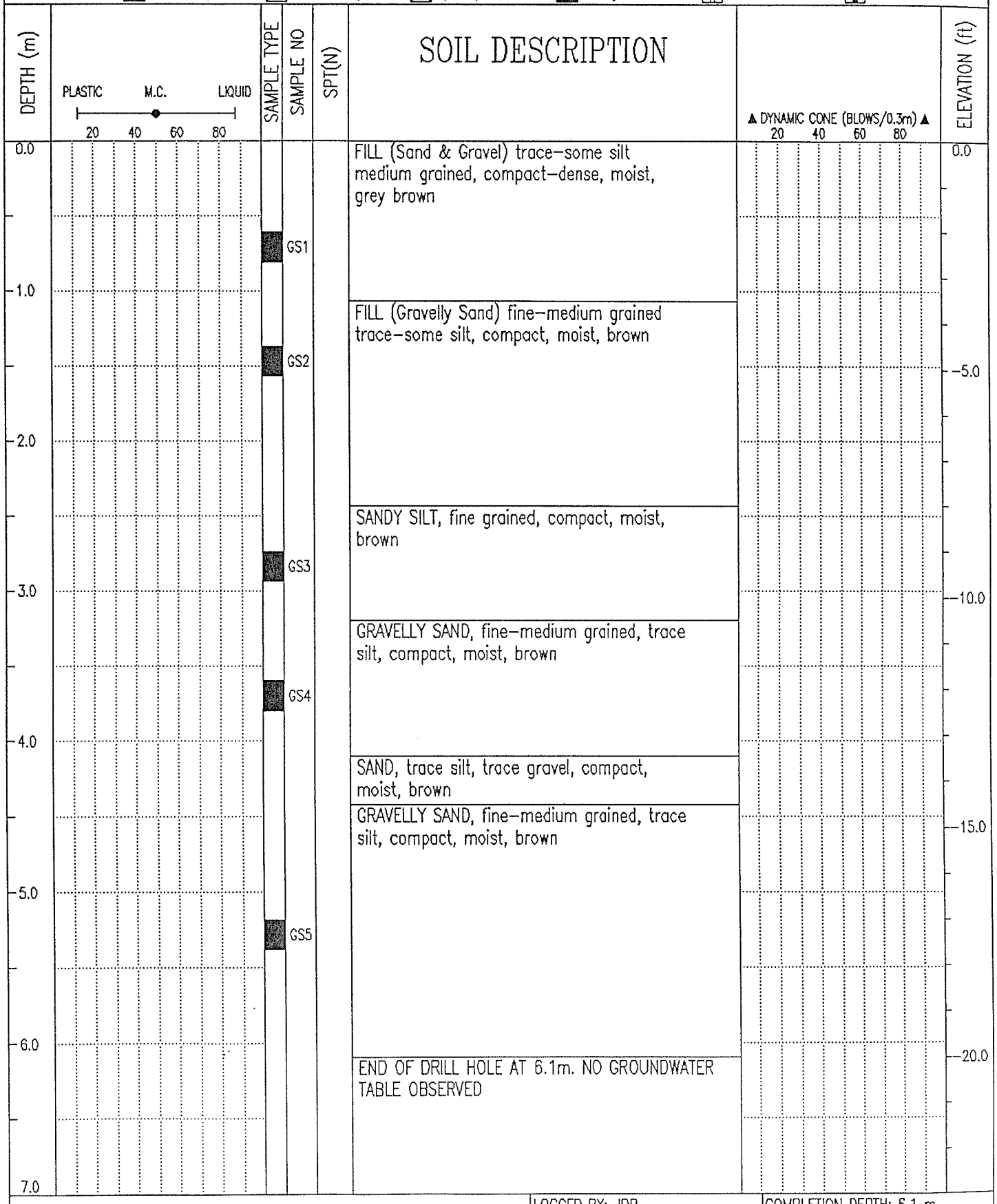
SAMPLE TYPE: Grab Sample | No Recovery | Split Spoon | Shelby Tube | Hollow Stem | Core



AMEC Earth & Environmental Limited	LOGGED BY: JDR	COMPLETION DEPTH: 6.1 m
	REVIEWED BY: AED	COMPLETE: 01/21/02
	Fig. No: 2	Page 1 of 1

Chilliwack Dyke	STA 4+000	BOREHOLE No: DH-3
DAYTON & KNIGHT		Project No: KA20481
START DATE: 01/21/02	See Drawing for Drill Hole Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample | No Recovery | Split Spoon | Shelby Tube | Hollow Stem | Core



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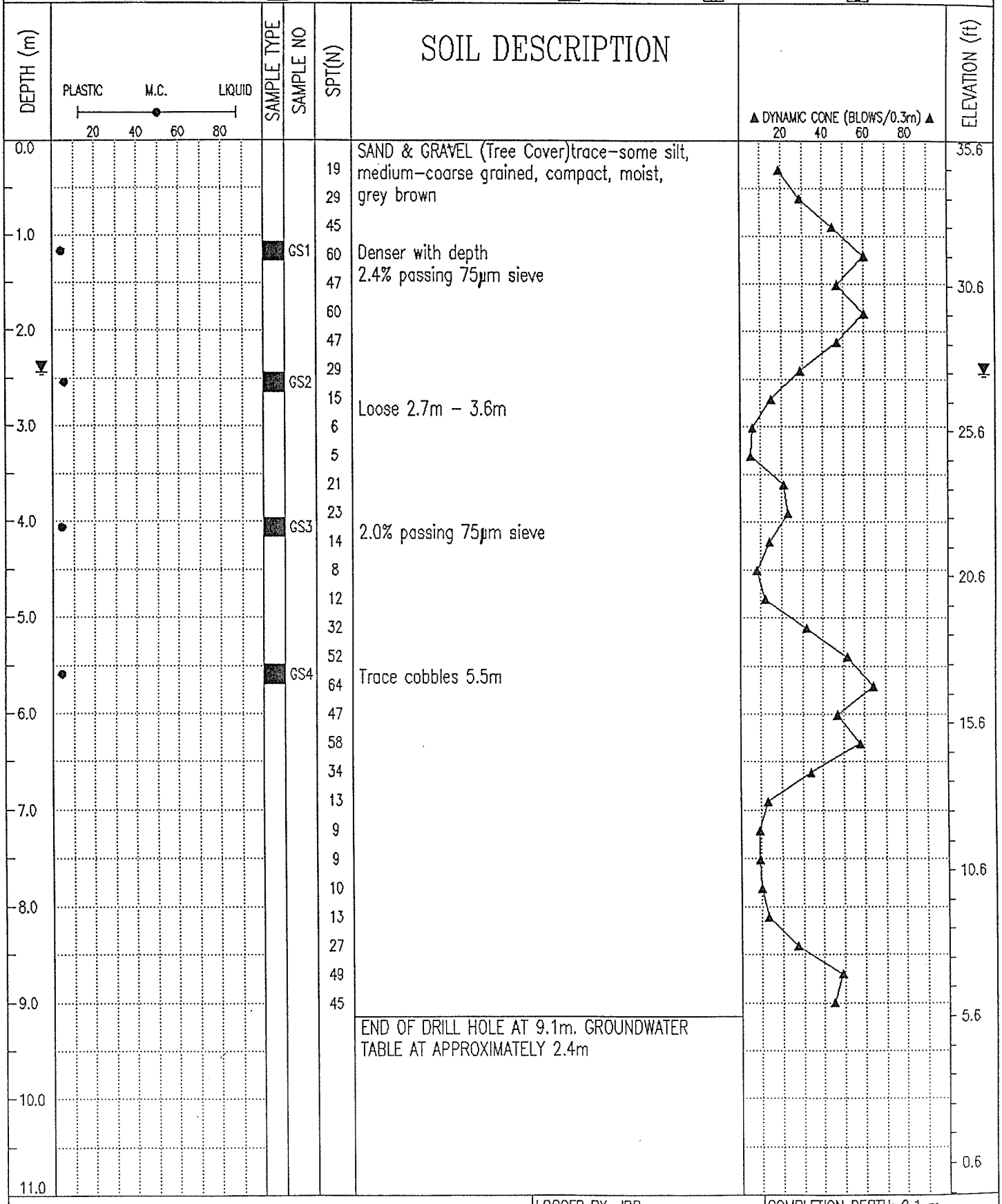
	REVIEWED BY: AED	COMPLETE: 01/21/02
	Fig. No: 3	Page 1 of 1

DYKE STABILITY ASSESSESSMENT	Nelson Slough at Carey Road	BOREHOLE No: AH-1/DCPT-1
CITY OF CHILLIWACK	STA # 9+875	Project No: KA20468
START DATE: 01/10/12	See Drawing for Drill Hole Location	ELEVATION: 10.86 (m)

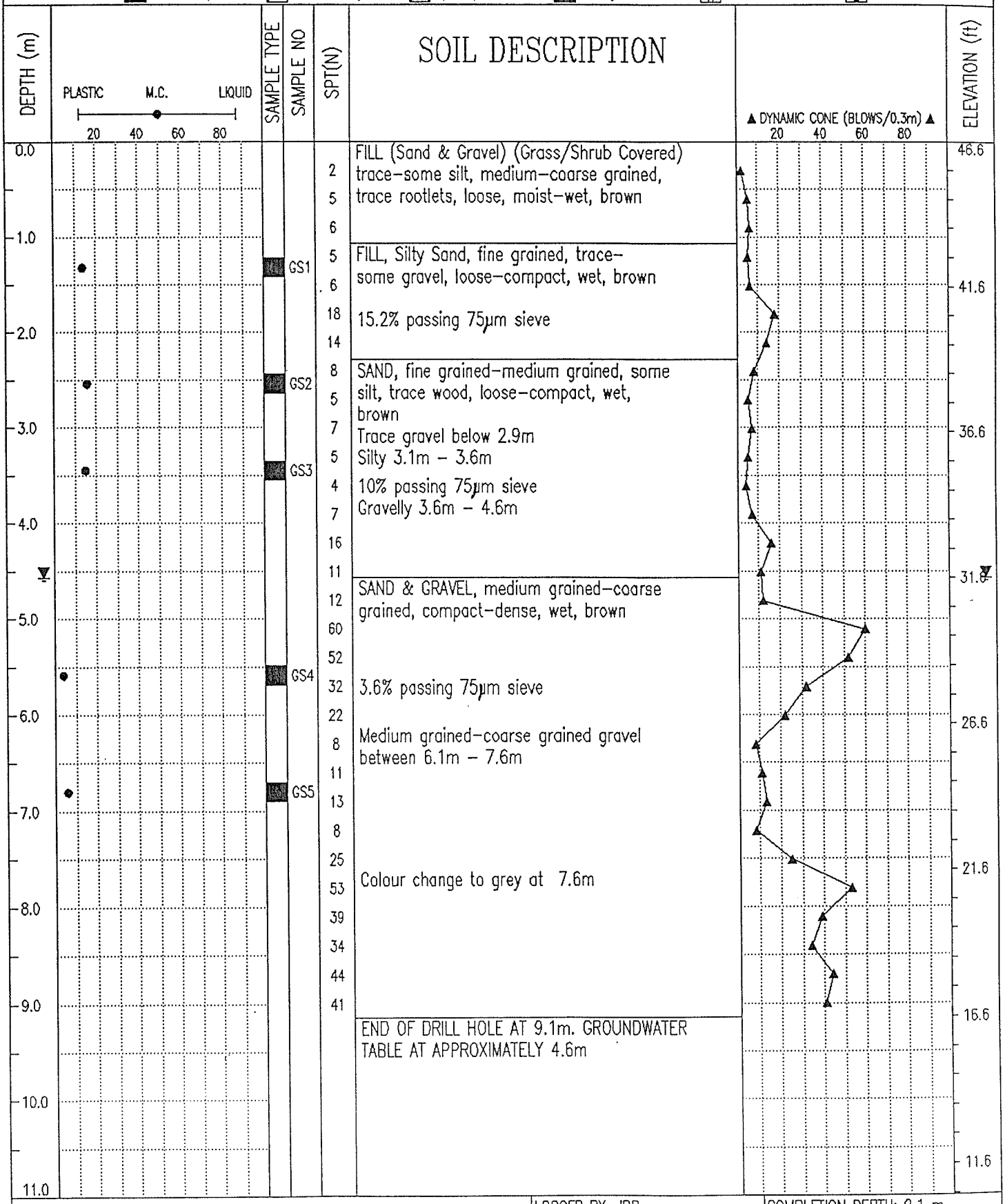
SAMPLE TYPE: Grab Sample | No Recovery | Split Spoon | Shelby Tube | Hollow Stem | Core



AMEC Earth & Environmental Limited	LOGGED BY: JDR	COMPLETION DEPTH: 9.1 m
	REVIEWED BY: AED	COMPLETE: 01/10/12
	Fig. No: 1	Page 1 of 1

DYKE STABILITY ASSESSMENT	Nelson Slough at Carey Road	BOREHOLE No: AH-2/DCPT-2
CITY OF CHILLIWACK	STA #9+875 Landside top of Slope	Project No: KA20468
START DATE: 01/10/12	See Drawing for Drill Hole Location	ELEVATION: 14.19 (m)

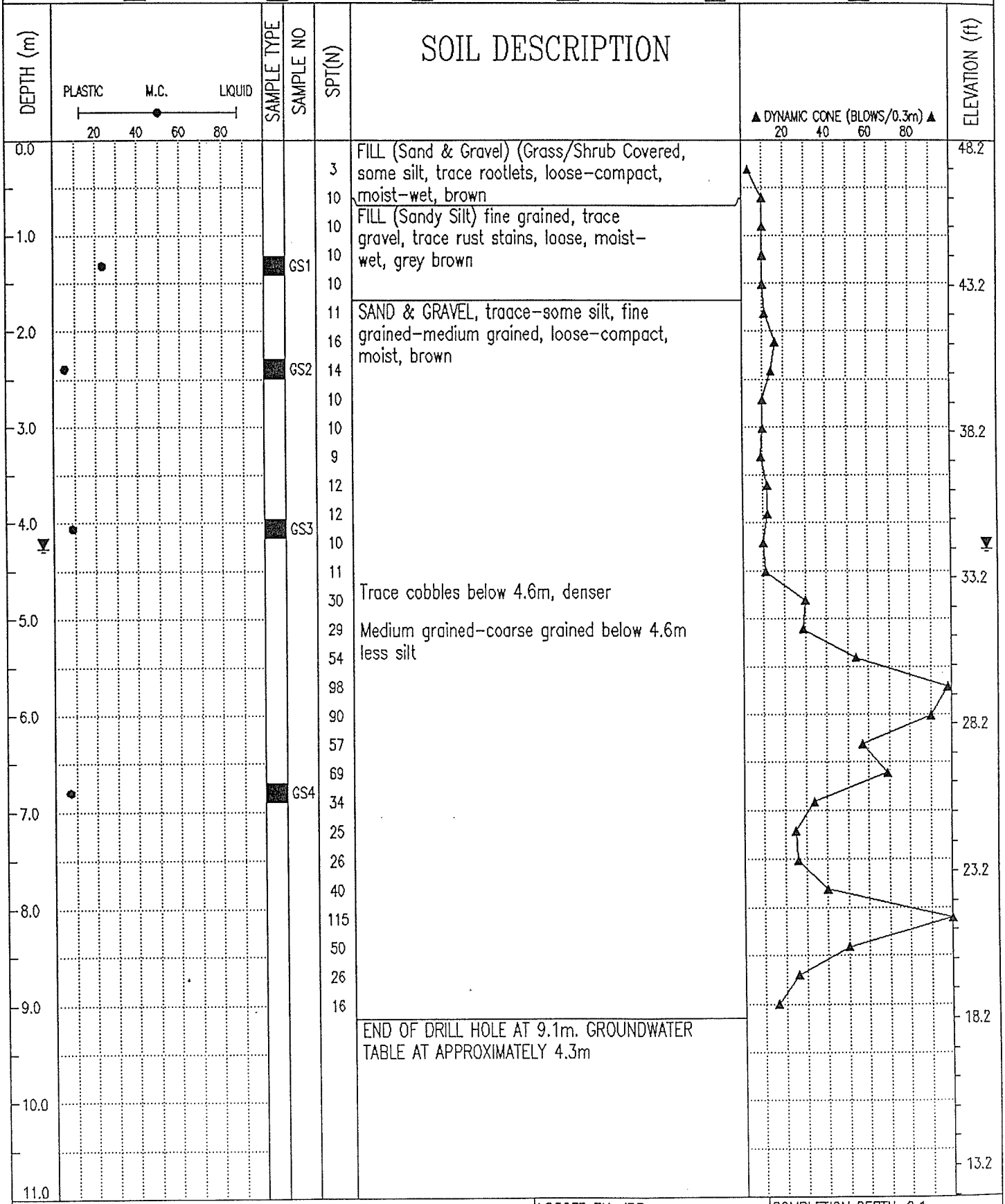
SAMPLE TYPE: Grab Sample | No Recovery | Split Spoon | Shelby Tube | Hollow Stem | Core



AMEC Earth & Environmental Limited	LOGGED BY: JDR	COMPLETION DEPTH: 9.1 m
	REVIEWED BY: AED	COMPLETE: 01/10/12
	Fig. No: 2	Page 1 of 1

DYKE STABILITY ASSESSMENT	Nelson Slough at Carey Road	BOREHOLE No: AH-3/DCPT-3
CITY OF CHILLIWACK	STA #9+875 Riverside at top of Slope	Project No: KA20468
START DATE: 01/10/12	See Drawing for Drill Hole Location	ELEVATION: 14.70 (m)

SAMPLE TYPE: Grab Sample | No Recovery | Split Spoon | Shelby Tube | Hollow Stem | Core



AMEC Earth & Environmental Limited	LOGGED BY: JDR	COMPLETION DEPTH: 9.1 m
	REVIEWED BY: AED	COMPLETE: 01/10/12
	Fig. No: 3	Page 1 of 1

APPENDIX B

AMEC Test Pit Logs

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-1
DAYTON & KNIGHT	"clean gravel" presence STA 13+500	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample; No Recovery; Split Spoon; Shelby Tube; Hollow Stem; Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SAND & GRAVEL, trace–some silt, loose–compact, rounded, trace cobbles, moist, brown		0.0
			GS1		11.1% passing 75mm sieve		
					END OF TEST PIT AT 0.8m.		
1.0							
							-5.0
2.0							

AMEC Earth & Environmental Limited

LOGGED BY: JDR	COMPLETION DEPTH: 0.8 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 1	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-2
DAYTON & KNIGHT	"clean gravel" presence STA 13+250	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE ■ Grab Sample ⊘ No Recovery ⊠ Split Spoon ▤ Shelby Tube ▥ Hollow Stem ▮ Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					Crushed Gravel surface, grey		0.0
			GS1		SAND & GRAVEL, trace-some silt, loose-compact, rounded, trace cobbles, moist, brown		
			GS2		SAND & GRAVEL, trace silt, rounded, loose-compact, moist, brown 7.2% passing 75μm sieve		
1.0					END OF TEST PIT AT 0.9m		-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 0.9 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 2	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-3
DAYTON & KNIGHT	"clean gravel" presence STA 13+000	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample | No Recovery | Split Spoon | Shelby Tube | Hollow Stem | Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					Crushed Gravel surface, grey		0.0
			GS1		SAND & GRAVEL, trace–some silt, compact, trace organics, moist, brown		
					SAND & GRAVEL, trace silt, compact, loose–compact, moist, brown		
1.0					END OF TEST PIT AT 0.9m. GS2 @ 0.3m DEPTH FROM RIVERSIDE ~~SILT BLANKET~~ Toe		–5.0
2.0							

AMEC Earth & Environmental Limited

LOGGED BY: JDR	COMPLETION DEPTH: 0.9 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 3	Page 1 of 1

Chilliwack Dyke	Test Pits on Landside to assess "Relief	BOREHOLE No: TP-4
DAYTON & KNIGHT	Pit" presence STA 13+000	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample | No Recovery | Split Spoon | Shelby Tube | Hollow Stem | Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SILT sandy, trace to some gravel, some roots/rootlets		0.0
			GS1				
					SILTY SAND & GRAVEL (Trace Rounded Gravel) some organics		
			GS2				
					SILT, some fine grained sand, mottled		
					Sandier with depth		
			GS3		No evidence of Relief Pit		
1.0					END OF TEST PIT AT 1.0m		
							-5.0
2.0							

AMEC Earth & Environmental Limited

LOGGED BY: JDR	COMPLETION DEPTH: 1.0 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 4	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-5
DAYTON & KNIGHT	"clean gravel" presence STA 12+750	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample; No Recovery; Split Spoon; Shelby Tube; Hollow Stem; Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SAND & GRAVEL, trace-some silt, rounded, trace rootlets, compact, moist, brown		0.0
			GS1				
					SAND, fine grained, some silt, compact, moist, brown		
			GS2				
1.0					END OF TEST PIT AT 1.0m		-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 1.0 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 5	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-6
DAYTON & KNIGHT	"clean gravel" presence STA 12+500	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE ■ Grab Sample ⊘ No Recovery ⊠ Split Spoon ▤ Shelby Tube ▥ Hollow Stem ▮ Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SAND & GRAVEL, trace-some silt, trace rootlets, compact, moist, brown		0.0
		■	GS1				
1.0					END OF TEST PIT AT 0.8m		-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 0.8 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 6	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-7
DAYTON & KNIGHT	"clean gravel" presence STA 12+250	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample | No Recovery | Split Spoon | Shelby Tube | Hollow Stem | Core

DEPTH (m)	PLASTIC / M.C. / LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SAND & GRAVEL, trace-some silt, trace rootlets, compact, moist, brown		0.0
			GS1				
					SAND & GRAVEL, trace silt, compact, moist, brown		
			GS2				
					END OF TEST PIT AT 0.8m		
1.0							
							-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 0.8 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 7	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-8
DAYTON & KNIGHT	"clean gravel" presence STA 11+940	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample; No Recovery; Split Spoon; Shelby Tube; Hollow Stem; Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0			GS1		SAND & GRAVEL, trace-some silt, trace rootlets & roots, trace cobbles, compact, moist, brown 2.7% passing 75μm sieve		0.0
					SAND & SILT, compact, moist, brown		
1.0			GS2		48% passing 75μm sieve		
					END OF TEST PIT AT 1.3m		-5.0
2.0							

AMEC Earth & Environmental Limited

LOGGED BY: JDR	COMPLETION DEPTH: 1.3 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 8	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-9
DAYTON & KNIGHT	"clean gravel" presence STA 11+900	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample; No Recovery; Split Spoon; Shelby Tube; Hollow Stem; Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					GRAVELLY SAND, some silt, trace rootlets, compact, moist, brown 14.1% passing 75μm sieve		0.0
			GS1				
					SAND & GRAVEL, trace silt, trace cobbles, compact, moist, brown		
			GS2				
1.0					END OF TEST PIT AT 1.0m. GS3 @ TOE ON LAND SIDE, 2.6% PASSING 75μm SIEVE (SAND, FINE GRAINED, TRACE SILT)		
							-5.0
2.0							

AMEC Earth & Environmental Limited

LOGGED BY: JDR	COMPLETION DEPTH: 1.0 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 9	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-10
DAYTON & KNIGHT	"clean gravel" presence STA 11+750	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample; No Recovery; Split Spoon; Shelby Tube; Hollow Stem; Core

DEPTH (m)	PLASTIC / M.C. / LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0			GS1		GRAVELLY SAND, some silt, trace rootlets, compact, moist, brown		0.0
					END OF TEST PIT AT 0.9m		
1.0							
							-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 0.9 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 10	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-11
DAYTON & KNIGHT	"clean gravel" presence STA 11+500	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample; No Recovery; Split Spoon; Shelby Tube; Hollow Stem; Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SAND & GRAVEL, trace-some silt, trace rootlets, compact, moist, brown		0.0
			GS1				
					GRAVELLY SAND, some silt, trace cobbles, compact, moist, brown		
			GS2				
					END OF TEST PIT AT 0.9m		
1.0							
							-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 0.9 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 11	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-12
DAYTON & KNIGHT	"clean gravel" presence STA 11+250	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample; No Recovery; Split Spoon; Shelby Tube; Hollow Stem; Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SAND & GRAVEL, trace-some silt, trace rootlets, compact, moist, brown		0.0
			GS1				
					GRAVELLY SAND, some silt, trace cobbles, compact, moist, brown		
1.0					END OF TEST PIT AT 0.9m		-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 0.9 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 12	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-13
DAYTON & KNIGHT	"clean gravel" presence STA 11+000	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample; No Recovery; Split Spoon; Shelby Tube; Hollow Stem; Core

DEPTH (m)	PLASTIC / M.C. / LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SAND & GRAVEL, trace-some silt, trace rootlets, compact, moist, brown		0.0
			GS1				
					SILT, some fine grained sand, firm, moist, brown		
			GS2		SAND & GRAVEL, some silt, medium grained, compact, moist, brown		
1.0					END OF TEST PIT AT 1.0m		-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 1.0 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 13	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-14
DAYTON & KNIGHT	"clean gravel" presence STA 10+750	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample; No Recovery; Split Spoon; Shelby Tube; Hollow Stem; Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SAND & GRAVEL, trace–some silt, trace rootlets, trace cobbles, compact, moist, brown		0.0
			GS1				
					SAND, fine grained, some silt, trace gravel, compact, moist, brown		
			GS2				
1.0					END OF TEST PIT AT 1.0m		
							-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 1.0 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 14	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-15
DAYTON & KNIGHT	"clean gravel" presence STA 10+500	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample; No Recovery; Split Spoon; Shelby Tube; Hollow Stem; Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SAND & GRAVEL, trace-some silt, trace rootlets, trace cobbles, compact, moist, brown		0.0
		Grab	GS1				
1.0					SAND, fine grained, some silt, trace gravel, compact, moist, brown		
					END OF TEST PIT AT 1.0m		-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 1.0 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 15	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-16
DAYTON & KNIGHT	"clean gravel" presence STA 10+250	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample | No Recovery | Split Spoon | Shelby Tube | Hollow Stem | Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SAND & GRAVEL, trace–some silt, trace rootlets, trace cobbles, compact, moist, brown		0.0
			GS1				
1.0					SAND, fine grained, some silt, trace gravel, compact, moist, brown		
					END OF TEST PIT AT 1.2m		
							-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 1.2 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 16	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-17
DAYTON & KNIGHT	"clean gravel" presence STA 10+000	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample; No Recovery; Split Spoon; Shelby Tube; Hollow Stem; Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					Asphalt Covered Surface		0.0
			GS1		Crushed Granular Base, 19mm minus, compact-dense, moist, brown		
					SAND & GRAVEL, trace-some silt, compact, trace cobbles, moist, brown		
1.0					END OF TEST PIT AT 0.8m		-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 0.8 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 17	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-18
DAYTON & KNIGHT	"clean gravel" presence STA 9+735	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE ■ Grab Sample ⊘ No Recovery ⊠ Split Spoon ▤ Shelby Tube ▥ Hollow Stem ▮ Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					0.3m Crushed Gravel Surface SAND & GRAVEL, tracesilt, trace cobbles, roots & rootlets, compact, moist, brown		0.0
		Grab	GS1		1.0% passing 75μm sieve		
1.0					END OF TEST PIT AT 1.0m		-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 1.0 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 18	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-19
DAYTON & KNIGHT	"clean gravel" presence STA 9+500	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE: Grab Sample | No Recovery | Split Spoon | Shelby Tube | Hollow Stem | Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SAND & GRAVEL, trace silt, trace cobbles, roots & rootlets, compact, moist, brown		0.0
			GS1		1.6% passing 75µm sieve		
					GRAVELLY SAND, fine grained, trace–some silt, compact, moist, brown		
1.0					END OF TEST PIT AT 0.9m		
							-5.0
2.0							

AMEC Earth & Environmental Limited

LOGGED BY: JDR	COMPLETION DEPTH: 0.9 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 19	Page 1 of 1

Chilliwack Dyke	Test Pits on Riverside Crest to assess	BOREHOLE No: TP-20
DAYTON & KNIGHT	"clean gravel" presence STA 9+250	Project No: KA20481
START DATE: 01/14/02	See Drawing for Test Pit Locations	ELEVATION: 0.00 (m)

SAMPLE TYPE ■ Grab Sample ⊘ No Recovery ⊠ Split Spoon ▤ Shelby Tube ▥ Hollow Stem ▮ Core

DEPTH (m)	PLASTIC — M.C. — LIQUID (20 40 60 80)	SAMPLE TYPE	SAMPLE NO	SPT(N)	SOIL DESCRIPTION	▲ DYNAMIC CONE (BLOWS/0.3m) ▲ (20 40 60 80)	ELEVATION (ft)
0.0					SAND & GRAVEL, trace-some silt, trace rootlets, trace cobbles, compact, moist, brown		0.0
		■	GS1				
					GRAVELLY SAND, fine grained, trace-some silt, compact, moist, brown		
		■	GS2				
1.0					END OF TEST PIT AT 0.9m		
							-5.0
2.0							

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LOGGED BY: JDR	COMPLETION DEPTH: 0.9 m
REVIEWED BY: AED	COMPLETE: 01/14/02
Fig. No: 20	Page 1 of 1

APPENDIX C

Grain Size Distribution Test Plots

Brian Walker, P. Eng.

