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3-3588

nhc

April 25, 2002

District of Hope
PO Box 609
Hope, BC
V0X 1L0

ATTN: Mr. Scott Misumi
Director of Development Services

Dear Mr. Misumi:

RE: JOHNSON ROAD FLOOD HAZARD ASSESSMENT
Final Report – Hydrotechnical Assessment

INTRODUCTION

In early January, a large rain-on-snow event triggered a series of debris flows and floods along Johnson Road, just east of Kawkawa Lake. The combination of high runoff and debris overwhelmed the ditches and culverts along Johnson Road, which caused significant damage to District and private properties. At that time, Northwest Hydraulic Consultants Ltd. (**nhc**) was requested by the District of Hope to assess the magnitude of the rain-on-snow event and to provide recommendations as to how to manage runoff and sedimentation along Johnson Road to Kawkawa Lake.

Following submission of **nhc**'s draft report, a subsequent storm event in February again produced significant runoff volumes along the ditch and confirmed the inadequate capacity of the culverts along the Johnson Road ditch.

This report contains **nhc**'s original assessment following the January storm event, as well as the additional assessment and design work carried out following the February event.

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BACKGROUND

Johnson Road runs in an east-west direction, perpendicular to the eastern shore of Kawkawa Lake, within the District of Hope. The road lies across an alluvial fan at the base of Ogilvie Mountain to the north, and is the northern-most road within the East Kawkawa Lake community. The road drops from approximately El. 100 m at its top end to El. 60 m at its intersection with Kawkawa Lake Road.

A steep creek (referred to as Camilos Creek in earlier reports) flows from Ogilvie Mountain and intersects Johnson Road approximately 275 m up from Kawkawa Lake Road at El. 90 m. The creek has been diverted from its natural course by Johnson Road and most of its flow is directed along a drainage ditch that runs along the northern edge of the road toward Kawkawa Lake. The remaining portion of the flow is directed via a 48-inch culvert below Johnson Road into a concrete lined channel that runs between private properties on the south side of the road. Three 30-inch culverts have been placed along the ditch to accommodate driveways for private properties on the north side of the road. A 42-inch culvert runs beneath Kawakawa Lake Road directing flow from the ditch into Kawkawa Lake.

During the period from January 6 – 8, 2002, a large rain-on-snow event occurred in the Hope area (and across much of the Lower Fraser Valley). The event produced high runoff along Camilos Creek and other creeks in the Ogilvie Mountain drainage basin. Based on District reports, initial flooding in the Johnson Road area occurred at approximately 5:45 pm on Monday January 7. At approximately 6:30 pm, residents were asked to voluntarily evacuate their homes due to the severity of the flooding. Sometime between 11:00 pm and 6:30 am the following morning, a large debris flow or series of debris flows occurred along Camilos Creek. The event overwhelmed the ditch and culverts along Johnson Road, filling the ditch and plugging culverts with talus debris.

A significant volume of debris also spilled across Johnson Road and onto adjacent properties, partially burying automobiles and causing significant property damage. Flood damage also occurred due to the ditch and culvert blockages. It was estimated that approximately 2000 m^3 of debris was trucked away from the site during clean-up operations.

A second major storm event occurred in the area during the period from February 21 – 22, 2002. The event duration was not as long as the January event and it does not appear that a debris torrent accompanied this event. However, significant runoff again transported a large volume of debris into the Johnson Road ditch, over the road and onto private property. The combination of high runoff and debris again overwhelmed the ditch and caused some minor flooding. Debris volumes and flood damage was not as significant as for the January event. It is likely that the majority of the debris was sourced from the unconsolidated bed and banks of the Camilos Creek channel (downstream of the logging road) as a result of the debris event in January. The District estimates that about 160 m^3 of material was removed from the site after this event.

HYDROLOGY

Climate Data

Climate records for the two storm events were obtained for the gauging station at the Hope Airport (Hope A - 1113540). The station collects hourly data and is located at El. 39.0 m.

January 6 – 8, 2002 Storm Event

Figure 1 plots the hourly precipitation and temperature recorded at the Hope A gauge for the period from January 6 – 8, 2002. The plot indicates that precipitation began at about 4:00 pm on January 6 and was followed by a sharp rise in temperature. An initial rainfall intensity peak of 10 mm/hr occurred at 2:00 am on January 7, followed by a second peak of 12 mm/hr at 6:00 pm. Air temperature continued to rise and peaked at 12°C at 7:00 pm on January 7. By this time, 92 mm of rain had fallen at the gauge station and residents along Johnson Road were being evacuated.

Intensity-duration-frequency (IDF) curves were obtained for the Hope Airport gauge and used to estimate the magnitude of the storm event. The most recent published curves are based on gauge data for the period 1964 – 1990. Table 1 lists the return periods associated with various storm duration maximums.

Table 1 – Return Periods for January 6 – 8, 2002 Storm

Duration (hr)	Maximum Total Rainfall (mm)	Return Period (yr)
1	12	< 2
2	22	~ 2
4	36.5	~ 4
6	47	~ 4
12	68.5	~ 3
24	114.5	~ 5

The data in Table 1 indicate a return period of about 2 years for the shorter duration rainfall intensities, with a return period of up to 5 years for the maximum 24-hour duration rainfall. The storm return periods are consistent with the findings of several authors investigating debris torrent events in coastal B.C. (e.g. Miles and Kellerhals, 1981; Church and Miles, 1987). In general, past debris torrents in coastal B.C. have been triggered by concentrated cells of high precipitation, with return periods of 2 – 5 years. Significant snowmelt is also usually combined with the storms, given the rapid temperature increase and rise in freezing levels associated with the storms. A significant rise in temperature was observed for the January 6 – 8 storm, as shown in Figure 1. Orographic effects leading to higher short-term precipitation intensities at higher basin elevations are also suspected during these storms, which would not necessarily be detected in the rain gauge network, particularly for low elevation gauges such as at Hope.

To further investigate the potential orographic effects and snowmelt component of the January 6 – 8 storm, data was obtained from the Wahleach Lake climate and snow pillow gauging stations. Wahleach Lake is located approximately 20 km southwest of Hope. One station records hourly temperature and cumulative precipitation (in mm equivalent of water) at El. 640 m, while the other records snow water equivalent (SWE) data for the snow pillow at El. 1480 m. The SWE is a measure of the equivalent amount of water in a given amount of snow. The gauge only records an increase in the SWE during periods of snowfall.

Figure 2 plots the Wahleach Lake data for both gauges and Figure 3 is a plot of the Wahleach data superimposed onto the Hope Airport data. Figures 2 and 3 indicate that the Wahleach Lake data follows the same general trends during the storm as the Hope data, for both temperature and precipitation, however the Hope data lags the Wahleach data by about 8 hours for precipitation and about 12 hours for temperature.

Although a direct comparison cannot be made between the Wahleach and Hope gauge sites, the sites are close enough to allow for some general observations to be drawn.

Figure 2 indicates that the temperature at the El. 640 m Wahleach gauge was high enough during most of the storm duration that precipitation was falling in the form of rain at that elevation and likely at higher elevations as well (a fully saturated adiabatic lapse rate of 4°C per 1000 m can be assumed). This indicates that there was likely a significant rain-on-snow component to the storm event at the Wahleach gauge.

Precipitation at the gauge ranged from a maximum intensity of 20 mm/hr to a maximum 24-hour rainfall total of 149 mm (compared with 12 mm/hr and 114.5 mm for the Hope A gauge). In addition, Figure 2 indicates a total increase in the SWE at El. 1140 m of 183 mm for the same 24-hour period. This indicates that higher intensity precipitation was occurring at higher elevations.

Thus, based on the data observed at the Wahleach gauges, the following can be inferred for the Camilos Creek basin:

- air temperatures were likely above zero at all elevations in the basin during the storm, so that an additional rain-on-snow component would have added to the storm runoff and increased return period frequencies above those reported in Table 1 for the Hope Airport, and;

- orographic effects likely would have increased precipitation intensities and cumulative precipitation amounts at higher elevations in the basin, again resulting in increased return period frequencies than for the low elevation Hope Airport gauge.

An estimate of the snowmelt contribution to the total runoff in the Camilos Creek basin can be made using the temperature and precipitation data recorded at the Hope Airport.

The fully saturated adiabatic lapse rate of 4 °C per 1000 m is applied to estimate temperatures at higher basin elevations. Melt is then estimated for elevation bands in the basin using semi-empirical snowmelt equations developed by the US Army Corps of Engineers, based on energy budget methods. Figure 4 plots the snowmelt amounts estimated for each hour during the 24-hour period of maximum rainfall accumulation during the storm. Melt estimates as high as 2.6 mm/hr were estimated at the storm peak. Table 2 lists the total precipitation amounts for various maximum rainfall durations, including snowmelt amounts.

Table 2 – Snowmelt Estimation for January 6 – 8, 2002 Storm

Duration (hr)	Maximum Recorded Rainfall (mm)	Estimated Snowmelt (mm)	Total (mm)	Increase (%)
1	12	2.2	14.2	18
2	22	4.8	26.8	22
4	36.5	8.2	44.7	22
6	47	11.0	58	23
12	68.5	18.6	57.1	27
24	114.5	28.6	143.1	25

As indicated in Table 2, precipitation totals increase as much as 27 % when snowmelt is accounted for. These totals would also likely increase further with consideration of orographic effects at the higher basin elevations.

