Environment Canada
Inland Waters Directorate

B. C. Environment
Water Management Division

Floodplain Mapping Program

Serpentine and Nicomekl Rivers

Design Brief

File: 2997

Original Completion Date March 1993
Revised March 1994

KPA Engineering Ltd.
200 - 776 Cloverdale Avenue
Victoria, B. C.
V8X 2S7

ENVIRONMENT CANADA
INLAND WATERS DIRECTORATE

B. C. ENVIRONMENT
WATER MANAGEMENT DIVISION

FLOODPLAIN MAPPING PROGRAM

SERPENTINE AND NICOMEKL RIVERS

DESIGN BRIEF

VOLUME I

Table of Contents

Table of Contents - Continued

TABLES

Table of Contents - Continued

FIGURES

APPENDICES

VOLUME II

ENVIRONMENT CANADA-INLAND WATERS DIRECTORATE
B. C. ENVIRONMENT-WATER MANAGEMENT DIVISION
FLOODPLAIN MAPPING PROGRAM
SERPENTINE AND NICOMEKL RIVERS

DESIGN BRIEF

1.0 INTRODUCTION

This Design Brief and the Floodplain Maps for the lowlands of the Serpentine and Nicomekl Rivers were prepared under the Canada - British Columbia Floodplain Mapping Agreement by KPA Engineering Ltd. One subconsultant, Pacific Meteorology Inc., was retained to conduct an analysis of a severe historic precipitation event.

The principal contact persons for this project were K. W. Wilson, P. Eng., Project Manager for B. C. Environment, and M. Sydor, Head, Ecosystem Risk Forecasting and Analysis Division, Environment Canada. The KPA project team was managed by Y. Shumuk, P. Eng.

The drainage basins of the Serpentine and Nicomekl Rivers are located in the District of Surrey, the City of Langley and the Township of Langley, near Vancouver, B. C. In their lower reaches these rivers traverse a common lowland area before discharging into Mud Bay. Because of the interdependent hydraulics of these two river systems, this delineation study required the analysis of simultaneous flood discharges in both rivers. Two major sources of floodwater threaten the lowlands. One is runoff from the uplands caused by extreme storm events; the other is dyke overflow from the sea during extremely high storm surge and tide combinations. Flood events from both sources were analyzed in this study.

This floodplain delineation study differed from most earlier studies in that hydrodynamic modelling with Environment Canada's ONE-D computer program was used, rather than steady-state backwater analysis with HEC-2. This approach required that basin runoff modelling be employed to produce the inflow hydrographs to the hydrodynamic model, rather than peak flow frequency analyses alone.

The fundamental reason the hydrodynamic approach was used for the Serpentine-Nicomekl basin was that a steady-state model cannot simulate the very large impact that storage of floodwaters in the lowlands has on downstream discharges, nor can it adequately simulate the combination of tidal backwater and storm runoff. Also, the determination of the depth and extent of flooding caused by an extremely high tide and storm surge in Boundary Bay that would overtop the sea dykes is fundamentally a dynamic problem; a steady-state model would not have been applicable at all.

The floodplain delineation study contained the following components:

- storm precipitation analyses to develop hyetographs for historic storm events in each subbasin
- development and calibration of an SCS Method runoff model, incorporating land use and soil type information to generate storm hydrographs for each subbasin
- development and calibration of a ONE-D model to simulate hydraulic response in the lowland to upland flood runoff
- development of a ONE-D model to simulate the results of dyke overflows from extreme high water levels in Boundary Bay
- determination of the governing flood levels and an appropriate "freeboard" quantity
- delineation of the land below the flood level plus freeboard as the 200-year floodplain.

River cross section and bridge data were provided by B. C. Environment, Water Management Division, based on surveys they conducted in 1984, 1985 and 1991. The base mapping for the floodplain delineation, derived from April 1989 aerial photography, was also provided by B. C. Environment.

The 1:5000 scale Floodplain Maps produced by this study appear on 14 sheets entitled, "Floodplain Mapping - Serpentine and Nicomekl Rivers" (Drawing Numbers 91-5-1 to 91-5-14). These maps are located in the pockets in Volume 2 of this Design Brief.

2.0 BACKGROUND

2.1 The Serpentine-Nicomekl Drainage Basin

Physical Features

The combined drainage area of the Serpentine and Nicomekl Rivers totals 334 km^2, of which 59 km^2 is lowland with much of that at elevations between -1.0 m and +2.0 m[1]. As shown on Figure 1, the lowland area extends for a considerable distance inland. For example, on the Serpentine River, the lowlands extend more than 16 km up-valley, or 21 km upriver, from the mouth. The remaining 82% of the catchment is located on the upland plateau or on the slopes between the uplands and the lowlands. The highest point in the combined basin, located near the eastern end of the Nicomekl basin, has an elevation of approximately 140 m.

The Serpentine River is entirely situated within the boundaries of the District of Surrey, as is almost all of its upland drainage area. The Serpentine is fed by three major tributaries. Latimer Creek drains the Port Kells area from the east. Mahood Creek, also known as Bear Creek, is the largest tributary and drains much of the developed land from the west. A 2.4 km man-made cutoff channel exists near the mouth of Mahood Creek, creating Bose Island. Hyland Creek, which also enters from the west, drains an area south of the Mahood basin. The cumulative channel length of the Serpentine River system analyzed for this study was about 31 km.

The downstream reaches of the Nicomekl River, and their adjacent lowlands, are also located within the District of Surrey. The headwaters of the Nicomekl River and its two major tributaries, Murray Creek and Anderson Creek, are located within the Township of Langley and the City of Langley. The cumulative channel distance of the study reaches in the Nicomekl system totals 34 km.

---

[1] All elevations in this report are related to Geodetic Survey of Canada Datum, unless otherwise stated.

Unlike the Serpentine basin, where the slopes of the upland drainage courses abruptly changes from steep to very flat on the lowlands, the Nicomekl riverbed profile changes gradually from a moderate slope in its upstream reaches to a low gradient west of 192nd Street. These differences in profiles are corroborated by observations of the tidal effect on water levels, which extends much farther inland on the Serpentine River than it does on the Nicomekl River.

Before European settlers inhabited the area, much of the lowland area was inundated at every high tide. A system of sea dykes and two sea dams, one on each river, now protects the area from flooding by normal tides. The rivers are also dyked, however at several locations the upstream ends of the river and tributary dykes are not effectively tied to the upland slopes. As a result, much of the runoff during large floods bypasses the river and inundates the lowlands outside the river dyke system. The dykes were built to an agricultural standard and dyke failures are common during major floods.

The lowlands are traversed by two major railroads, five highways and a grid pattern of smaller roads. These are all situated on top of elevated embankments, which together with the dykes and upland slopes, divide the lowlands into at least 46 distinct floodplain storage enclosures, or storage cells.

During floods these cells may receive water from the following sources:

- upland runoff
- flows through culverts from adjacent cells
- road overflows from adjacent cells
- flows through dyke breaches
- sea dyke overflows
- direct precipitation
- seepage.

The cells may drain in the following ways:

- through floodboxes to the river system
- over roads to adjacent cells
- through culverts to adjacent cells
- over dykes to the river
- through dyke breaches
- through pumpstations
- evaporation.

Delineation of such a floodplain requires a thorough understanding of the hydraulic behaviour of the system under flood conditions. Analysis of a major flood through the existing arrangement of rivers, tributaries, dykes that may breach, roads, railroads and storage cells in the Serpentine-Nicomekl lowlands represents a very complex hydraulic problem. The primary aim of the methodology used in this study was to unravel the complexity by simulating the entire system using dynamic models.

Development History

Civilian settlement of Surrey by Europeans began in the 1860s. At that time the Serpentine and Nicomekl Rivers were used extensively for transportation. With settlement came ditching, small creek diversions and small, hand-built dykes in the lowlands to prevent saltwater from inundating the land. The first dyking district was incorporated in 1889.

The Great Northern Railway, which now forms part of the sea dyke system, was built by 1910. The seadams on the Serpentine and Nicomekl Rivers were originally built in 1913. The system of dykes along each river has expanded and dykes have been raised and improved in stages since these early beginnings(1).

During this period, the drainage basins have undergone dramatic changes and continue to change at an unprecedented rate. The tidal flats in the lowlands have become farmlands. The once-forested uplands are now a patchwork of residential subdivisions. Several small lakes which once existed in the basins have been drained. Storm drains and ditches, many with small detention basins, now convey runoff from the uplands to the lowlands, altering the hydrologic response of the basin from its predevelopment regime.

In 1981 Surrey had 147,100 residents. Its population doubled from 1961 to 1981, and is expected to at least double again by 2001. It has been recognized in recent years as the fastest growing municipality in Canada. Although Langley is less developed than Surrey, it is also subjected to similar development pressure. In response to the rapid urbanization, continuing changes to the hydrologic regime of the Serpentine-Nicomekl basin must be anticipated. In this study, the runoff estimates for floodplain delineation purposes have been based on future land uses, as identified in the Official Community Plans for Surrey and Langley.

2.2 Historic River Flood Events

Floods occur in the Serpentine-Nicomekl basins between the months of October and March, but most frequently in December or January. These events result from heavy rainfall events associated with frontal disturbances travelling onshore from the Pacific Ocean. On some occasions, a snow cover exists prior to the onset of rain, which melts and adds to the runoff. However, the majority of flood events are caused by rainfall alone.

Parts of the lowlands of the Serpentine-Nicomekl basin are subjected to some degree of inundation during most years. The dates of three past rain events that resulted in unusually widespread flooding are listed below:

21 January, 1935
20 January, 1968
18 December, 1979.

Although other floods have occurred in the area, these three are particularly relevant to this study. Each of these events is described in the following sections.

1935 Flood

The storm of 20 - 25 January, 1935 produced the heaviest 5-day precipitation amount ever recorded in Vancouver, Surrey and vicinity. The return period of the 5-day precipitation was estimated to be in the order of 1500 years (2) although shorter duration amounts in the same storm were much less extreme.

In combination with very heavy rainfall, severe antecedent conditions existed in the basin which increased the ultimate flood levels. These are identified in the following excerpts from a 1963 drainage report by the Surrey Engineering Department (1).

> "During the night of January 20th, 18 inches of wet snow fell onto the already frozen ground which was covered with 8 inches of snow from a previous snowfall. On the morning of January 21st it started to rain and over 5 inches of rain fell in the first 24 hours of the storm. A total of nearly 12 inches of rain fell in the 4 days that the storm lasted."

> "The flooding in the valleys was compounded by two other serious factors. Firstly, the storm occurred during the season of high tides and secondly, the two rivers were frozen over at the start of the storm. The ice broke up as the rivers rose and blocked the flood gates at the dams. Attempts were made to clear the ice through the floodboxes by dynamiting. These attempts were not successful and the rivers overflowed the dykes and increased the flooding in the lowlands to over four feet deep. During the worst part of the flooding the water flowed across the Elgin Road causing extreme flooding in the Mud Bay Dyking District. This road was raised when the King George Highway was built. At the height of the flooding nearly all of the lowland areas including most of the roads in these areas were under water. In addition, many of the interior dykes were submerged."

The sequence of remarkable weather extremes in the second and third weeks of January 1935, which culminated in widespread flooding, formed lasting impressions on the residents of south coastal British Columbia (3). For more than a week a deep layer of stationary Arctic air resulted in very cold conditions on the coast. Ice impeded navigation on the Fraser River, and the ground froze. Minor snowfalls occurred during this period, but these accumulations were dwarfed by the large fall on the 20th, which paralyzed traffic, stranded commuters and collapsed the roofs of several large buildings.

The falling snow changed to rain before noon on the 21st with the arrival of mild Pacific air associated with a synoptic pattern that included two troughs moving eastward in close proximity to one another (2). Air temperatures in Vancouver rose 21°C in 24 hours. Very heavy rain continued through the 21st, easing slightly on the 22nd. More rain then fell through the next three days, resulting in the extremely rare 5-day total. East of Langley, in the Fraser Valley, freezing rain falling through pools of cold air resulted in thick ice coatings on every exposed surface and felled powerlines, fences and tree branches (3).

The record rainfall, combined with melting of the heavy snowpack, occurred on frozen ground which resisted infiltration and produced extremely high rates and volumes of runoff. In the lowlands of the Serpentine and Nicomekl Rivers the flooding caused by the runoff was exacerbated by the partial blockage of the sea dams by pieces of ice. Because of this unusual combination of extreme factors, the 1935 storm caused the worst flooding in most parts of the Serpentine-Nicomekl basin since settlements began in the area.

A few photographs taken during the 1935 flood appear in Appendix 1.

1968 Flood

Beginning on the 18th of January 1968, a storm occurred over the Serpentine-Nicomekl basin, which contained 2-day precipitation totals similar to the 1935 storm 2-day amounts, and caused the most extensive flooding in the lowlands since 1935. The longer duration amounts, however, were much lower than those recorded for the 1935 storm. The rain was accompanied by warm temperatures, but was not preceded with the snow or frozen ground conditions of the 1935 storm. The basin was, however, significantly more urbanized by 1968. This storm was also associated with a double trough system (2).

Although there were not many water level recorders operating in the basin, and none in the lowlands, the storm provided essential data for this study by virtue of the excellent aerial photography taken of the inundated areas (examples appear in Appendix 1). This photography provided a basis for calibrating the models for the areal extent of flooding in the lowlands.

1979 Flood

On the 17th and 18th of December 1979, a rainfall which contained an intense period lasting approximately 24 hours, caused flooding in the Serpentine and Nicomekl lowlands. The flooding was not as extensive as that which occurred in 1968 or 1935. Nevertheless, large expanses of the lowlands were inundated. This flood was selected for calibration because it was the largest flood for which several water level gauges were operating throughout the basin, providing actual stage hydrographs against which simulated results could be compared.

2.3 Historic Flooding from the Sea

Before development, much of the Serpentine-Nicomekl lowland area was flooded at every high tide. On several occasions since the sea dams and sea dykes were built, water from Boundary Bay has overtopped or breached the sea dykes and inundated farmlands. Prior to 1950 such events were not well documented. Three of the more significant events which have occurred since then are briefly described in the following paragraphs.

In December 1951, the sea dyke near the Serpentine Fen failed completely, inundating the Mud Bay Dyking District. The dyke contained a considerable quantity of peat and was difficult to repair. After approximately one month the breach was repaired, using two rows of sheet piling and fill placed between them.

In the winter of 1972/73, the sea dyke along the Nicomekl River was breached, again inundating Mud Bay District. This time the breach was plugged much more quickly using rock transported to the site by barge.

The largest of the recent events occurred on 16 December, 1982, when high winds generated a large storm surge. The peak of the surge coincided with a high tide, resulting in water levels which overtopped the lowest parts of the dykes surrounding the Mud Bay and Colebrook Dyking Districts. Water approximately 1 m deep was reported covering the fields in the Mud Bay District, and the inundation extended inland to the King George Highway. In the Colebrook Dyking District, only isolated ponding occurred. In both areas, the dykes did not wash out completely, instead overtopping flows gradually eroded the surface material from the dyke crest at many locations. A quick response to the emergency by a fleet of trucks delivering gravel to the overtopped areas was credited for preventing major breaching of the dykes and subsequent greater damage.

The same storm event created the combination of high water levels and large waves at Crescent Beach. Water and logs overtopped the seafront boulevard, which forms the sea dyke, and washed into the front yards of some of the seafront homes (4). In an emergency effort to limit the damage, municipal crews dumped riprap and placed sandbags on the dyke.