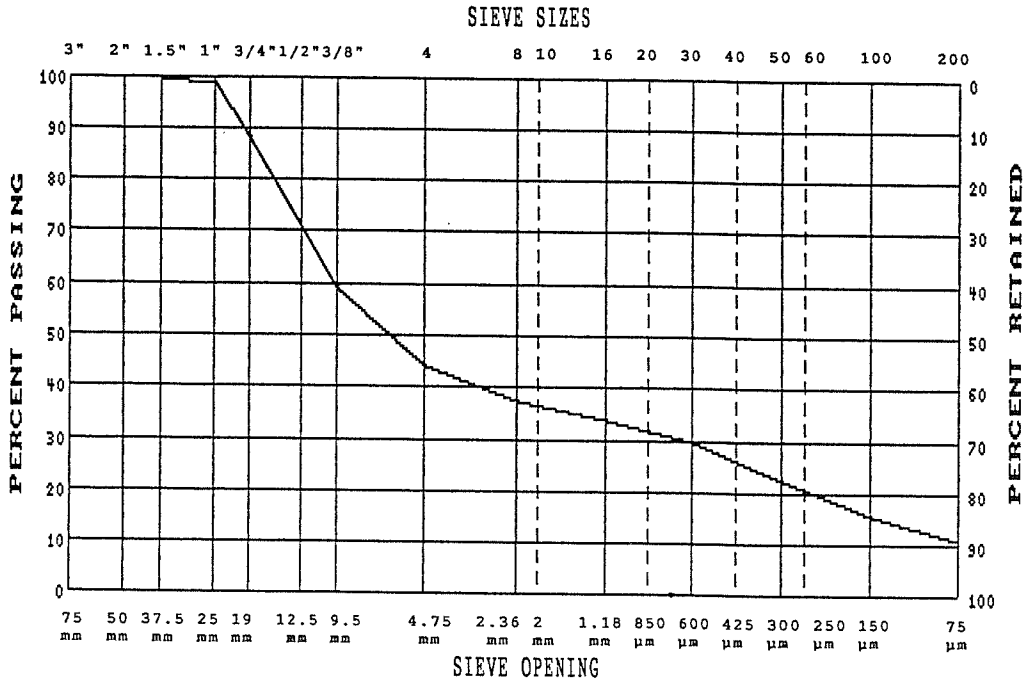
Dayton & Knight
612 Clyde Avenue
West Vancouver, BC V7V 3N9

PROJECT NO: KA20481
DATE: 02.Jan.24
CLIENT P.O.:
CC:

PROJECT: Chilliwack Dyke
Chilliwack, BC

TESTNO: L2035 SOURCE: TP#1, GS#1 SAMPLE TYPE: bag

DATE SAMPLED: 02.Jan.18 BY: J. David Ross DATE TESTED: 02.Jan.24



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm		
1 1/2"	37.5	mm	100.0	
1"	25	mm	98.9	
3/4"	19	mm	88.3	
1/2"	12.5	mm	70.8	
3/8"	9.5	mm	59.5	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No. 4	4.75	mm	44.5	
No. 8	2.36	mm	37.5	
No. 16	1.18	mm	34.0	
No. 30	600	µm	29.9	
No. 50	300	µm	22.5	
No. 100	150	µm	15.6	
No. 200	75	µm	11.1	

COMMENTS: Sand & Gravel, Trace-Some Silt
STA 13+500

AMEC - PER:

Reporting of these test results constitutes a testing service only.
Engineering interpretation or evaluation of the test results is provided only on written request.

Hamid Yousefbeigi

AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Brian Walker, P. Eng.

Dayton & Knight
612 Clyde Avenue
West Vancouver, BC V7V 3N9

PROJECT NO: KA20481
DATE: 02.Jan.24
CLIENT P.O.:
CC:

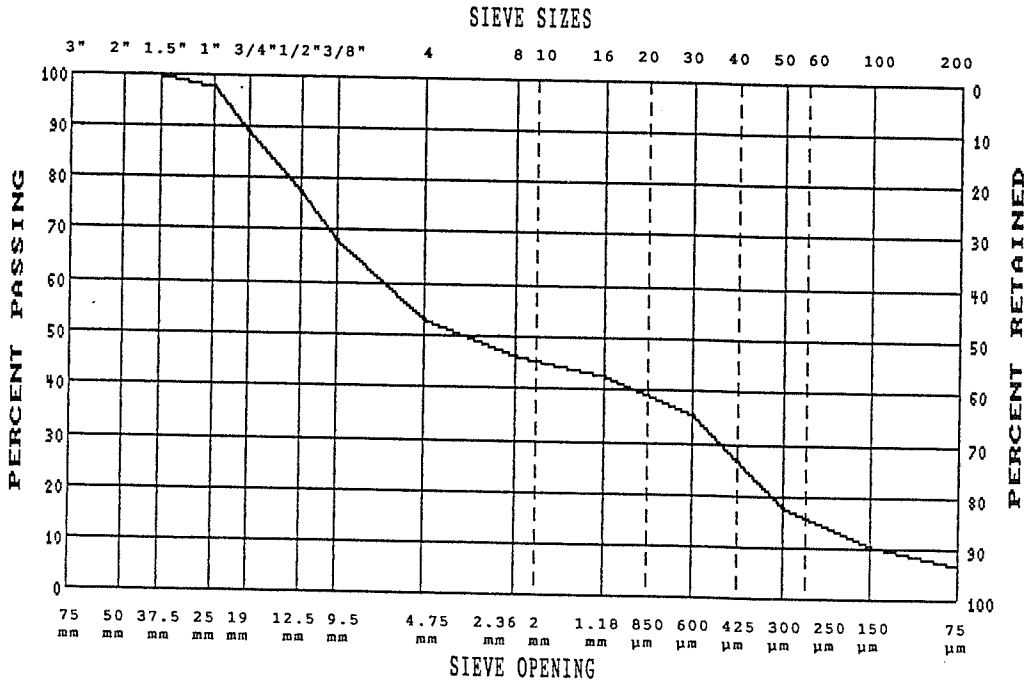
PROJECT: Chilliwack Dyke
Chilliwack, BC

TESTNO: L2035 SOURCE: TP#2, GS#2

SAMPLE TYPE: bag

DATE SAMPLED: 02.Jan.18 BY: J. David Ross

DATE TESTED: 02.Jan.24



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm		
1 1/2"	37.5	mm	100.0	
1"	25	mm	97.6	
3/4"	19	mm	88.7	
1/2"	12.5	mm	77.6	
3/8"	9.5	mm	68.2	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No.	4	4.75 mm	53.3	
No.	8	2.36 mm	46.4	
No.	16	1.18 mm	42.6	
No.	30	600 µm	35.6	
No.	50	300 µm	18.0	
No.	100	150 µm	10.8	
No.	200	75 µm	7.2	

COMMENTS: Sand & Gravel, Trace Silt
STA 13+250

AMEC - PER:

Hamid Yousefbeigi

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Engineering interpretation or evaluation of the test results is provided only on written request.

AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Brian Walker, P. Eng.

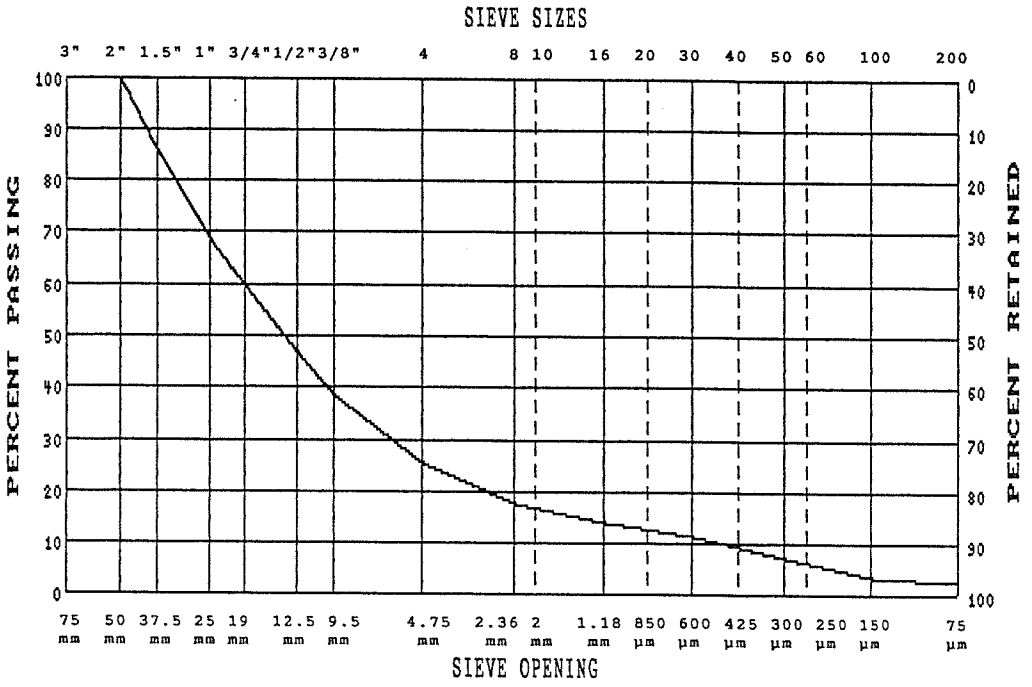
Dayton & Knight
612 Clyde Avenue
West Vancouver, BC V7V 3N9

PROJECT NO: KA20481
DATE: 02.Jan.24
CLIENT P.O.:
CC:

PROJECT: Chilliwack Dyke
Chilliwack, BC

TESTNO: L2035 SOURCE: TP#8, GS#1 SAMPLE TYPE: bag

DATE SAMPLED: 02.Jan.18 BY: J. David Ross DATE TESTED: 02.Jan.24



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm	100.0	
1 1/2"	37.5	mm	85.9	
1"	25	mm	68.9	
3/4"	19	mm	59.6	
1/2"	12.5	mm	47.1	
3/8"	9.5	mm	38.4	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No.	4	4.75 mm	25.4	
No.	8	2.36 mm	17.6	
No.	16	1.18 mm	14.4	
No.	30	600 µm	11.5	
No.	50	300 µm	7.3	
No.	100	150 µm	4.0	
No.	200	75 µm	2.7	

COMMENTS: Sand & Gravel, Trace Silt
STA 11+940

AMEC - PER:

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Engineering interpretation or evaluation of the test results is provided only on written request.

Hamid Yousefbeigi

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Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Brian Walker, P.Eng.

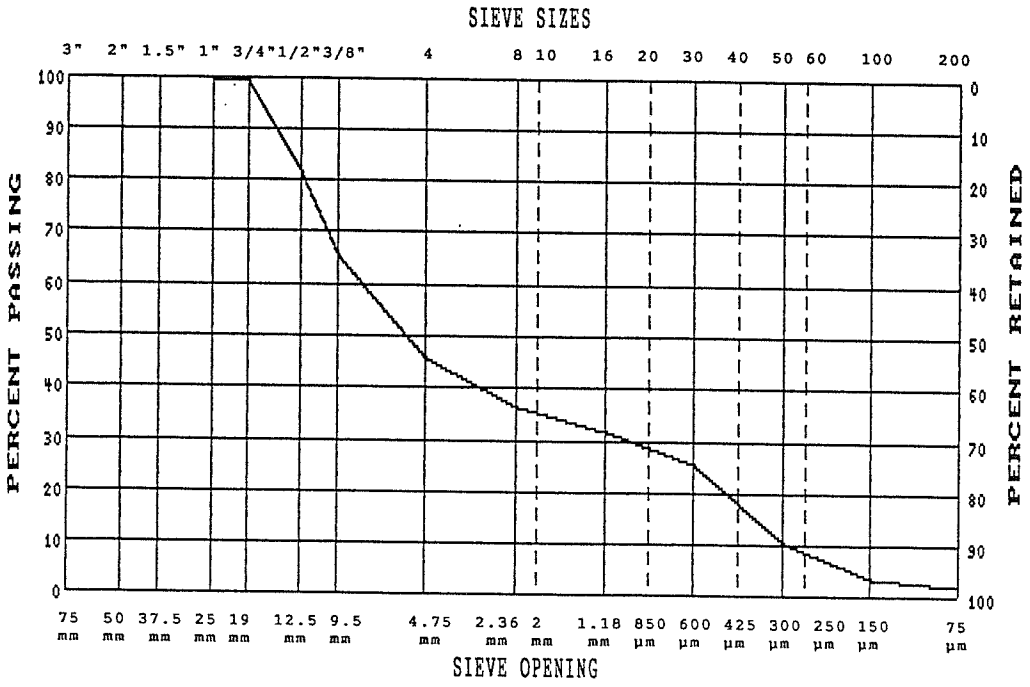
Dayton & Knight
612 Clyde Avenue
West Vancouver, BC V7V 3N9

PROJECT NO: KA20481
DATE: 02.Jan.24
CLIENT P.O.:
CC:

PROJECT: Chilliwack Dyke
Chilliwack, BC

TESTNO: L2035 SOURCE: TP#22, GS#1 SAMPLE TYPE: pail

DATE SAMPLED: 02.Jan.18 BY: J. David Ross DATE TESTED: 02.Jan.24



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm		
1 1/2"	37.5	mm		
1"	25	mm	100.0	
3/4"	19	mm	99.5	
1/2"	12.5	mm	81.8	
3/8"	9.5	mm	65.7	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No. 4	4.75	mm	45.9	
No. 8	2.36	mm	36.4	
No. 16	1.18	mm	31.9	
No. 30	600	µm	25.7	
No. 50	300	µm	10.5	
No. 100	150	µm	3.6	
No. 200	75	µm	2.1	

COMMENTS: Sand & Gravel, Trace Silt, Relief Pit Filter Gravel
STA 11+025

AMEC - PER:

Reporting of these test results constitutes a testing service only.
Engineering interpretation or evaluation of the test results is provided only on written request.

Hamid Yousefbeigi

AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Brian Walker, P.Eng.

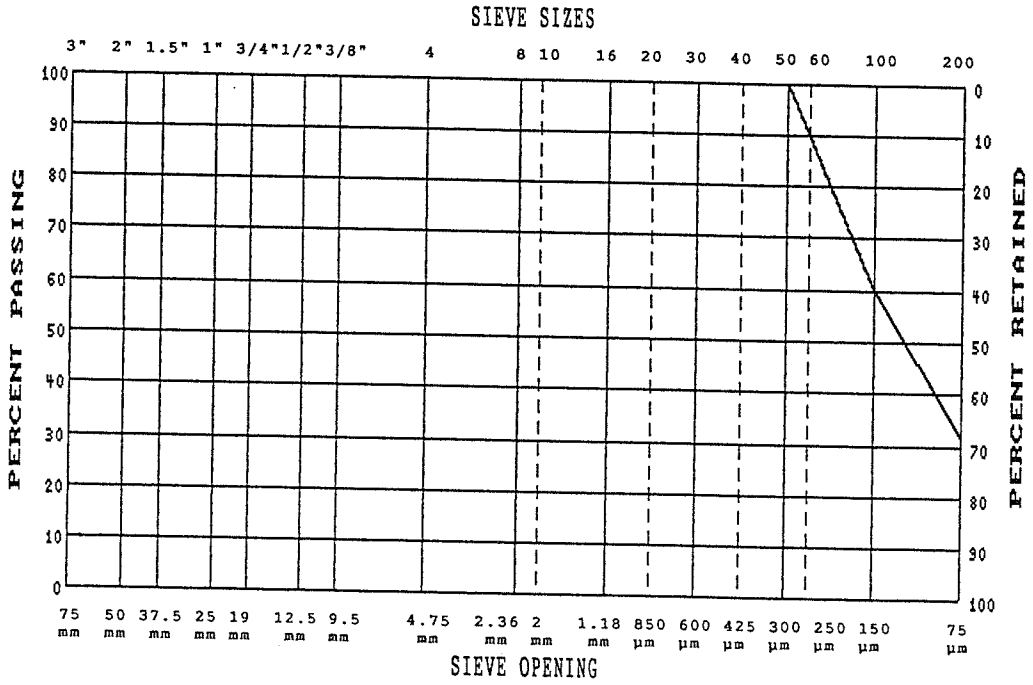
Dayton & Knight
612 Clyde Avenue
West Vancouver, BC V7V 3N9

PROJECT NO: KA20481
DATE: 02.Jan.24
CLIENT P.O.:
CC:

PROJECT: Chilliwack, Dyke
Chilliwack, BC

TESTNO: L2035 SOURCE: TP#25, GS#3 SAMPLE TYPE: pail

DATE SAMPLED: 02.Jan.18 BY: J. David Ross DATE TESTED: 02.Jan.24



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm		
1 1/2"	37.5	mm		
1"	25	mm		
3/4"	19	mm		
1/2"	12.5	mm		
3/8"	9.5	mm		

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No. 4	4.75	mm	100.0	
No. 8	2.36	mm	100.0	
No. 16	1.18	mm	99.9	
No. 30	600	µm	99.8	
No. 50	300	µm	99.6	
No. 100	150	µm	59.6	
No. 200	75	µm	32.0	

COMMENTS: Silty Sand
STA 4+490

AMEC - PER:

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Hamid Yousefbeigi

AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Brian Walker, P. Eng.

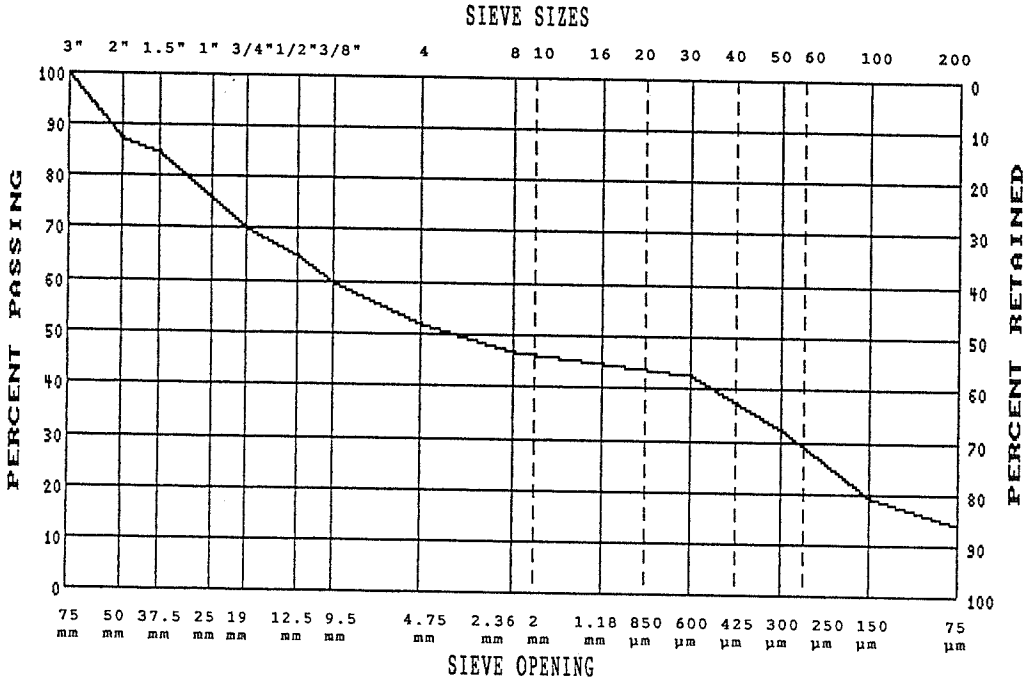
Dayton & Knight
612 Clyde Avenue
West Vancouver, BC V7V 3N9

PROJECT NO: KA20481
DATE: 02.Jan.24
CLIENT P.O.:
CC:

PROJECT: Chilliwack Dyke
Chilliwack, BC

TESTNO: L2035 SOURCE: TP#9, GS#1 SAMPLE TYPE: bag

DATE SAMPLED: 02.Jan.18 BY: J. David Ross DATE TESTED: 02.Jan.24



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm	100.0	
2"	50	mm	87.5	
1 1/2"	37.5	mm	84.5	
1"	25	mm	75.7	
3/4"	19	mm	70.4	
1/2"	12.5	mm	64.8	
3/8"	9.5	mm	60.1	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No. 4	4.75	mm	52.1	
No. 8	2.36	mm	46.9	
No. 16	1.18	mm	44.7	
No. 30	600	µm	42.5	
No. 50	300	µm	32.6	
No. 100	150	µm	19.3	
No. 200	75	µm	14.1	

COMMENTS: Sand & Gravel, some silt
STA 11+900

AMEC - PER:

Hamid Yousefbeigi

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Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Brian Walker, P.Eng.