February 21 - 22, 2002 Storm Event

Figure 5 plots the hourly precipitation and temperature recorded at the Hope A gauge for the period from February 20 – 22, 2002. The plot indicates a single rainfall intensity peak of 11.5 mm/hr at 8:00 pm on February 21. Air temperature began to rise during the storm, but did not peak until after the rainfall peak and was not as high as during the January event. Unlike the January event, it is not suspected that a significant snowmelt component contributed to the storm runoff.

Table 3 lists the estimated return periods associated with the February event for various storm duration maximums.

Table 3 – Return Periods for February 21 - 22, 2002 Storm

Duration (hr)	Maximum Total Rainfall (mm)	Return Period (yr)
1	11.5	< 2
2	22	~ 2
4	35	~ 3
6	47	~ 4
12	80	~ 7
24	126	~ 10

The data in Table 3 indicate a return period of about 2 years for the shorter duration rainfall intensities, rising to a return period as high as 10 years for the maximum 24-hour duration rainfall.

A comparison of the two storm events shows that the February event was shorter than the January event and thus total precipitation amounts were less - 147.5 mm over 43 hours for the February 21 – 22 event compared to about 180 mm over 56 hours for the January 6 – 8 event. However, the longer duration rainfall intensities (12 and 24 hour) were somewhat higher for the February event, leading to higher return period estimates for these durations.

Discharge Estimates

Peak discharge was estimated for the storm runoff for Camilos Creek. The creek has a drainage area of 0.81 km^2. The rational method for estimating peak flows can be applied, based on the time of concentration for the basin, and the IDF curves for the Hope Airport. Time of concentration was estimated at 1 hour, based on Provincial guidelines developed for mountain watersheds. The discharge coefficient was estimated as 1.0, given that saturated basin conditions would prevail during a multi-day storm, as was the case for the January 6 – 8 storm (Figure 1). Table 4 lists the computed peak flow estimates for the January 7 and February 21 storm peaks and for 1-hour precipitation rates for various return periods.

Table 4– Peak Flow Estimates – Hope Airport IDF Curves (1 hr Duration)

Precipitation (mm)	Peak Discharge (m^3/s)	Return Period (yr)
12	2.7	Jan. 7 storm peak
14.2	3.2	Jan. 7 peak incl. snowmelt
11.5	2.6	Feb. 21storm peak
14.3	3.2	2
17	3.9	5
19.5	4.4	10
23	5.2	25
26	5.9	50

The data in Table 4 indicate that the peak discharge for both storm events was just under 3 m^3/s, slightly less than a 2-year discharge. As outlined earlier, the actual peak instantaneous value may be higher due to orographic effects producing higher intensity precipitation at the upper basin elevations.

In a 1987 report by Stanley Associates Engineering Ltd., the Creager method was used to estimate peak discharges for the Camilos Creek basin. Estimates for the 2-year peak flow ranged from 1.2 to 2.8 m^3/s, which compares well with the estimates in Table 3. The study also estimated the 200-year peak flow would fall between the likely range of 4.1 to 9.7 m^3/s.

HYDRAULICS

Drainage

The capacity of the drainage ditch and culverts along the north side of Johnson Road was assessed following the January storm event, given the predicted peak flows computed in Table 4. The ditch measures approximately 1.2 m wide along the invert and 1.6 m deep, with side slopes varying from 0.5H:1V to 1H:1V. Invert slopes along the ditch vary from approximately 3% at the bottom end to 11% at the upstream end of the ditch. Three 30-inch culverts are spaced along the 275 m length of the ditch.

Based on the dimensions of the ditch, the maximum capacity of the ditch (without culverts) is approximately 6 m^3/s, constrained by the minimum slope at the downstream end of the ditch. This assumes a clean ditch, with no bed aggradation. Based on the flow estimates in Table 4, the ditch itself would have enough capacity to handle high return period flows.

However, the culverts along the ditch and under Kawkawa Lake Road govern the present drainage capacity of the ditch. The existing capacity for each of the 30-inch culverts is calculated at 1 m^3/s, assuming no debris blockage. The culverts significantly reduce the drainage capacity of the ditch, as evidenced during the both storm events.

From Table 4, it is evident that the current culvert configuration can pass less than a two-year event. However, according to the District, there had not been any significant problems with drainage along the ditch prior to the debris flow event. This discrepancy suggests that a portion of storm runoff had either been infiltrating into the alluvial fan complex above the ditch or was diverted through the culvert across Johnson Road, where the creek first enters the ditch. This 48-inch diameter culvert, which is currently blocked, has a capacity of 2 m^3/s and directs flow into a concrete lined channel that runs between properties on the south side of Johnson Road (although the District believes that significant flows were not being directed thru this culvert and would prefer that this is the case for future mitigation).

It is possible that the debris flow event in January may have had an impact on the drainage patterns of Camilos Creek at the logging road. It appears that runoff is now concentrated along the creek channel below the road, rather than being allowed to disperse along two or more channels identified below the road during our site visit. The event deposited debris across the road, essentially channelizing the creek across the road and into the channel feeding the Johnson Road Ditch. This would appear to be the case for the February storm event, which again saw high runoff volume directed into the Johnson Road ditch.

Culvert Replacement

In order to increase the drainage capacity of the ditch, the existing culverts along the ditch and below Kawkawa Lake Road will need to be replaced with larger culverts. Alternatively, small bridge crossings could be constructed over the ditch to accommodate property driveways which would significantly improve the capacity of the ditch. The culvert below Kawkawa Lake Road would still have to be upgraded or replaced.

Circular Culverts

The largest practical circular culvert that can fit within the existing ditch is 48-inches. As stated above, the capacity of 48-inch culverts (similar to the one presently below Johnson Road) is only about 2 m^3/s, and would therefore not significantly improve drainage along the ditch, given the peak flows calculated in Table 4.

Box Culverts

An alternative to circular culverts would be the use of pre-cast concrete box culverts along the ditch and below Kawkawa Lake Road. The largest practical size that would fit the existing ditch dimensions would be a 4 x 6 foot (1.2 x 1.8 meter) culvert (rise x span).

As with circular culverts, the capacity of a box culvert depends on the entrance conditions, with slightly more capacity available if the culvert entrance has flared wingwalls on either side. However, given the existing ditch width, culvert wingwalls would not be feasible and a vertical headwall entrance would be used. Given this type of inlet, the maximum capacity of the culvert that would give a headwater depth-to-rise ratio (HW/D) of 1.0 (i.e. inlet water level just at top of culvert rise) is 3.4 m^3/s. This is just over the 2-yr discharge calculated in Table 4.

HW/D ratios corresponding to the 5-yr and 10-yr discharges in Table 4 are 1.1 and 1.2, respectively. Thus, to pass flows higher than the 2-yr discharge without overtopping of the culvert would require additional headwall height above the top of the culvert. For the 10-yr discharge for example, the HW/D ratio of 1.2 would result in an inlet water level approximately 0.25 m above the top of the culvert. Use of lock-blocks for the headwall (0.75 m high) would provide sufficient height as well as an additional 0.5 m of freeboard.

Hydraulic Modeling

To simulate the box culverts along the ditch and to examine resulting water levels and flow velocities along the ditch, a simple hydraulic model was developed using HEC-RAS. Data for the model was obtained from a survey conducted by the District along the lower 120 m of the ditch, between Kawkawa Lake and the furthest downstream ditch culvert. Flows up to the 10-yr discharge (Table 4) were examined in the model.

Figure 6 shows the modeled water surface elevation profile along the surveyed ditch. Refer to Figures 7 – 11 for conceptual sketches (superimposed onto the survey plan provided by the District) of the proposed culvert replacement and channel modifications described.

Water Levels

Model results indicate that the drainage ditch upstream of the Kawkawa Lake Road culvert would have sufficient capacity to handle a 10-yr discharge, assuming the channel cross-sectional profile remains as shown in the District's survey plan.

The plot in Figure 6 shows that water levels will not go overbank between culverts, but will rise at the headwalls and spill onto Johnson Road. As a result, the left edge of the ditch would need to be raised and tied into the headwall (Figure 7). The right bank should also be raised to provide some freeboard. The protection should extend approximately 5 m upstream of the inlet. This could be in the form of lock-blocks along the road (left bank) and a berm along the right bank. Protection should extend to 1.8 m above the culvert invert elevation, which would provide 0.3 m of freeboard for the 10-yr discharge.

As indicated in Figure 6, water levels along the ditch downstream of the Kawkawa Lake Road culvert would generally go overbank for flows greater than the 2-yr discharge, given the narrower cross-section profile along this section of channel and the lower bank heights, particularly the right bank. It is recommended that the channel section be widened to 1.2 m and that bank heights be raised to about 1.1 m above the bed level along this section. Banks should be sloped no steeper than 1.5H:1V (Figure 9).

An alternative to raising the banks of the widened channel downstream of the Kawkawa Lake Road culvert would be to lower the bed to contain high flows. The bed would need to be lowered uniformly for the entire length of channel to the lake. Lowering the bed by about 0.3 m (1 ft) below its present level should be sufficient, although a small berm should still be provided (about 0.2 m high) for freeboard allowance along low banks, such as along the low right bank around STA 0+80 (Figure 9). Existing bank heights around STA 0+60 should be sufficient for the lowered bed/widened channel.