A hindcast of the December 1982 surge was conducted by Seaconsult (6). They reported a surge magnitude of 0.89 m at Point Atkinson, and 1.18 m at Crescent Beach. The difference between these peak surge values was primarily attributed to local wind effects at Boundary Bay, where factors such as wind setup (discussed in Section 4.3) are much more pronounced.

3.0 HYDROLOGIC ANALYSIS

This section summarizes the procedures and findings of the hydrology study. A more detailed report on this topic is presented as a Technical Appendix under separate cover entitled "Development of Flood Hydrographs."

3.1 Methodology

The Serpentine-Nicomekl drainage basin was divided into 36 subbasins. The objective of the hydrologic analysis was to develop 36 subbasin hydrographs for each of three historic flood events and for a 200-year runoff event. The subbasin hydrographs were later used to represent inflows to the ONE-D hydrodynamic model, and some of the hydrographs were subsequently apportioned to several inflow points as required by the hydrodynamic modelling. The subdivision of the drainage basin is illustrated in Figure 2.

In typical floodplain mapping projects, design flows were generated from frequency analysis of streamflow data. However, in this case the available surface water database was deemed to be insufficient, therefore an analysis of historic rainfall patterns was conducted. Also, an entire flood hydrograph, rather than a single discrete peak discharge, was required for the hydrodynamic modelling. This involved the selection of storm events for calibration purposes, for which good precipitation, streamflow, water level and photographic data was available. The hydrologic model was calibrated with six events which occurred between 1972 and 1990. The confidence gained in this effort indicated that reasonable estimates of flood hydrograph could be generated for other major events for use in calibrating the hydrodynamic model.

The January 1935 storm stood out as the storm of record, and a detailed analysis of this event was conducted. Pacific Meteorology Inc. was retained for this project to reconstruct this storm from the sparse database available. Estimates of precipitation with 2-hour time increments for seven locations in the Serpentine-Nicomekl catchment area were determined. Another major event (the largest of record at many of the tipping bucket rain gauges) occurred in January 1968 and was relatively well documented. It was also used for calibration of the hydraulic model. These two events formed the basis for development of the 200-year flood hydrograph.

The procedure used to develop hydrographs for input to the hydrodynamic model is outlined below:

- selection of relevant precipitation and streamflow data for analysis
- analysis of hourly precipitation data to determine return periods for significant storm events
- flood frequency analysis of available stream gauge data
- compilation of land use and soil classification maps for computation of SCS curve numbers and times of concentration for each of the gauged calibration basins and for the 36 subbasins
- calibration of the HYDSYS hydrologic model by comparing recorded streamflow data to hydrographs generated from estimates of curve numbers, times of concentration and land use factors
- application of the calibrated HYDSYS model to compute flood hydrographs for January 1968 and December 1979 storms for all 36 catchments, thereby providing flows with which to calibrate the hydrodynamic model.
- determination of 200-year flood hydrographs for all catchments using HYDSYS. Precipitation values for a modified January 1935 storm were scaled to match the intensity-duration frequency characteristics of a 200-year return period rain event.

The rainfall-runoff simulation program known as HYDSYS, which is based on the U. S. Soil Conservation Service Hydrograph method (9, 10), was used for the hydrologic analyses of this study. The SCS Hydrograph Method requires inputs of drainage area, curve number and time of concentration. A modification for HYDSYS allows input of a parameter known as the land use factor, which alters the unit hydrograph to reflect general land use. The program output consists of a discharge hydrograph for which runoff volume, peak instantaneous discharge and time to peak are summarized.

3.2 Available Data

Information from federal, provincial and municipal government agencies was collected for the purpose of this study. The main sources and types of data are listed below:

1. Atmospheric Environment Services (AES)

- Hourly and daily precipitation data for all tipping bucket rain gauges (TBRG) operated in the study area and for regional daily observing stations. Precipitation records consisted of electronically-stored hourly precipitation data for the years 1962 to 1991, and daily total precipitation for longer periods. It should be noted that not all stations operated on a continuous basis during the 1962 to 1991 period.

2. Water Survey of Canada (WSC)

- Hourly discharges for gauged basins within the study area.

3. District of Surrey

- 1:12,500 scale mapping showing drainage subbasins
- 1:12,500 scale maps with 25-ft. contour intervals
- 1:25,000 scale 1976 zoning map
- 1:25,000 scale 1985 Official Community Plan.
- TBRG strip chart data from the period 1976 to 1990
- Streamflow recorder charts and rating curves

4. Township of Langley

- 1:25,000 scale 1986 Official Community Plan
- Strip chart precipitation for Langley STP TBRG.

5. City of Langley

- 1:2,000 scale detailed land use maps of the City of Langley.

6. B. C. Environment

- 1:5,000 scale 1-m contour topographic plans (1991) of the Serpentine-Nicomekl floodplain
- Stage strip chart data for gauges on Serpentine and Nicomekl Rivers
- Oblique aerial and ground flood photography.

7. Maps B. C.

- Aerial photography covering Serpentine-Nicomekl Rivers' drainage basin:
  - 1989 at 1:24,000 scale (BC89001 Nos. 97 - 208)
  - 1972 at 1:62,000 scale (BC5492)
  - 1971 at 1:30,000 scale (BC5406).

Locations of the key streamflow and precipitation gauges used in the hydrologic analysis are shown on Figure 3.

3.3 Storm Precipitation Analysis

The objectives of the storm analysis were to:

- relate magnitudes and return periods of flood-producing storms
- select appropriate storms for calibration of the hydrodynamic One-D model
- select storms for development of 200-year flood hydrographs.

Hydrograph data from all TBRG stations located inside or near the basin was used in this analysis. It was observed that precipitation intensities varied throughout the basin. The highest precipitation accumulations for each storm were usually recorded in the northern part of Surrey and the lowest values occurred in the south. Because of this variability, data from gauges within the basin was considered to be more reliable than data from longer term but more distant gauges.

The following TBRGs with hourly total precipitation amounts were selected for storm frequency analysis:

White Rock Sewage Treatment Plant	26 years of record
Surrey Municipal Hall	28 years of record
Surrey Kwantlen Park	30 years of record
Pitt Meadows Sewage Treatment Plant	16 years of record
Langley Lochiel	28 years of record
Langley Prairie	29 years of record.

Precipitation data from the District of Surrey and Langley Township had periods of record that were too short for frequency analysis, so this data was used for storm calibration only.

The precipitation database was processed with software which identified the significant storm events contained in the record. Then the maximum 3-day and 5-day precipitation amounts were determined for each significant storm event, and the return period interval for each amount was determined from the intensity-duration frequency tables provided for each gauge by AES. These return periods are summarized in Table 1, on the following page.

The rainstorms which occurred in January 1968 and December 1979 were selected for calibration of the ONE-D hydrodynamic model. The reasons for this selection are explained in Section 5.4 of this report.

Table 1
Return Periods for Recorded Storms
(3-Day and 5-Day Precipitation Totals)

Storm Event	White Rock STP	Surrey Municipal Hall	Surrey Kwantlen Park	Pitt Meadows STP	Langley Lochiel	Langley Prairie
20/01/68	10/2-5	50/10	>100/100	N/R	N/R	100/25
26/12/72	5/10	10/100	25/25	N/R	25/25	N/R
19/10/75	5/5	2-5/5	5/5	<2/<2*	5/5	N/R
02/12/75	*	2/<2	*	<2/<2*	25/2	N/R
17/01/77	<2/<2	<2/<2	<2/<2	<2/<2	*	N/R
18/12/79	5/25	N/R	10/25	10/100	N/R	N/R
21/11/80	5/2	5/5	5/2*	5/2	N/R	N/R
04/01/84	10/5	10/5*	5/2*	<2/<2	10-25/5	N/R
26/02/86	5/2	<2/<2	5/2	N/R	*	N/R
24/11/86	<2/<2	2/5*	<2/10	<2/<2*	N/R	N/R
10/11/89	5/5	2/<2	5/2	N/R	N/R	N/R
11/11/90	10/10	5/5	5/5	10/10	N/R	N/R
21/11/90	<2/<2*	<2/<2	<2/<2	<2/<2	N/R	N/R

Legend:
* - partially missing data
N/R - No Record
e.g. 50/10 means 50-year return period for 3-day precipitation amount
and
10-year return period for 5-day precipitation amount

3.4 Flood Frequency Analysis

Traditionally, flood frequency analyses using recorded peak streamflow data was the preferred method to determine 200-year peak discharges. For this study, however, this approach was found to be inadequate because of limitations in the recorded data. The analysis was performed as part of the study, and results are presented herein for comparison.

The Consolidated Frequency Analysis program (CFA), prepared by Environment Canada, was used to analyze flood frequencies in gauged basins located within the study area. Four probability distributions are provided in CFA, as follows:

- Generalized Extreme Value (includes Gumbel)
- Three Parameter Log Normal
- Log Pearson Type III
- Wakeby.

Historic surface water data was recorded by Water Survey of Canada for a number of gauging stations inside the Serpentine-Nicomekl basin. Only stations with at least 15 years of record were selected for this analysis, as follows:

•	08MH018[2]	Mahood Creek near Newton (including 08MH154 Mahood Creek at 144 Street)	30 years of record[1]
•	08MH020[2]	Mahood Creek near Sullivan	24 years of record
•	08MH129	Murray Creek at 216 Street, Langley	15 years of record
•	08MH104[3]	Anderson Creek at Mouth	21 years of record
•	08MH105[2]	Nicomekl River below Murray Creek (including 08MH155 Nicomekl River at 203 Street)	25 years of record[1]

1 combined period of record, developed by including values from the shorter record by prorating on the basis of drainage area.
2 parts of the record are manual gauge data
3 manual gauge data for entire period of record.

These periods of record indicate the number of mean daily peak flows contained in the annual series. Instantaneous peak flow data was available for shorter periods of record.

Maximum 1- and 3-day discharges were determined for all of the above basins and analyzed using all four probability distributions. A plot of each flood frequency curve showing the individual data points was reviewed before a "best fit" probability distribution was selected. In general, the first three distributions exhibited similar estimates and good fit. The Wakeby curves displayed significant slope change at both extremes, hence they provided very different estimates for greater return intervals.

Figures 4 and 5 are log-log plots of peak day and peak 3-day unit flows versus drainage area. The Mahood Creek flows are significantly higher than the others, probably due in part to the higher degree of urbanization of the watershed. The Mahood Creek data also showed a significant trend of increasing peak flows with time, probably due to the increasing proportion of urbanization of the watershed during the gauge's period of record.

Inconsistencies were found in the WSC data for two Nicomekl River gauges; Nicomekl River below Murray Creek (08MH105) and at 203 Street, Langley (08MH155). The gauge below Murray Creek recorded flows from 1965 to 1984. The daily peak flow for the January 1968 flood was estimated at 28.3 m^3/s and for the December 1979 event it was 24.6 m^3/s. When this gauge was discontinued in 1984, the gauge at 203 Street was established downstream, with a 7.3% larger catchment area. In the seven years of data recorded since, the maximum daily discharge was reported as 60.2 m^3/s for a storm in November of 1990, a storm which had less rain and much less flood damage reported than the 1968 and 1979 floods.

The lowest annual peak daily flow reported in these seven years for the 203 Street gauge was 23.2 m^3/s, which is almost equal to the discharges reported at the upstream gauge for the 1968 and 1979 floods. The reasons why the flow records between the two gauges were so disproportionate were not found, however it was concluded that some of this data was probably erroneous. Later calibration runs of the ONE-D hydrodynamic model for the 1968 and 1979 floods yielded simulated daily peak flows of 63.2 m^3/s and 59.4 m^3/s, respectively, at the upstream gauge location. These are significantly greater than the discharges reported by WSC for these two events at the upper gauge. Therefore, it would appear that the major discrepancy lies in the WSC data for the Nicomekl River below Murray Creek gauge.

Because of the weaknesses in the streamflow data described above, determination of a 200-year event was based on the precipitation intensity-duration frequency characteristics rather than computed streamflow flood frequencies. The results of the flood frequency analyses for some streamflow gauges were useful for comparison purposes.

3.5 Calibration of the HYDSYS Model

Land Use Mapping

Historic and future land uses within the Serpentine-Nicomekl drainage basin were assessed for the purpose of calculating curve numbers for input to the hydrologic model. The basin was divided into the land use classes which correspond to groupings established by the U. S. Department of Agriculture (11).

A map showing land uses in the 1970s is presented in Figure 6. This map, required for generating 1968 and 1979 flood hydrographs, was prepared using the District of Surrey 1976 zoning map and 1972 aerial photography of the watershed. The aerial photography was mainly used to determine land use for the Township of Langley portion of the drainage basin. The Township does not maintain land use maps from past decades.

The future land use plan (Figure 7), required to estimate the design flood events, was derived from the District of Surrey Official Community Plan (OCP) (1985), the City of Langley OCP (1991) and the Township of Langley OCP (1986).

Soils Mapping

A soils classification map (Figure 8) was prepared using a 1981 B. C. Ministry of Environment report (12), which contained a comprehensive description of the soils of the study area. A 1939 B. C. Department of Agriculture soils map (13) was used as a source of soil descriptions for study areas not classified in the 1981 report. The unclassified areas included the region north of 80th Avenue and west of 160th Street, and the area south of the Nicomekl River and west of Highway 99.

Storm Selection

The following criteria were used to select storm events and gauging stations for calibration:

- magnitude of the storm,
- availability of observed hydrograph data,

- availability of precipitation data,
- perceived accuracy of data,
- proximity of rain gauges, recording during the storm, to gauged basins.

Consistent with the above considerations, storms of December 1972, December 1979, November 1980, December 1980, February 1986 and November 1990 were selected for calibration. Many significant storms were excluded due to lack of hourly precipitation or hourly discharge data and/or little confidence in some of the strip chart data. For example, there were no hourly discharges for the January 1968 event.

The gauged basins selected for calibration were distributed throughout the upland Serpentine-Nicomekl drainage basin and represented the diverse catchment conditions within the study area. Recorded hydrographs for selected storms were obtained from WSC and the District of Surrey. The precipitation data was made available by AES and the District of Surrey. The Thiessen Polygon method was applied for all calibrated storms to determine the radius of influence of a rain gauge within the study area.

HYDSYS Input Data

Drainage areas of each subbasin were delineated using the District of Surrey's 1:12,500 scale storm drain maps and 1:25,000 scale National Topographic Series maps. A multilayered AutoCAD Version 12 drawing was developed which permitted overlay of the various subbasins with land use, soil classification and Thiessen Polygon information, and which automated the measurement of the numerous subbasin subdivisions that resulted from the overlays.

Curve numbers required for the SCS Method were related to soil groups and land uses as outlined in the following table:

Table 2

Curve Numbers for Various Land Uses and Hydrologic Soil Groups

Land Use	Hydrologic Soil Group			
	A	B	C	D
Agricultural without conservation treatment	86	92	95	97
Pasture or range - good hydrologic condition	59	78	88	91
Forested with good cover	43	74	85	89
Industrial	92	95	97	98
Commercial and business	96	97	98	98
Urban - ¼ acre lots	78	88	93	95
Suburban - ½ acre lots	73	85	91	94
Open spaces - 75% grass cover and more	59	78	88	91
Paved areas	98	98	98	98

Composite or weighted curve numbers for each subbasin were determined by summing the products of each land use area and each soil grouping, then dividing by the total area of the subbasin.