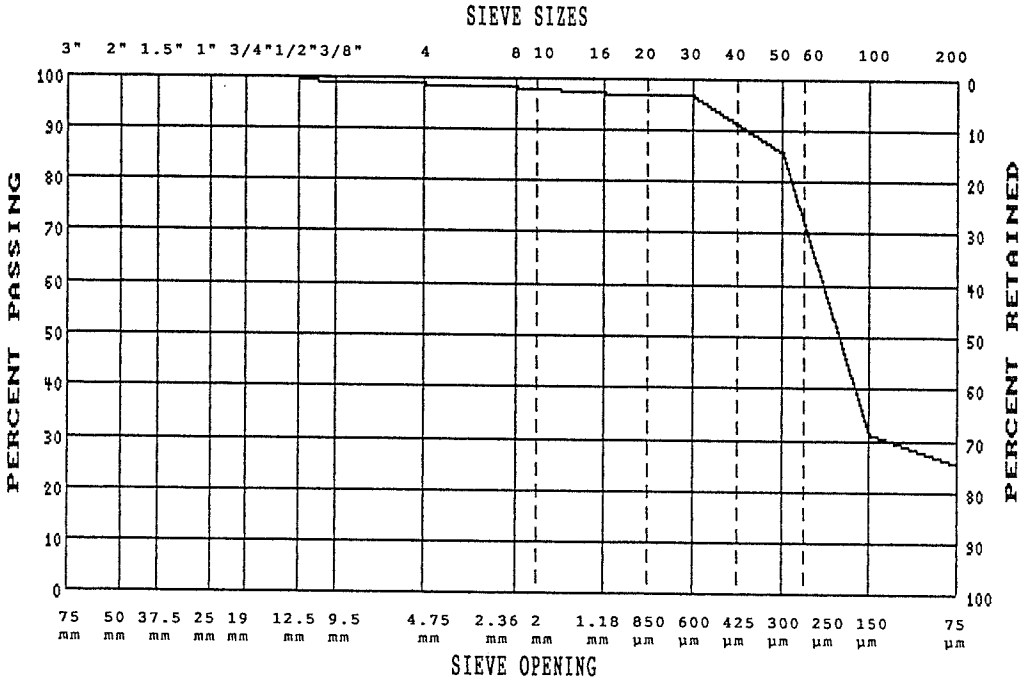
Dayton & Knight
612 Clyde Avenue
West Vancouver, BC V7V 3N9

PROJECT NO: KA20481
DATE: 02.Jan.24
CLIENT P.O.:
CC:

PROJECT: Chilliwack Dyke
Chilliwack, BC

TESTNO: L2035 SOURCE: TP#9, GS#3 SAMPLE TYPE: pail

DATE SAMPLED: 02.Jan.18 BY: J.David Ross DATE TESTED: 02.Jan.24



GRAVEL SIZES		PERCENT PASSING	GRADATION LIMITS
3"	75 mm		
2"	50 mm		
1 1/2"	37.5 mm		
1"	25 mm		
3/4"	19 mm		
1/2"	12.5 mm	100.0	
3/8"	9.5 mm	98.8	

SAND SIZES AND FINES		PERCENT PASSING	GRADATION LIMITS
No. 4	4.75 mm	98.6	
No. 8	2.36 mm	98.4	
No. 16	1.18 mm	97.1	
No. 30	600 µm	96.6	
No. 50	300 µm	85.6	
No. 100	150 µm	31.1	
No. 200	75 µm	25.6	

COMMENTS: Silty Sand
STA 11+900

AMEC - PER:

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AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Brian Walker

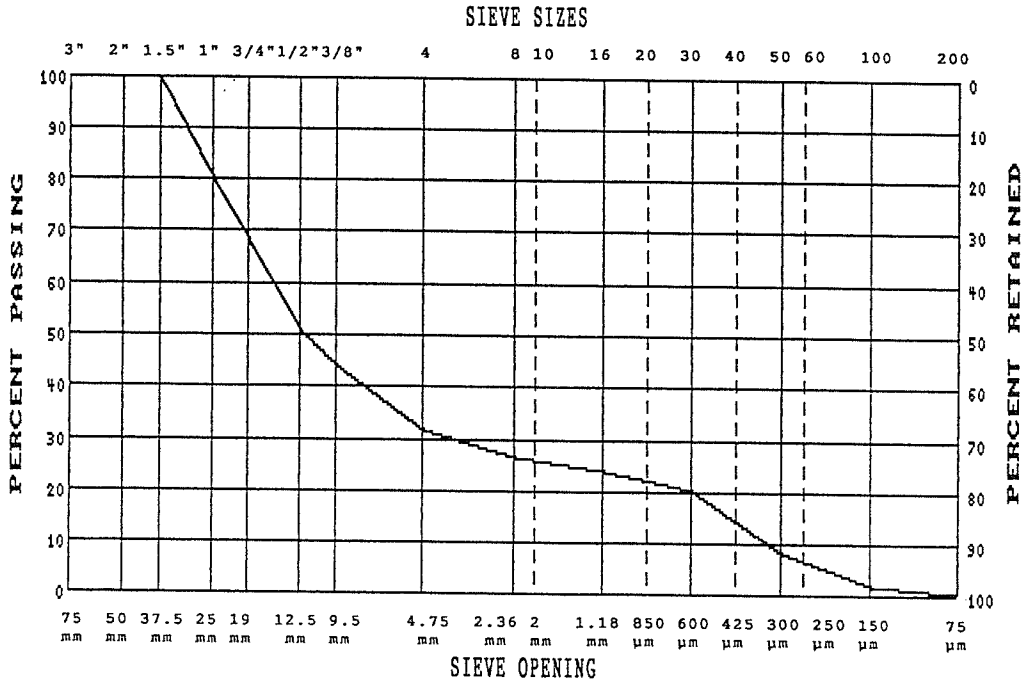
Dayton & Knight
612 Clyde Avenue
West Vancouver, BC V7V 3N9

PROJECT NO: KA20481
DATE: 02.Jan.30
CLIENT P.O.:
CC:

PROJECT: Chilliwack Dyke
Chilliwack, BC

TESTNO: L2035 SOURCE: TP#18, GS#1 SAMPLE TYPE: pail

DATE SAMPLED: 01.Jan.18 BY: J. David Ross DATE TESTED: 01.Jan.24



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm		
1 1/2"	37.5	mm	100.0	
1"	25	mm	80.5	
3/4"	19	mm	68.7	
1/2"	12.5	mm	50.8	
3/8"	9.5	mm	44.1	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No. 4	4.75	mm	32.0	
No. 8	2.36	mm	26.7	
No. 16	1.18	mm	24.2	
No. 30	600	µm	20.4	
No. 50	300	µm	8.6	
No. 100	150	µm	2.3	
No. 200	75	µm	1.0	

COMMENTS: Sand & Gravel, Trace Silt
STA 10+735

AMEC - PER:

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AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Brian Walker

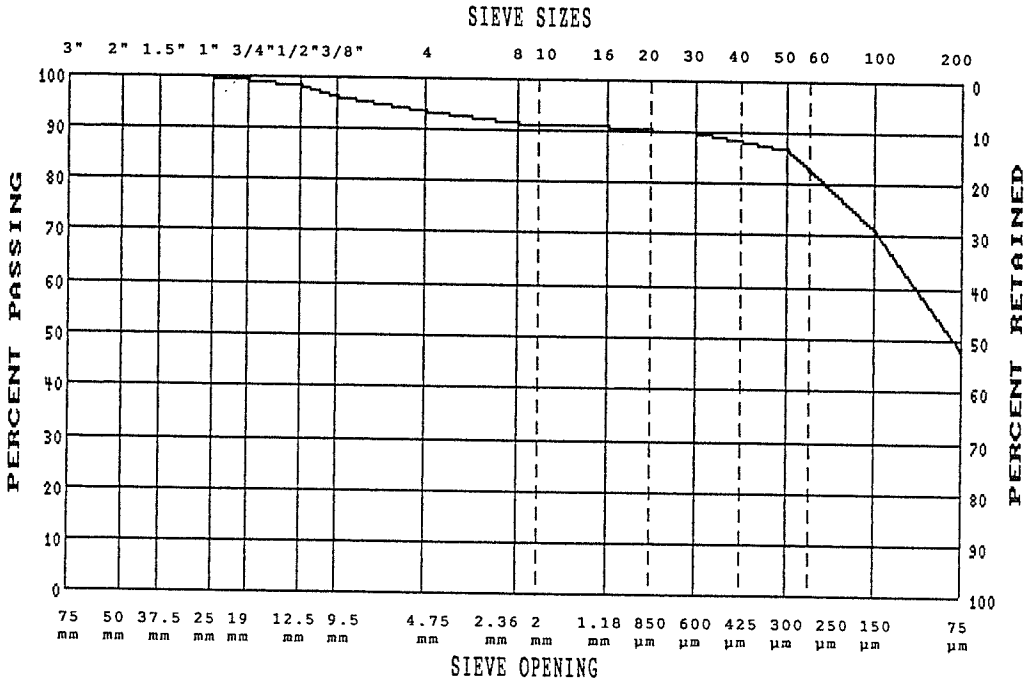
Dayton & Knight
612 Clyde Avenue
West Vancouver, BC V7V 3N9

PROJECT NO: KA20481
DATE: 02.Jan.30
CLIENT P.O.:
CC:

PROJECT: Chilliwack Dyke
Chilliwack, BC

TESTNO: L2035 SOURCE: TP#8, GS#2 SAMPLE TYPE: pail

DATE SAMPLED: 02.Jan.18 BY: J. David Ross DATE TESTED: 01.Jan.24



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm		
1 1/2"	37.5	mm		
1"	25	mm	100.0	
3/4"	19	mm	99.4	
1/2"	12.5	mm	98.0	
3/8"	9.5	mm	96.4	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No.	4	4.75 mm	93.3	
No.	8	2.36 mm	91.6	
No.	16	1.18 mm	90.8	
No.	30	600 µm	89.7	
No.	50	300 µm	86.7	
No.	100	150 µm	71.2	
No.	200	75 µm	48.0	

COMMENTS: Sand & Silt, Trace Gravel
STA 11+940

AMEC - PER:

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AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Brian Walker, P.Eng.

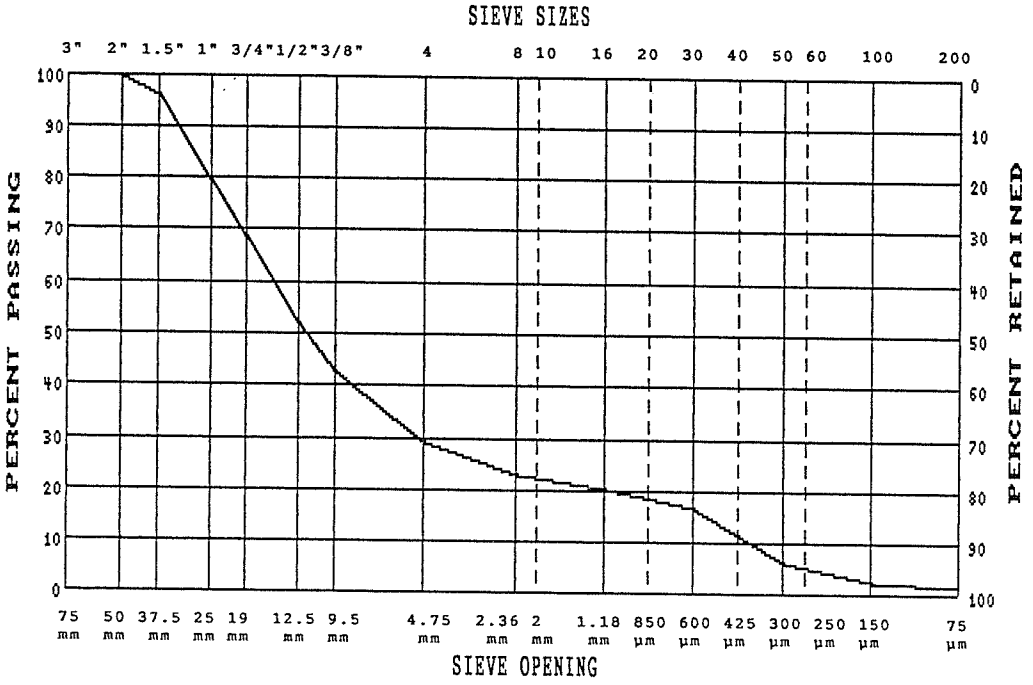
Dayton & Knight
612 Clyde Avenue
West Vancouver, BC V7V 3N9

PROJECT NO: KA20481
DATE: 02.Jan.30
CLIENT P.O.:
CC:

PROJECT: Chilliwack Dyke
Chilliwack, BC

TESTNO: L2035 SOURCE: TP#19, GS#1 SAMPLE TYPE: pail

DATE SAMPLED: 02.Jan.18 BY: J. David Ross DATE TESTED: 02.Jan.24



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm	100.0	
1 1/2"	37.5	mm	96.0	
1"	25	mm	79.3	
3/4"	19	mm	68.6	
1/2"	12.5	mm	52.2	
3/8"	9.5	mm	43.0	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No. 4	4.75	mm	29.2	
No. 8	2.36	mm	23.2	
No. 16	1.18	mm	20.4	
No. 30	600	µm	16.7	
No. 50	300	µm	6.6	
No. 100	150	µm	2.6	
No. 200	75	µm	1.6	

COMMENTS: Sand & Gravel, Trace Silt
STA 9+500

AMEC - PER:

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Hamid Yousefbeigi

AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Brian Walker, P. Eng.

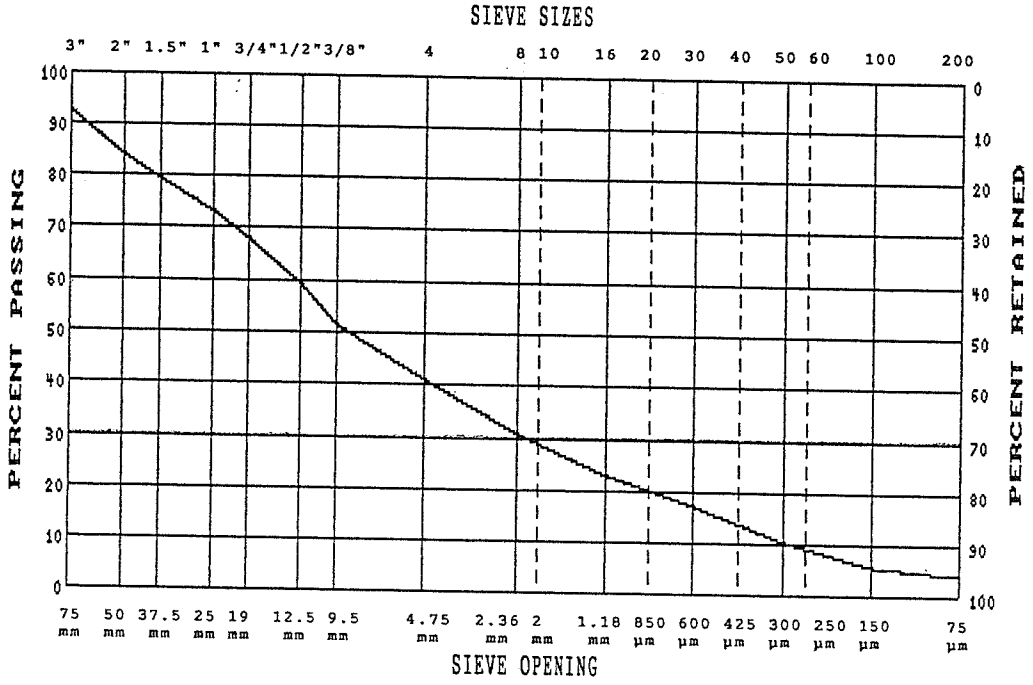
Dayton & Knight
612 Clyde Avenue
West Vancouver, BC V7V 3N9

PROJECT NO: KA20481
DATE: 02.Jan.30
CLIENT P.O.:
CC:

PROJECT: Chilliwack Dyke
Chilliwack, BC

TESTNO: L2034 SOURCE: Steelhead, Chilliwack SAMPLE TYPE: bag

DATE SAMPLED: 01.Jan.15 BY: Steelhead DATE TESTED: 01.Jan.16



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm	92.4	
2"	50	mm	84.0	
1 1/2"	37.5	mm	79.4	
1"	25	mm	73.1	
3/4"	19	mm	68.2	
1/2"	12.5	mm	59.2	
3/8"	9.5	mm	51.5	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No.	4	4.75 mm	41.2	
No.	8	2.36 mm	30.6	
No.	16	1.18 mm	22.9	
No.	30	600 µm	17.3	
No.	50	300 µm	10.4	
No.	100	150 µm	6.0	
No.	200	75 µm	4.4	

COMMENTS: Recycled Materials - Sand & Gravel, Trace Cobbles, Trace Silt

AMEC - PER:

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Hamid Yousefbeigi

AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Tara Flundra

City of Chilliwack
8550 Young Road
Chilliwack, BC
V2P 8A4

PROJECT NO: KA20468
DATE: 01.Oct.30
CLIENT P.O.:
CC:

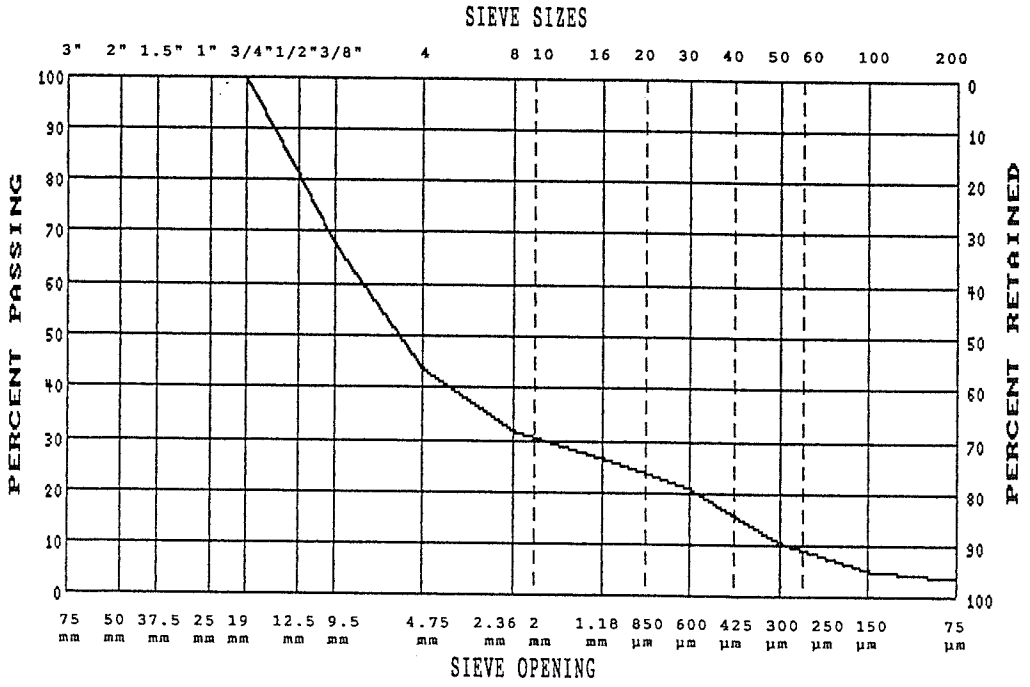
PROJECT: Dyke Stability Assessment
Chilliwack, BC

TESTNO: L1984-5 SOURCE: AH-2/GS-4

SAMPLE TYPE: bag

DATE SAMPLED: 01.Oct.12 BY: J. David Ross

DATE TESTED: 01.Oct.22



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm		
1 1/2"	37.5	mm		
1"	25	mm		
3/4"	19	mm	100.0	
1/2"	12.5	mm	81.2	
3/8"	9.5	mm	67.6	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No.	4	4.75 mm	43.9	
No.	8	2.36 mm	32.0	
No.	16	1.18 mm	26.5	
No.	30	600 µm	21.0	
No.	50	300 µm	10.6	
No.	100	150 µm	5.4	
No.	200	75 µm	3.6	

COMMENTS: Nelson Slough at Carey Road, STA 9+875
Sandy Gravel, Trace Silt

AMEC - PER:

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Hamid Yousefbeigi

AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Tara Flundra, EIT .

City of Chilliwack
8550 Young Road
Chilliwack, BC V2P 8A4

PROJECT NO: KA20468
DATE: 01.Oct.30
CLIENT P.O.:
CC:

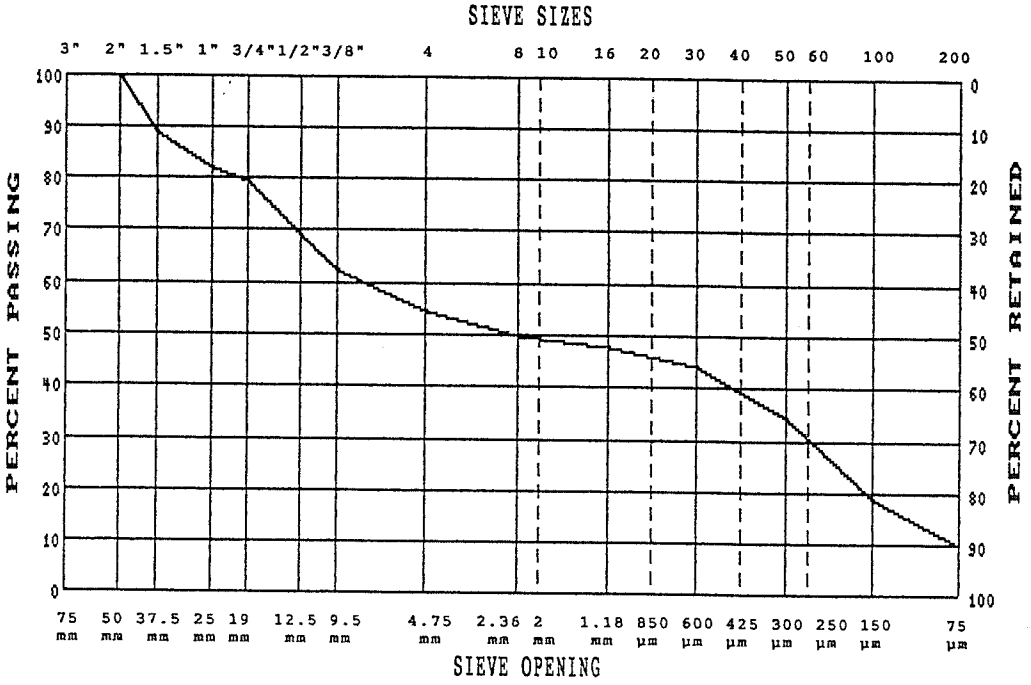
PROJECT: Dyke Stability Assessment
Chilliwack, BC

TESTNO: L1984-4 SOURCE: AH-2/GS-3

SAMPLE TYPE: bag

DATE SAMPLED: 01.Oct.12 BY: J. David Ross

DATE TESTED: 01.Oct.22



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm	100.0	
1 1/2"	37.5	mm	88.8	
1"	25	mm	81.9	
3/4"	19	mm	79.5	
1/2"	12.5	mm	69.0	
3/8"	9.5	mm	62.3	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No.	4	4.75 mm	54.7	
No.	8	2.36 mm	50.2	
No.	16	1.18 mm	47.8	
No.	30	600 µm	44.4	
No.	50	300 µm	34.3	
No.	100	150 µm	18.8	
No.	200	75 µm	10.0	

COMMENTS: Nelson Slough at Carey Road, STA#9+875
Gap Graded Sand & Gravel, Trace-Some Silt

AMEC - PER:

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Hamid Yousefbeigi

AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Tara Flundra, EIT .

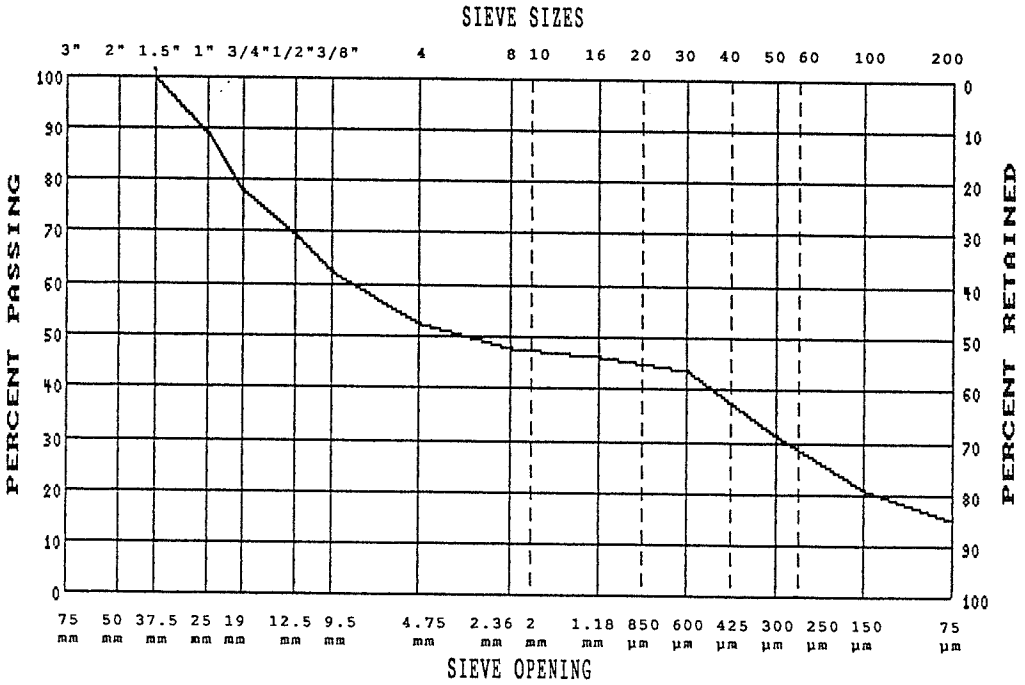
City of Chilliwack
8550 Young Road
Chilliwack, BC V2P 8A4

PROJECT NO: KA20468
DATE: 01.Oct.30
CLIENT P.O.:
CC:

PROJECT: Dyke Stability Assessment
Chilliwack, BC

TESTNO: L1984-3 SOURCE: AH-2/GS-1 SAMPLE TYPE: bag

DATE SAMPLED: 01.Oct.12 BY: J. David Ross DATE TESTED: 01.Oct.22



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm	157.6	
1 1/2"	37.5	mm	100.0	
1"	25	mm	88.9	
3/4"	19	mm	77.8	
1/2"	12.5	mm	69.6	
3/8"	9.5	mm	62.6	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No.	4	4.75 mm	52.5	
No.	8	2.36 mm	48.0	
No.	16	1.18 mm	46.3	
No.	30	600 µm	43.6	
No.	50	300 µm	31.4	
No.	100	150 µm	20.9	
No.	200	75 µm	15.2	

COMMENTS: Nelson Slough at Carey Road, STA#9+875
Gap Graded Sand & Gravel, Some Silt

AMEC - PER:

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Hamid Yousefbeigi

AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Tara Flundra, EIT .

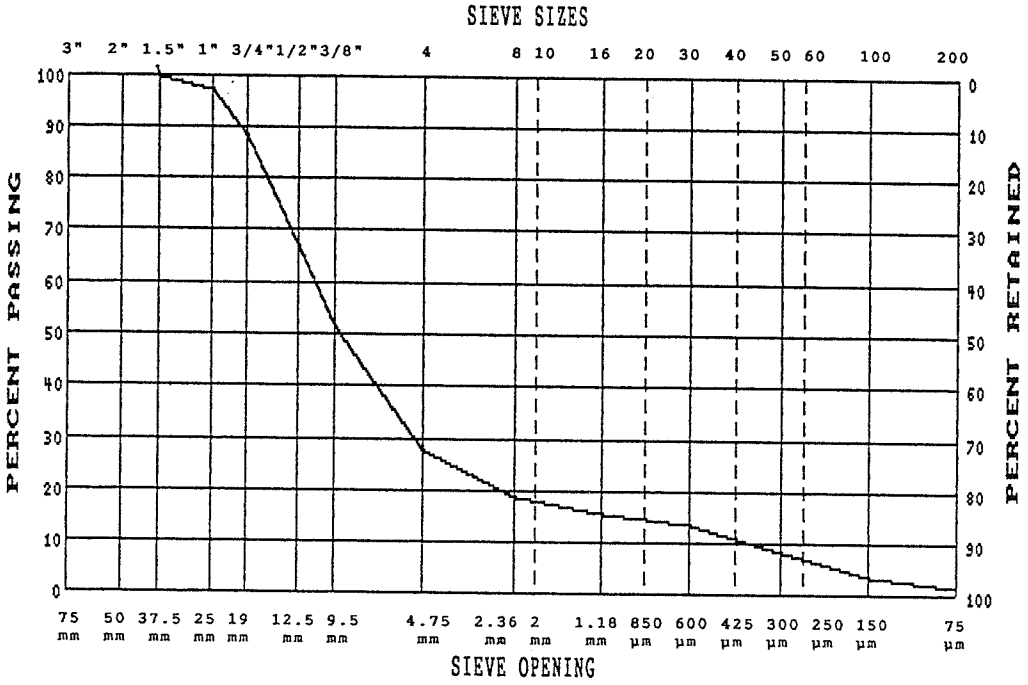
City of Chilliwack
8550 Young Road
Chilliwack, BC V2P 8A4

PROJECT NO: KA20468
DATE: 01.Oct.30
CLIENT P.O.:
CC:

PROJECT: Dyke Stability Assessment
Chilliwack, BC

TESTNO: L1984-2 SOURCE: AH-1/GS-3 SAMPLE TYPE: bag

DATE SAMPLED: 01.Oct.12 BY: J. David Ross DATE TESTED: 01.Oct.22



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm	132.1	
1 1/2"	37.5	mm	100.0	
1"	25	mm	97.2	
3/4"	19	mm	88.5	
1/2"	12.5	mm	67.2	
3/8"	9.5	mm	51.6	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No. 4	4.75	mm	27.8	
No. 8	2.36	mm	18.8	
No. 16	1.18	mm	16.0	
No. 30	600	µm	13.7	
No. 50	300	µm	8.3	
No. 100	150	µm	3.7	
No. 200	75	µm	2.0	

COMMENTS: Nelson Slough at Carey Road, STA#9+875
Sand & Gravel, Trace Silt

AMEC - PER:

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Hamid Yousefbeigi

AMEC
Abbotsford, British Columbia

SIEVE ANALYSIS REPORT

8 16 30 50 SERIES

Tara Flundra, EIT .