One concern with lowering the bed is the possibility of infilling over time, particularly at the downstream end of this section of channel as it flows into the lake. Small gravels and fines will inevitably deposit here during low flow periods (even if the bed were not lowered) and the upstream extent of the deposition will depend on lake levels. Higher flows may flush the fines downstream, but it is envisioned that routine monitoring and occasional maintenance will be required if deposition begins to significantly raise bed levels.

Velocities and Erosion Protection

Model results indicate channel velocities in the order of 2.5 – 3.5 m/s for the 10-yr discharge, with velocities as high as 5 m/s at the culvert outlets. To prevent channel erosion, the invert and banks of the channel should be lined with rock in the order of 300 mm (12 in), particularly along the steeper upstream section of the ditch, with slightly smaller material required for the lower gradients at the downstream end of the ditch and below Kawkawa Lake Road.

Theoretical calculations of stable stone sizes for the upper section of the ditch vary widely from approximately 300 mm – 800 mm at the 10-yr discharge. However, given the size of the ditch, it would not be feasible to use large boulders to line the channel. It is suggested that the lower end of the range be used (i.e. 300 mm) and maintenance to the channel be carried out should larger flow events initiate significant bed movement.

At culvert outlets, 500 mm boulders should be keyed into the bed and banks for at least 5 m of length, to prevent undermining of the culvert and/or headcutting upstream if the bed below the Kawkawa Lake Road culvert is lowered.

Sediment Issues

Sediment issues will be an ongoing concern for the near future since the channel bed and banks between Johnson Road and the forestry road are loose, unconsolidated and exposed. Sediment sizes along the channel below the forestry road range from gravel up to approximately 0.8 m diameter boulders. Gravel and small cobbles will be transported towards the ditch/culvert system, which will reduce the capacity of the ditch and culverts, and affect the outflow at the lake.

To deal with the constant sediment influx toward the ditch, a sediment trap is proposed above the ditch. A conservative estimate of the volume of the available mobile sediment along the channel below the forestry road is approximately 400 m^3. Sediment volumes above the forestry road were not assessed, but it was observed that the lower 250 m of the steep gully was essentially devoid of accumulated sediment and that several bowl/cascade complexes dominate the upper creek profile, providing some long-term sediment storage capacity.

Assuming a trap depth of 1.5 m and width of 4.5 m, the total available volume of the trap would then depend on its length. A length of 15 m would give a trap volume of about 100 m^3, providing storage for about 25 % of the long-term sediment volume.

Regular maintenance to remove accumulated sediment would be required on an on-going, as-needed basis. Maintenance frequency is difficult to estimate, but will depend on the frequency of flood events during the year.

Figure 12 presents a sketch of the proposed sediment trap. The downstream end of the trap would consist of a porous boulder berm. The berm material should be uniform in size at 600 – 800 mm, which would prevent small gravel and cobble material from passing through, but would allow runoff to exit the trap. Boulders would also be required at the upstream end of the trap to prevent upstream headcutting. Part of the trap maintenance would require temporarily removing the rocks that form the downstream berm, and then replacing them to re-establish the porous outlet.

The sediment trap will significantly reduce the regular supply of sediment to the ditch, but inevitably small gravels and fines may still pass through the trap and any small gravels and fines below the trap will move downstream. The smaller material should be flushed downstream at higher flows or will tend to cement the larger material along the channel. Velocities through the culverts are generally high enough that significant deposition within the culverts should not be a concern. Material may deposit at low flows, but should be flushed out at higher discharges. Theoretical calculations for sediment deposition through the lower gradient (3%) Kawkawa Lake Road culvert indicate that material below about 30 mm should pass through at a discharge of 0.5 m^3/s ranging to about 250 mm and below at the 10-yr discharge of 4.4 m^3/s.

Channel Alignment

It is recommended that the channel alignment approaching the ditch be modified, as shown in Figure 12. The re-alignment would improve flow transition into the ditch and reduce the potential for flow surges up onto the road at high flows. The bed and banks of the ditch at the channel entrance should be armoured with 0.5 m diameter (or larger) rock to prevent channel erosion and potential undermining of the road in this area.

To further reduce the potential for high flows to overtop the road at the channel entrance, a lock-block retaining wall is proposed along the left bank consisting of a line of 3 - 4 blocks. One row of blocks should be set below grade, with two rows above. This would provide approximately 0.3 m of freeboard to contain flow run-up at a 50-year discharge. The upper row of blocks would be stabilized by placing rebar or threaded rod through the top two rows of blocks.

To prevent significant flows from entering the 48" culvert below Johnson Road, the culvert inlet should be covered with a steel plate. A small vertical slot cut into the center of the plate could be provided to allow some flow to pass into the culvert. However, given the realignment of the channel entering the ditch, a small rock weir would also be required in the ditch downstream of the channel entrance to allow flow to pond up above the culvert invert (Figure 12). Estimates of slot width and weir height (to supply a given flow into the culvert) can be provided once the ditch invert elevation at the weir location is known, relative to the culvert invert elevation. Regular maintenance to remove small sediment build-up upstream of the weir will be required.

Debris Torrent Concerns

The recommendations given in this report do not address the potential for the occurrence of debris torrents along Camilos Creek. The 1987 report by Stanley and Associates provides a detailed assessment of debris torrent potential and magnitude along the creek and other creeks in the area and suggests several long-term mitigation measures. The report concludes that there is a high probability for a 7000 m^3 magnitude debris flow to occur along Camilos Creek.

The debris torrent that occurred on January 7, 2002 deposited about 2000 m^3 of material along Johnson Road, with a similar volume deposited on the upstream fan and at the forestry road. Computed velocity estimates of the peak debris flow based on field observations are in the order of 3 m/s, with a flow area estimated in the order of 22 m^2. This results in a peak flow estimate of 60 – 70 m^3/s for the event.

CONCLUSIONS

Based on the information collected during our site visit and the assessment carried out in this report, the following conclusions can be drawn:

- the January 6 – 8, 2002 storm event had a return period of approximately 2 - 5 years, based on rainfall data collected at the Hope Airport gauge. It is suspected that rain-on-snow increased the runoff at Camilos Creek by about 20 % above that predicted by the Hope Airport data and that orographic effects would likely have further increased this amount, based on climate data collected from Wahleach Lake;

- the February 21 – 22, 2002 event has similar return periods for the shorter duration rainfalls, however the return period was as high as 10 years for the 24-hour duration rainfall. Significant snowmelt is not suspected during this event;

- the existing 30-inch diameter culverts in the drainage ditch have a maximum capacity of 1 m^3/s, limiting drainage along the ditch to less than a 2-year event, assuming that all the runoff is directed into the ditch. Installation of 48-inch diameter culverts along the ditch, would increase drainage capacity to 2 m^3/s;

- replacing the circular culverts along the ditch and below Kawkawa Lake Road with 1.2 m by 1.8 m (rise x span) concrete box culverts increases drainage capacity along the ditch to approximately 3.4 m^3/s without overtopping, and to about 4.4 m^3/s (a 10-yr event) by providing headwalls at the culvert inlets; and,

- a sediment trap measuring 4.5 m wide by 15 m long by 1.5 m deep would provide storage for approximately 25% of the estimated available mobile sediment volume along the channel below the forestry road, and help to reduce sediment accumulation along the ditch and/or culverts.

* * *

I trust that this assessment meets your present requirements. Please call if we can provide any additional information or if you wish to discuss our report.

Yours truly,

northwest hydraulic consultants ltd.