Times of concentration were computed by summing travel times for overland and conduit flow along the longest flow path. Individual travel times were calculated by the following formula:

$$T = \frac{c\,n\,L}{12\,S^{1/2}}$$

Where:

T = travel time in minutes

L = drainage distance in metres

S = slope of land or drainage conduit

n = Manning's n

c = 1.4 for overland flow
0.5 for conduit.

It was found that times of concentration were very sensitive to the length of overland flow.

KPA has experimented with modifications to the unit hydrograph provided by the SCS model in order to make it more representative of the study area. The result was the introduction of a land use factor as an additional parameter to the model. This proved to be very useful in the model calibration. The land use factor alters the triangular unit hydrograph shape by either lengthening or shortening the time ordinate and correspondingly reducing or increasing the peak. This helps provide better simulations of the largely undeveloped and highly urbanized basins, respectively.

Baseflow was added to the direct runoff estimates provided by the SCS method. Based on previous work for Erickson Ditch (14, 15) a value of 0.69 L/s/ha was the criteria used for calculating baseflow in winter months.

Calibration Results

Mahood Creek at Newton and Mahood Creek at 144th Street, gauges No. 08MH018 and 08MH154, respectively, calibrated well with calculated curve numbers and times of concentration and the SCS land use factor. Similar observations were made for the District of Surrey gauged basins of Hyland Creek and Robson Creek. All of these basins were relatively highly urbanized. Latimer Creek, Erickson Creek and Elgin Creek gauges, situated in agricultural and forested/ suburban areas of Surrey, also calibrated well with calculated curve numbers and times of concentration, but with land use factors 3 and 4. Plots showing the comparison between observed and calculated hydrographs are illustrated in Figure 9.

Nicomekl River below Murray Creek, WSC basin No. 08MH105, could not be accurately simulated with the hydrologic model. Calculated runoff hydrographs for several storm events exhibited consistently higher runoff volumes and peak discharges than the observed hydrographs. Several calibration attempts failed to match observed runoff volumes and peak discharges. It appeared that channel storage upstream from the gauge played a significant role in attenuating runoff. Also, suspected errors in the WSC discharge data, discussed earlier in section 3.4, likely contributed to the poor matches.

3.6 Generation of Flood Hydrographs

January 1968 and December 1979 Storms

The AES stations of Surrey Kwantlen Park, Surrey Municipal Hall, White Rock STP and Langley Prairie were the only operating TBRGs in the study area during the January 1968 storm. The greatest precipitation occurred at the Surrey Kwantlen Park station and the least precipitation was recorded at the White Rock STP.

More abundant hourly precipitation data existed for the December 1979 storm. In addition to the AES stations of Surrey Kwantlen Park, Pitt Meadows STP, Langley Lochiel and White Rock STP, the District of Surrey provided data from TBRG stations at Cedar Hill (Robson), Fire Hall No. 10, Latimer Creek, East Kensington (Erickson) and Elgin Creek. As a result of the Thiessen polygon analysis some of the Surrey-operated gauges displaced the AES stations of Surrey Kwantlen Park and Langley Lochiel (Surrey Municipal Hall was inoperative) for the December 1979 storm.

To generate the hydrographs, precipitation amounts for each of the 36 subbasins were weighted on the basis of the Thiessen polygon analysis. Drainage areas, curve numbers, times of concentration and land use factors were calculated as described earlier for land use and development circa 1974. Electronic data files were created for each of the hydrographs and reformatted for input to the ONE-D program.

As part this project KPA retained Pacific Meteorology Inc. to conduct a study of precipitation patterns in the Serpentine-Nicomekl drainage basin for the January 1935 storm (2). The study investigated spatial and temporal variations of precipitation on the B. C. Lower Mainland. It estimated 5-day, total precipitation amounts at 2-hr. intervals for the following contemporary AES rain gauge locations: Surrey Kwantlen Park, Surrey Municipal Hall, Pitt Meadows STP, Langley Prairie, Langley Lochiel, White Rock STP and a centrally-located hypothetical "Station A." The above stations did not exist in 1935. However, they were used in analysis of the January 1968 and December 1979 storms.

Total precipitation, including snow in millimetres (mm) of water equivalent, was provided for the period January 20th to 25th, 1935. The snowfall period began at approximately 8:00 a.m. and lasted for 24 hrs. This resulted in a range of 39 to 61 mm of snow water equivalent from White Rock STP to Pitt Meadows STP, respectively. Similarly, the highest maximum 5-day precipitation of 421 mm was estimated for Pitt Meadows STP, and the lowest 5-day precipitation of 292 mm was calculated for White Rock STP.

A frequency analysis compared the total precipitation intensities for various durations of the 1935 and 1968 storms to 200-year values for the selected gauge locations. This information was used to develop 200-year design hyetographs by providing information for scaling storm precipitation amounts from these storms to respective 200-year levels. The 1968 event appeared to be the largest on record at many of the TBRG stations within the study area, therefore it was important in selecting a design event.

Actual maximum precipitation amounts and estimated 200-year return period values for 2-hr. to 5-day durations were determined for nine long-term AES climate stations locations. The IDF data was extended from the 100-year values provided to the 200-year interval by first plotting the provided information on Gumbel distribution paper then extending best fit lines to the 200-year return period. New Westminster, Abbotsford Airport, Stave Falls and Steveston stations are located outside the immediate study area. However, they were included in the analysis because of the available long-term precipitation record. Table 3 on the following page, is a summary of precipitation amounts for the 1935 and 1968 storms with 200-year values for each station. Table 4 uses this information to calculate ratios of storm precipitation to 200-year values.

Table 4 shows that 1- and 2-day accumulations for both events were in the order of the 200-year event. However, longer durations in 1935 significantly exceeded the 200-year level, while in 1968 they were generally less than 200-year. Short-duration intensities in 1935 were well under 200-year levels.

Table 3
Maximum Total Precipitation Amounts for Three Storm Events (mm)

Event	2-hr.	6-hr.	12-hr.	1-day	2-day	3-day	4-day	5-day
January 1935*								
Surrey Kwantlen Park	10.9	34.1	60.9	108.6	175.8	232.9	302.7	358.9
Surrey Municipal Hall	15.3	34.9	62.3	104.9	163.0	208.1	264.3	314.0
Pitt Meadows STP	13.9	42.6	77.1	130.2	196.6	264.2	352.7	418.4
Langley Lochiel	12.0	35.6	65.8	111.8	168.5	214.4	276.6	330.2
Langley Prairie	14.3	38.2	71.1	120.2	176.8	227.4	297.1	353.4
White Rock STP	14.2	34.3	61.7	102.4	158.0	198.2	224.3	289.5
New Westminster (daily)	--	--	--	105.4	180.3	261.1	324.6	367.5
Abbotsford Airport (daily)	--	--	--	--	--	--	--	--
Steveston (daily)	--	--	--	59.4	100.	139.4	176.2	216.1
January 1968								
Surrey Kwantlen Park	21.1	46.5	77.3	140.1	192.7	209.	215.4	223.1
Surrey Municipal Hall	12.4	27.4	49.7	91.7	126.0	142.9	147.3	149.7
Pitt Meadows STP	--	--	--	--	--	--	--	--
Langley Lochiel	--	--	--	--	--	--	--	--
Langley Prairie	16.7	40.5	70.5	124.5	171.1	184.5	187.5	193.2
White Rock STP	9.6	24.4	35.7	60.0	83.1	90.2	92.	95.9
New Westminster (daily)	--	--	--	--	--	--	--	--
Abbotsford Airport (daily)	--	--	--	78.2	126.2	148.6	148.9	158.6
Steveston (daily)	--	--	--	61.5	82.8	97.5	100.5	101.3
200-Year								
Surrey Kwantlen Park	33.3	58.2	92.4	135	189	205	221	241
Surrey Municipal Hall	28.8	51.3	81.6	102	144	169	187	201
Pitt Meadows STP	42.5	69.0	99.6	108	171	221	248	263
Langley Lochiel	27.7	54.9	84.0	96	132	160	190	218
Langley Prairie	--	--	--	129	189	202	216	238
White Rock STP	46.6	55.7	74.4	77	118	130	142	155
New Westminster	--	--	--	135	188	228	257	282
Abbotsford Airport	--	--	--	118	183	212	229	257
Steveston	--	--	--	77	117	134	155	175

* estimated values, excluding snowmelt

Table 4

Relative Magnitudes of the 1935 and 1968 Storms

Event	Total Storm Precipitation ÷ 200-Year Precipitation							
	2-hr.	6-hr.	12-hr.	1-day	2-day	3-day	4-day	5-day
January 1935								
Surrey Kwantlen Park	0.3	0.6	0.7	0.80	0.93	1.14	1.37	1.49
Surrey Municipal Hall	0.5	0.7	0.8	1.03	1.13	1.23	1.41	1.56
Pitt Meadows STP	0.3	0.6	0.8	1.21	1.15	1.20	1.42	1.59
Langley Lochiel	0.4	0.6	0.8	1.16	1.28	1.34	1.46	1.51
Langley Prairie	--	--	--	0.93	0.94	1.13	1.38	1.48
White Rock STP	0.3	0.6	0.8	1.33	1.34	1.52	1.58	1.87
New Westminster	--	--	--	0.78	0.96	1.15	1.26	1.30
Abbotsford Airport	--	--	--	--	--	--	--	--
Steveston	--	--	--	0.77	0.85	1.04	1.14	1.23
January 1968								
Surrey Kwantlen Park	0.6	0.8	0.8	1.04	1.02	1.02	0.97	0.93
Surrey Municipal Hall	0.4	0.5	0.6	0.90	0.88	0.85	0.79	0.74
Pitt Meadows STP	--	--	--	--	--	--	--	--
Langley Lochiel	--	--	--	--	--	--	--	--
Langley Prairie	--	--	--	0.97	0.91	0.91	0.87	0.81
White Rock STP	0.2	0.4	0.5	0.78	0.70	0.69	0.65	0.62
New Westminster	--	--	--	--	--	--	--	--
Abbotsford Airport	--	--	--	0.66	0.69	0.70	0.65	0.62
Steveston	--	--	--	0.80	0.71	0.73	0.65	0.58

Snowmelt Estimates for the 1935 Storm

Approximately 20 cm of snow covered the basin prior to commencement of the storm on January 20th, 1935. In addition, snowfall in the first 24 hrs. of the storm brought snow accumulations of 59 to 81 cm, depending on location. It was assumed that a 3-hr. ripening period was required before the snow contributed to runoff after the rain began. The calculated total snowmelt consisted of the 20 cm layer of old snow plus the 24-hr. accumulation. For the old snow a factor of 0.15 was used to convert snow to water equivalent. For the fresh snow a value of 0.10 was used.

A semi-empirical basin-wide snowmelt formula (16), which requires wind, air temperature and rainfall information, was used to calculate the rate of snowmelt. An average wind velocity of 5 mph was measured at Vancouver Jericho, the only reporting station, during January 20 - 25, 1935. Also, an average atmospheric temperature of 5°C was recorded in the White Rock area within this time period. These values and 1-day rain amounts for each station were used in snowmelt calculations.

Hyetographs provided by Pacific Meteorology Inc. for this event were modified on the basis of these calculations to represent the additional snowmelt input to the SCS model. Then the 2-hr. increments provided by Pacific Meteorology were converted to 1-hr. increments by simply dividing each ordinate by two. This provided hyetographs which could be input to the SCS model in a manner consistent with the other storm event data. Resulting hyetographs for five hypothetical stations are illustrated in Figure 10.

200-Year Flood Hydrographs

Flood hydrographs corresponding to WSC stream gauge locations were calculated using January 1935 storm data. The input data included snowmelt and basin parameters previously calculated for the mid-1970s development scenario.

Instantaneous peaks, 1-day, 3-day and 5-day runoff volumes, estimated for the 1935 event, were compared to the results of the flood frequency analysis. The comparison is presented in Table 5. For further comparative information, synthetic 200-year hyetographs were calculated using the intensity-duration frequency curve conversion facility contained within the HYDSYS model. This feature utilizes statistical

200-year IDF precipitation amounts from the selected AES gauge to generate a synthetic rainstorm pattern having 200-year intensities for all durations from 1 hr. to 5 days. The calculated hydrograph values derived from the synthetic rainstorms are also shown in Table 5.

TABLE 5
COMPARISON OF 200-YEAR HYDROGRAPH ESTIMATES

Calculation Method	Runoff (L/s/ha)			
	Inst. Peak	1-Day	3-Day	5-Day
MAHOOD CREEK AT NEWTON (08MH018)				
200-Year by WSC/CFA	40.9	23.0	11.3	8.4
1935 Storm Unmodified (with snowmelt)	18.3	14.9	11.5	9.0
200-Year Synthetic Storm	35.5	15.0	7.9	5.6
1935 Storm Modified	35.1	17.1	11.1	7.5
NICOMEKL RIVER BELOW MURRAY CREEK (08MH105)				
200-year by WSC/CFA	19.4	15.0	9.0	5.7
1935 Storm Unmodified (with snowmelt)	13.8	11.6	9.2	7.2
200-Year Synthetic Storm	18.6	10.1	5.8	4.3
1935 Storm Modified	19.2	13.2	8.7	6.2
MURRAY CREEK AT 216 STREET, LANGLEY (08MH129)				
200-Year by WSC/CFA	25.5	12.0	6.0	4.0
1935 Storm Unmodified (with snowmelt)	13.7	11.6	9.1	7.1
200-Year Synthetic Storm	20.5	9.19	5.5	4.2
1935 Storm Modified	20.2	13.0	8.6	6.2

Based on comparison of estimated January 1935 storm hydrographs, 200-year synthetic hydrographs and 200-year stream discharge frequency analysis values, the following observations were made:

1. The January 1935 storm had the greatest runoff values for 3- to 5-day durations and lowest instantaneous peaks.

2. Instantaneous discharges from the 200-year synthetic storm and from the 200-year frequency analysis were similar, except for Murray Creek.

3. The synthetic storm had the lowest runoff values for 3- to 5-day durations.

In order to improve the estimate of 200-year runoff provided by the January 1935 event, the hourly rain amounts were modified in two ways. The purpose of the first modification was to increase short-duration runoff intensities of the 1935 storm, which would more reasonably simulate 200-year peak runoff from small watersheds and in the floodways. This was accomplished by replacing approximately 7 hrs. of 1935 hyetograph data with maximum precipitation values of the synthetic 200-year storm. The second modification consisted of the adoption of the synthetic storm precipitation amounts for the last two days of the storm, with a smooth transition from the 1935 storm to the synthetic storm during the third day. This produced hydrographs with 3- to 5-day runoff values that were generally closer to the 200-year discharge frequency analysis amounts. Table 5 lists summaries of flows calculated using the modified January 1935 data for selected WSC streamflow basins. The three types of hydrographs are presented in Figure 11: the January 1935 flood, modified and unmodified, and the synthetic 200-year flood.

A set of hydrographs for the 36 subbasins was generated using curve numbers representing historic land use (circa 1974) and the unmodified 1935 rainstorm data. These hydrographs were used as input to the ONE-D model in an attempt to achieve an approximate simulation of the 1935 flood event.

Using the modified 1935 rainstorm, and curve numbers representing future land use as outlined in the Official Community Plans, the 200-year flood hydrographs were generated for each of the 36 subbasins. These hydrographs became the inflow hydrographs for the ONE-D hydrodynamic model simulation of a 200-year runoff event.