City of Chilliwack
8550 Young Road
Chilliwack, BC V2P 8A4

PROJECT NO: KA20468
DATE: 01.Oct.30
CLIENT P.O.:
CC:

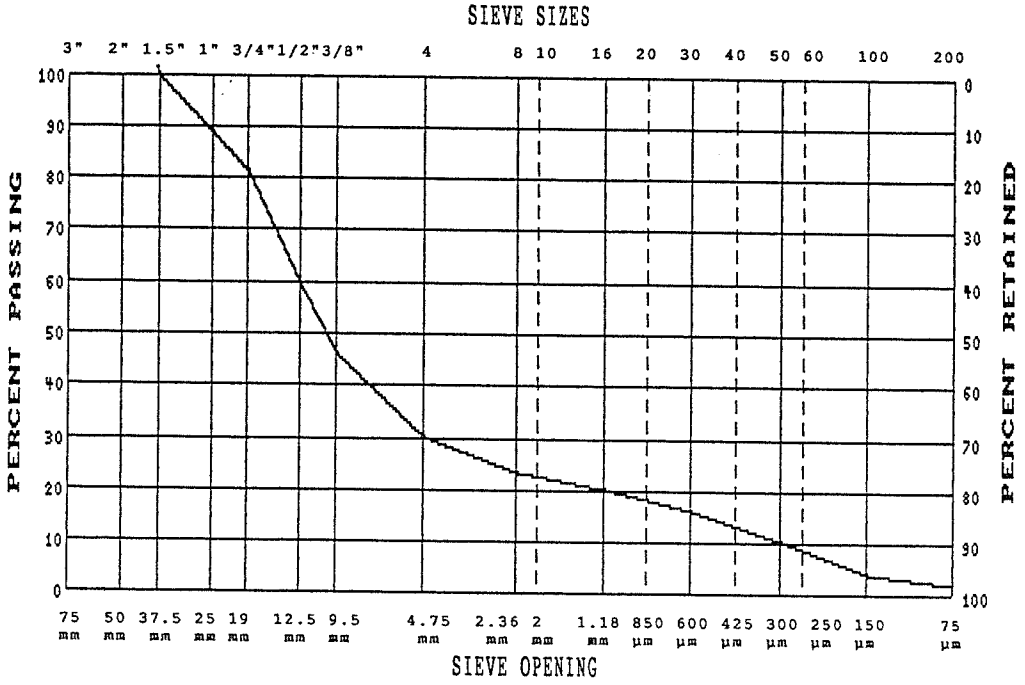
PROJECT: Dyke Stability Assessment
Chilliwack, BC

TESTNO: L1984-1 SOURCE: AH1/GS-1

SAMPLE TYPE: bag

DATE SAMPLED: 01.Oct.12 BY: J. David Ross

DATE TESTED: 01.Oct.22



GRAVEL SIZES			PERCENT PASSING	GRADATION LIMITS
3"	75	mm		
2"	50	mm	130.1	
1 1/2"	37.5	mm	100.0	
1"	25	mm	88.8	
3/4"	19	mm	81.6	
1/2"	12.5	mm	59.9	
3/8"	9.5	mm	46.5	

SAND SIZES AND FINES			PERCENT PASSING	GRADATION LIMITS
No. 4	4.75	mm	30.1	
No. 8	2.36	mm	23.6	
No. 16	1.18	mm	20.3	
No. 30	600	µm	16.4	
No. 50	300	µm	10.4	
No. 100	150	µm	4.2	
No. 200	75	µm	2.4	

COMMENTS: Nelson Slough at Carey Road, STA#9+875
Sand & Gravel, Trace Silt

AMEC - PER:

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APPENDIX D

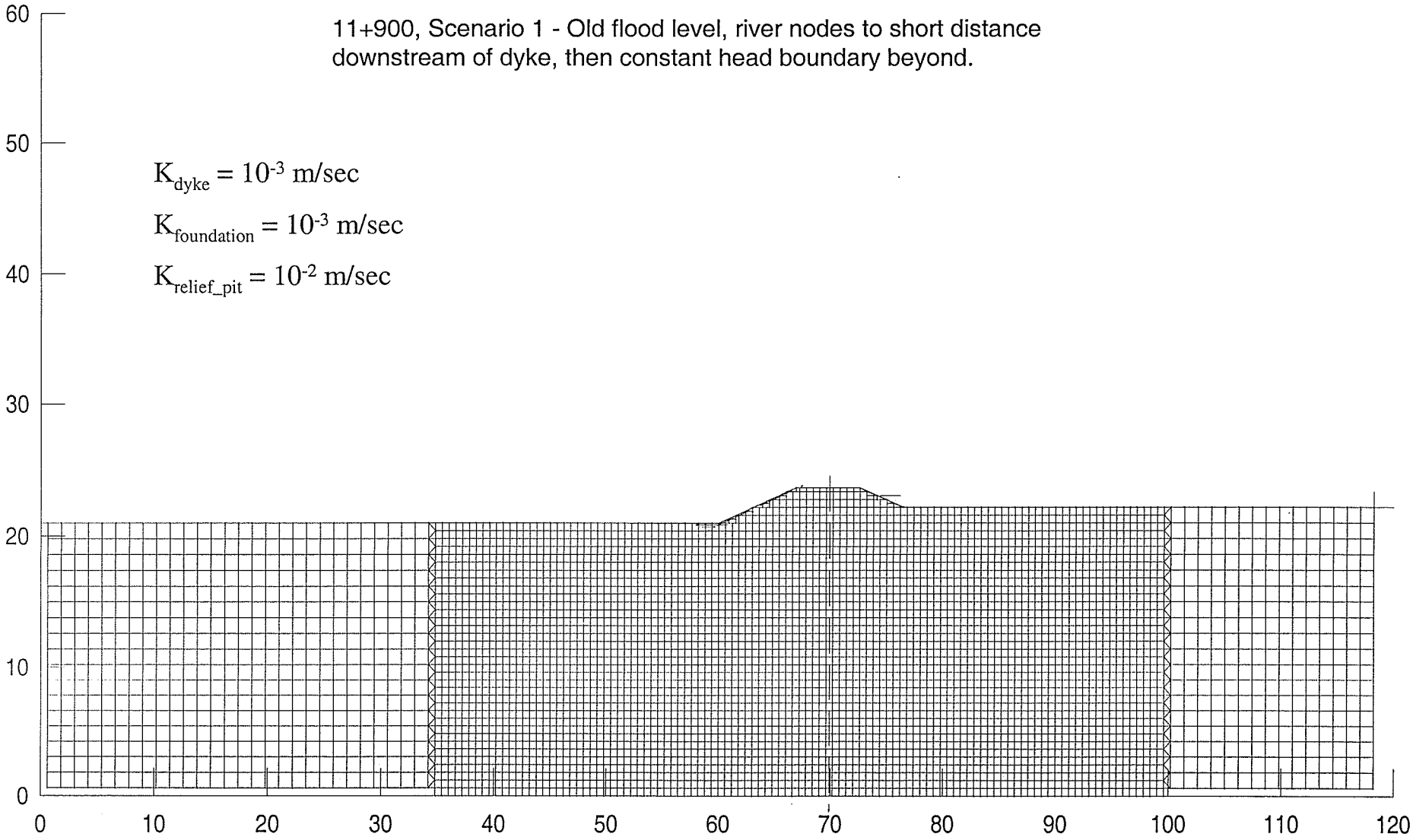
SEEP/W Modeling – Sta. 11+900 m

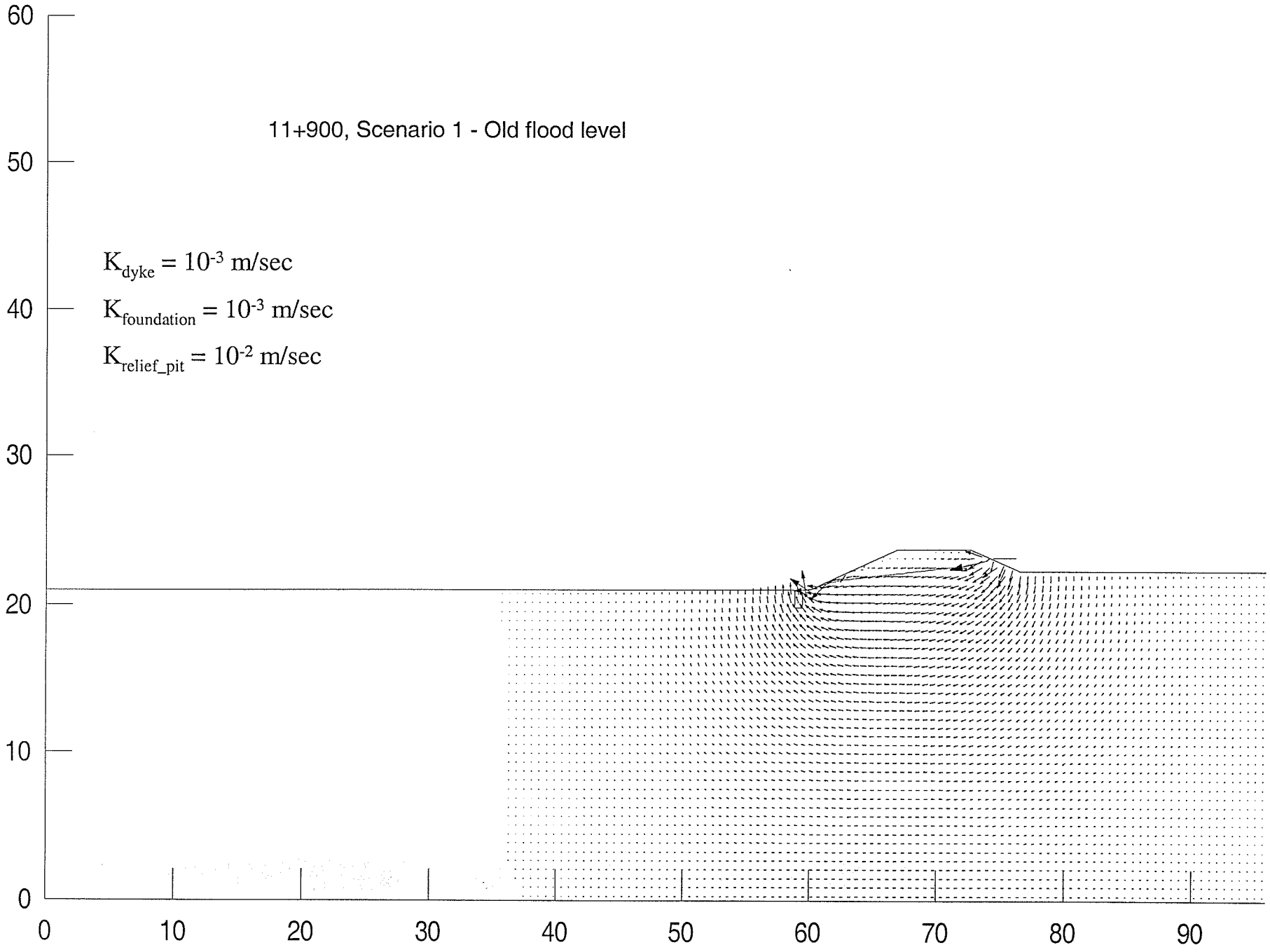
11+900, Scenario 1 - Old flood level, river nodes to short distance downstream of dyke, then constant head boundary beyond.

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-3}$ m/sec

$K_{relief\_pit} = 10^{-2}$ m/sec



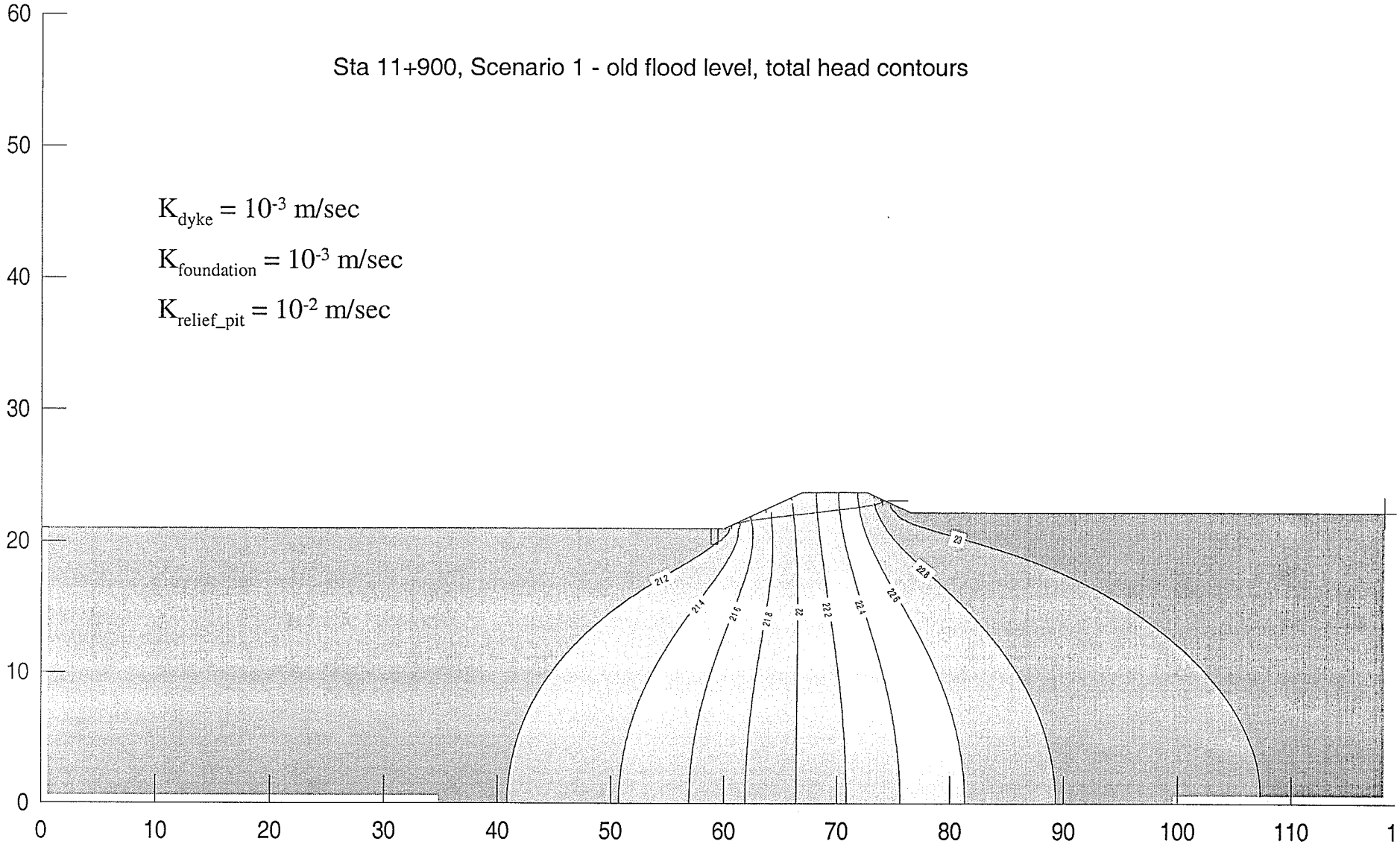

11+900, Scenario 1 - Old flood level
$K_{dyke} = 10^{-3}$ m/sec
$K_{foundation} = 10^{-3}$ m/sec
$K_{relief\_pit} = 10^{-2}$ m/sec
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
80
90

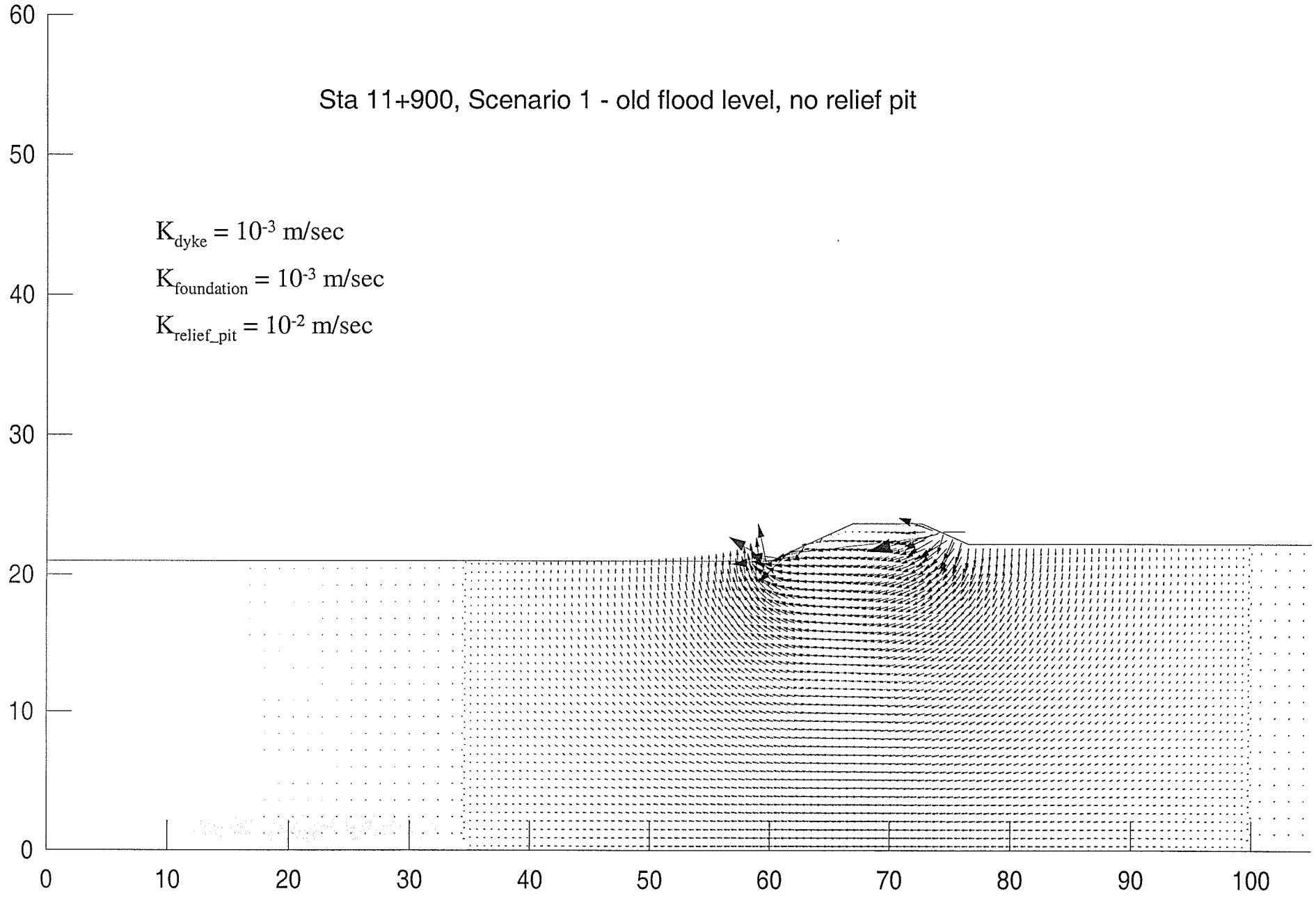
Sta 11+900, Scenario 1 - old flood level, total head contours

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-3}$ m/sec

$K_{relief\_pit} = 10^{-2}$ m/sec



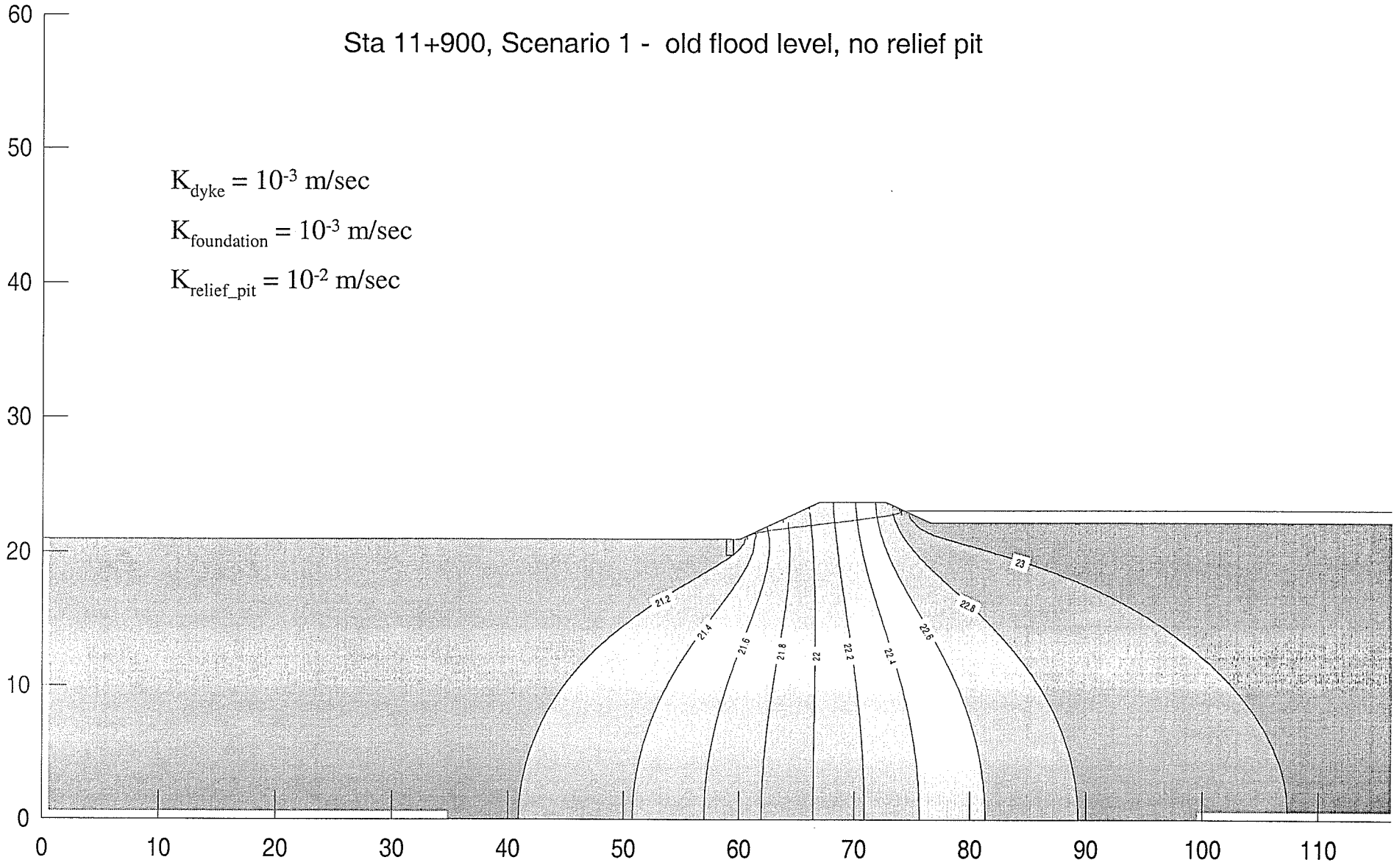

Sta 11+900, Scenario 1 - old flood level, no relief pit
$K_{dyke} = 10^{-3}$ m/sec
$K_{foundation} = 10^{-3}$ m/sec
$K_{relief\_pit} = 10^{-2}$ m/sec
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
80
90
100