B. Walsh

PROFESSIONAL PROVINCE OF B. W. WALSH ... COLUMBIA ENGINEER

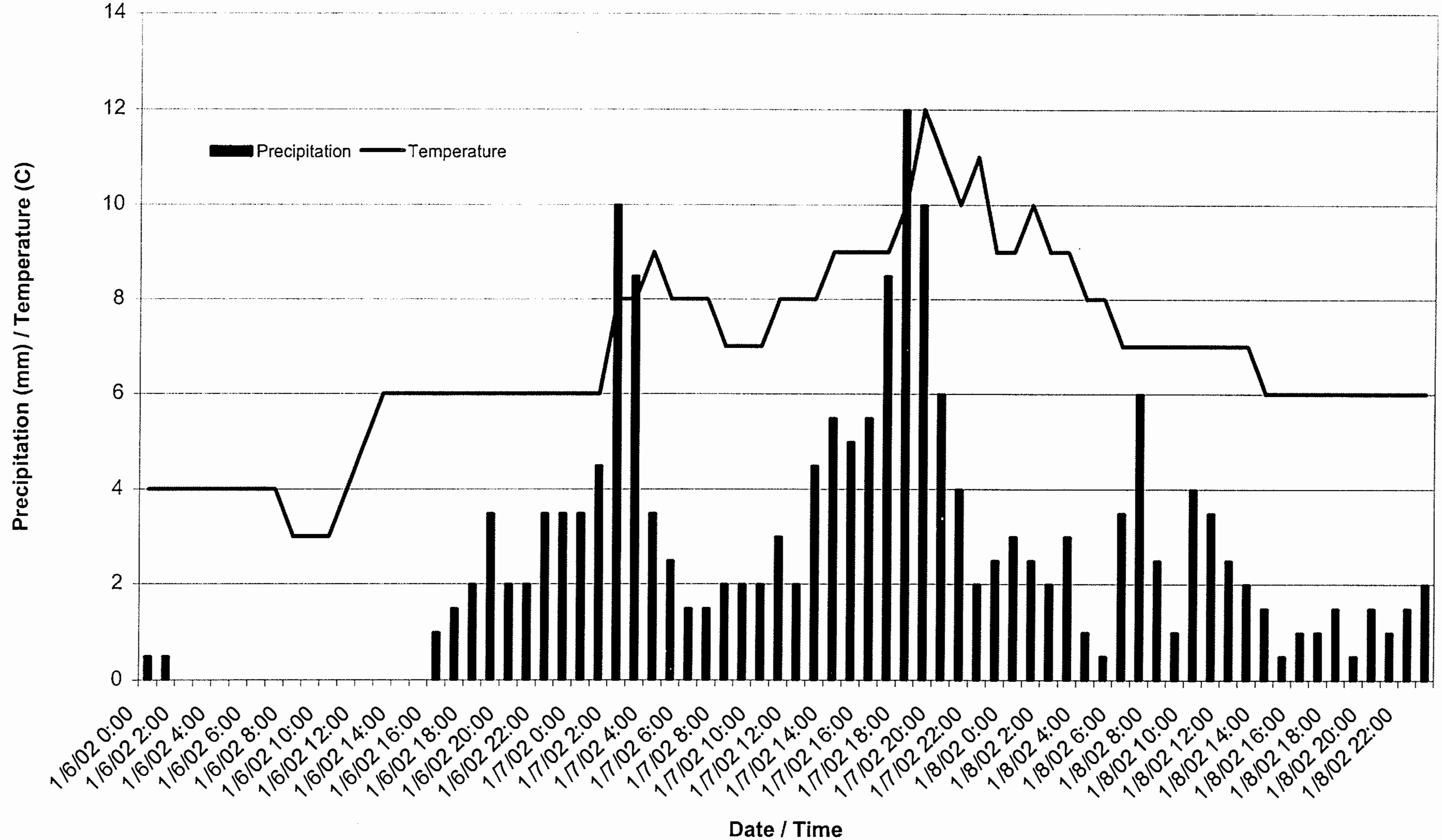
Bruce Walsh, P.Eng.
Principal

Dave Strajt, E.I.T.
Project Engineer

Figures

FIGURE 1

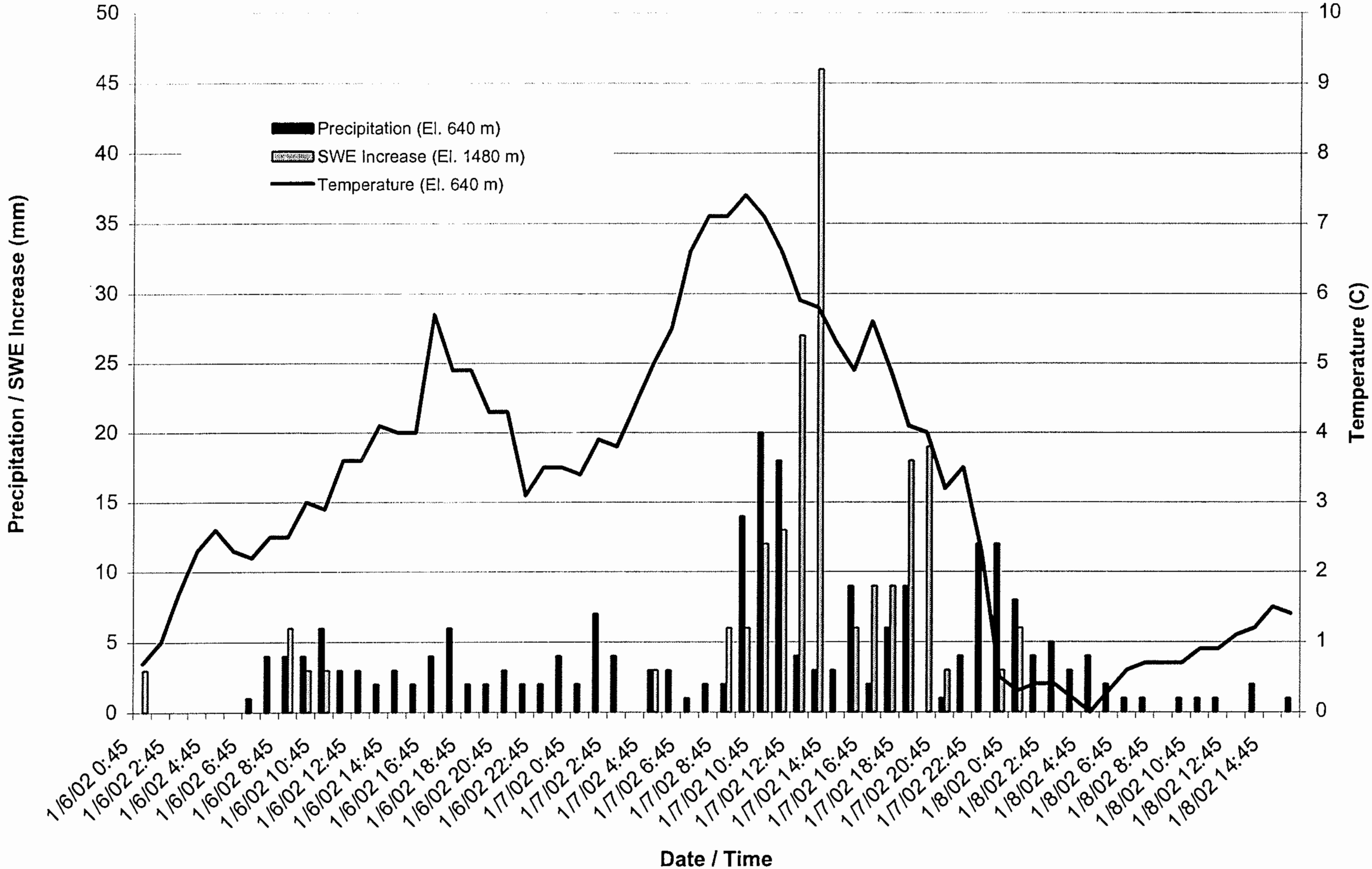
Climate Data - Hope Airport
January 6 - 8, 2002



Hope A.xls

FIGURE 2

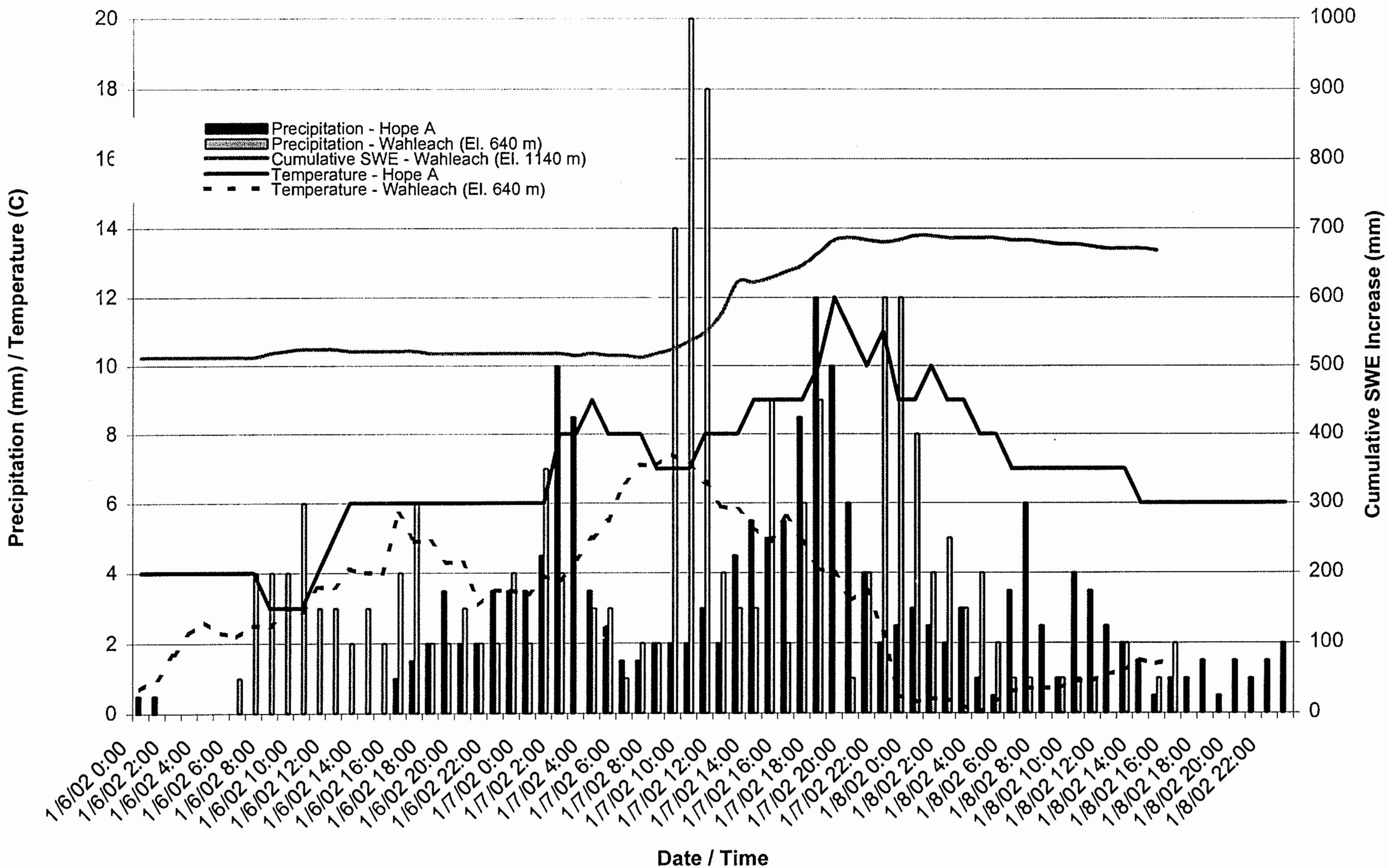
Climate Data - Wahleach Lake
January 6 - 8, 2002