No adjustments were made to the model to account for possible future detention basin capacity in Surrey. The rationale for this approach is outlined below:

1. Runoff volumes of large floods are not significantly altered by detention, and detention basins will have less impact on the 200-year flood peak than on the 10-year flood peak for which they were designed.

2. Detention basins are planned to only serve the urban and suburban areas of Surrey and urban areas of Langley, approximately 43% of the total catchment.

3. Only areas developed after 1980 (approximately) have detention.

4. Detention basins delay the centroid of the hydrograph by 3 to 4 hrs., approximately.

5. The gauged basins, such as Mahood Creek and Hyland Creek, are receiving detained flow and this fact is reflected in some of the calibrations which were performed.

6. It is conservative to make no alteration to the hydrographs from urban land to account for detention.

Therefore, an alteration to the 200-year hydrograph was considered unnecessary to account for detention basins.

4.0 BOUNDARY BAY WATER LEVELS

4.1 Available Data

Reliable Boundary Bay water level data was a necessity for the realistic simulation of Serpentine and Nicomekl River hydraulics. There were primarily two types of water level information required:

- tidal data for simulating historic and 200-year river flood levels
- extreme water level information for the simulation of a sea dyke overtopping event.

Except for a crest gauge near Crescent Beach, which recorded only peak water levels, there was no long-term tide gauge located in Boundary Bay. Therefore, all data used in the simulations was derived from tide gauge records at other locations. The nearest long-term tide gauge was located at Point Atkinson (Figure 1), which recorded tidal information continuously since 1944, and intermittently before then. Longer periods of continuous records existed for Victoria and Seattle, but both of these gauges were much more distant than Point Atkinson from Boundary Bay.

To estimate historic Boundary Bay tidal data, the following transfer function relating Point Atkinson water levels to those at Boundary Bay was used:

$$B = 0.894 \times A - 2.69$$

where

A = Point Atkinson water level in metres referenced to local chart datum

B = Boundary Bay water level in metres referenced to Geodetic datum.

This transfer function was derived from one of two functions provided by the Canadian Hydrographic Service (CHS) at the Institute of Ocean Sciences, and was originally developed using data collected in 1992. It was selected for use in this study because it provided results which better fit a set of recorded crest gauge values for Crescent Beach.

A short-term tide gauge was operated at Tsawwassen. Boundary Bay is much nearer to Tsawwassen than to Point Atkinson. This record was used for estimating Boundary Bay water levels for one storm event in January 1968. The following transfer function was based on an equation provided by the CHS and used in this study:

$$B = 0.2774 \times T - 2.629$$

where

T = Tsawwassen water level in feet referenced to local chart datum

B = Boundary Bay water level in metres referenced to Geodetic datum.

Some additional water level data which was not directly measured in Boundary Bay, but provided valid estimates of Mud Bay water levels in the upper part of the tidal range, was measured by B. C. Environment at the downstream side of the Nicomekl sea dam from 1974 to 1982. At high tide, the sea dams were almost always closed, and the substantial depth of water in the Nicomekl River between the sea dam and Boundary Bay would have ensured very similar water levels at both ends of this segment of river channel. One variable which may have introduced differences however, is wind.

Most of the information presented herein related to extreme water levels in Boundary Bay was obtained from the four studies on this subject by Seaconsult Marine Research Ltd. In the first study Dunbar and Hodgins (5) presented estimates of the 200-year water level in Boundary Bay based on extreme value analysis of surges at Point Atkinson, Victoria and Seattle, and extrapolated to Boundary Bay using a transfer function for extreme water levels derived from numerical modelling of five severe storms. The model, named GF7, is an explicit finite difference model that simulates tidal and storm surge conditions in the Straits of Georgia and Juan de Fuca for a rectangular grid with a 2-km grid spacing. It was developed by the Institute of Ocean Sciences.

The second study, by Dunbar, Hodgins and Stronach (6) introduced several improvements to the numerical modelling procedures. A nested fine-grid storm surge model, named C2D, with a 400-m spacing, was applied to the Boundary Bay area. Also, refinements were made to the wind database and the wind stress term in the model. To improve confidence in the extreme water level transfer function from Point Atkinson to Boundary Bay, ten additional storms and the five earlier storms, were hindcast. Special emphasis was directed to reconciling crest gauge data and model predictions for the 16 December, 1982 storm surge event, which caused some of the most extreme water levels on record at the mouths of the Serpentine and Nicomekl Rivers.

The third and fourth studies (7, 8) examined probabilities of the joint occurrence of individual components which could combine to create an extreme water level in Boundary Bay. The final result of the Phase IV study was a set of estimates for extreme water levels, excluding wave setup, for return periods up to 200 years.

4.2 Tidal Data Used for River Flood Simulations

The water levels in Boundary Bay affect the water levels of the Serpentine and Nicomekl Rivers in the lowlands, even though the rivers upstream of the two sea dams are usually protected from high tide levels. During a typical tidal cycle, the river channels fill during high tides and drain through the side-mounted flapgates of the sea dams during low tides.

During average and low runoff periods, the daily variation in the river levels upstream of the sea dam is typically less than 1 m. During very intense runoff periods, however, the variation on the upstream side of the sea dam can be almost as large as the tidal range in Boundary Bay, which at times approaches 4 m. In such instances, the rate of rise of the tide in Boundary Bay is matched or exceeded by the rate of rise of river levels caused by high rates of runoff filling the river channels behind the closed sea dams. When the river level upstream of the sea dam rises higher than the level on the downstream side, the gates open, linking the rate of rise on both sides of the sea dam.

To calibrate the river models using historic flood events, historic Boundary Bay water level hydrographs were required. These were derived using recorded Point Atkinson hourly tidal data for the January 1935 and December 1979 floods, and Tsawwassen data for the January 1968 event, with the transfer functions defined in Section 4.1. Figure 12 shows the tidal data estimated for Boundary Bay which spanned each of these rainstorm periods and was used in the river models.

For the 200-year river flood simulations, the same tide data which occurred during the January 1968 storm was used. This tidal water level sequence was selected because it represented a historical condition which coincided with a major flood event, during a period when large astronomical tides typically occur.

It was considered prudent and reasonable to select a tidal sequence for simulation of the 200-year event which contained several high peaks and relatively high daily mean levels. The maximum recorded tide for the month of January 1968 at Tsawwassen occurred a few hours before the upland runoff peak occurred on the 19th of January. However, the peak flood stages reached in the Serpentine-Nicomekl lowlands are not sensitive to the exact timing of daily tidal and runoff peaks, since the lowland flooding resulting from a major storm event has a long time base that will span several tidal cycles.

4.3 Extreme Boundary Bay Water Levels

The first two studies by Seaconsult Marine Research Ltd. (5, 6), identified the following components which could occur in combination to create extremely high water levels in Boundary Bay.

- external storm surge
- local wind setup
- astronomic tide
- low-frequency mean sea level variations
- wave setup
- waves and runup

Each of these components is discussed in the following paragraphs, followed by an explanation of how the various components were combined and transformed to yield stage hydrographs which contained the estimated extreme Boundary Bay water levels. These hydrographs were subsequently used as input to the ONE-D model simulations which governed the delineation of a large downstream portion of the Serpentine-Nicomekl floodplain.

Some of the estimates presented in the following sections are referenced to a datum, either mean sea level (MSL) or Geodetic datum (GSC). It is noteworthy that these are not the same. Their relation in Boundary Bay is expressed by the following:

$$MSL - GSC = 0.05 \text{ m.}$$

External Storm Surge

The external storm surge refers to the rise above normal water levels in the Pacific Ocean caused by atmospheric pressure differences and wind stresses of major weather systems. These propagate into the Strait of Georgia and affect water levels in Boundary Bay. In the Seaconsult studies, the external surge did not include the effects of local wind stress; these were included under the wind setup component.

Seaconsult modelled the external surge wave for 15 historic events in the Georgia-Fuca waterway and, from the results, determined a transfer coefficient of 1.09 between Point Atkinson and Boundary Bay for the external surges in general. In the first Seaconsult study the 200-year surge level at Point Atkinson was determined to be 1.15 m. Therefore, the 200-year external surge, S_e, at Boundary Bay was estimated to be:

$$S_e = 1.09 \times 1.15 = 1.24 \text{ m.}$$

Local Wind Setup

Storm surge levels are generally greater in Boundary Bay than at Point Atkinson because of local wind stress on the shallow waters of the Bay. Seaconsult used their numerical models to measure the response of Boundary Bay to a 200-year return period wind from each of three directions: southeast, south and southwest. The 200-year wind speeds were determined by an extreme value frequency analysis using wind records measured at Saturna Island. The wind setup values calculated by their model for each wind direction at two locations in Boundary Bay are presented in Table 6, below. The location on the north shore of Mud Bay was nearest to the mouth of the Serpentine River, and the location at Crescent Beach was nearest to the Nicomekl River mouth.

Table 6

Wind Setup for 200-Year Wind*

Wind Direction:	Southeast	South	Southwest
200-year return period wind speed (km/h):	106	115	97
wind setup at north shore of Mud Bay (m):	0.395	0.812	0.477
wind setup at Crescent Beach (m):	0.221	0.467	0.272

*Data obtained from Seaconsult (6).

The wind setup values show that the response of Boundary Bay is sensitive to wind direction, and the largest response is at the north shore of Mud Bay. The authors of the Seaconsult study stated that this is a logical outcome given the shape of Boundary Bay.

Astronomic Tide

The astronomic tide is a cyclical process with generally two high-tide peaks and two low-tide minimums each day. The magnitude of the maximum high water differs each day throughout the year, but has a fundamental periodicity of 18.6 years. A time series of the length, using 15-minute intervals, was generated by Seaconsult, for Boundary Bay, excluding the low-frequency mean sea level variations. The maximum high tide water level was found by this method to be 1.60 m above MSL.

Low-Frequency Mean Sea Level Variations

There are several processes which act to vary the mean sea level over long periods. Seasonal sea level changes occur as a result of changes in estuarine circulation, offshore processes such as coast upwelling and large-scale changes in atmosphere circulation. These seasonal variations were determined from Tsawwassen tide records to have a magnitude of 0.22 m, with the highest levels of 0.116 m above MSL recurring every winter, during the storm surge season. Therefore, this was deemed to be additive to the other extreme water level components.

The El Nino phenomenon can also have a significant effect on the mean sea level. In 1983, a strong El Nino year, the mean sea level rose approximately 0.10 m at Seattle, Victoria and Point Atkinson. The effect of El Nino was not, however, included in the low-frequency sea level variation component, because at any one time there is a higher chance of the mean sea level being lower than the long-term mean. In other words, at any given time there is a greater probability that the El Nino phenomenon is not occurring. To limit the degree of conservatism in the extreme water level estimates, the El Nino contribution to the low-frequency sea level variations was taken as zero.

In addition to the above, there is a long-term trend of rising mean sea level in the Seattle and Victoria tide records. Whether this is due more to climate change or to land subsidence caused by tilting of the earth's crust in this region has not been determined. The effect is small in the recent tide records for Point Atkinson, therefore it has been disregarded by Seaconsult in their extreme water level estimates. In summary, the component of low-frequency sea level variation was taken to be 0.116 m relative to MSL.

Wave Setup

Wind-generated waves propagating into Boundary Bay shoal and break near the shoreline. This onshore mass transport of water by wave action alone causes an increase in the water level near the shore called wave setup. The second study by Seaconsult (6) estimated wave setup magnitudes by hindcasting three severe storms, which had peak wind speeds ranging from 82 to 97 km/h. A subsequent analysis by Hay & Company (17) produced the following estimates for wave setup:

North Shore of Mud Bay	0.30 m
At Crescent Beach	0.05 m

There is some uncertainty regarding the applicability of the above estimates to a situation where these dykes are overtopped during an extreme surge, wind setup and tide event. A necessary condition for the wave setup to attain the above magnitudes is that the waves must break towards a shore or small impoundment. It was not possible in this study to quantify this refinement to the wave setup estimates, therefore it could only be addressed in a subjective manner. This is discussed further in the section on the summation of the components.

Waves and Runup

A maximum wave height of 2.5 m was hindcast by Seaconsult for their wave setup calculation. This represents a wave crest at 1.25 m above the still water level. However, these waves would occur on the ocean side of the sea dykes. The shallow depths near the existing shoreline and the sea dykes would cause these large waves to break, even if the sea dykes were to be overtopped to the depths predicted by this study. On the land side of the sea dykes, the numerous roads, railroads and river dykes would act as breakwaters, limiting the maximum wave heights to much smaller values by restricting the available fetch lengths for waves regenerating on the Serpentine-Nicomekl floodplain. Similarly, even if the necessary ground slope conditions existed, any wave runup, which is a function of wave height and ground slope, would be small in areas on the land side of the sea dykes. Therefore, wave height and runup components were not included in the extreme water level components. These are identified on the mapping as a special flood hazard that would apply only to land or buildings that are exposed to a wave threat. For example, large storm waves from Boundary Bay could pose a significant additional hazard to the exposed buildings along Crescent Beach, but would not affect buildings in sheltered locations. The floodplain maps are based on estimates of the still water level only, with no waves or runup considered.

Summation of the Components

In the initial Seaconsult study (5), the components identified above were simply added. Table 7, below, presents a summary of these components and their totals for the two locations and three wind directions considered in the modelling.

Table 7
Summation of the Extreme Water Level Components*

Wind Direction: / Wind Speed (km/h):	Southeast 106	South 115	Southwest 97
Location 1 - North Shore of Mud Bay			
external storm surge	1.24	1.24	1.24
local wind setup	0.395	0.812	0.477
astronomic tide	1.60	1.60	1.60
low-frequency sea level variation	0.116	0.116	0.116
wave setup	<0.10	<0.30	<0.35
datum shift, MSL to GSC	0.05	0.05	0.05
Totals	3.50	4.12	3.83
Location 2 - At Crescent Beach			
external storm surge	1.24	1.24	1.24
local wind setup	0.221	0.467	0.272
astronomic tide	1.60	1.60	1.60
low-frequency sea level variation	0.116	0.116	0.116
wave setup	<0.10	<0.25	<0.35
datum shift, MSL to GSC	0.05	0.05	0.05
Totals	3.33	3.72	3.63

*Data obtained from Seaconsult (6).

The results of the summation indicate that the highest extreme Boundary Bay water levels near Mud Bay would occur in conjunction with a local wind from the south. The calculated maximum still water levels, relative to Geodetic datum, for a south wind scenario, are 4.12 m along the north shore of Mud Bay and 3.72 m at Crescent Beach.

The totals presented in Table 7 are conservative estimates of overall 200-year return period extreme water levels, because summing the components requires the assumption that all would occur simultaneously. In other words, for the water surface to attain these levels, it would be necessary for the 200-year storm surge, the 200-year wind setup from the south, the peak high tide, the calculated sea level variation and the wave setup to all peak at the same time. Although the occurrence of all of these factors are highly interdependent, it is obvious they also have some degree of independence. The joint probability of their coincidence was not addressed in the first two Seaconsult studies.

The likelihood that the estimates in Table 7 are conservative was recognized by Seaconsult (7, 8), and by B. C. Environment engineers reviewing the Seaconsult studies (18). As a result, attempts were made to quantify the degree of conservatism present in the totals in Table 7, and to reduce them accordingly.

In their fourth study (8), Seaconsult conducted extreme value frequency analyses on the tide, external surge and wind setup components. One set of analyses was based on the assumption that all three of these components were statistically independent. In the other set of analyses, it was assumed that the external surge and wind setup were coupled. The results from both approaches were similar, differing only 0.03 m at Crescent Beach and 0.13 m along Mud Bay for a 200-year return period. At both locations, the assumption of independent components produced the higher extreme water level estimates and these higher estimates were adopted for this study.