Sta 11+900, Scenario 1 - old flood level, no relief pit

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-3}$ m/sec

$K_{relief\_pit} = 10^{-2}$ m/sec

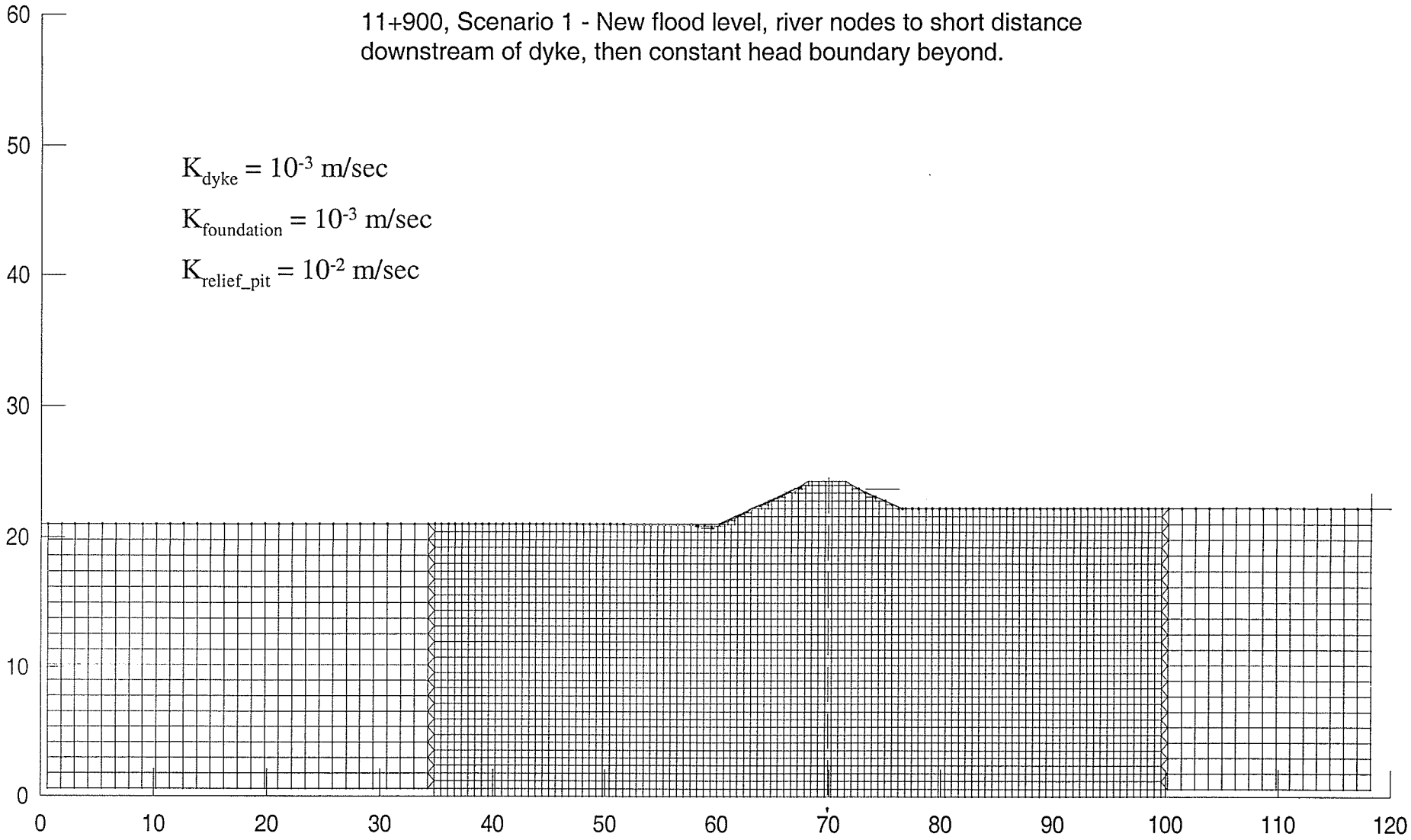


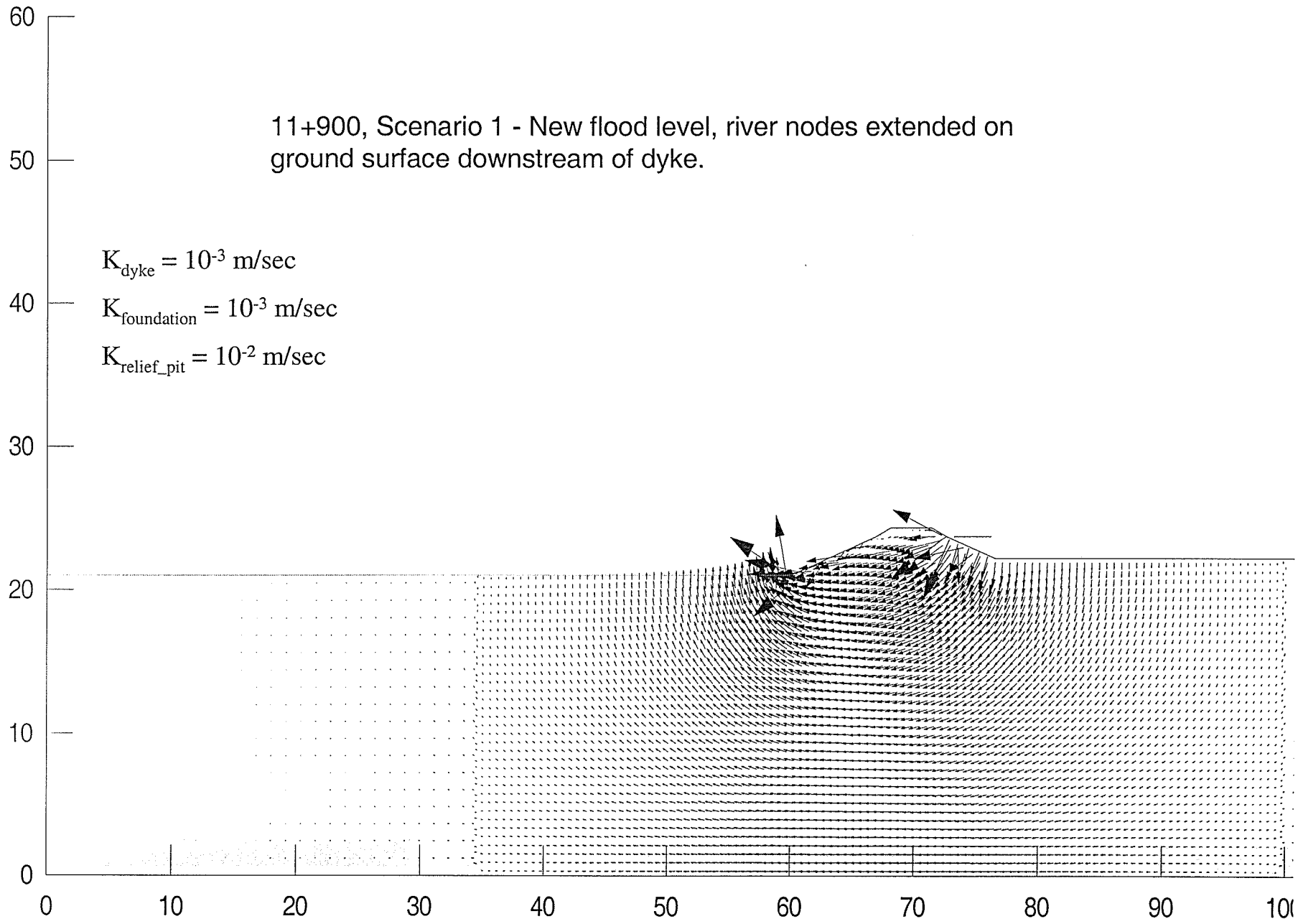
11+900, Scenario 1 - New flood level, river nodes to short distance downstream of dyke, then constant head boundary beyond.

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-3}$ m/sec

$K_{relief\_pit} = 10^{-2}$ m/sec



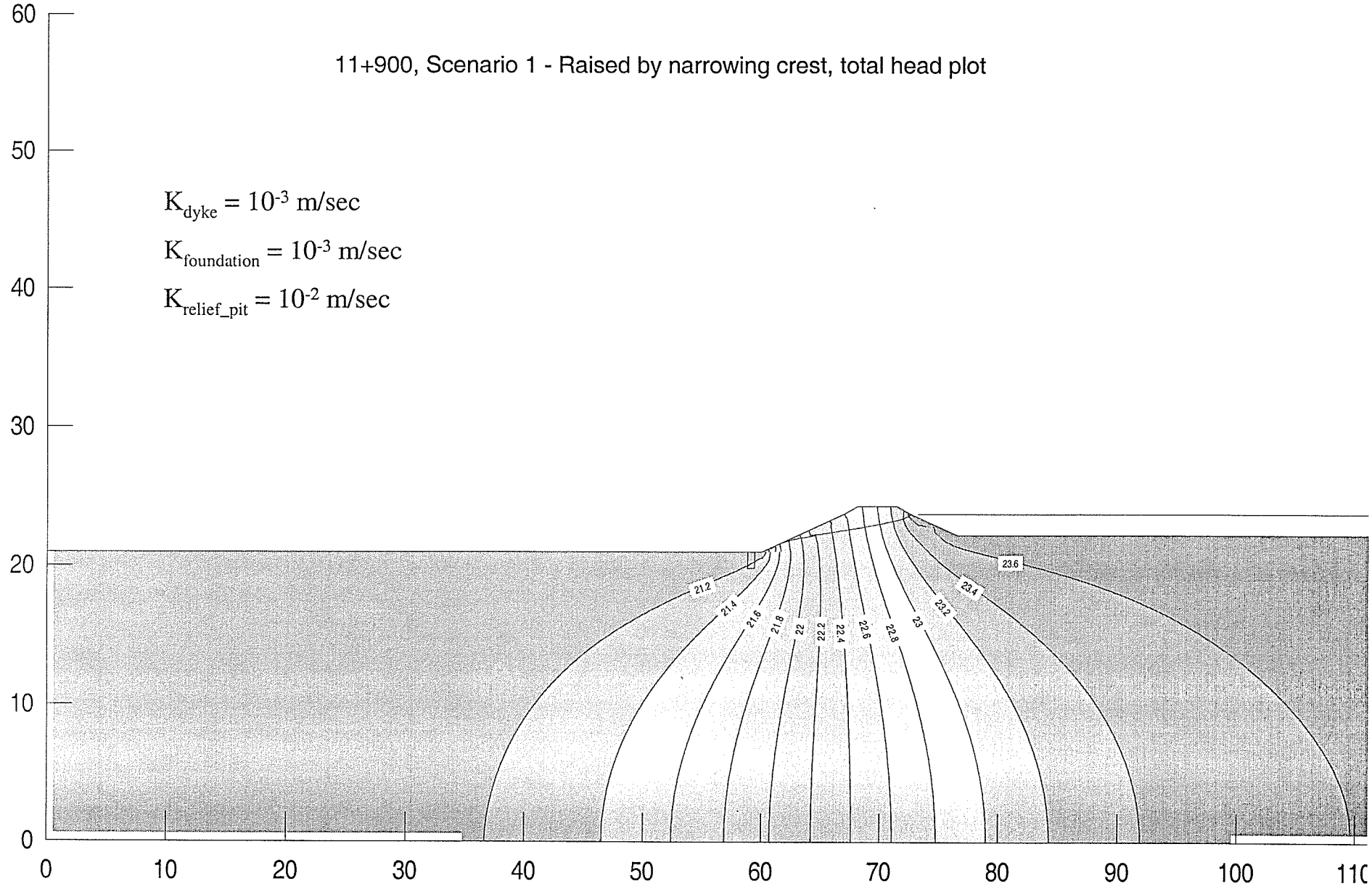

11+900, Scenario 1 - New flood level, river nodes extended on ground surface downstream of dyke.
$K_{dyke} = 10^{-3}$ m/sec
$K_{foundation} = 10^{-3}$ m/sec
$K_{relief\_pit} = 10^{-2}$ m/sec
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
80
90
100

11+900, Scenario 1 - Raised by narrowing crest, total head plot

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-3}$ m/sec

$K_{relief\_pit} = 10^{-2}$ m/sec

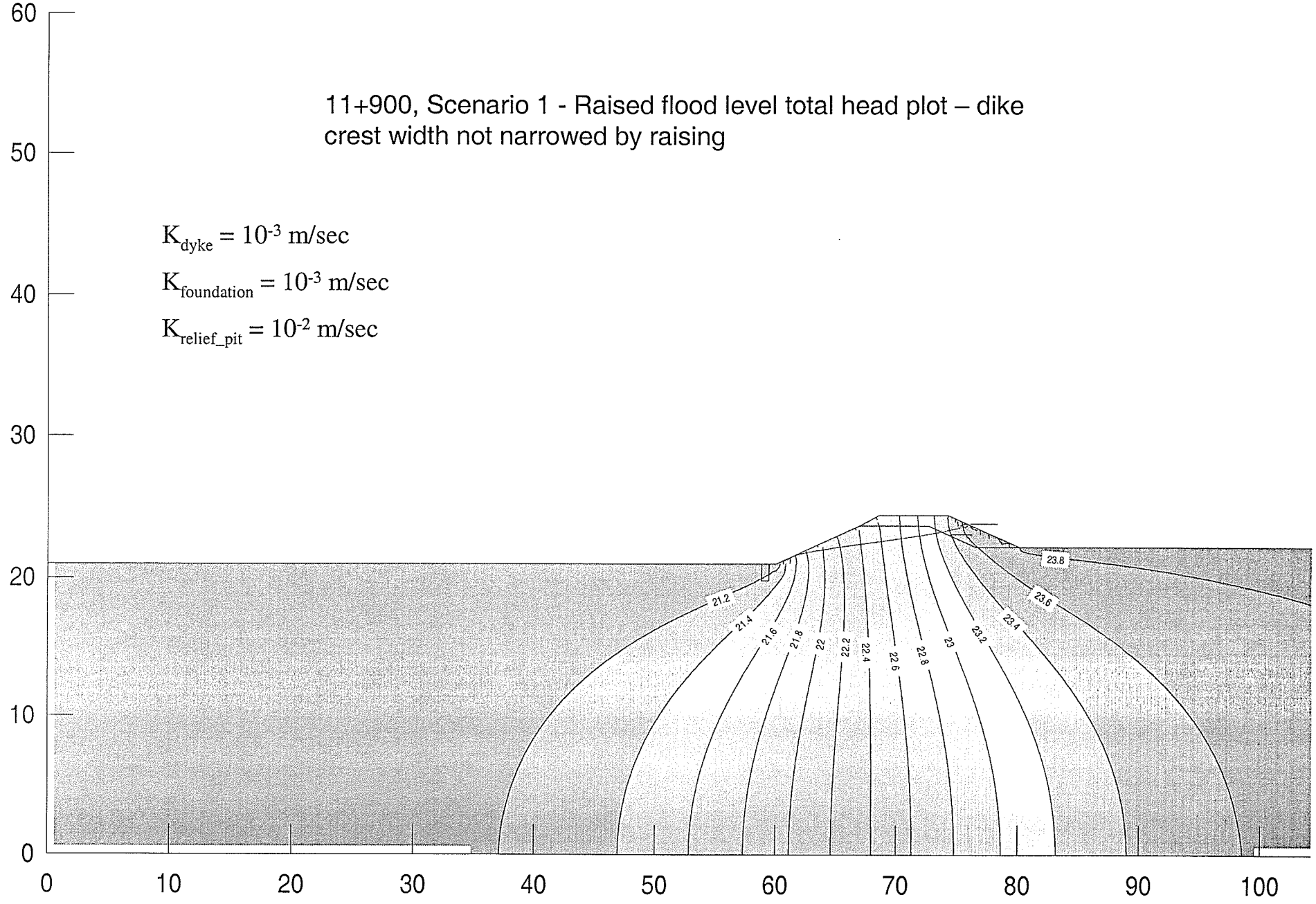


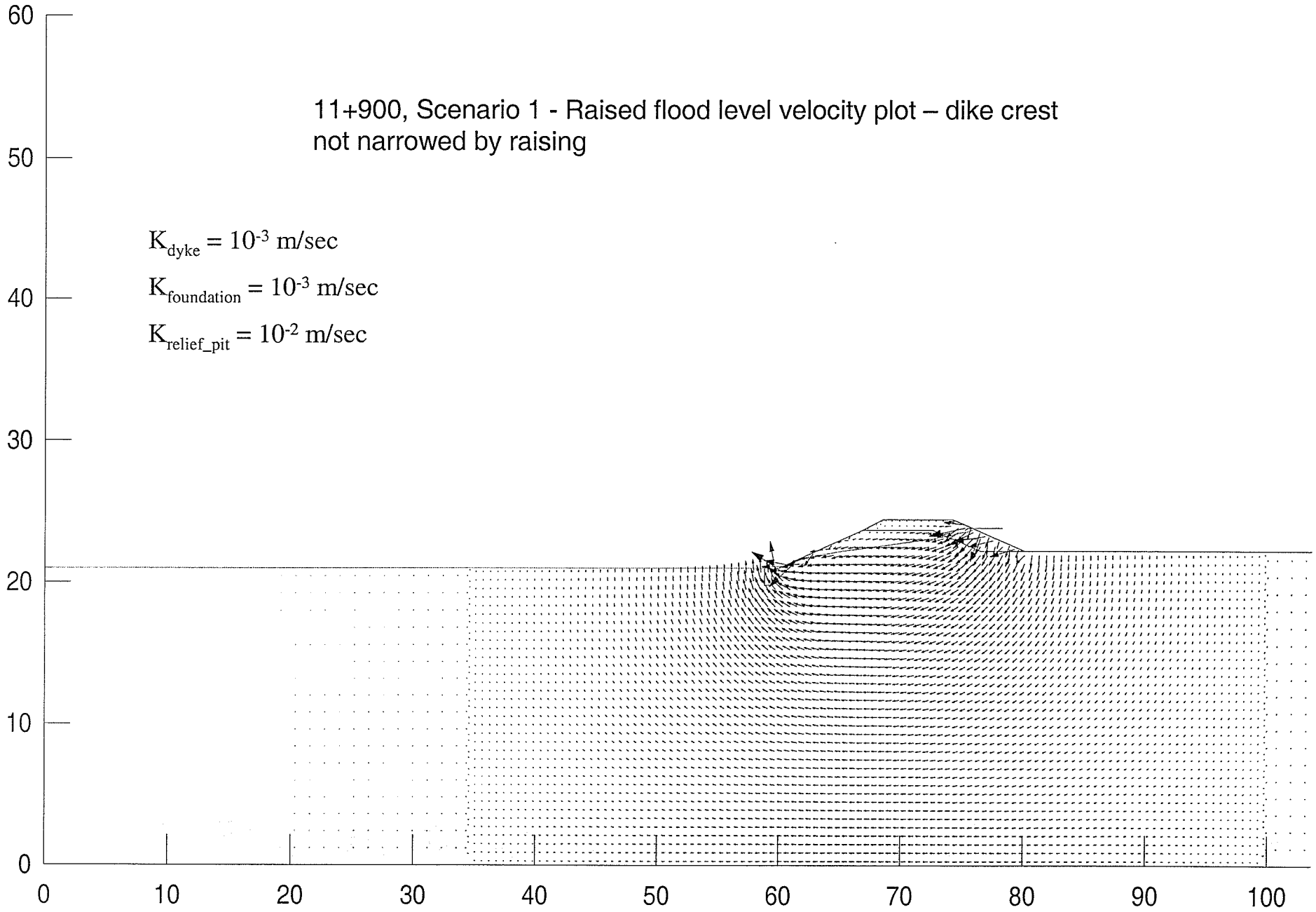
11+900, Scenario 1 - Raised flood level total head plot – dike crest width not narrowed by raising

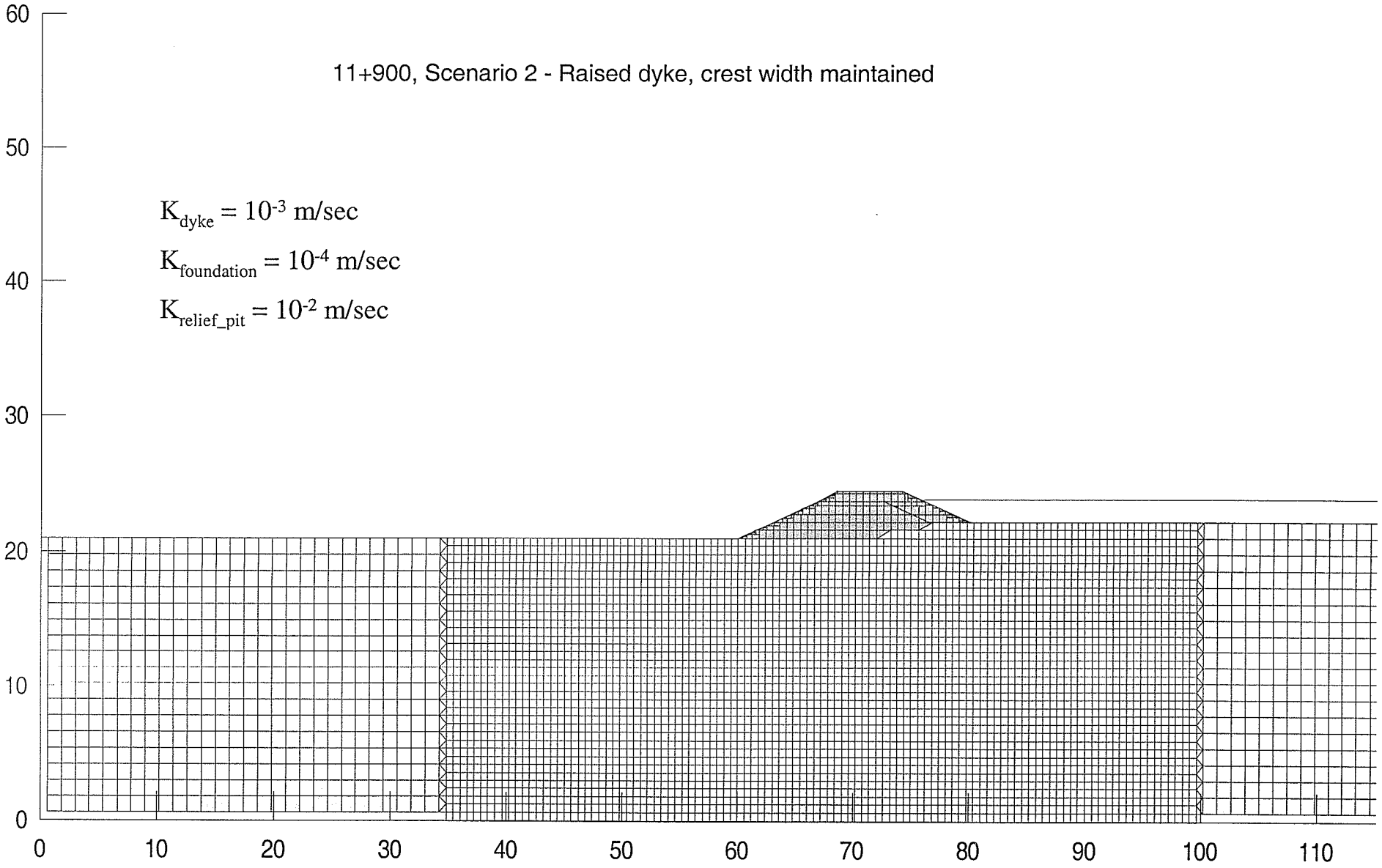
$K_{dyke} = 10^{-3}$ m/sec

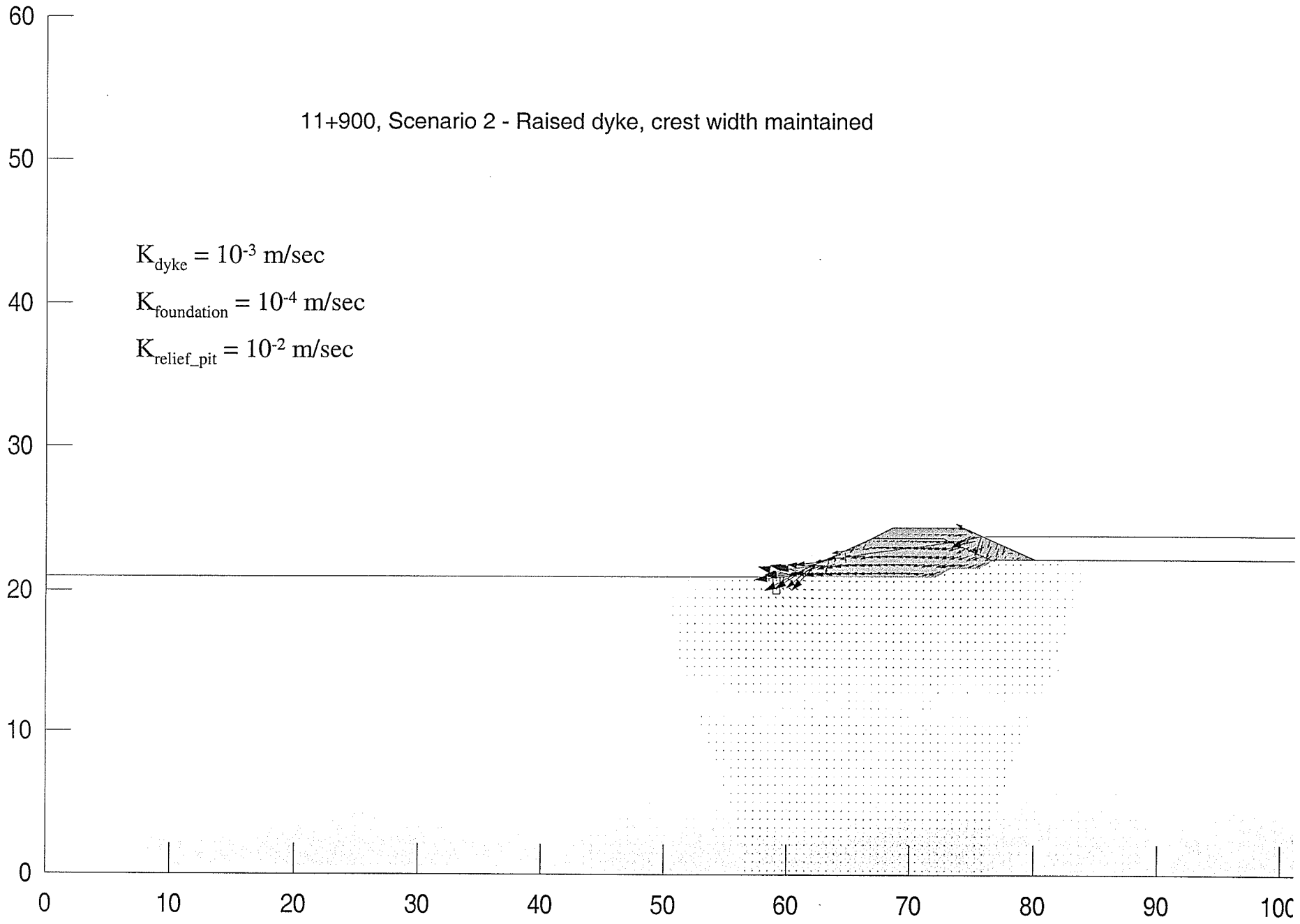
$K_{foundation} = 10^{-3}$ m/sec

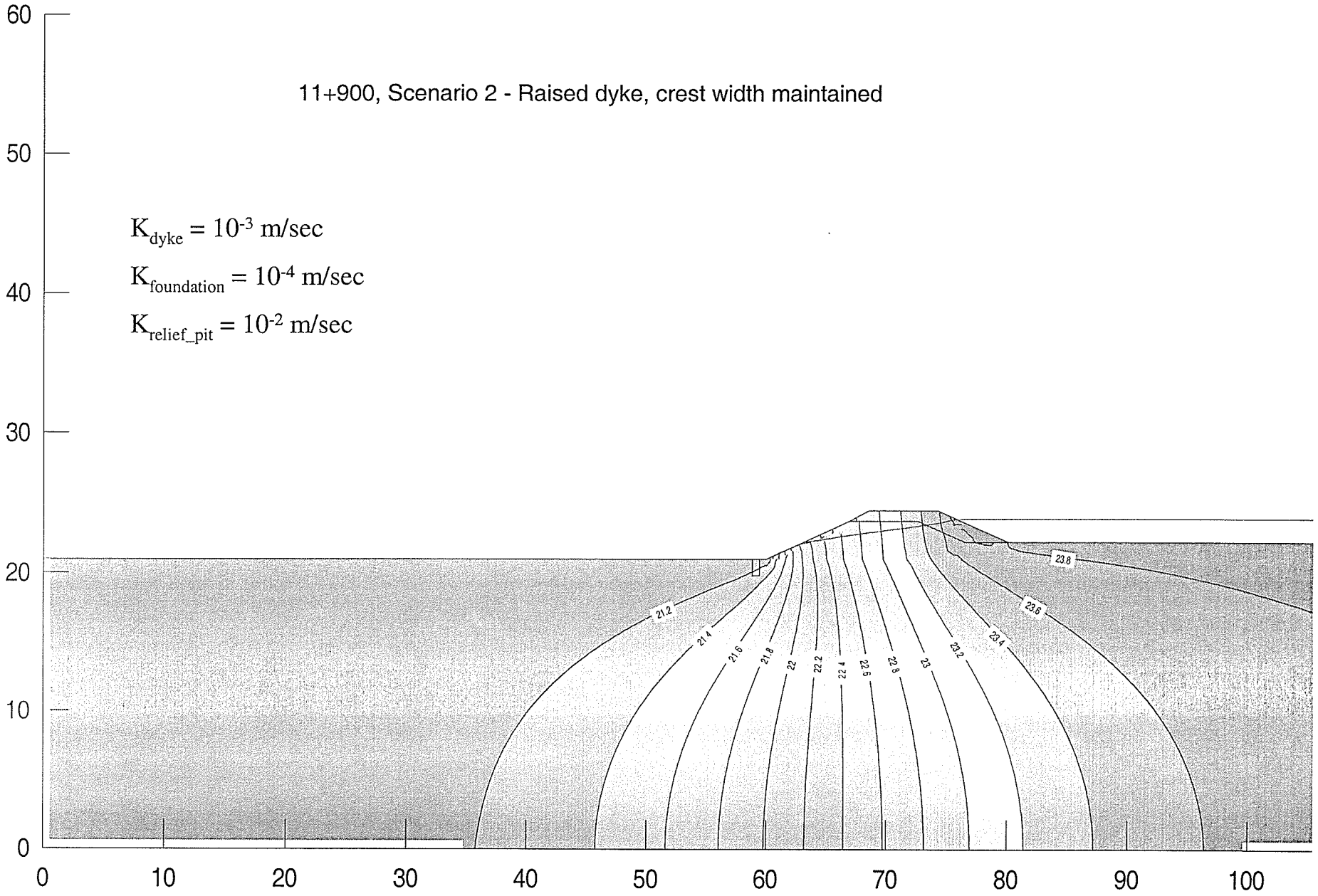
$K_{relief\_pit} = 10^{-2}$ m/sec




11+900, Scenario 1 - Raised flood level velocity plot – dike crest not narrowed by raising
$K_{dyke} = 10^{-3}$ m/sec
$K_{foundation} = 10^{-3}$ m/sec
$K_{relief\_pit} = 10^{-2}$ m/sec
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
80
90
100