Hope A.xls

FIGURE 3

Combined Climate Data - Hope Airport and Wahleach Lake
January 6 - 8, 2002



Hope A.xls

FIGURE 4

Snowmelt Estimates - Hope Airport Data

January 7, 2002

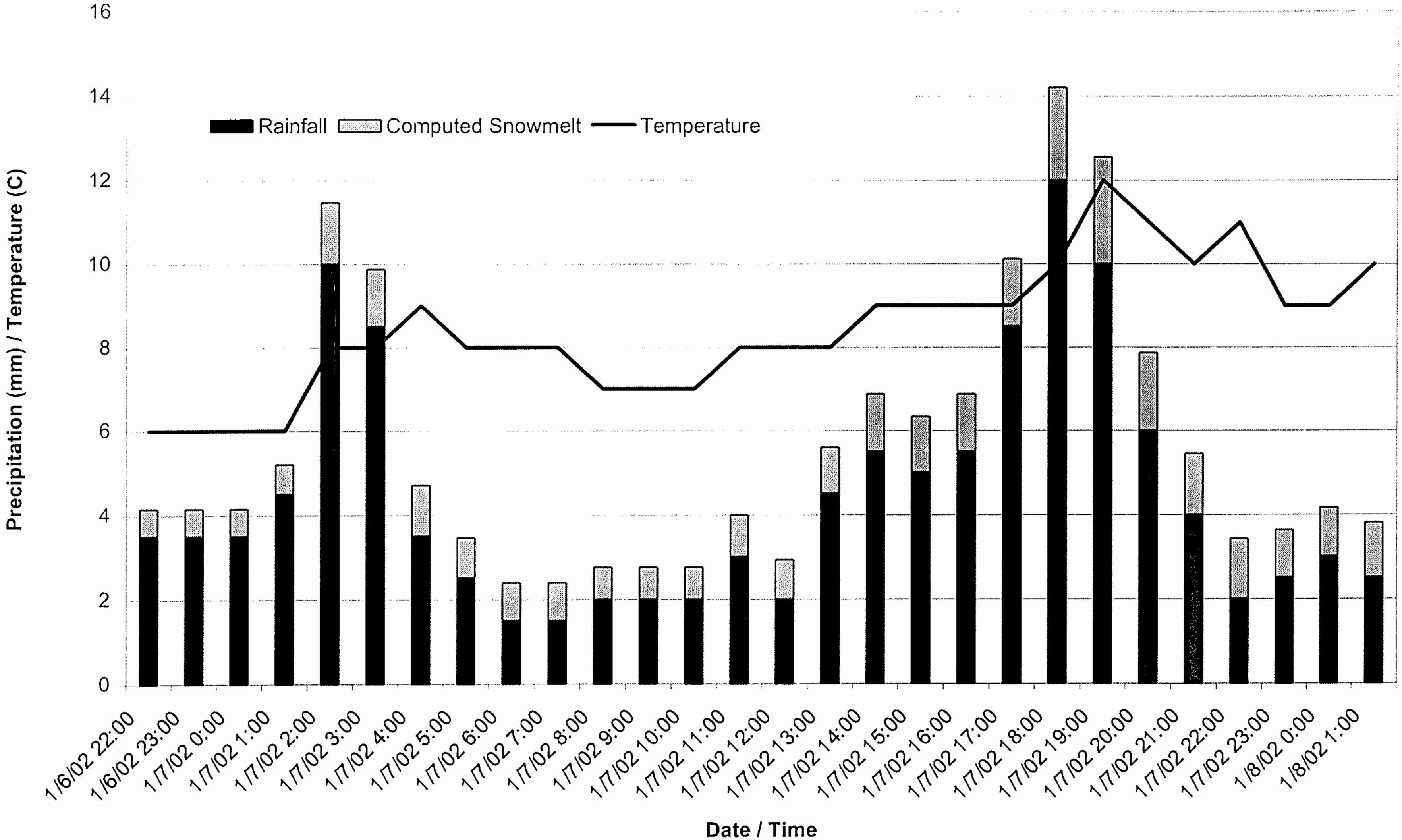
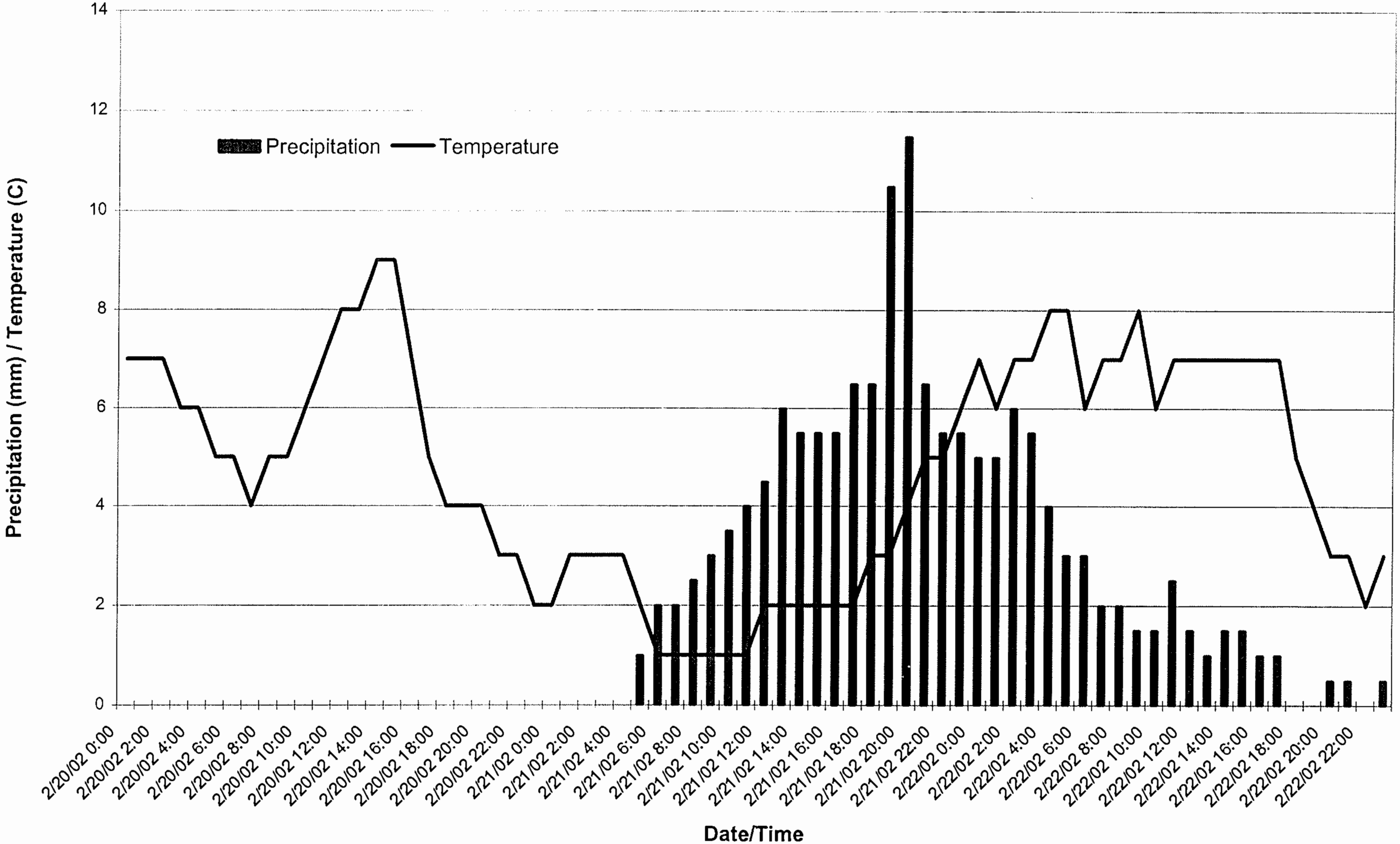