The adjusted 200-year extreme water levels for Boundary Bay are listed in Table 8. Wave setup was not included in Seaconsult's analyses, and was therefore added here.

Table 8
Final Extreme Water Level Estimates

Location	Extreme Water Level Excluding Wave Setup (m)	Wave Setup (m)	Extreme Water Level (m)
North Shore of Mud Bay	3.09	0.30	3.39
At Crescent Beach	2.93	0.05	2.98

The wave setup component was not reduced in the above estimates even though, as stated previously, there are indications that the wave setup component itself appears to be conservative where the water from breaking waves becomes less confined once the sea dykes are overtopped. Another factor related to local wind, which was not considered in the analysis, is the wind setup that would occur on the inundated cells in the floodplain. Omitting this factor would tend to make the results less conservative. Neither the reduction to the Boundary Bay wave setup, nor the increase due to wind setup on individual cells could be reasonably quantified for this study. To some extent these two adjustments would cancel one another, and the resultant difference was expected to be within the range of uncertainty covered by the freeboard applied to the calculated 200-year floodplain levels. Therefore, the adjusted 200-year water levels listed above were used in the simulations of flooding from the sea.

Tsunamis

A tsunami is a seismic sea wave generated by a submarine or coastal landslide or earthquake. It is not possible with the current state of knowledge and available data, to assign a probability to magnitudes of peak water levels caused by tsunamis.

For the Strait of Georgia, the tsunamis can be classified into two groups; those propagated from the Pacific Ocean into the Strait, and those generated within the Strait of Georgia. For the tsunamis which propagate into the Strait, there have been model studies which suggest that their amplitudes within the Strait of Georgia would be less than 1 m (19).

Within the Strait of Georgia, a tsunami could be generated by a local earthquake centred in the Strait or by a submarine landslide of the foreslope of the Fraser Delta (20, 21). Such a slide may or may not be generated by an earthquake. The presence of submarine hills indicates that a major foreslope slide has occurred in the past. No estimates could be found of the amplitude range that such a slide would generate in Boundary Bay, however, current research at the Institute of Ocean Sciences is addressing this question.

Recorded amplitudes of tsunamis caused by submarine slides in other parts of the world suggest that such waves in the Strait of Georgia might create peak water levels that would be much higher than the extreme levels estimated in the previous section of this report. These waves could have devastating consequences on areas of low relief and high population density (21).

For the floodplain mapping produced by this study, flood levels caused by tsunamis have not been considered, since the wave characteristics themselves cannot yet be quantified. Users of the mapping should be aware of the potential special hazard posed by tsunamis to the Serpentine-Nicomekl lowlands and adjacent foreshore areas, such as Crescent Beach.

4.4 Development of Time Series Containing the Extreme Levels

The simulation of sea dyke overtopping and flooding of the Serpentine-Nicomekl valley from the sea required a times series of Mud Bay water levels as a boundary condition at the mouth of each river. Therefore, stage hydrographs were developed for these locations, which contained as maximum peaks the extreme water levels described in the foregoing sections. These stage hydrographs were developed by scaling up the December 1982 storm surge recorded at Point Atkinson, and adding it to the predicted astronomic tide for the same period at Crescent Beach. The procedure comprised the following steps:

1. The December 1982 storm surge at Point Atkinson was derived by subtracting hourly values of the predicted astronomic tide from the observed tide at this gauge.

2. The predicted astronomic tide at Crescent Beach for the same period was estimated by applying the transfer function defined in Section 4.1 to the predicted astronomic tide for Point Atkinson.

3. The Point Atkinson storm surge values (step 1) were multiplied by a scale factor, and the results were added to the Crescent Beach predicted astronomic tide (step 2) to create an estimated Crescent Beach tide plus storm surge data set. The scale factor was adjusted until the peak value of this data set matched the extreme water level for the north shore of Mud Bay (3.39 m above Geodetic datum). This data set became the stage hydrograph boundary condition for the Serpentine River for use in the ONE-D modelling of the flooding from the sea.

4. Step 3 was repeated with a different scale factor to match the extreme water level determined for Crescent Beach (2.98 m above Geodetic datum). This resulting stage hydrograph was applied as the downstream boundary condition for the Nicomekl River in the ONE-D model.

Figure 13 shows the 4-day time series at Crescent Beach for the predicted astronomic water levels (step 2), and for the two data sets containing the extreme levels (steps 3 and 4).

5.0 HYDRAULIC MODELLING

5.1 The ONE-D Hydrodynamic Model

A computer program developed by Environment Canada and known as ONE-D, was used to calculate the maximum water levels which would occur throughout the Serpentine-Nicomekl study area when subjected to flood-producing conditions. The program allows the simulation of gradually varied, unsteady, one-dimensional flows in open channels by solving the St. Venant equations using an implicit finite difference technique developed at the Massachusetts Institute of Technology by Gunaratnam and Perkins (22). It was designed for application to rivers and tidal estuaries, and to divided flow multiple-channel situations where steady-state solutions are not applicable.

The ONE-D program can be used to model the following:

- channels with irregular cross sections
- off-channel storage, as occurs in the non-conveying portion of a floodplain cross section
- embayment storage, as occurs in small bays or lakes
- bridge waterways
- culverts
- floodboxes
- sea dams
- pumpstations
- overtopping of embankments, such as road or dykes
- time-dependent external boundary conditions (discharges or water levels) and lateral inflows
- internal boundary conditions, such as would occur at a waterfall, weir, gate structure or spillway
- effect of ice cover
- several water quality parameters.

Other features of the program include:

- automatic interpolation and extrapolation of cross sections at computation mesh points based on the input cross sections

- variable computational mesh spacing from reach to reach
- multiple reaches joining at a single node
- variable roughness, expressed as Manning's "n," with depth, and from cross section to cross section..

The ONE-D program also allows the user to define the output at the end of one run as the initial conditions for a subsequent run, provided the basic structure of the model is not altered. This makes it possible for certain parameters, including cross section shapes, culverts, dyke overtopping weirs and the time step, to be modified at any time during a simulation.

The program is written in FORTRAN and is organized in a modular structure to allow subroutines to be easily added in order to extend the program's capabilities. The program is documented (23, 24, 25, 26) and supported by Environment Canada to Canadian users.

The program has been successfully used for several projects across Canada and overseas. It was applied in a flood study for Truro, Nova Scotia, where tides, dyke overtopping and floodbox flows required modelling. It was used on the Fraser and Pitt Rivers where flow reversals were an important aspect of hydraulic behaviour. Flows through the complex network of channels at the Peace-Athabasca Delta have been modelled with ONE-D. Other applications have been completed for the Mackenzie River Delta in the Northwest Territories and the Red River in Manitoba.

Several important features were added to the ONE-D program by Environment Canada for its application to the Serpentine-Nicomekl River system. The ability to simulate the behaviour of the sea dams was added to the program. Although sea dams are essentially a large set of floodboxes, the program did not have the ability, before this project, to properly simulate a floodbox in a mainstem river channel. The ability to model culverts as reaches such that their hydraulics were directly included in the dynamic solution, was introduced in this project. A series of tests was performed by KPA to confirm that bridge waterways, with partial or total inundation of the deck, could be reasonably modelled.

As part of the Terms of Reference for this study, KPA wrote and tested a subroutine for the simulation of pumpstations in a channel system. The subroutine, which Environment Canada has incorporated into the ONE-D program, can handle single or multiple pumps with local or remote sensing switches, including emergency shut-off during flood conditions. The subroutine was not used for the Serpentine-Nicomekl study, because the effect of pumping was estimated to have an insignificant effect on final flood levels for the extreme events considered in this study.

An attempt was made by Environment Canada to develop a subroutine to simulate erodible dyke breaches. However, this was not successfully completed within the time frame for this study. Dyke breaches were simulated by stopping a simulation at a time when the water level began to exceed the dyke crest elevation, modifying the data to effect an instantaneous dyke breach, then continuing with the run to completion.

The ONE-D program required other preprocessing software to enable more efficient model development. These programs were used to prepare most of the data for execution by ONE-D. The program XSECTION converted channel cross sections from the common HEC-2 GR format to that required for COORD1. The programs COORD1 and COORD2 converted cross section geometry to hydraulic tables required by ONE-D. Once each initial model was built, there was no need to use these preprocessing programs for later data modifications or program runs.

The ONE-D program did not have any graphical postprocessing capability, and provided only tabular numerical output. To create graphical summaries for this study, blocks of numerical data were extracted from the ONE-D output and imported into computer-aided drafting and spreadsheet programs. Figure 14 provides a summary of the family of programs and data files associated with the ONE-D program.

The size of the executable ONE-D program could be controlled by the user, allowing a small application to be run on a computer with limited memory, without restricting the program from running large, complex models on a different computer. This was done by user specification of 22 dimensions in the source code prior to compilation of the program. The size of the final compiled code depended on the dimensions required for each particular application.

The Serpentine-Nicomekl ONE-D models were developed, tested, calibrated and run on a DOS-based 486 computer with a clock speed of 50 MHz. The computer had an 80487 math coprocessor and 16 Mb of extended memory. The available hard disk memory was 380 Mb, using the DOS 6.0 DoubleSpace facility. The ONE-D source code was compiled using the Lahey FORTRAN compiler F77L-EM/32, Version 5.0. This compiler used extended memory to create executable programs larger than 640 kb in size.

With this combination of hardware and software, it was possible to model all the necessary channels, floodplain cells and hydraulic elements in the entire Serpentine-Nicomekl study area. The maximum run time was approximately 25 hours for a 5-day simulation of the entire linked model, using a 7.5 minute time step. The size of the executable ONE-D program necessary to contain the model was approximately 10 Mb. The Serpentine-Nicomekl model was, by far, the largest and most complex application of the ONE-D program to date.

5.2 Input Data

To create a Serpentine-Nicomekl ONE-D model, the following types of input data were required:

- definition of how reaches, nodes and other hydraulic elements are connected
- dimensions and elevations, of all relevant physical features
- discharge hydrographs at all inflow points
- water level hydrographs at Boundary Bay
- hydraulic parameters such as Manning's "n" and loss coefficients.

Reach-Node Network

The basic framework of the model was a network of river reaches connected to each other at common endpoints, or nodes. Floodplain enclosures were typically modelled as single reaches, or a series of a few reaches. In most instances these floodplain enclosures were not directly joined to the river network through reach-node links. Instead, interconnections were made using hydraulic elements, such as culverts, floodboxes or embankment overflow weirs. A schematic diagram of the network used for simulating the 200-year runoff event is presented on Figure 15. Another network developed for modelling floods from the sea is shown on Figure 16.

The reaches shown in Figures 15 and 16 are generally numbered according to their location. Most of the reach numbers less than 1000 are on the Serpentine River and its tributaries. Reaches along the Nicomekl River and its tributaries are numbered between 1000 and 2000. Reaches that represent floodplain storage cells are numbered between 5000 and 6000.

The arrangement of reaches and nodes was specified in the input data by a reach-node connectivity table. The other hydraulic elements were located in the network by specifying the particular reach number and distance along the reach where the upstream and downstream end of each element was to be connected.

Physical Data

Dimensions and elevations of the principal physical features which governed behaviour of the Serpentine-Nicomekl hydraulic system, were obtained from several sources. Description of these features in sufficient detail to create a representative model of the entire system required a large volume of numerical data acquired from a variety of sources. The sources of the data used to define these physical features are listed in Table 9, on the following page.

Discharge Data at Inflow Points

Inflow hydrographs used in the hydrodynamic modelling for upstream boundary conditions and for lateral inflow entry points were derived as part of the hydrologic analysis for this study. The procedures and results are described in detail in Section 3.0 of this Design Brief.

Boundary Bay Water Level Hydrographs

The tidal downstream boundary conditions for the Serpentine and Nicomekl Rivers were derived for this study as described in Section 4.0 of this Design Brief.

Table 9

Sources of Measured Physical Data

Physical Feature	Data Source	Date of Measurement
• Surveyed River Cross Sections	B. C. Environment	
- 28 near Serpentine and Lower Nicomekl Bridges		1984
- 6 between sea dams and ocean		1985
- 128 throughout study area		1991
• Cell storage volumes from 1:5000 Mapping	B. C. Environment (mapping only)	1992
• Bridge Dimensions/Elevations	B. C. Environment	1984
• Seadams	Associated Engineering Services Ltd.	1975
• Culverts:		
- ditch crossings	District of Surrey, drainage mapping	1992
- major streams	B. C. Environment	1984, 1991
- miscellaneous	KPA site investigations	1992
• Floodboxes:		
- within Surrey Dyking District	Surrey Dyking District	1992
- within Mud Bay and Colebrook	B. C. Environment	1987
• Dyke Crest Elevations:		
- at cross section locations	B. C. Environment	1984, 1991
- within Mud Bay and Colebrook	B. C. Environment	1987
- Nicomekl R. upstream of 192nd Street	Westcraft Construction Ltd.	1991
- from spot elevations on 1:5000 mapping	B. C. Environment	1992
• Top of Road and Railway Profiles:		
- 21 surveyed profiles	B. C. Environment	1991
- from spot elevations on 1:5000 mapping	B. C. Environment	1992
• 168th Street Linear Ponds	District of Surrey	1989
• 168th Street Ditch south of Cloverdale	Dayton & Knight	1992

Hydraulic Parameters

Initially hydraulic data required for modelling, such as Manning's "n" and entrance and exit loss coefficients for bridges and culverts, was selected on the basis of experience of the model developers, hydraulics manuals and textbooks. Some of these parameters were adjusted during the calibration process to achieve a match with measured information.

Manning's "n" values for the channels varied from 0.055 for the narrow channels flowing on moderately steep slopes near the upstream boundaries of the study reaches, to 0.030 for the broad channels near the river mouths. The roughness coefficients for the culvert barrels ranged from 0.024 to 0.035. Entrance loss coefficients for the culvert reaches were typically set to a value of 0.9. For culverts and floodboxes that were modelled as steady-state hydraulic elements between adjacent floodplain cells, it was necessary to provide an equivalent Manning's "n" that accounted for entrance losses. For these structures the roughness coefficients typically ranged from 0.035 to 0.050.

5.3 Modelling Strategy

The general approach taken in the simulation of flood-producing events with the ONE-D program was to first develop and calibrate a model for flooding caused by extreme runoff, then to revise the data to develop a second model for flooding from the sea. The rationale for this approach was that a considerable quantity of calibration data existed for runoff floods, but very little quantitative information could be found for floods from the sea. Developing the first model on the basis of the superior calibration data allowed many elements to be verified and used with confidence in the second.

For the runoff flood simulations, submodels of the Serpentine and Nicomekl Rivers were first developed and tested separately and, to some extent, calibrated prior to linking the two models together. The final calibrations were executed using linked models to properly simulate the effects of flooding in the downstream part of the valley. The configuration of the rivers, dykes and floodplain is such that for all flows up to a moderate flood, the rivers behave independently. For very large floods, however, the hydraulic behaviour of each river begins to affect the other in the lowland area as cells between the two rivers become inundated. This interdependence could only be dynamically simulated by cross-linking the two individual river models.

The steps taken to develop and calibrate each river model are outlined below.

1. Input data for the river channel reaches, with mainstem culverts and bridges included, was prepared and tested in sequential stages, starting from the upstream end. Assumed initial conditions were coded for each reach.