11+900, Scenario 2 - Raised dyke, crest width maintained
$K_{dyke} = 10^{-3}$ m/sec
$K_{foundation} = 10^{-4}$ m/sec
$K_{relief\_pit} = 10^{-2}$ m/sec
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
80
90
100
110


11+900, Scenario 2 - Raised dyke, crest width maintained
$K_{dyke} = 10^{-3}$ m/sec
$K_{foundation} = 10^{-4}$ m/sec
$K_{relief\_pit} = 10^{-2}$ m/sec
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
80
90
100

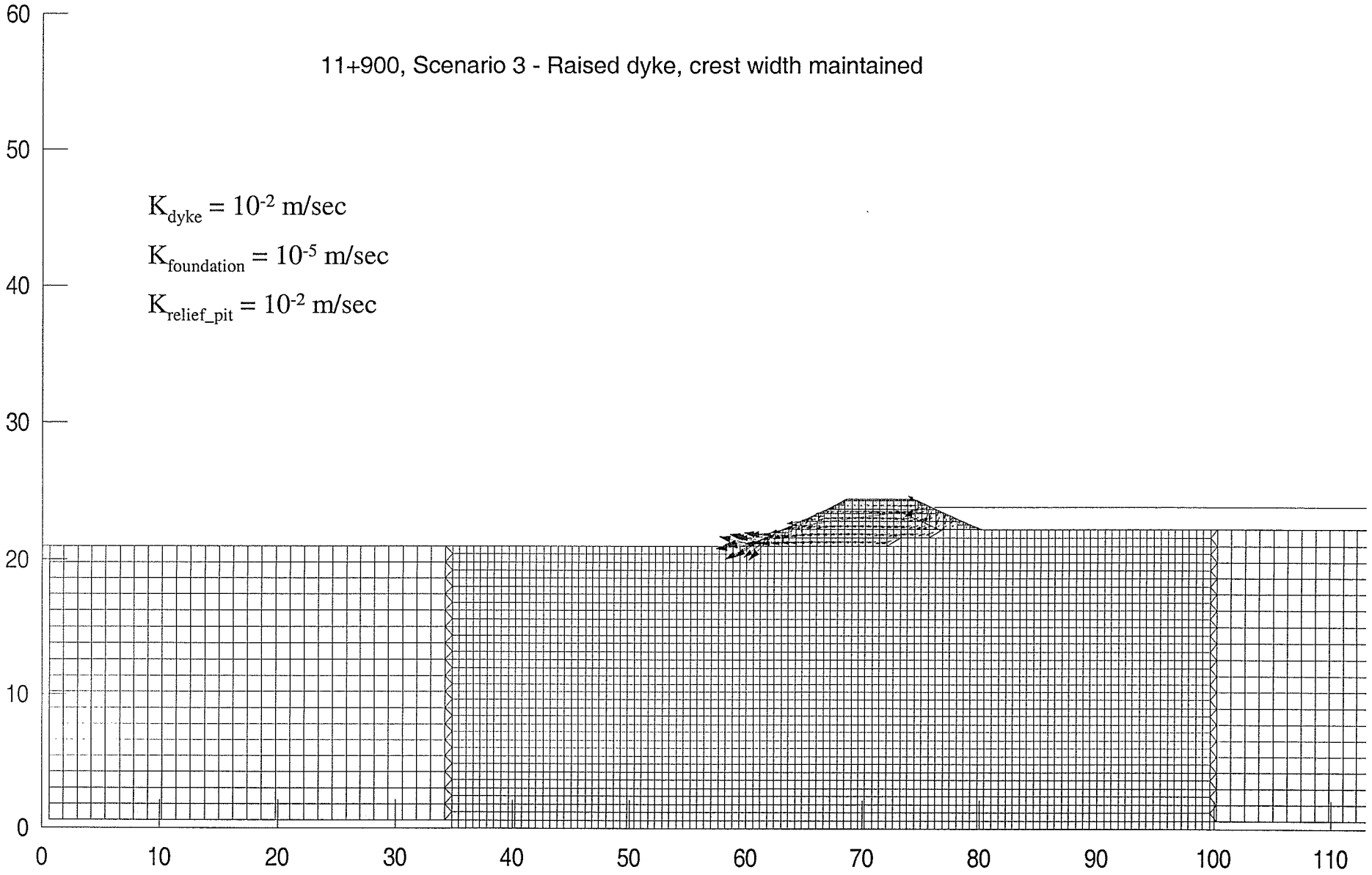

11+900, Scenario 2 - Raised dyke, crest width maintained
$K_{dyke} = 10^{-3}$ m/sec
$K_{foundation} = 10^{-4}$ m/sec
$K_{relief\_pit} = 10^{-2}$ m/sec
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
80
90
100
21.2
21.4
21.6
21.8
22
22.2
22.4
22.6
22.8
23
23.2
23.4
23.6
23.8

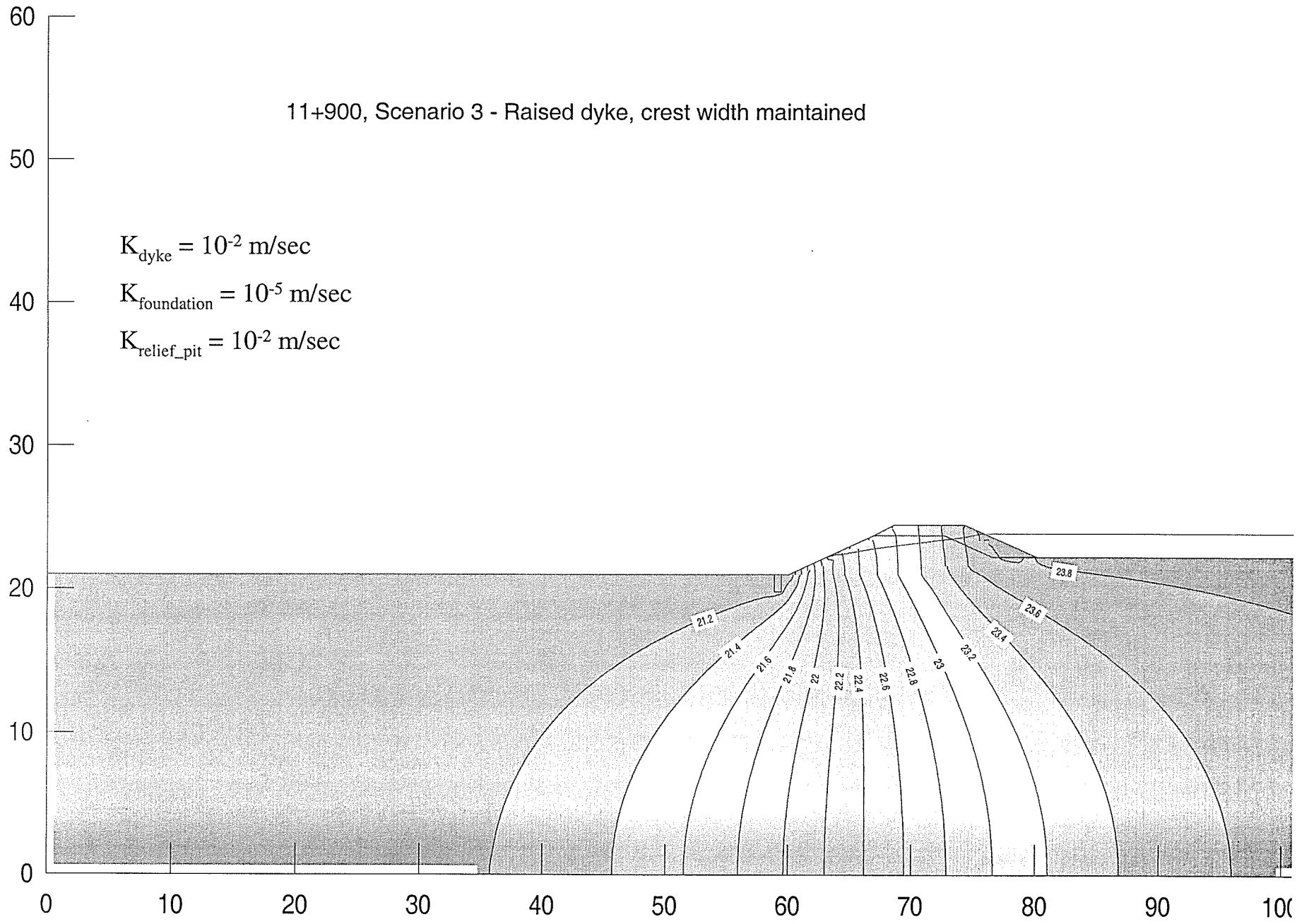
11+900, Scenario 3 - Raised dyke, crest width maintained

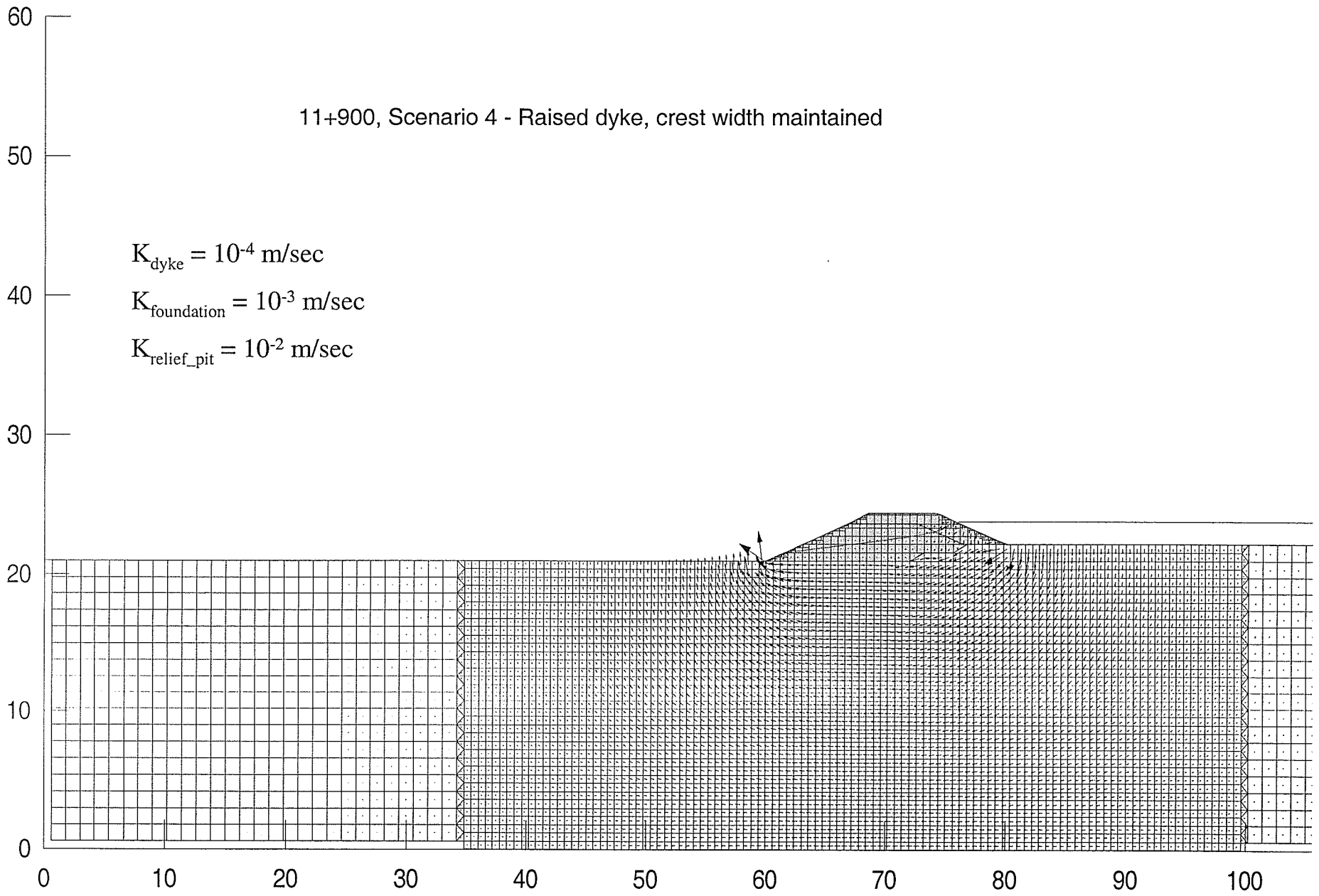
$K_{dyke} = 10^{-2}$ m/sec

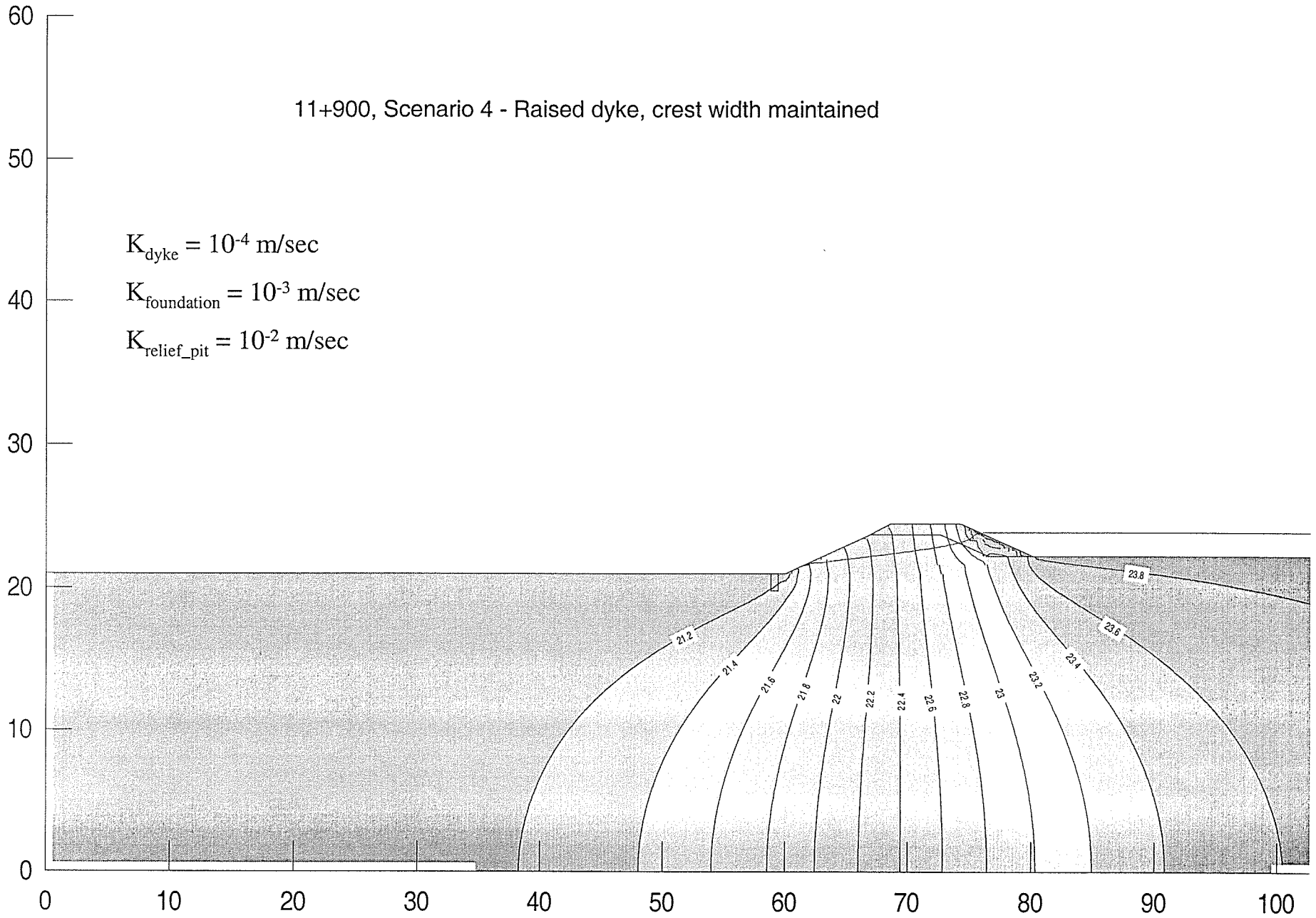
$K_{foundation} = 10^{-5}$ m/sec

$K_{relief\_pit} = 10^{-2}$ m/sec




11+900, Scenario 3 - Raised dyke, crest width maintained
$K_{dyke} = 10^{-2}$ m/sec
$K_{foundation} = 10^{-5}$ m/sec
$K_{relief\_pit} = 10^{-2}$ m/sec
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
80
90
100
21.2
21.4
21.6
21.8
22
22.2
22.4
22.6
22.8
23
23.2
23.4
23.6
23.8


11+900, Scenario 4 - Raised dyke, crest width maintained
$K_{dyke} = 10^{-4}$ m/sec
$K_{foundation} = 10^{-3}$ m/sec
$K_{relief\_pit} = 10^{-2}$ m/sec
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
80
90
100


11+900, Scenario 4 - Raised dyke, crest width maintained
$K_{dyke} = 10^{-4}$ m/sec
$K_{foundation} = 10^{-3}$ m/sec
$K_{relief\_pit} = 10^{-2}$ m/sec
21.2
21.4
21.6
21.8
22
22.2
22.4
22.6
22.8
23
23.2
23.4
23.6
23.8
0
10
20
30
40
50
60
70
80
90
100

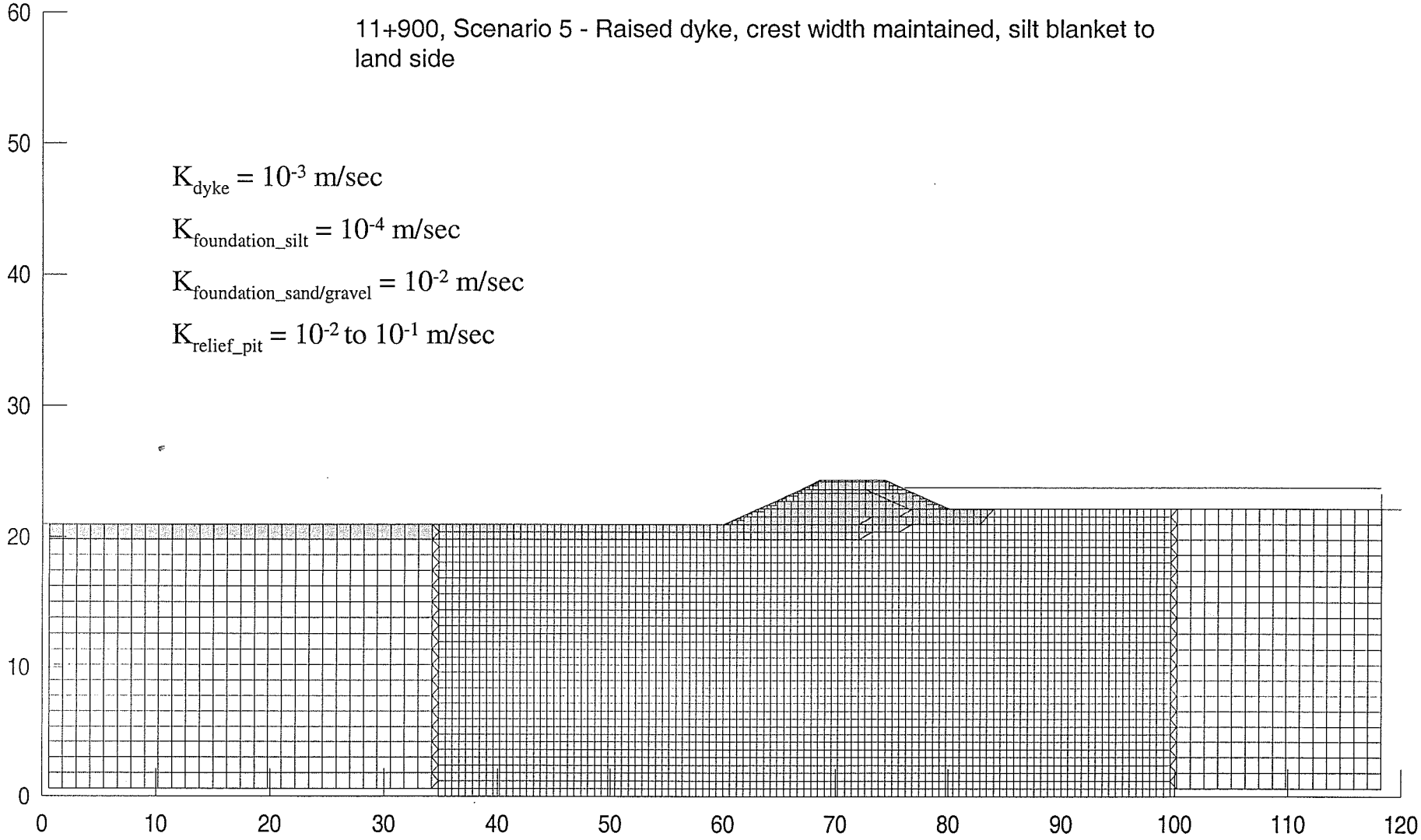
11+900, Scenario 5 - Raised dyke, crest width maintained, silt blanket to land side

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation\_silt} = 10^{-4}$ m/sec

$K_{foundation\_sand/gravel} = 10^{-2}$ m/sec

$K_{relief\_pit} = 10^{-2}$ to 10^{-1} m/sec



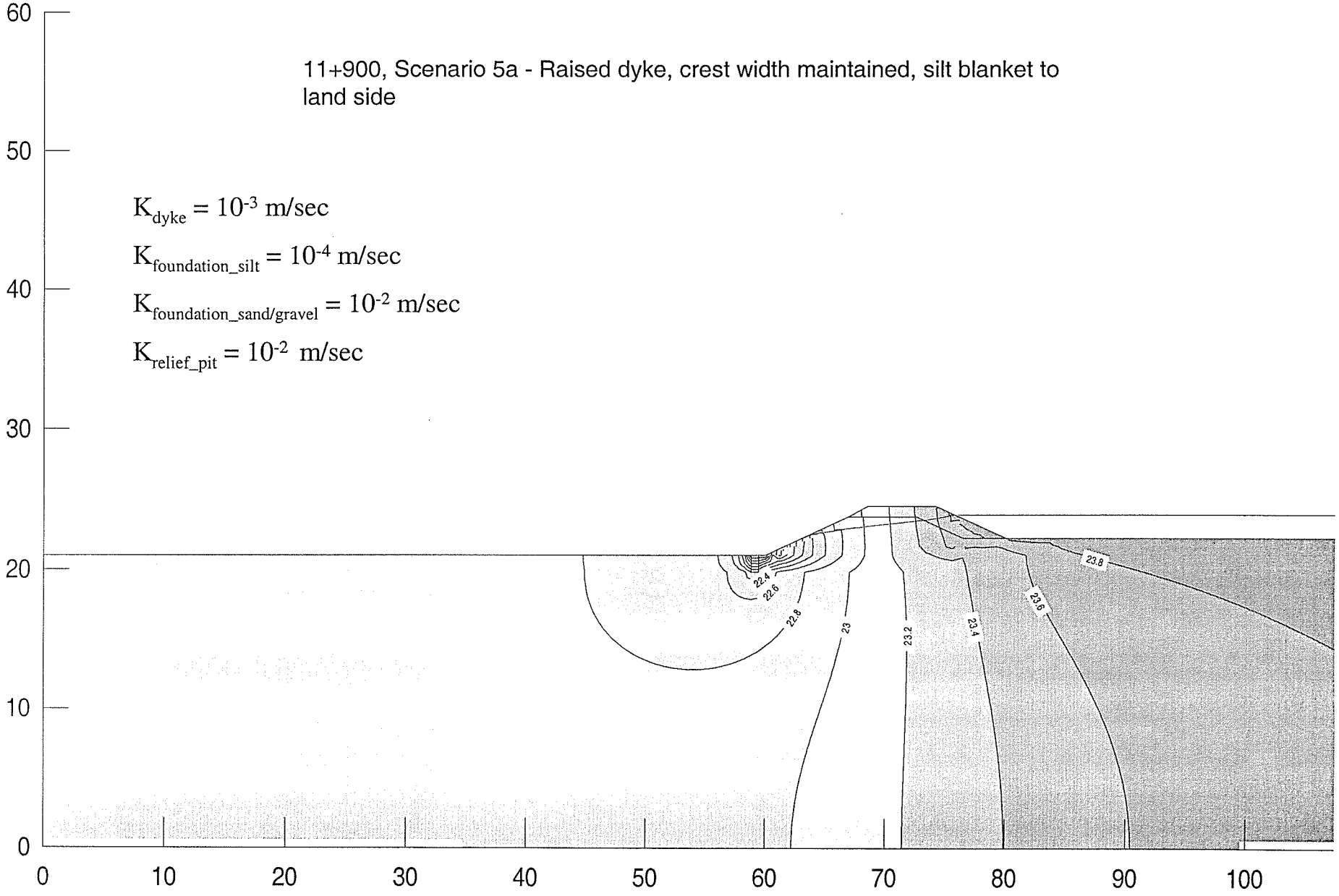
11+900, Scenario 5a - Raised dyke, crest width maintained, silt blanket to land side