FIGURE 5

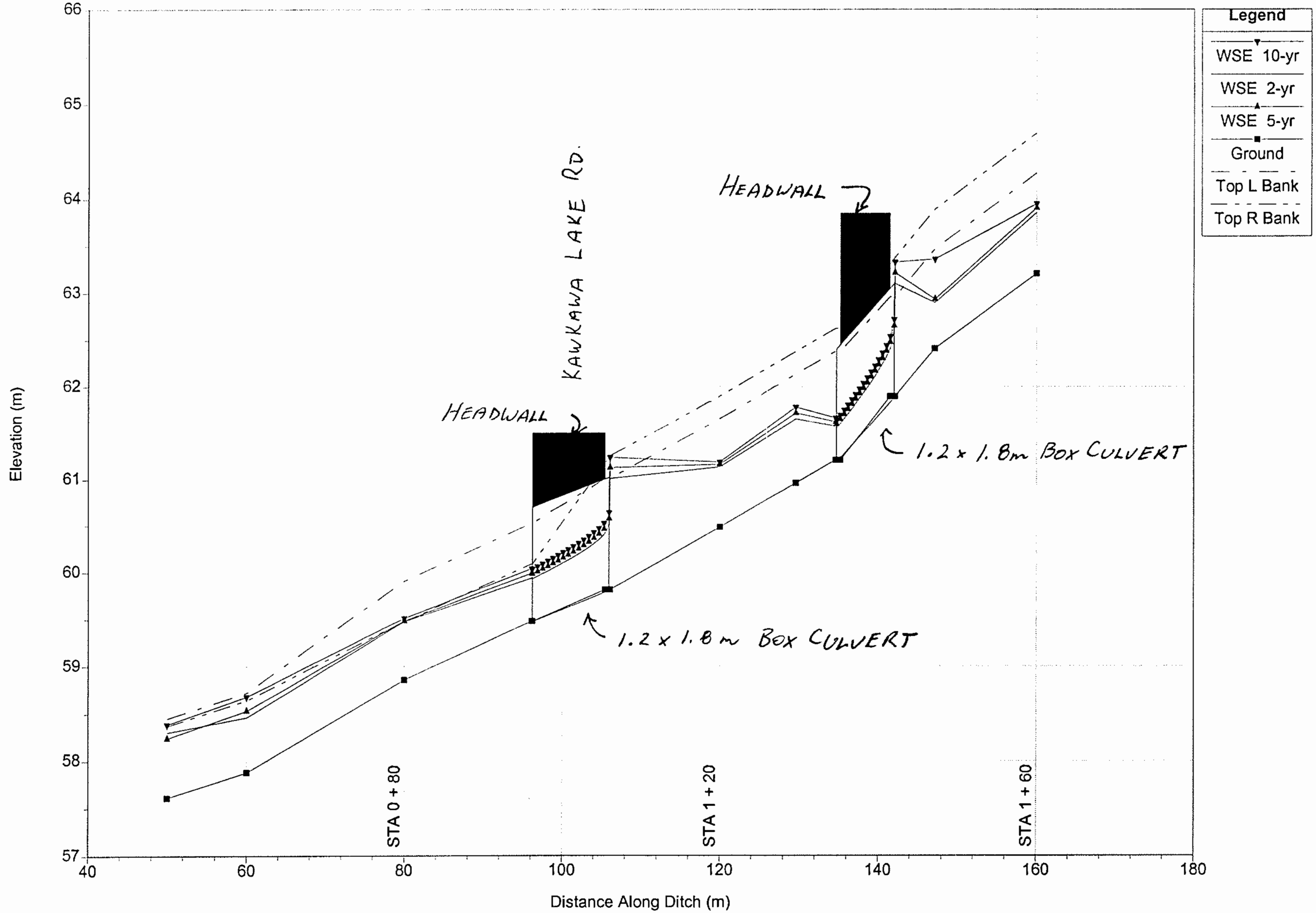
Climate Data - Hope Airport
February 20 - 22, 2002

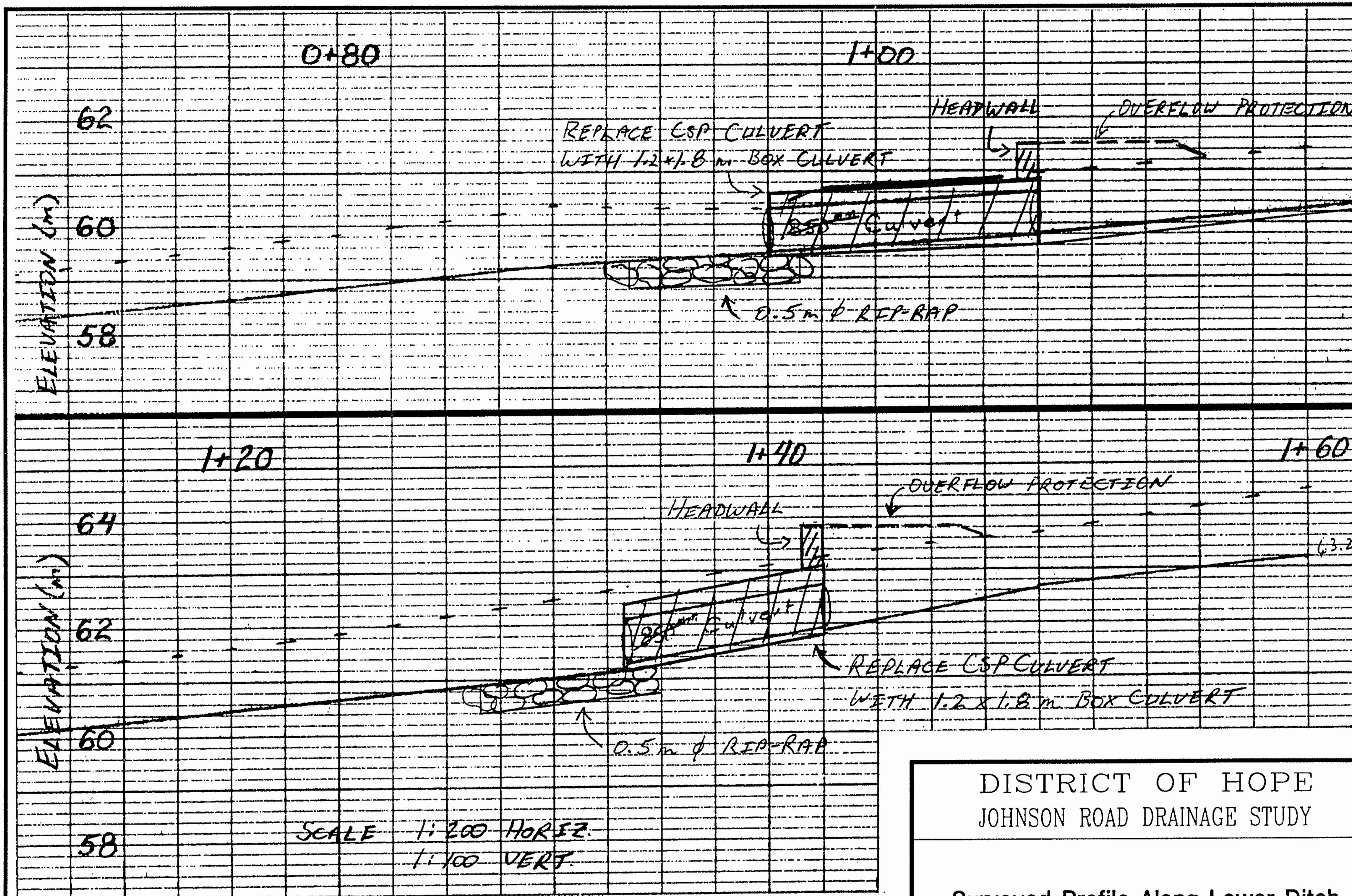


Feb Event.xls

FIGURE 6

Modeled Water Surface Profile





NOTES:

1. DRAWING BASED ON 03/11/02 SURVEY PLAN PROVIDED BY THE DISTRICT OF HOPE.

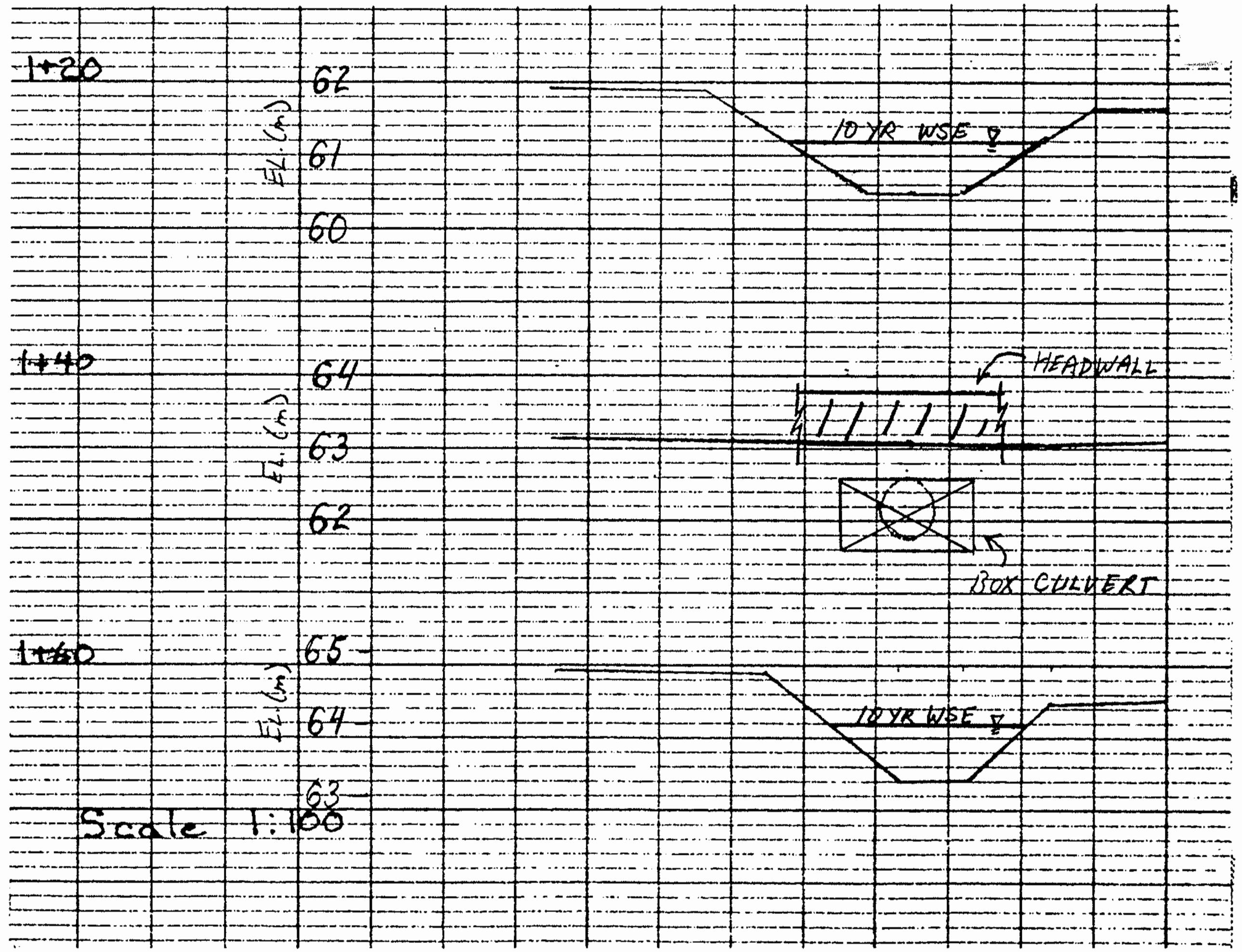
NHCV

DISTRICT OF HOPE
JOHNSON ROAD DRAINAGE STUDY

Surveyed Profile Along Lower Ditch

northwest hydraulic consultants

FIGURE 7



SCALE
1:100

NOTES:

1. DRAWING BASED ON 03/11/02 SURVEY PLAN PROVIDED BY THE DISTRICT OF HOPE.

NHCV

DISTRICT OF HOPE
JOHNSON ROAD DRAINAGE STUDY

Surveyed Cross-Sections
STN. 1+20 - 1+60

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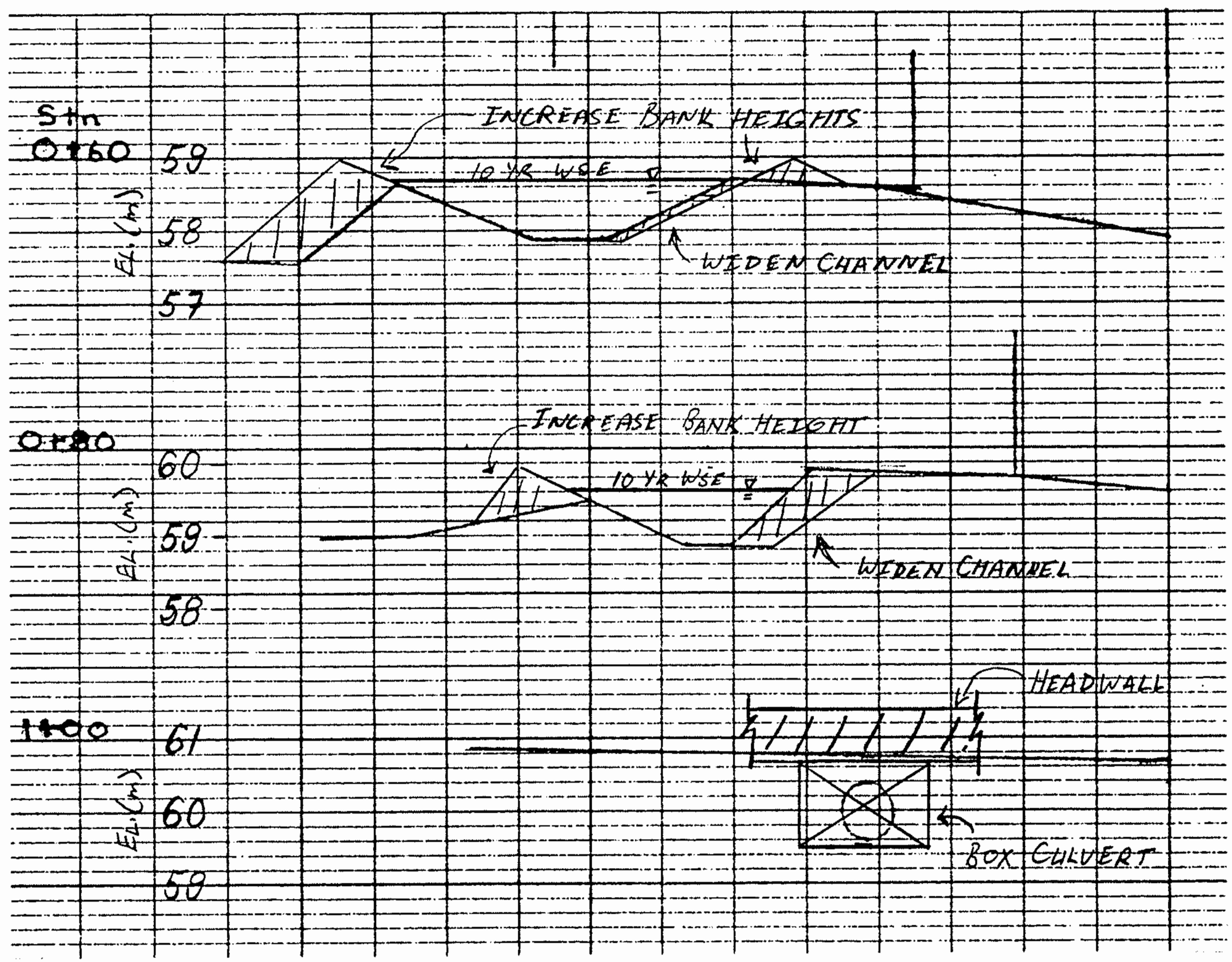
FIGURE 8



SCALE
1:100

NOTES:

1. DRAWING BASED ON 03/11/02 SURVEY PLAN PROVIDED BY THE DISTRICT OF HOPE.

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DISTRICT OF HOPE
JOHNSON ROAD DRAINAGE STUDY

Surveyed Cross-Sections
STN. 0+60 - 1+00

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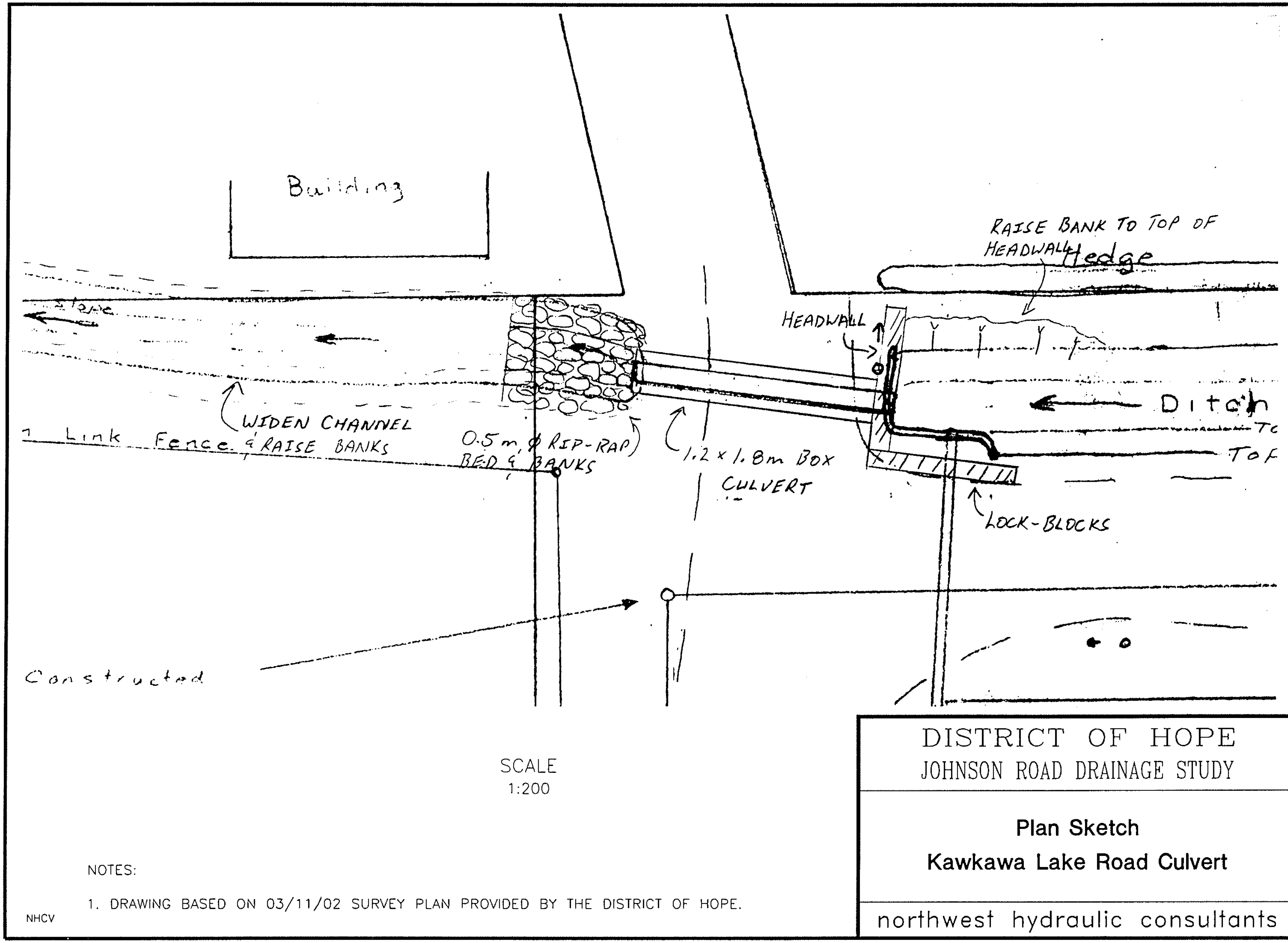
FIGURE 9