2. Floodplain storage enclosures, or cells, were coded as single reaches with very large storage capacities and added to the model. As each cell was added, the hydraulic elements which connect it to the river and to adjacent cells, such as culverts, floodboxes or embankment overflow weirs, were also added.

3. Discharge hydrographs for the January 1968 flood were added as boundary conditions or as lateral inflows to the appropriate cell reach or river reach.

4. A sea dam was added into each river model.

5. The tidal stage hydrograph for the flood period in January 1968 was added as the downstream boundary condition.

6. Initial conditions were reviewed for consistency, and several input data checks were conducted.

7. The upstream portion of the Serpentine and Nicomekl Rivers were calibrated by adjusting input data for successive runs for the 1968 flood.

8. The models were linked and 1968 calibration runs for the combined Serpentine-Nicomekl system were executed. Successive modifications were made until a satisfactory match of computed and estimated actual water levels was achieved.

9. Starting with digital copies of the 1968 models, the 1968 data was replaced with 1979 discharge and tide data for the boundary and initial conditions in the individual river models, and the calibration process was repeated. It was not necessary to link the models for the 1979 calibration because flooding in the downstream part of the floodplain was not widespread, and the water levels in the two rivers appeared to be varying independently from one another.

10. A linked model was prepared using 1935 discharge and tide data and one calibration run was executed. Due to the uncertainties regarding the physical condition of the basin and floodplain in 1935, and due to the paucity and very approximate nature of the 1935 calibration data, this exercise provided little, if any, benefit to the overall calibration of the model.

11. The 200-year discharge data was prepared for a linked Serpentine-Nicomekl model, using tide data for the 1968 historic event, and an initial series of runs was executed. The results of these are presented in Appendix 2.

12. The late discovery of high water data associated with four floods since 1972 in the Upper Nicomekl area indicated that a recalibration was necessary for this part of the model. Following recalibration, a final run was executed for a case in which no river dykes breached. Then three additional scenarios involving river dyke breaches were simulated.

13. The maximum water levels attained in each reach and cell of the model were plotted on a map. These levels, with freeboard, formed the basis for floodplain delineation in those parts of the study area where they were not exceeded by model results for flooding from the sea.

14. Sensitivity runs were executed during and after the calibration phase to estimate the effects that varying certain parameters had on flood levels.

For the simulation of flooding from the sea, only a linked model, developed as described by the following steps, was used:

1. To simulate flooding from the sea, it was necessary to substantially modify the linked model used for the 200-year run. Details pertaining to large upstream portions of the Serpentine and Nicomekl Rivers that would not be affected by flooding from the sea were removed from the model to reduce the program execution times.

2. The tidal stage hydrographs containing the initial estimates of the extreme Boundary Bay water levels (Table 7) became the downstream boundary conditions.

3. The hydraulic elements which defined the embankment overtopping weirs were redefined in greater detail, since these would have a major influence in determining the extent and depth of the flooding inland.

4. The December 1979 flood discharge data, which represented about a 10-year return period event, was used for the sea flood simulation because it was considered likely that an extreme windstorm would likely be associated with heavy rain.

5. After completion of an initial model run which simulated overtopping of the sea dykes, a second run was executed which contained three 100-m long sea dyke breaches. Two breaches were into the Mud Bay Dyking District, with one near the mouth of the Nicomekl River and the other near the mouth of the Serpentine. A third breach was located along the north shore of Mud Bay into the Colebrook Dyking District. All subsequent runs contained these three sea dyke breaches. Six scenarios involving different combinations of river dyke breaches in conjunction with the sea dyke failures were simulated. These results are presented in Appendix 2.

6. The results of the fourth study by Seaconsult became available after the work in Step 5 was completed. The downstream boundary conditions were replaced with revised extreme Boundary Bay water level estimates based on the peaks listed in Table 8. The base run with sea dyke breaches, but no river dyke breaches, was repeated. Then three scenarios of different combinations of river dyke breaches were simulated.

7. The maximum water levels obtained in each of the runs were displayed on a map, and compared with the map of the highest runoff flood levels. The highest maximum water level in each floodplain cell and in each river reach was identified. Judgement was used to reconcile differences across dykes or roads where dyke breach scenarios were not tested with the model.

5.4 Calibration of the Runoff Flood Model

The ONE-D model used to simulate the 200-year rainfall runoff event was calibrated on the basis of two past flood events. One of these events, the January 1968 flood, provided the basis for calibrating the model spatially, due to numerous aerial photographs taken during the flood. The other event, which occurred in December 1979, was used to calibrate the model temporally, since good-quality stage hydrographs were recorded in 1979 at five river locations in the study area.

Model Limitations Affecting Calibration

Once all the physical data for each river system was entered into the input file, together with the tide and discharge data for the 1968 flood event, a series of model runs was executed to overcome numerical instabilities, which often resulted in premature termination of the run and yielded incomplete results. One of the principal causes of the instabilities was the combination of large depths of inundation over long lengths of road or other low barriers in the floodplain, relatively small volumes of storage in a cell adjacent to the road and long time steps. The ONE-D program routine which handles embankment overtopping was based on the broad-crested weir formula and was not an integral part of the finite difference solution procedure. Instead it operated as a steady-state function between each time step's finite difference solution. The most common type of instability occurred when a relatively small head difference across a long, deeply-inundated road resulted in the calculation of a large discharge. This discharge was multiplied by the time step and the resulting volume of water was transferred to the downstream side of the embankment. When the storage volumes were small and time steps were long, the water level on the downstream side rose higher than the upstream water level, resulting in a greater head difference in the opposite direction. The cyclical process repeated itself with increasing amplitude until a balance was found or program execution was aborted due to a "division by zero" error.

The obvious solution to the above problem was to decrease the time step. However, with a model as large and as complex as the combined Serpentine-Nicomekl model, there were practical limits to how small the time step could be, since the program execution time is inversely proportional to the time step duration. The time step used for the runoff flood calibration runs was 15 minutes, and for the final runoff flood simulations, 7.5 minutes.

Another approach to resolving this problem was to reduce the length of the overtopping along the embankment. Because the instabilities were caused by excess discharge capacity over the weirs that occurred after the adjacent cells had filled, there was very little effect on final results introduced by reducing the weir length. The cells still filled to essentially the same peak level, but the filling required a slightly longer time. The instabilities were eliminated or reduced to tolerable amplitudes by this technique.

There were two embankments in the upper Serpentine lowlands that were not surveyed for this project, but which were pivotal in determining peak water levels in nearby cells. One of these was 83rd Avenue east of the Serpentine River, over which large volumes of south-bound flood waters passed in 1968 and other years to maximize flood levels from 83rd Avenue to 176th Street (see Photo 6, Appendix 1). This street divides Cell 5043 from 5037. To the north of 83rd Avenue, a low berm east of the radio towers (by 80th Avenue and 176th Street) appeared to control flood inflows into Cell 5037.

The crest lengths of the weirs used by the model to simulate these two embankments were not reduced. The elevation of the embankment crests were estimated by calibration, using the four spot heights on the mapping to indicate initial trial values for the weir crest elevations. It was found that the maximum flood levels were sensitive to the crest elevations used at these two barriers.

1968 Flood Calibration

On 20 January, 1968, two days after the rainstorm began, George Allen Aerial Photos Ltd. took approximately 40 low-level oblique aerial photographs of the flooding in the Serpentine and Nicomekl basins. Examples of these photos appear in Appendix 1. The date was noted on each photo, but the time of day when they were taken was not recorded. The sky was overcast for all but a few of the photographs. By transferring the direction of shadows of some tall hydro poles on one photograph to maps, the azimuth of the sun was measured, and the time of photograph was calculated. It was determined that most of the photos were taken between 2:00 and 2:30 p.m. P.S.T. Therefore, for calibration, the model results at 2:15 p.m. were compared to levels estimated from the photos.

The accuracy of the water surface elevations estimated from the photos varied considerably from one area to another, depending on the scale and clarity of the image, the presence of structures with known elevations and the accuracy and representativeness of spot heights in the 1992 1:5000 scale mapping. For example, in the upper Serpentine lowlands a partly-inundated bridge deck allowed a very accurate water level estimate to be made. In contrast the area west of King George Highway was distant in the photo and the accuracy of the estimate was much poorer. Another characteristic of this downstream area, which was not as deeply inundated as the floodplain farther inland, was that the flooding was not contiguous within each cell. Therefore, separate ponded areas would likely have had different water levels, but the model estimated only one average water level for each cell. Although most of the study area was visible in the set of photographs, the Nicomekl River upstream of the Fraser Highway bridge was not photographed at all. Therefore, part of the model which simulated flooding of this area was not calibrated for the 1968 event.

Ranges of water levels in the photos were estimated for each cell in the lowlands and for each major reach of the upper Nicomekl River to the Fraser Highway. The size of the range reflected the accuracy of the estimates. These ranges are listed in Table 10.

The results of the final calibration run for 2:15 p.m., 20 January, 1968, are also listed in Table 10 for comparison with the photographed water level estimates. The final calibration was achieved in two stages. First, the model was calibrated for the 1968 event only. Then, after the calibration for the 1979 storm event was completed, the 1979 model's discharge and tide data was replaced with 1968 data. Except for some road and dyke elevation changes and dyke breaches that occurred only in 1968, all other calibration parameters used in the 1979 model remained the same. All dyke breaches and road elevation changes were known to have occurred, but two dyke elevation changes could not be confirmed. These dyke changes were necessary to match water levels for both the 1968 and 1979 events. Thus, the second stage results, which appear in Table 10, represent a final 1968 simulation with a model essentially calibrated for both flood events.

Table 10

Observed and Simulated Water Levels for the 1968 Flood

Lowland Cell No.	Range Containing Photographed Water Levels Low	High	Water Level Computed in Final Calibration Run (Run L107)
5001	0.5	1.0	0.60
5002	0.4	0.8	0.43
5003	0.4	0.8	0.29
5004	0.7	0.9	0.73
5005	0.7	1.0	0.75
5006	0.5	1.0	0.51
5007	0.6	0.9	0.42
5008	0.3	0.5	0.42
5009	0.6	0.9	0.81
5010	0.1	0.5	0.48
5011	0.4	0.6	0.49
5012	0.4	0.6	0.42
5013	0.1	0.3	0.40
5014	0.7	1.0	0.81
5015	0.0	0.9	0.03
5016	0.5	0.9	0.81
5017	0.0	0.2	-0.01
5018	-0.1	0.2	-0.05
5019	0.3	0.5	0.32
5020	0.8	1.6	1.07
5021	0.8	1.0	0.88
5022	1.3	1.5	1.43
5023	0.6	1.0	0.90
5024	0.9	1.4	0.92
5025	1.3	1.5	1.34
5026	0.3	0.7	0.39
5027	0.0	0.2	0.15
5028	0.8	1.0	0.80
5029	0.8	1.1	0.81
5030	0.1	0.4	0.19
5031	0.8	1.1	0.81
5032	1.5	1.7	1.68
5033	1.4	1.7	1.68
5034	0.8	1.1	1.02
5035	0.8	1.1	1.02
5036	0.8	1.1	1.02
5037	1.6	1.9	1.64
5038	1.5	1.9	1.77
5039	1.5	2.2	1.72
5040	0.3	0.6	0.46
5041	0.3	0.6	0.37
5042	1.2	1.5	1.45
5043	1.5	1.7	1.62
5044	1.0	1.2	1.03
5045	2.2	2.5	2.27
Upper Nicomekl Reach No.			
1060	6.0	6.4	6.31
1080	5.8	6.4	6.26
1090 @XS-47	5.0	6.1	5.24
1110 @ XS-44	4.2	4.8	4.03
1120 @XS-410	3.8	4.5	3.50
1150	2.0	3.0	2.45
1160	2.0	3.0	2.14

Figure 17 shows the cells in the lowlands where these calibrated model results fell within the range of observed water levels and where they were higher or lower than the observed range. A possible reason for the underestimation in cell 5003 and the low value for cell 5002, is that the model contains data for an existing floodbox for Clearwater Creek, the principal drainage channel for cells 5002 and 5003, that was enlarged in 1984. Therefore the actual flooding in 1968 would have reflected a smaller floodbox capacity and possibly higher water levels than those computed by the model for cells 5002 and 5003.

Table 10 also lists ranges and computed water levels for the Nicomekl River between the lowlands at 184th Street and the Fraser Highway. The model computed lower-than-observed water levels in the area between the 200th Street and 203rd Street bridges, and for about 500 m upstream of the 203rd Street bridge. This discrepancy might have been due to the improvements made to these bridges since 1968, which are included in the model, but not reflected by the observed levels.

Despite these few areas where the differences between observed water levels and model results could not be entirely resolved, the model provided a reasonable simulation of spatial distribution of floodwaters which occurred in the 1968 flood. Even though the observed water level data was not precise, the calibration of the 1968 event established a basis for confidence in the model's ability to simulate real flood events with generally accurate results.

1979 Flood Calibration

Although some aerial photography existed for the 1979 flood, the photos covered less than half of the lowland area. Unlike the 1968 flood, when all of the water level recorders in the study area were inoperative, there were five continuous chart recorders operating throughout the 1979 event. The gauges and the agencies responsible for them are listed below:

Table 11 Water Level Recorders Operating during 1979 Flood		
Map Symbol*	Gauge Location	Agency Responsible
L	Nicomekl River below Murray Creek	Water Survey of Canada
K	Serpentine River above Sea Dam	B. C. Environment
D	Nicomekl River below Sea Dam	B. C. Environment
A	Serpentine River at Fraser Highway	District of Surrey
B	Serpentine River at Highway 10	District of Surrey

***See Figure 17.**

All these gauges were surveyed to Geodetic datum, allowing direct comparisons of the recorded water level hydrographs with the simulated ones. Four of the gauges, all except the Nicomekl River below Murray Creek recorder, were in areas that were affected by tidal conditions, and three of these were upstream of the sea dams.

Results from the initial calibration runs indicated a consistent shortfall in the simulation of the peak water levels at all four gauges for most of the high tide periods in the three-day simulation period. This seemed particularly unusual for the Nicomekl River below Sea Dam gauge, because there were no barriers between this gauge and Boundary Bay. At high tide the lower Nicomekl River channel was filled with water and would have had a very large flow capacity even at very shallow river gradients. During the part of the tidal cycle when the gates of the sea dam were closed, there would have been very little flow in the channel downstream of the sea dam. Therefore, at such times water levels at the gauge below the sea dam should have equalled water levels in Boundary Bay. However, the Crescent Beach water levels, estimated from Point Atkinson data using a transfer function (see Section 4.1), were an average of 0.2 m lower than the water levels recorded below the sea dam during the upper half of the tide cycles on all three days of the 1979 storm event.

No reason for this difference could be confirmed. A check was made which indicated that this difference did not consistently occur between estimated Crescent Beach and recorded Nicomekl River levels for other storm events. It is possible that wind setup across the Boundary Bay mud flats may have been a factor during the 1979 flood, but unlikely that the wind setup should result in such a consistently even difference over three high tide cycles.

The calibrations were completed using high tide data based on the Nicomekl River below Sea Dam records. Following the calibration procedure, an additional simulation was executed using the unaltered Crescent Beach estimated tide data. The effect of the different sets of tide data on upstream water levels is evident by comparing the pairs of hydrographs in Figures 18, 19, 20 and 21.

These four figures show how closely the model replicated the actual water levels through the 1979 storm event. Figure 18 shows the simulated and recorded water levels on the Nicomekl below the Sea Dam.