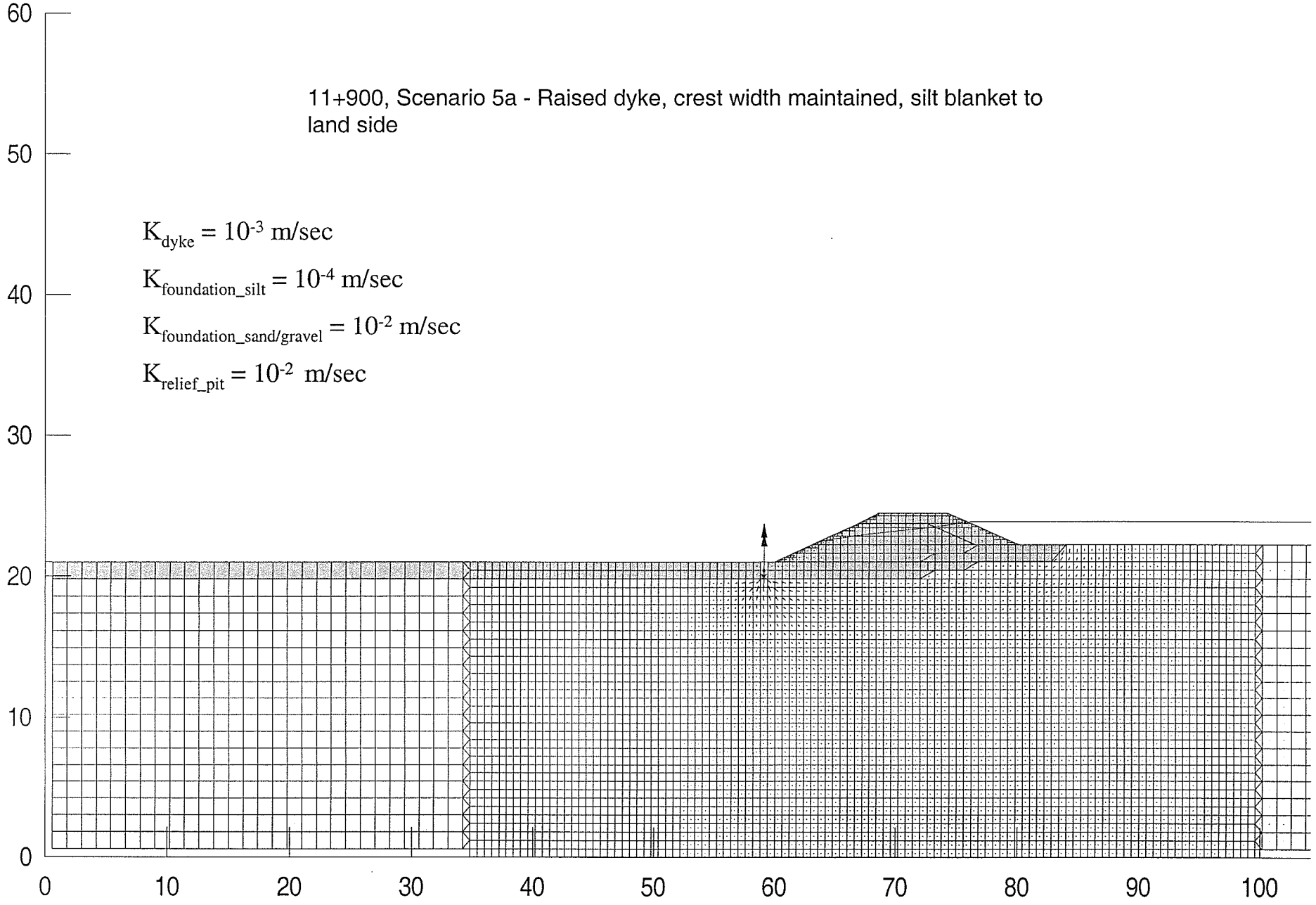
$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation\_silt} = 10^{-4}$ m/sec

$K_{foundation\_sand/gravel} = 10^{-2}$ m/sec

$K_{relief\_pit} = 10^{-2}$ m/sec




11+900, Scenario 5a - Raised dyke, crest width maintained, silt blanket to land side
$K_{dyke} = 10^{-3}$ m/sec
$K_{foundation\_silt} = 10^{-4}$ m/sec
$K_{foundation\_sand/gravel} = 10^{-2}$ m/sec
$K_{relief\_pit} = 10^{-2}$ m/sec
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
80
90
100

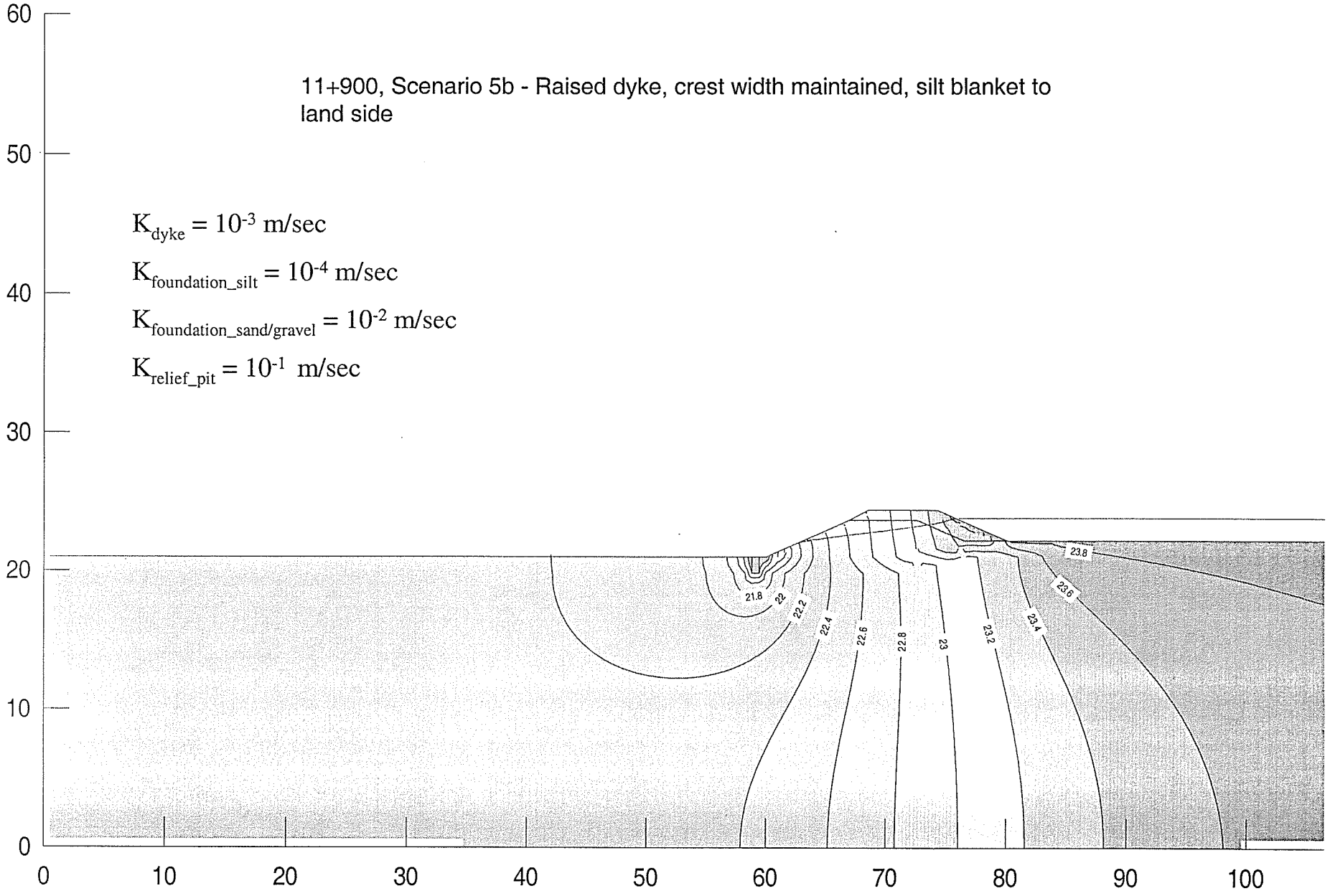
11+900, Scenario 5b - Raised dyke, crest width maintained, silt blanket to land side

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation\_silt} = 10^{-4}$ m/sec

$K_{foundation\_sand/gravel} = 10^{-2}$ m/sec

$K_{relief\_pit} = 10^{-1}$ m/sec



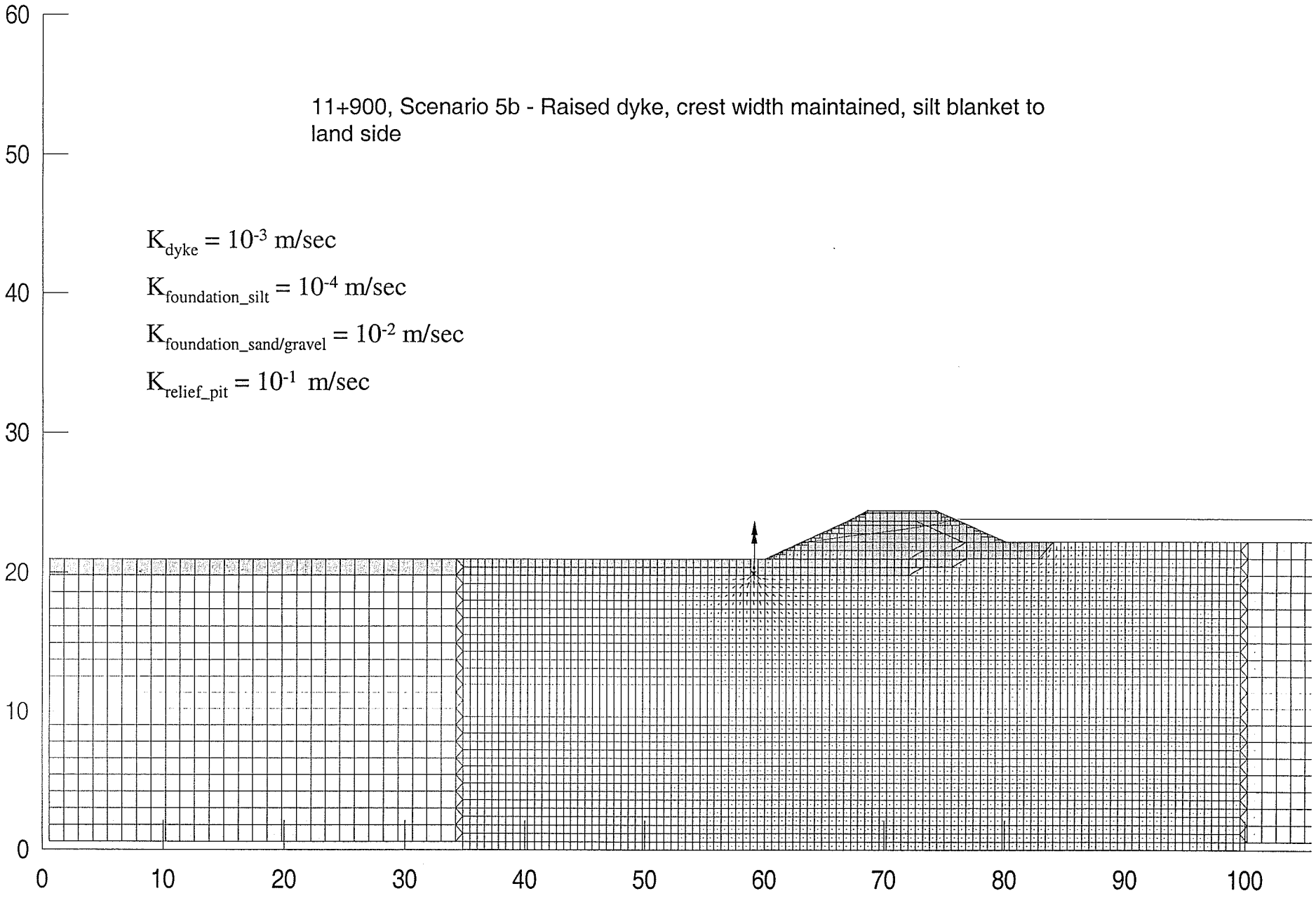
11+900, Scenario 5b - Raised dyke, crest width maintained, silt blanket to land side

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation\_silt} = 10^{-4}$ m/sec

$K_{foundation\_sand/gravel} = 10^{-2}$ m/sec

$K_{relief\_pit} = 10^{-1}$ m/sec



APPENDIX E

SEEP/W Modeling – Sta. 11+940 m (Nelson Slough)

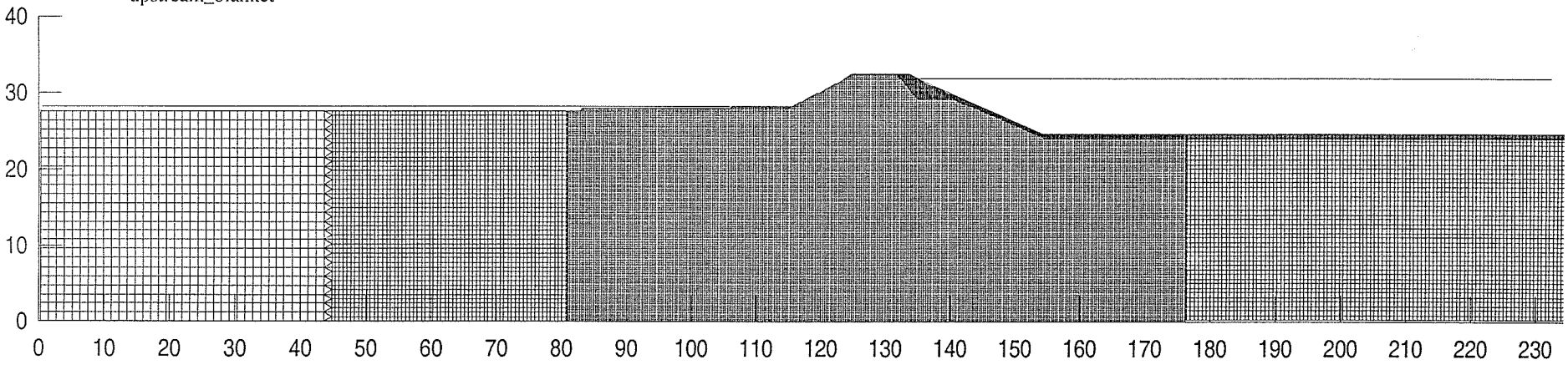
11+940, Scenario 1 - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-3}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 4 \times 10^{-4}$ m/sec



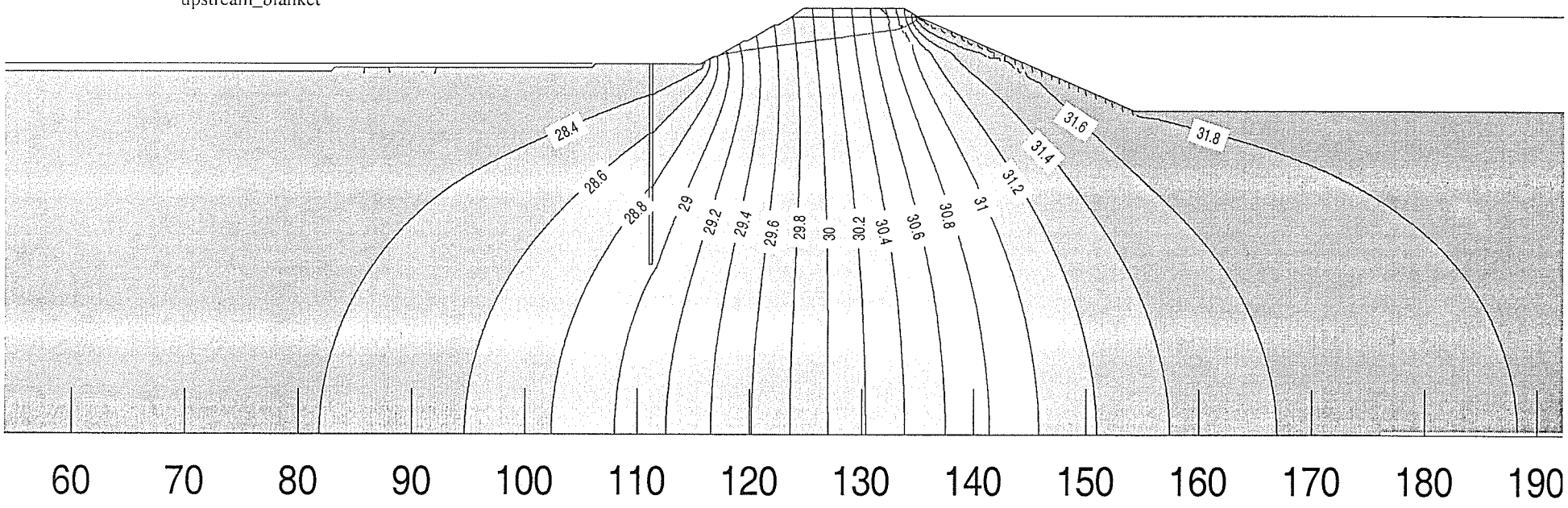
11+940, Scenario 1 - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-3}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 4 \times 10^{-4}$ m/sec



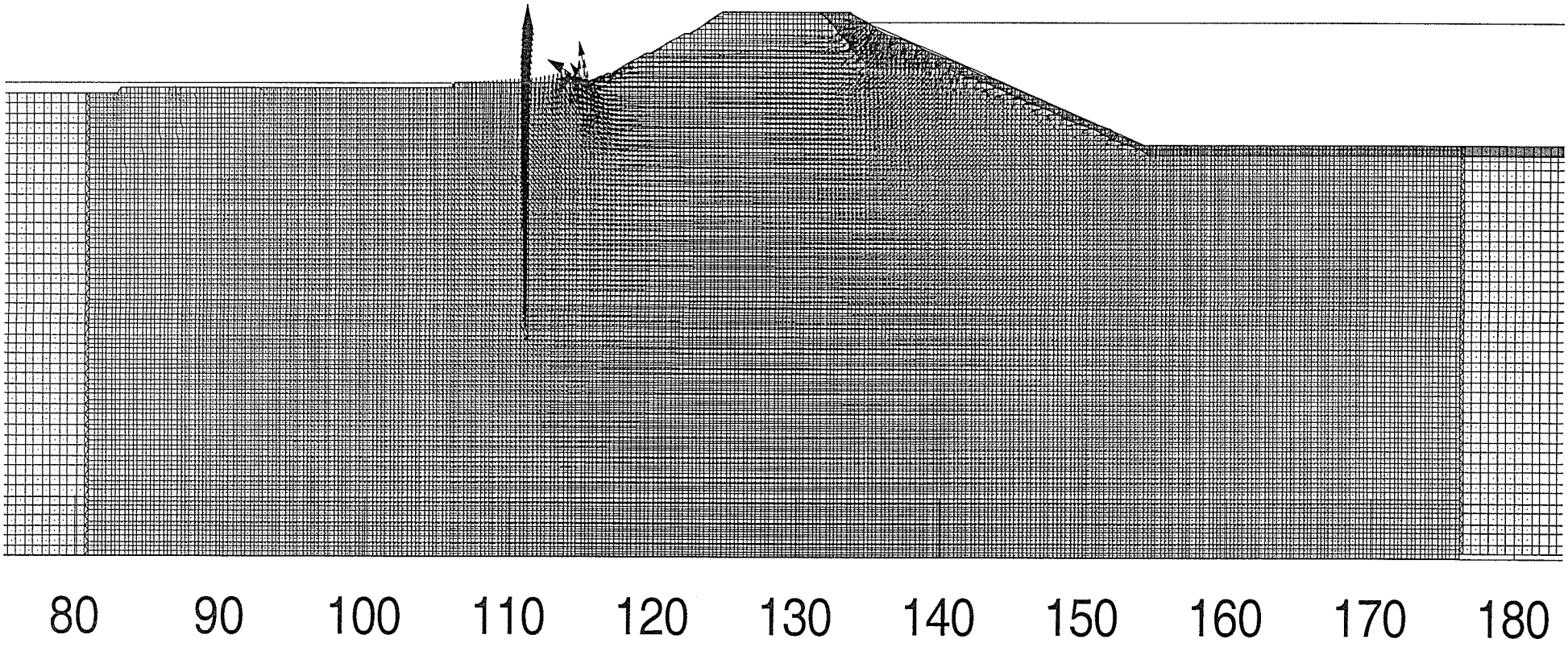
11+940, Scenario 1 - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-3}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 4 \times 10^{-4}$ m/sec



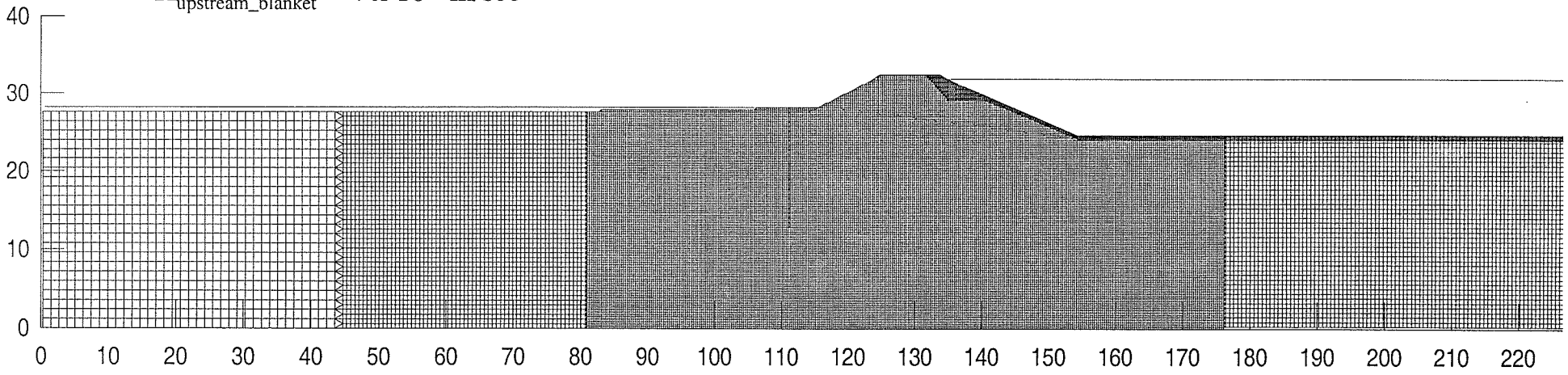
11+940, Scenario 2 - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-4}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 4 \times 10^{-4}$ m/sec



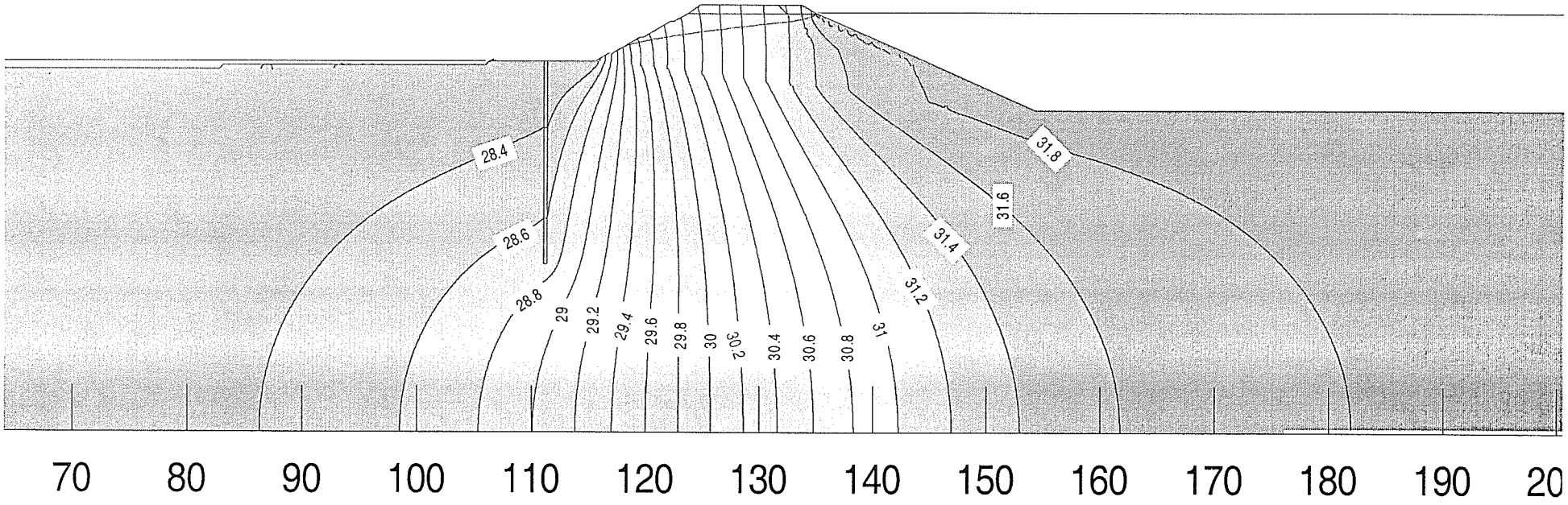
11+940, Scenario 2 - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-4}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 4 \times 10^{-4}$ m/sec



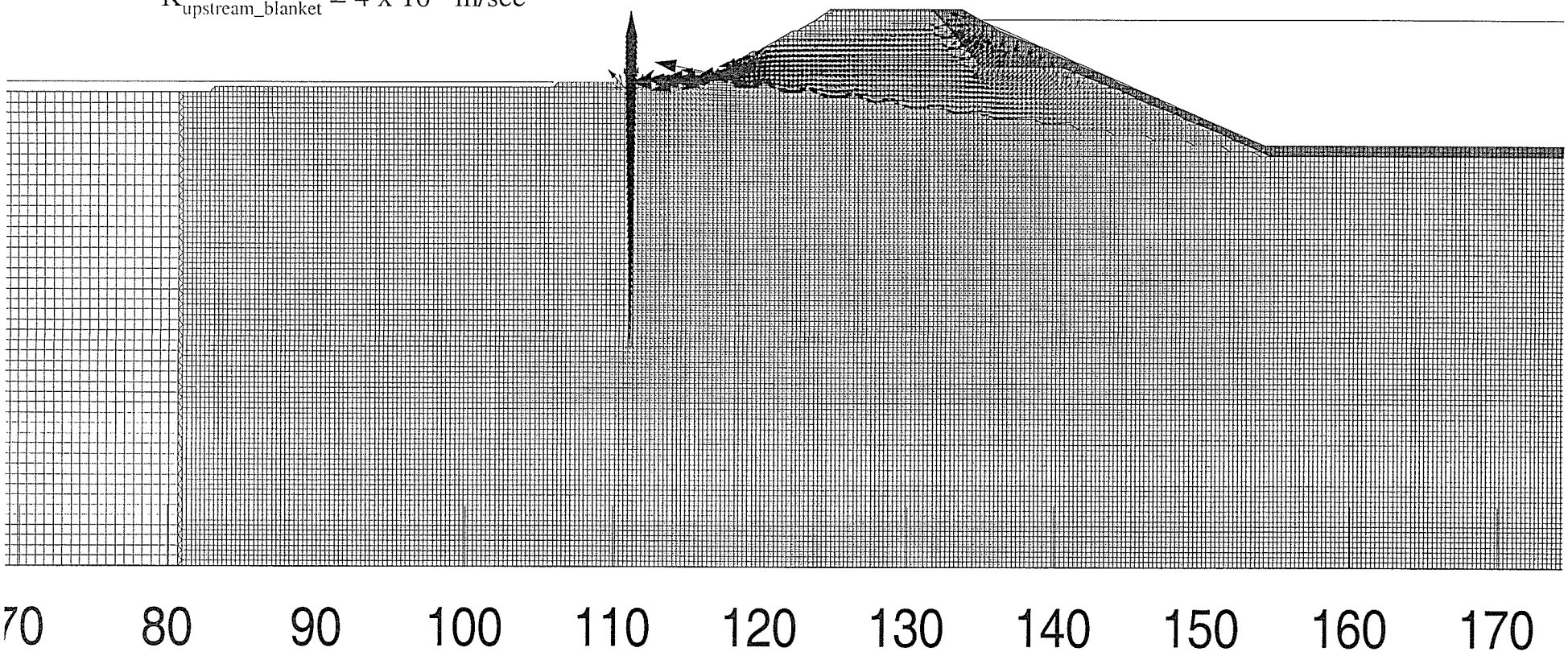
11+940, Scenario 2 - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-4}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 4 \times 10^{-4}$ m/sec