Building
RAISE BANK TO TOP OF HEADWALL
Hedge
HEADWALL
Ditch
LOCK-BLOCKS
WIDEN CHANNEL & RAISE BANKS
Link Fence
0.5m ϕ RIP-RAP BED & BANKS
1.2 x 1.8m BOX CULVERT
Constructed
SCALE
1:200
NOTES:
1. DRAWING BASED ON 03/11/02 SURVEY PLAN PROVIDED BY THE DISTRICT OF HOPE.
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DISTRICT OF HOPE
JOHNSON ROAD DRAINAGE STUDY
Plan Sketch
Kawkawa Lake Road Culvert
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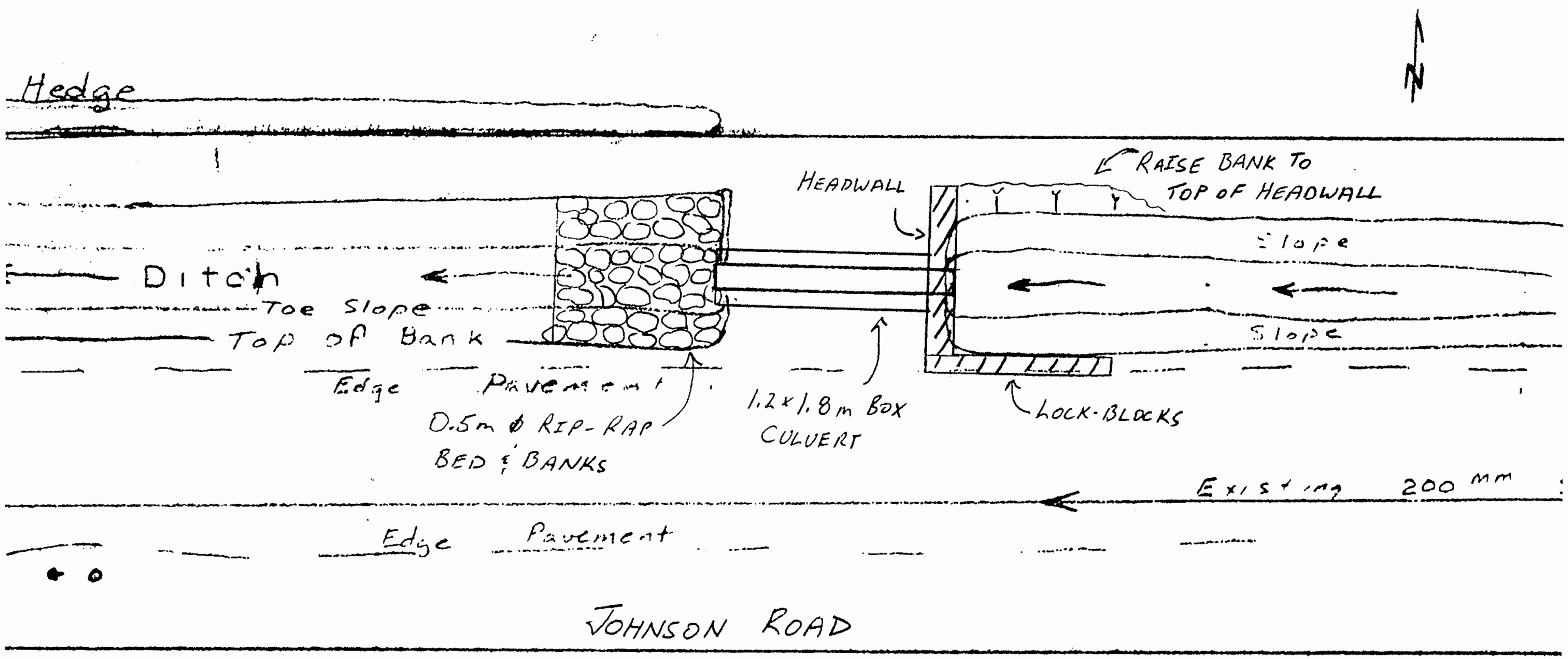
FIGURE 10



SCALE
1:200

NOTES:

1. DRAWING BASED ON 03/11/02 SURVEY PLAN PROVIDED BY THE DISTRICT OF HOPE.

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DISTRICT OF HOPE
JOHNSON ROAD DRAINAGE STUDY

Plan Sketch
Downstream Ditch Culvert

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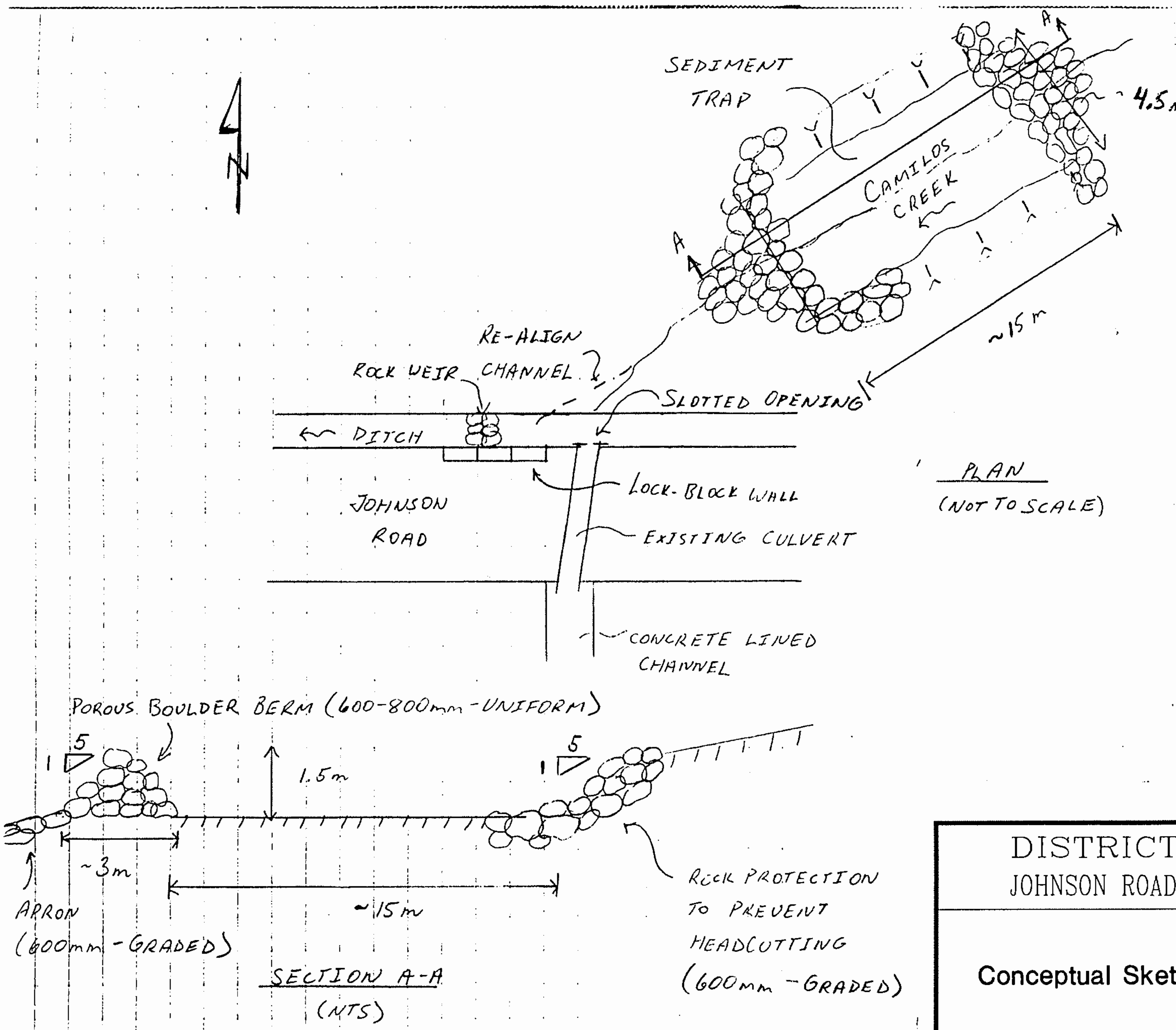
FIGURE 11



DISTRICT OF HOPE
JOHNSON ROAD DRAINAGE STUDY

Conceptual Sketch - Sediment Trap

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FIGURE 12