The close match for the high tide parts of the cycles for the hydrograph using Nicomekl recorded data indicated a negligible head loss between the sea dam and Boundary Bay. A good match was obtained for the low tide periods, indicating that the model was simulating the Nicomekl River channel between the sea dam and Boundary Bay reasonably well. The water levels at this location were fundamentally controlled by the tide, and the comparison in Figure 18 demonstrated calibration of only a small part of the entire modelling exercise.

Figure 19 represents hydrographs for the Serpentine River upstream of the sea dam. The water levels at this location were also primarily controlled by the tide. This calibration confirmed that the hydraulics and the openings and closings of the sea dam, were simulated with reasonable accuracy.

Approximately 7.6 km upstream of the sea dam, the Serpentine River passes under Highway 10. The recorded and simulated hydrographs at this location appear in Figure 20. In general, the model replicated the water level variations well. The four simulated peak levels on the 18th and 19th of December were lower than the recorded ones by about 0.1 m. It was found during calibration that these peak levels could be raised by constricting the key hydraulic elements that controlled overflows into the floodplain. Conversely, the peaks could be lowered by enlarging their capacity. These elements are located near the upstream limits of the Serpentine River and Mahood Creek lowlands where the dyke system is not well tied into the high ground of the uplands. Accurate topographic data was not available for these overflow areas, and it was not possible to confirm where the current ground configuration would have been representative of the 1968 and 1979 situations, since the dykes were frequently upgraded and modified in the lowlands. The calibration exercise did emphasize the critical role these overflow areas played in determining how much water spilled into the fields, and how much was contained in the Serpentine River channel.

Figure 21 presents the hydrographs for the Serpentine River at Fraser Highway, which is 6.2 km upstream, in channel distance, from Highway 10. After the initial peak, the simulated hydrograph mimicked the shape of the recorded hydrograph very well, but was about 0.1 m low throughout the latter two days of the simulation. Whether this small underestimation was due to underestimation of flow capacity in the aforementioned overflow areas or imperfections in the shape of the inflow hydrographs could not be determined. The difference was, however, relatively small.

A characteristic of the discharge hydrographs of Figure 8 for Hyland Creek and Erickson Creek was that the initial peak of the 1979 storm was modelled as being higher and earlier than the records show actually occurred. For Latimer Creek, the discharge was also modelled as rising too early. This appears to be reflected in the water level hydrographs in Figures 19 and 20. Therefore, the ONE-D model was not adjusted to remove this deviation, since its origin appeared to be in the hydrologic modelling.

Figure 22 compares the simulated water levels and simulated discharges for the Serpentine River at Fraser Highway gauge location through the 1979 flood. The complete lack of a relationship between water level and discharge underscores the need for a hydrodynamic modelling approach to analyze hydraulic behaviour in the lowlands.

A fifth hydrograph was recorded in the upper Nicomekl basin, well upstream of tidal influences. The calibration hydrographs are presented in Figure 23. They indicate an overestimation of the peak water level, but a substantial underestimation of water levels during the latter part of the recession limb of the hydrograph. This was most likely due to limitations in the hydrologic modelling and suspected errors in the recorded discharge data. The discharge hydrographs presented in Figure 9 for the 1979 event show an excessively rapid recession for the calculated hydrographs at several of the gauges. This is a common problem associated with the SCS method.

Calibration was less successful in the upper Nicomekl River basin, upstream of 192nd Street, partly because of limitations in the available calibration data. The only WSC gauge operating in the study reaches of the Upper Nicomekl was the Nicomekl River below Murray Creek gauge, and the 1979 flood data for this gauge appeared to contain significant errors (see discussion in Section 3.4).

During the latter phases of this floodplain mapping study, it was discovered that the City of Langley had measured high water levels at various locations in the City during the 1972, 1975, 1979 and 1986 flood events. It was not certain whether these were peak or near-peak water levels and some minor discrepancies indicated that the levels might not have always accurately represented the peak water levels. However, they were very useful in calibrating the model.

The Nicomekl River below Murray Creek gauge was discontinued in 1984 and the Nicomekl River at 203rd Street gauge was established soon after. The latter gauge appears to be providing more reliable data than its predecessor. The high water levels recorded by Langley indicated that the 1979 flood peak on the Nicomekl River was slightly higher than the 1986 flood peak. Using the 1986 flood peak data at the 203rd Street gauge and both 1979 and 1986 high water levels measured by Langley, the upper Nicomekl portion of the model was recalibrated. The final simulated water levels were similar to or higher than the measured 1986 and 1979 levels in all locations where such comparisons could be made, except one area near 200th Street. As a result of this anomaly, a higher freeboard was applied to the calculated 200-year water levels for this area prior to floodplain delineation.

Despite these imperfections, the end result of the modelling was that the 1979 flood was replicated very well over most of the study area, with the maximum peak water level at four of the five gauge locations estimated to within 0.05 m of the recorded peak, using Nicomekl below sea dam recorded high tides, and within 0.25 m using Crescent Beach tide data estimated from Point Atkinson Records.

1935 Flood Simulation

An attempt to simulate the 1935 flood event was made using the model which represented the configuration of roads and dykes prepared for the 1968 event. As discussed in Section 3.6, the hydrographs used in this simulation were based on a reconstructed January 1935 rainstorm, but a land use representative of the mid-1970s. The extensive effort required to research the 1935 road and dyke configurations and the 1935 land use was not warranted, since there were no precise high water marks. A few approximate high water levels were determined, but it was impossible to ascertain the time they were observed and whether or not the datum to which they were compared has moved significantly since 1935.

An attempt had been made in 1968 by the B. C. Ministry of Environment to determine the peak water level that occurred in the Cloverdale area during the 1935 flood (27), concluding that the peak flood elevation was probably less than 1.2 m. This was based on reports that the B. C. Electric Co. tracks at 168th Street were not covered by floodwaters. The lowest track elevation near 168th Street was surveyed in 1991 to be at elevation 1.33 m. However, a January 24, 1935 Vancouver Sun newspaper article (28) stated that:

> "The B. C. Electric Railway tracks at Cloverdale are under 20 inches of water."

Using the estimate of 1.2 m for the track elevation, this report would indicate a peak elevation of 1.71 m. However, if the lowest point surveyed along the tracks in 1991, which was measured 250 m east of the Serpentine River bridge at an elevation of 1.05 m, is used, then a peak water elevation of 1.56 m is indicated. Both of these peak water level estimates are higher than the B. C. Environment estimate of 1.2 m, maximum.

This contradiction was not resolved. One unknown factor is the magnitude of elevation change of the railway track since 1935. Some settlement and/or some addition of ballast material would not be unusual in the past 56 years, although it was reported that these tracks had not been raised or lowered between 1935 and 1968 (27). It is also possible that the newspaper article erred in the statement quoted above.

The ONE-D simulation of the 1935 event produced the following peak water level estimates:

Cell 5023	1.37 m
Cell 5010	1.20 m
Cell 5014	1.26 m
Cell 5015	1.19 m
Cell 5016	1.23 m
Cell 5017	0.41 m
Serpentine Reach 249	1.69 m

These simulated water levels compared reasonably well with the 1.2 m estimate by B. C. Environment. It would appear likely that the dyke and road configuration used in the model resulted in more independent cell water levels than actually occurred in 1935. With lower dyke and road crests, the modelled peak levels at the locations listed above would have been more similar to one another.

Given the uncertainty in the water level observations and the unrepresentative aspects of the model for the 1935 flood, the calculated results compared reasonably well with the observations.

200-Year Flood Model Development

The Serpentine and Nicomekl models calibrated for the 1979 flood provided the basis for the 200-year flood model. A linked model was created using the 1979 calibration parameters, changing only those parameters which represented physical features known to have changed between 1979 and 1991. The 1979 inflows were replaced with the 200-year flood inflow hydrographs, and the resulting input file was used to simulate the 200-year flood event. As a result of the recalibration of the 1979 event, the 200-year flood model was updated with the changes resulting from this recalibration and a second set of model runs was executed.

Another change added to the second set of runs was a reduction in the cross-sectional area of the sea dams to simulate the effect of partial ice blockage. Ice jamming at the sea dams did occur during the 1935 flood. For these simulations it was assumed that two of the seven barrels of each sea dam were completely blocked with ice. A sensitivity run indicated that, although a larger head difference resulted across the sea dams during the falling tide periods, the peak flood levels upstream of the sea dams were not significantly affected.

5.5 Development of the Sea Dyke Breach Model

As described in Section 5.3, a separate model was developed to simulate overtopping and breaching of the sea dykes. Starting with the linked model developed for the 200-year flood simulations, the detail in the Nicomekl River model upstream of 192nd Street was removed. Similarly, the detailed portion of the Serpentine model upstream of 168th Street was simplified, substituting the individual cells with a single large cell and removing upstream reaches and other details. The areas removed from the model were outside the influence of a large sea dyke breach flood.

Cells 5001 and 5002 were extended westward to accommodate the existence of a length of sea dyke which, when overtopped, could contribute to flooding in the study area. The overtopping elements in the model which represented roads, railroads and dykes were defined in more detail, because the flooding inland was very sensitive to crest elevations and lengths of these embankments. It was assumed that roads and railroads would not wash out during the overtopping period, but that earth dykes probably would. A typical overtopping sequence over an earth dyke was expected to occur as follows:

1. The embankment would retain water until overtopped by rising water levels on the ocean side.

2. The initial shallow overflows would not have sufficient depth and velocity to erode the vegetation mat on the earth dykes. The cell on the inland side would be near empty and starting to fill.

3. As the depth of overtopping increases, the earth dykes would begin to erode. Once the surface vegetation is removed the rate of erosion would increase very rapidly, until the water levels on the downstream side of the dyke approaches the upstream water level. The decreased head difference would reduce discharge and velocity, slowing the rate of erosion or halting it altogether.

4. On the falling tide, flow would occur in the opposite direction through the breached dyke, draining the inland cell toward the sea. Velocities would be lower than they were during the initial overtopping. Enlargement of the breach would occur very slowly, if at all.

Dyke breaches were modelled as though they occurred suddenly and simultaneously at all locations in the system. They were introduced in the model at a time when the Boundary Bay water levels were just beginning to exceed the crest elevation of the sea dykes. Therefore the effect of these instantaneous breaches may have been to provide slightly conservative results compared to what might occur in reality, since a real breach could take some time to attain its near-ultimate dimensions. On the other hand, sudden dyke failures have occurred in the past and piping failures can occur before the rising flood level reaches the crest of the dyke.

One important estimate for which applicable data was very scare was the ultimate size of a dyke breach resulting from overtopping by the extreme water levels estimated for Boundary Bay. The sea dyke breaches and overtoppings which have occurred in the Mud Bay and Colebrook Dyking Districts were not measured. Recollections were sketchy and no photographs of these breaches could be found.

Hay & Company completed a floodplain management study for Richmond (29) in which they investigated dyke breaches in order to estimate a breach length for flood level estimation in Richmond. Based on descriptions of dyke failures which occurred during the 1948 flood on the Fraser River, they reported that breach widths varied from a few metres to 300 m, and that breaches widened very quickly to approximately half their ultimate width in a few minutes. On the basis of this information Hay & Company adopted a 300 m breach width for their flood study.

Severe overtopping conditions were indicated by the extreme water level estimates for Boundary Bay, with peak ocean levels exceeding the existing dyke crest elevation by 0.39 m. Such high peak water levels and the rapid water level changes associated with tides, combined with the low standard to which the dykes were initially built, would result in severe dyke breach conditions. Therefore large breaches would be expected to form under this scenario. In the absence of any more applicable precedents, the dyke breach width of 300 m observed on the Fraser River during the 1948 flood was adjusted for this study. Initially, this total width was divided into three breaches of 100 m each, occurring at the following locations:

- along the north shore of Mud Bay into Cell 5002
- along the south side of the Serpentine River near its mouth into Cell 5005
- along the east shore of Mud Bay at the low point of the profile of the Great Northern Railway into Cell 5005.

The extreme Boundary Bay water level hydrograph developed for the north shore of Mud Bay was the boundary condition applied to the first two dyke breach locations listed above, and the extreme tide hydrograph developed for Crescent Beach became the boundary condition for the third location. A consequence of this configuration of breaches and cells and the lack of a wind shear component in the ONE-D model was a circulatory flow through Cell 5005 once it had become filled in the model. Due to the higher water levels of the north breach into Cell 5005, flows entering at this location flowed through the cell and drained back to Boundary Bay through the more southerly breach. Although it is conceivable that a similar situation might occur in reality, the model clearly overestimated the magnitude of this circulatory flow. However, the net effect of this flow on flood levels inland was deemed to be negligible since it only occurred when Cell 5005 was completely filled and the total flow capacity between Cell 5005 and Boundary Bay was much greater than the available capacity over Highway 99, over which all eastbound floodwaters would have to pass.

Towards the end of the study a series of model runs were completed which tested the effect of two breaches into Cell 5005 that were both located near the mouth of the Serpentine River. For these simulations, the circulatory flow did not occur.

To complete the model, inflow hydrographs representing upland runoff were required. The selection of runoff event was not expected to have a large impact on the final flood levels, because the discharges associated with sea dyke overtopping and breaching were estimated to be much greater than the largest flood runoff discharges. However, prefilling cells with runoff floodwaters would affect the peak water level estimates toward the inland extent of the seawater flood.

A review of precipitation amounts associated with major windstorms in the vicinity of the study area revealed that rain almost always fell in conjunction with a major windstorm, however, the amount of rain did not appear to be dependent upon the duration of peak velocities of the wind. Conversely, major rainstorms such as the 1935, 1968 and 1979 events, did not generate exceptional winds. Although both heavy rain and strong winds are generated by deep low pressure systems, the data did not indicate any correlation in the severity of both factors.

In the absence of better knowledge relating to the joint probability of extreme rainfall amounts and extreme winds, it was decided to combine the 200-year extreme water level in Boundary Bay with a 10-year rain event. The 1979 flood was caused by rainfalls which approximated a 10-year return period event. Therefore, the inflow hydrographs generated for the 1979 calibrations were used in the simulation of flooding from the sea. For all of the simulations, these two events were sequenced such that the first sea dyke overtopping occurred approximately 24 hours after the initial runoff peak occurred in the inflows.

Initial runs of the model tested the sea dyke overtopping case only, then sea dyke breaches were introduced to the model. It was found that the overtopping elements used for road and dyke overflows were not applicable for calculating the flow through a major sea dyke breach. Instead, each breach was modelled using a series of three reaches to simulate the contracting flow approaching the breach, the flow through the breach and the expanding flow departing the breach.

The model was run with the breach reaches rendered ineffective for the first 52 hrs. of the 1979 runoff event. At the end of this period the model run was halted, with the water level along the north shore of Mud Bay about to exceed the sea dyke crest elevations of 3.0 m. The breach reaches were made to take effect, the time step was reduced from 7.5 minutes to 1 minute, and the run was restarted. In this manner six scenarios were simulated in the first set of final runs and four scenarios were modelled in the second set, as described in the following section.

5.6 Final Model Runs

In the first set of runs the 200-year upland runoff flood model was used for two key runs to determine maximum flood elevations in the study area. The first run, numbered 601, was the simulation of the 200-year flood model developed as described in Section 5.4. Run 601 did not simulate any river dyke breaches. Run 602 was identical to 601, except that a major breach of the south Nicomekl dyke was introduced downstream of 184th Street to determine whether such a breach would result in flood levels greater than those caused by flooding from the sea. More detailed descriptions of these runs and their results are presented in Appendix 2.