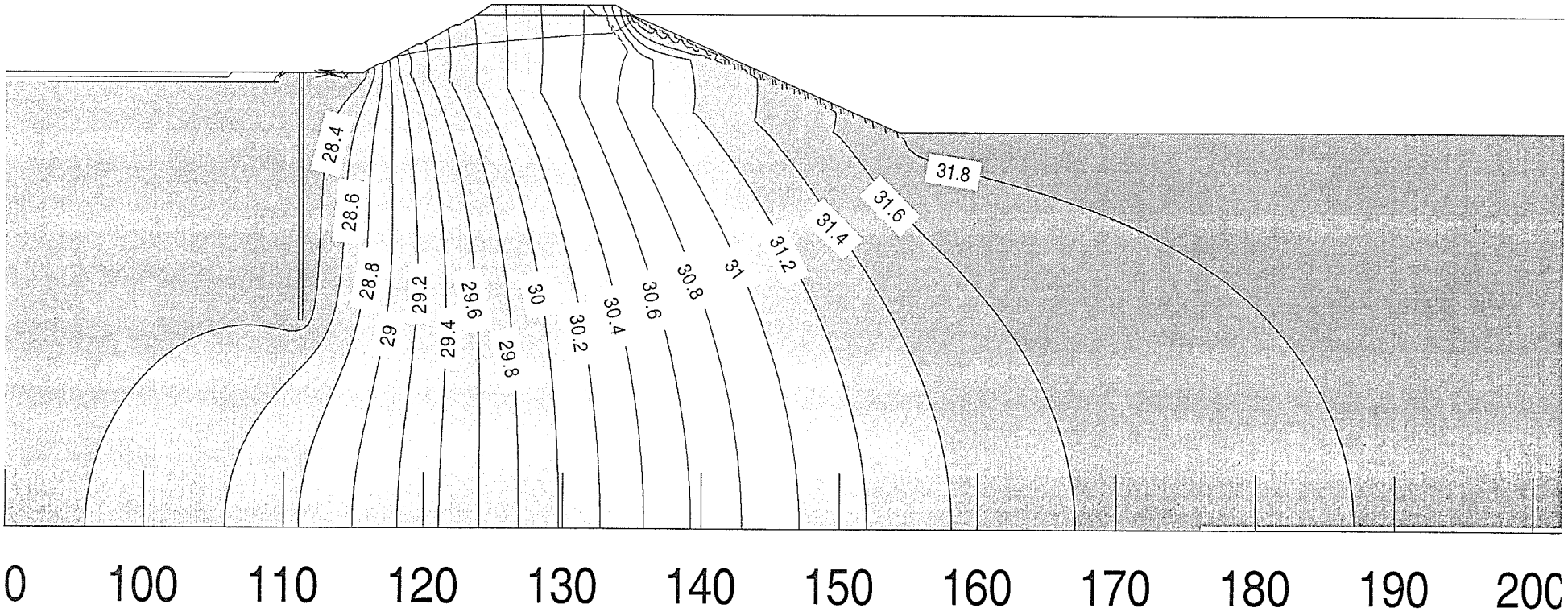
11+940, Scenario 3 - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-2}$ m/sec

$K_{foundation} = 10^{-5}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 4 \times 10^{-4}$ m/sec



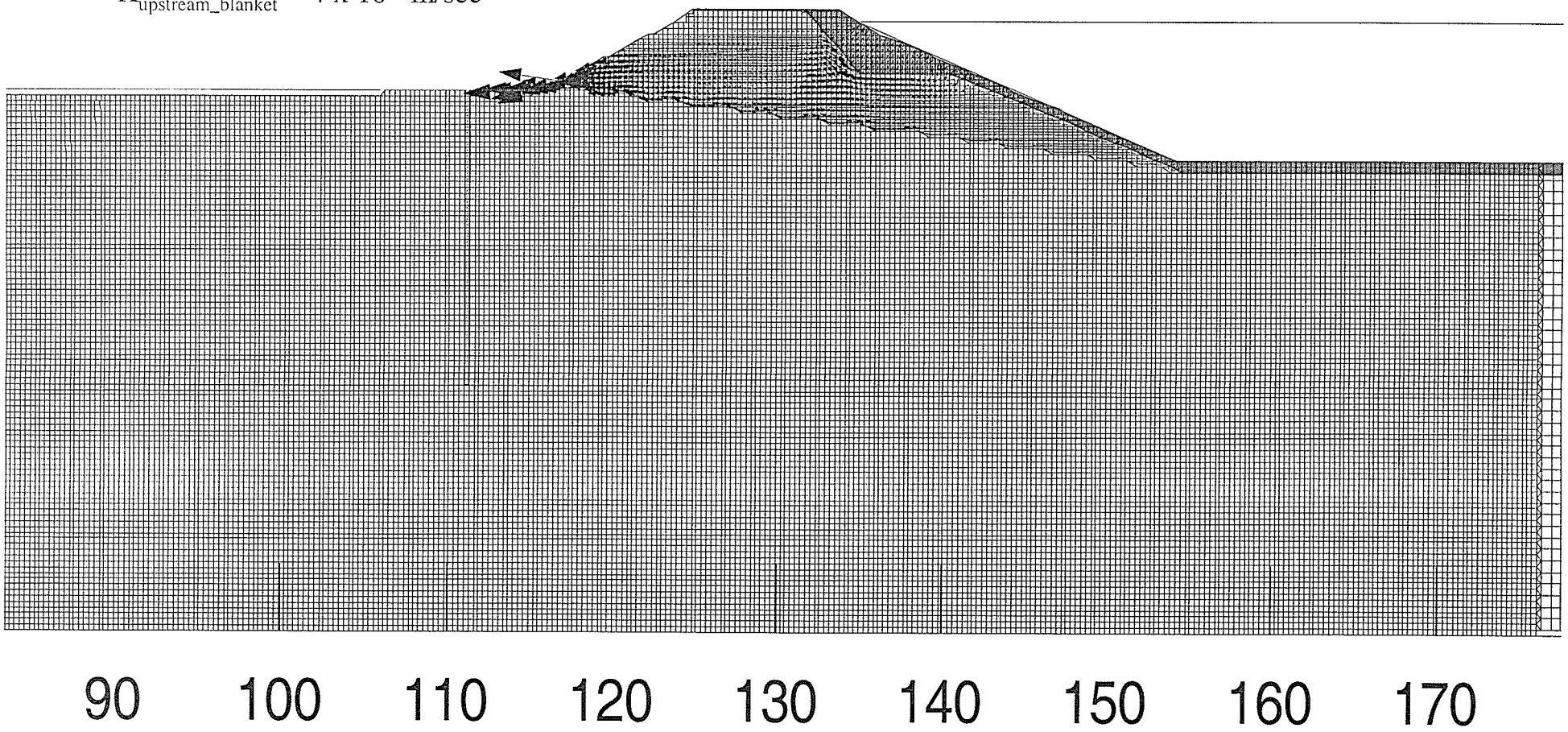
11+940, Scenario 3 - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-2}$ m/sec

$K_{foundation} = 10^{-5}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 4 \times 10^{-4}$ m/sec



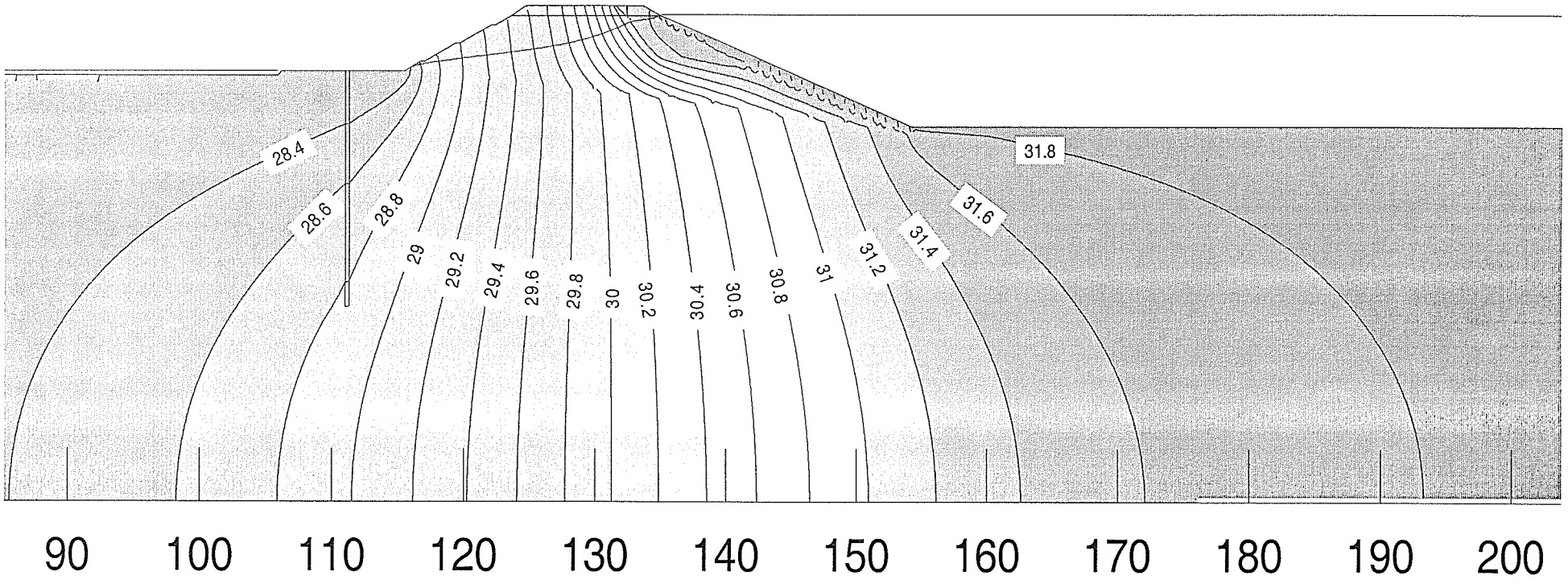
11+940, Scenario 4 - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-4}$ m/sec

$K_{foundation} = 10^{-3}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 4 \times 10^{-4}$ m/sec



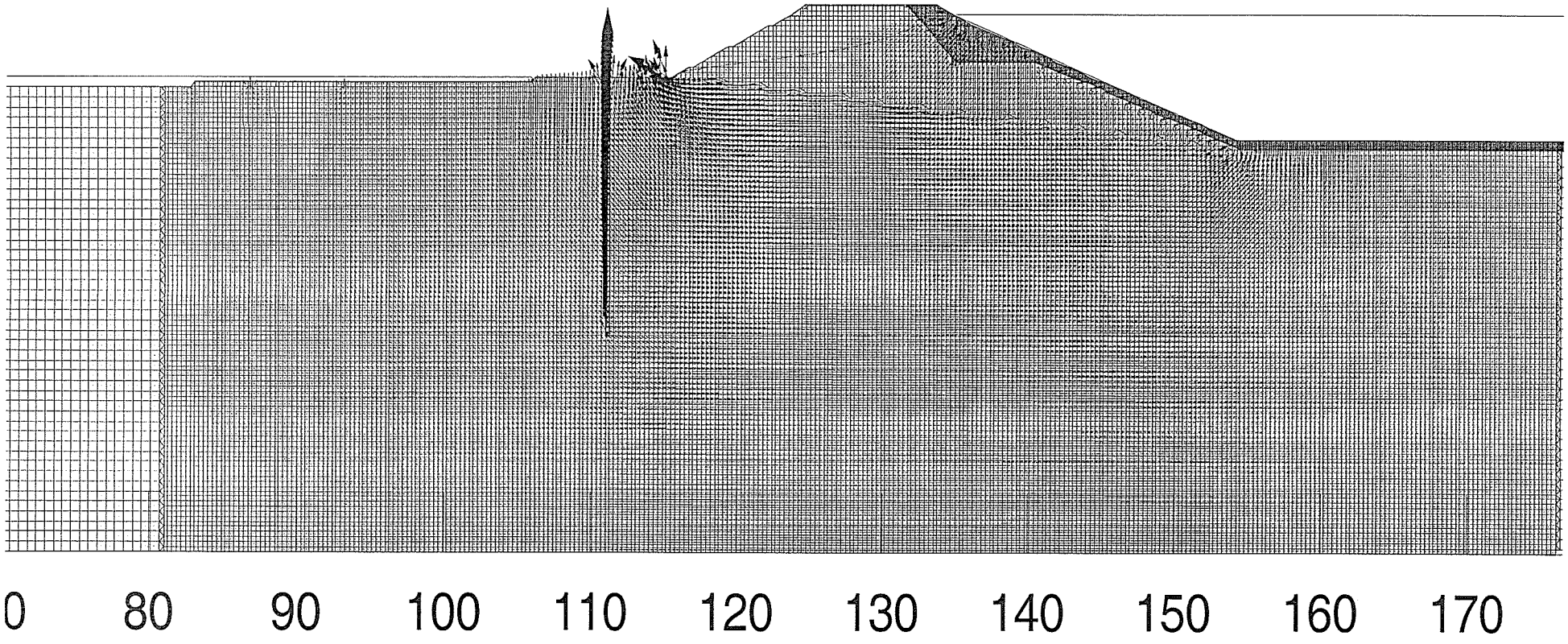
11+940, Scenario 4 - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-4}$ m/sec

$K_{foundation} = 10^{-3}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 4 \times 10^{-4}$ m/sec



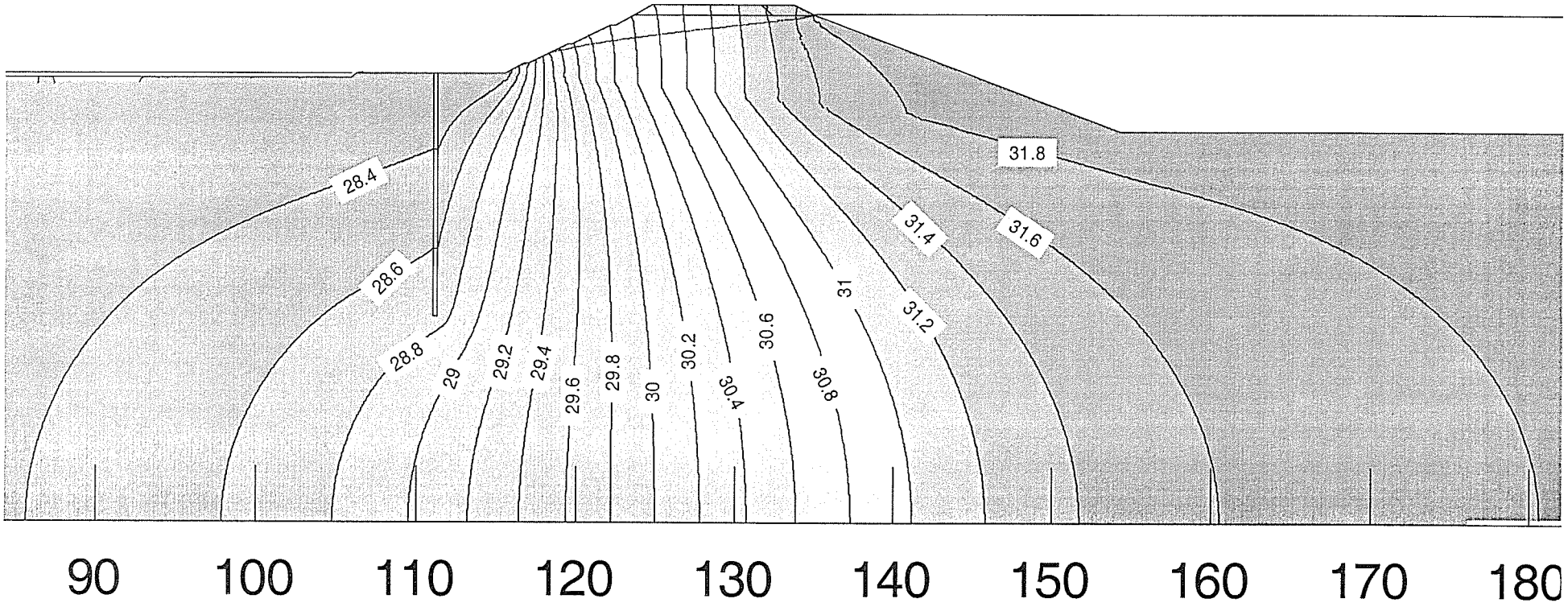
11+940, Scenario 5a - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-4}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 10^{-3}$ m/sec



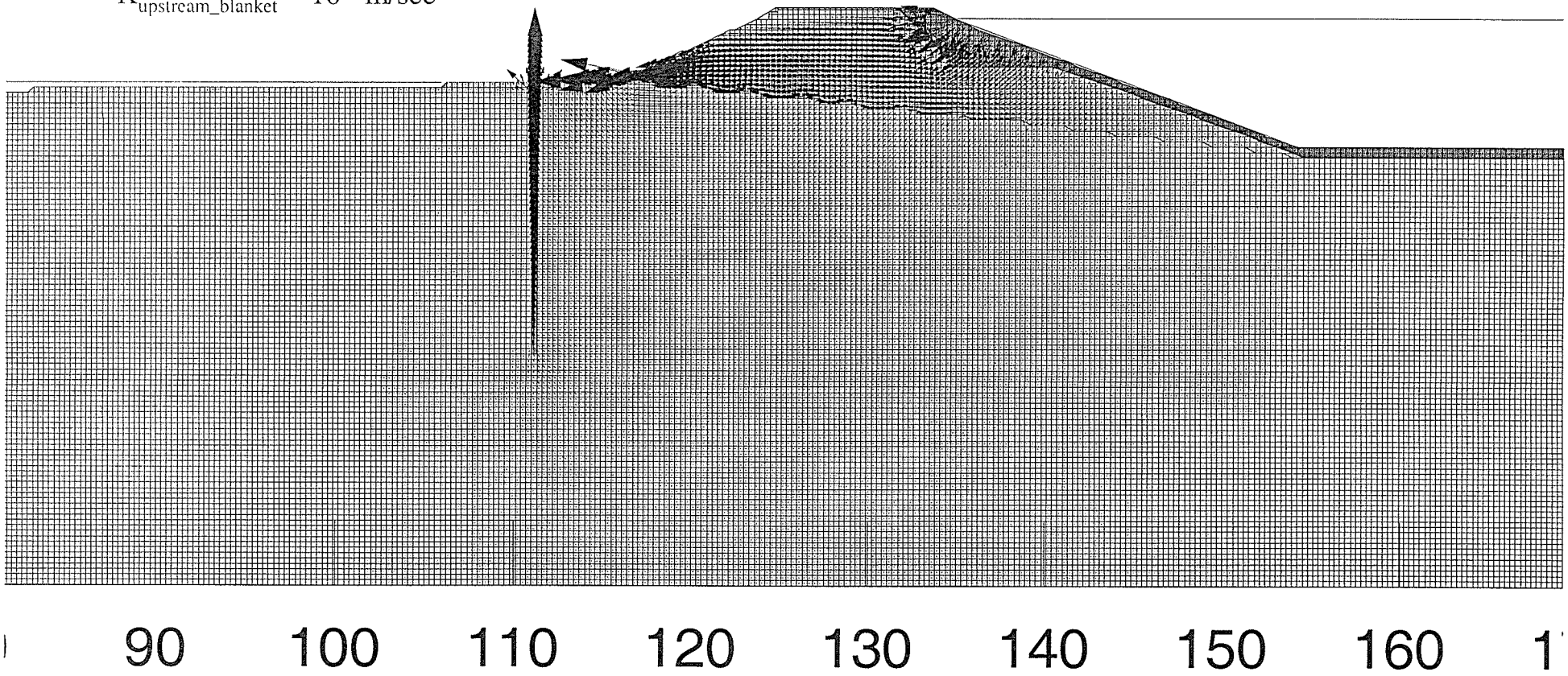
11+940, Scenario 5a - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-4}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 10^{-3}$ m/sec



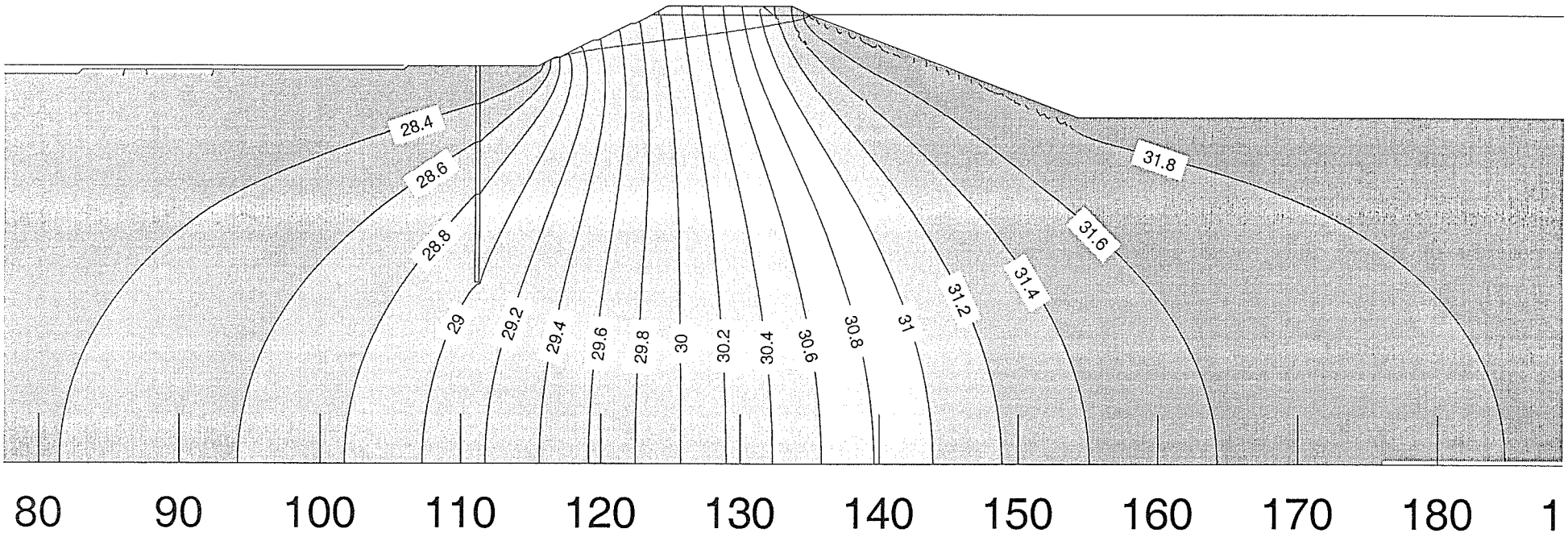
11+940, Scenario 5b - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-3}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

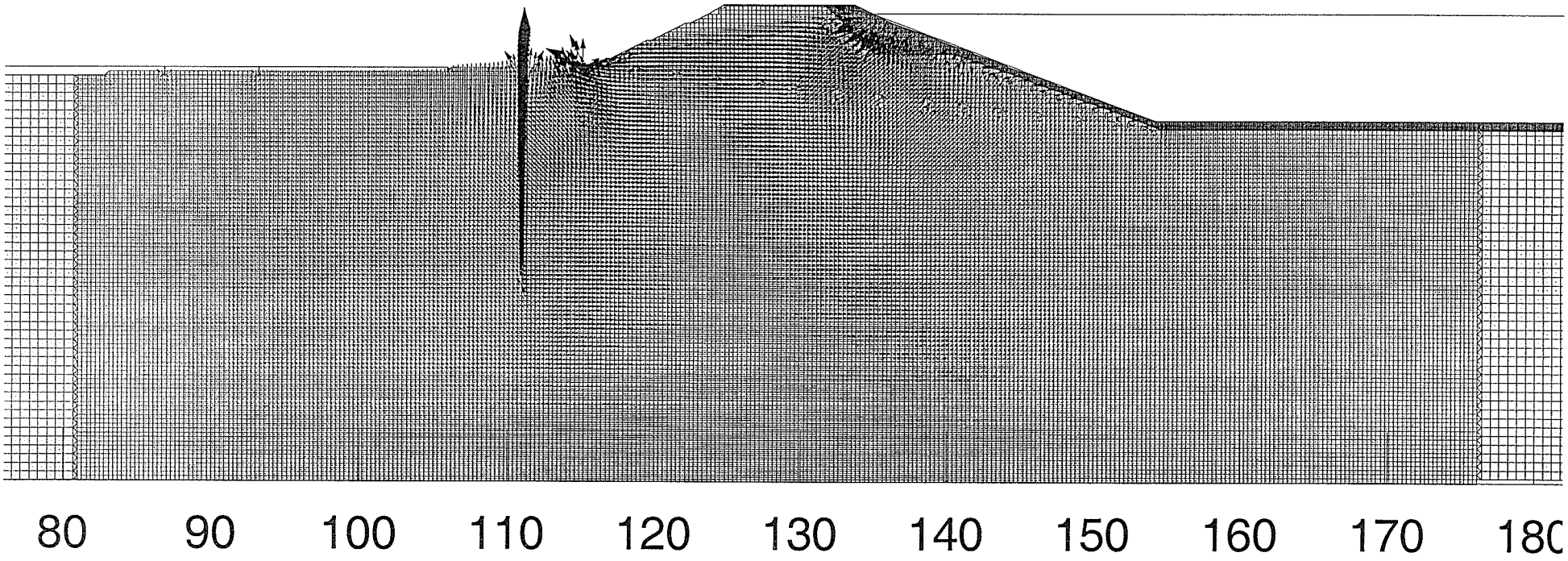
$K_{upstream\_blanket} = 10^{-3}$ m/sec



11+940, Scenario 5b - Nelson Slough – raised dyke and new flood level

$K_{dyke} = 10^{-3}$ m/sec

$K_{foundation} = 10^{-3}$ m/sec

$K_{relief\_well} = 10^{-2}$ m/sec

$K_{upstream\_blanket} = 10^{-3}$ m/sec