The second set of runs, which is described in Table 12, included the changes resulting from the 1979 flood recalibration and reduced sea dam capacity. The base run of this set, which had no river dyke breaches in it, was numbered 604. Three subsequent runs included various river dyke breaches to examine the effect such breaches would have on flood levels in the adjacent floodplain cells.

In the upper Nicomekl River, upper Serpentine River and tributary reaches above the lowlands, the results of Run 604 represented the 200-year flood profiles. These profiles are illustrated in Figures 24 and 25. Flood levels along these reaches are above the range of possible influence from a flood from the sea, and would also not be affected by dyke breaches in the lowlands. Therefore, for these reaches, Run 604 provided the governing definition of 200-year flood levels, lacking only the addition of freeboard.

In the lowlands, the determination of maximum flood elevation was much more complex. A vast number of possible dyke breach combinations could occur with the runoff flood event or with the sea flood event. By assessing the results of some of the initial model runs, it was possible to recognize that breaches in certain locations would result in upper bound flood levels in specific groups of cells in the lowlands. The models were modified to simulate these combinations, and the results of these simulations were assessed, often leading to subsequent revisions and additional runs.

The first set of sea flood simulations was based on the initial estimates for extreme water levels in Boundary Bay. The second set used the later estimate of the extreme water levels, which were substantially lower. In the first set, six scenarios of flooding from the sea were simulated. These are described in Appendix 2. For the second set of sea flood simulations, four river dyke breach cases were modelled. These are described in Table 12.

The instantaneous peak water levels in the lowland storage cells calculated by each of the runs in the second set are listed in Table 13. The calculated peak water levels at selected locations in the lowland river channels are presented in Table 14. The same information for the first set of runs is presented in Appendix 2.

From the results presented in these tables, the following conclusions were drawn:

1. The combination of the extreme water levels in Boundary Bay and major sea dyke breaches would cause higher flood levels than a 200-year runoff event would in the following areas:

   - all of the lowlands on the right bank of the Serpentine River that lie on the downstream side of Mahood Creek
   - all of the lowlands between the Serpentine and Nicomekl Rivers west of 184th Street.

   In all other lowland and upland areas, the 200-year runoff event would cause the higher flood level.

2. In such a flood event, the peak levels would be highest near the ocean and would decrease inland. The greatest peak water level differences would occur across the King George Highway south of the Serpentine River, across 152nd Street north of the Serpentine River and across the river dykes between these two roads.

3. In the event of two consecutive extreme high tides, river dyke breaches can, in some cases, cause lower, rather than higher, peak water levels in the adjacent storage cell(s) because the breach would allow more water to drain from the cells during the low-tide period between the two peaks.

Table 12.
Summary Descriptions of the Second Set of Final Model Runs

Run Number	Description
200-Year Runoff Flood Event Scenarios	
604	• simulation of the entire system within the study area • 200-year flood inflow hydrographs (5-day duration) • tidal data for period 17 - 21 January, 1968 • no dyke breaches • revisions to floodplain conveyance assumptions in Upper Nicomekl area included
605	• same as 604, but with a 60-m wide river dyke breach on the north side of Nicomekl River, a short distance downstream from 184th Street for entire 5-day duration • objective was to maximize flood levels north of Nicomekl River near the east end of the lowlands.
606	• same as 605, but with a second 60 m dyke breach, this one on the east side of the Serpentine River upstream of Highway 10 • objective was to maximize flood levels in the Cloverdale area
607	• same os 606, except that the b reach on the north side of the Nicomekl River was moved upstream of 184th Street and reduced to 30 m in width.
Sea Dyke Breach Event Scenarios	
385	• simulation of the lowland area channels and storage cells excluding the northeast Serpentine lowlands that are beyond the range of influence of the sea floods • tidal data containing the final estimates of extreme Boundary Bay water levels • 1979 flood inflows into the lowland area • Three 100-m wide breaches of the sea dykes located at: - the south side of Cell 5002 - the northwest corner of Cell 5005 - the southwest corner of Cell 5005 • no river dyke breaches
386	• same as 385, but with 60-m wide river dyke breaches added to both sides of Mahood Creek • objective was to maximize flood levels in Cell 5029
390	• same as 385, but with the 100-m sea dyke breach at the southwest corner of Cell 5005 moved northward to simulate two breaches near the northwest corner of Cell 5005 • objective was to maximize flood levels between the Serpentine and Nicomekl Rivers
392	• same as 390, but with the following river dyke breaches added: - a 30-m breach on the north side of the Nicomekl River upstream of 184th Street - a 60-m breach on the north side of the Nicomekl River downstream of 184th Street - two pairs of 60-m breaches across the Serpentine River downstream from Highway 10 (from Cell 5010 to Cell 5015 and from Cell 5011 to Cell 5012) • objective was to maximize flood levels south of Cloverdale

Table 13

Summary of Simulated Peak Water Levels in Lowland Cells for Eight Flood Scenarios

Lowland Cell Number	200-Year Flood Event				Sea Flood and Sea Dyke Breach Event			
	Run 604	Run 605	Run 606	Run 607	Run 385	Run 386	Run 390	Run 392
5001	0.97	0.97	0.97	0.97	3.23	3.23	3.23	3.23
5002	0.70	0.70	0.70	0.70	3.24	3.24	3.24	3.24
5003	0.65	0.65	0.65	0.65	3.31	3.31	3.32	3.32
5004	1.08	1.08	1.08	1.08	2.89	2.89	3.01	3.01
5005	1.06	1.06	1.06	1.06	3.06	3.06	3.23	3.23
5006	1.07	1.06	1.06	1.07	0.59	0.59	1.96	1.96
5007	0.99	0.99	0.99	0.99	2.83	2.83	2.83	2.82
5008	0.82	0.83	0.84	0.83	2.82	2.82	2.82	2.80
5009	0.90	0.93	1.03	0.91	1.98	1.98	2.13	2.13
5010	1.33	1.35	1.23	1.23	2.28	2.31	2.31	2.02
5011	1.09	1.11	1.08	1.06	2.30	2.30	2.30	2.00
5012	0.72	1.44	1.52	1.05	1.63	1.63	1.98	2.01
5013	1.23	1.00	1.02	1.16	0.41	0.41	0.42	0.39
5014	1.15	1.43	1.70	1.75	1.24	1.24	1.87	1.99
5015	0.84	1.41	1.74	1.75	1.17	1.18	1.87	2.00
5016	1.28	1.44	1.77	1.81	1.24	1.24	1.87	2.00
5017	0.32	1.40	1.72	1.74	0.10	-0.10	1.87	1.99
5018	0.24	1.44	1.53	1.05	1.50	1.50	1.96	2.00
5019	1.23	1.00	1.02	1.16	0.30	0.30	0.31	0.30
5020	2.18*	2.18*	2.18*	1.94	1.50	1.49	1.87	2.00
5021	1.27	1.84	1.84	1.51	1.50	1.50	1.96	1.99
5022	1.74	1.53	1.53	1.67	0.81	0.81	0.81	0.84
5023	1.34	1.37	1.24	1.24	2.28	2.28	2.29	2.01
5024	1.67	1.67	1.61	1.61	2.23	2.23	2.23	1.99
5025	2.00	2.00	2.00	2.00	2.19	1.85	2.20	1.53
5026	0.89	0.89	0.89	0.89	[2.15]	[2.12]	[2.16]	[1.99]

Table 13

Summary of Simulated Peak Water Levels in
Lowland Cells for Eight Flood Scenarios

Lowland Cell Number	200-Year Flood Event				Sea Flood and Sea Dyke Breach Event			
	Run 604	Run 605	Run 606	Run 607	Run 385	Run 386	Run 390	Run 392
5027	0.46	0.46	0.46	0.46	[2.14]	[2.11]	[2.15]	[1.98]
5028	1.61	1.61	1.61	1.61	[2.12]	[2.09]	[2.13]	[1.98]
5029	1.81	1.82	1.80	1.80	0.55	1.65	0.55	1.54
5030	1.81	1.81	1.76	1.79	(NS)	(NS)	(NS)	(NS)
5031	0.36	0.36	0.36	0.36	(NS)	(NS)	(NS)	(NS)
5032	1.97	1.96	1.94	1.97	(NS)	(NS)	(NS)	(NS)
5033	1.98	1.98	1.97	2.00	(NS)	(NS)	(NS)	(NS)
5034	1.84	1.84	1.80	1.84	(NS)	(NS)	(NS)	(NS)
5035	1.83	1.84	1.80	1.83	(NS)	(NS)	(NS)	(NS)
5036	0.44	0.44	1.43	0.44	(NS)	(NS)	(NS)	(NS)
5037	1.99	1.99	1.96	1.97	(NS)	(NS)	(NS)	(NS)
5038	2.02	2.03	2.01	2.01	(NS)	(NS)	(NS)	(NS)
5039	2.74	2.74	2.74	2.74	(NS)	(NS)	(NS)	(NS)
5040	1.23	1.01	1.02	1.16	0.42	0.42	0.42	0.40
5041	1.23	1.02	1.03	1.16	0.31	0.31	0.31	0.30
5042	1.85	1.85	1.85	1.85	0.81	0.81	0.81	0.84
5043	2.05	2.03	2.03	2.15	(NS)	(NS)	(NS)	(NS)
5044	2.52	0.79	0.79	3.37*	0.35	0.35	0.35	2.01
5045	3.51*	3.34	3.34	3.46	1.05	1.05	1.05	1.03

* These numbers were superseded by results from more detailed modelling or manual adjustments to mitigate the effects of model assumptions and simplifications.

[ ] Numbers in square brackets are based on the assumption that the cell is filled to the adjacent river level through a dyke breach.

(NS) Not Significant - these cells in the northeast Serpentine lowlands were not modelled in detail because they were beyond the upstream limit of significant influence of the sea flood event and the peak levels from the 200-year runoff event will govern here.

Table 14

Summary of Simulated Peak Water Levels
in Lowland River Channels for Eight Flood Scenarios

Lowland River Location	200-Year Runoff Flood Event				Sea Flood and Dyke Breach Event			
	Run 604	Run 605	Run 606	Run 607	Run 385	Run 386	Run 390	Run 392
Serpentine River								
A	1.97	1.97	1.90	1.91	2.07	2.04	2.08	1.97
B	1.97	1.97	1.78	1.78	2.15	2.13	2.16	1.99
C	1.97	1.98	1.79	1.79	2.14	2.13	2.15	2.01
D	1.97	1.97	1.79	1.79	2.30	2.30	2.31	2.10
E	1.96	1.96	1.96	1.96	3.28	3.28	3.29	3.29
Nicomekl River								
F	2.82	2.29	2.29	2.85	2.53	2.53	2.53	2.00
G	2.26	2.00	2.00	2.23	2.46	2.46	2.46	1.99
H	2.15	1.99	1.99	2.13	2.45	2.45	2.45	1.99
I	2.09	1.98	1.98	2.08	2.45	2.45	2.45	1.99
J	1.97	1.97	1.97	1.97	3.01	3.01	3.11	3.11

Description of River Locations

Serpentine River:

A Upstream side of Fraser Highway Bridge
B Upstream side of Highway 10 Bridge
C Upstream side of 152nd Street Bridge
D Upstream side of King George Highway Bridge
E Upstream side of Highway 99 Bridge

Nicomekl River:

F Upstream side of 184th Street Bridge
G Upstream side of 168th Street Bridge
H Upstream side of 40th Avenue Bridge
I Upstream side of King George Highway Bridge
J Midway between King George Highway and the Mouth.

These locations are also shown on Figure 17.

4. West of the King George Highway, peak flood levels would be virtually unaffected by river dyke breaches inland.

5. In the lowlands north and west of the Serpentine River and south of Mahood Creek, the model results indicate that the highest peak water levels would occur if no river dyke breaches were to occur in the lowlands.

6. In the central lowlands between the rivers and south of Cloverdale, the governing peak flood level would occur from a sea dyke breach combined with several major river dyke breaches.

7. South of the Nicomekl River the governing peak water level condition would occur from a 200-year runoff event combined with a major breach of the south river dyke. The peak levels for this area were derived from the model runs presented in Appendix 2, which remained valid for this area.

8. The lowest peak flood levels would occur in the northeast Serpentine lowlands. The governing peak water level condition for this area would occur from a 200-year runoff flood with or without river dyke breaches, since the dyke system in this area is not contiguous to high ground.

5.7 Sensitivity Analyses

Sensitivity analyses were undertaken to quantify the impact of changes to certain input parameters and assumptions. Unlike a conventional steady-state backwater analysis, the hydrodynamic model of the complex Serpentine-Nicomekl system contains a vast number of degrees of freedom and, therefore, a similar number of possibilities for sensitivity testing. In this study, only a few model runs were made that were specifically designed to test the sensitivity of a single particular change. In effect, the model development and calibration runs revealed the sensitivity of many parameter changes to the analysts and the knowledge gained was reflected in the subjective judgements made by them.

Sensitivity analyses can be used to indicate an appropriate freeboard quantity, or to test the effect of a proposed physical change to the system. Three aspects of the modelling for which the sensitivities were independently quantified were dyke breaches, Boundary Bay flood levels and road grade changes.

Dyke Breaches

Three pairs of model runs described in Table 12 demonstrated the sensitivity of water levels to three different river dyke breach scenarios. These pairs of runs and the associated breaches were:

Runs 604 and 605	-	a 60-m long breach on the north Nicomekl dyke during the 200-year runoff event
Runs 605 and 606	-	an additional 60-m long breach on the south Serpentine dyke during the 200-year runoff flood event
Runs 385 and 386	-	two 60-m long breaches on both sides of Mahood Creek during the ocean flood event.

The impacts of these breaches on peak water levels in the storage cells are listed in Table 13, and changes in the river levels are presented in Table 14. In the first breach scenario (Runs 604 and 605), the 60-m long breach caused peak water levels to increase in the cells north of the Nicomekl River by 0.15 to 1.20 m. In the Nicomekl River channel, the peak level decreased by 0.70 m near the breach, and by smaller amounts downstream. The largest magnitude change was the calculated decrease in the peak water level in Cell 5044, on the north side of the Nicomekl River, by 1.73 m. This is a secondary result of the breach, which lowered the peak river water levels, thus reducing the opportunity for water to overtop the dyke adjacent to Cell 5044. Cells opposite the breach on the south side (Cells 5022 and 5019) also showed decreased peak levels for the breach scenario. Peak water levels elsewhere in the system were essentially unchanged.

The addition of a second breach into this area (Run 606), this one from the Serpentine River, had the effect of raising peak water levels near Cloverdale approximately 0.3 m higher than the maximum levels resulting from the single Nicomekl River breach modelled in Run 605. More distant cells in the central area between the rivers, such as 5012 and 5009, had their peaks increased by less than 0.1 m. Distant cells adjacent to the Serpentine River showed lower peak levels in Run 607 due to the general drawdown of the peak Serpentine River water levels caused by the additional breach.

Another simulated breach scenario, which featured the double breach of the Mahood Creek dykes during a flood event from the sea (Runs 385 and 386) provided the expected results of lowering peak water levels in Cell 5025 by 0.35 m, and raising the peak level in Cell 5029 by 1.11 m. Water levels in other parts of the lowlands remained essentially unaffected.

In the first set of final model runs, reported in Appendix 2, some unexpected and interesting consequences of dyke breaches were revealed by the modelling. For this first set of runs, the extreme sea flood levels were higher than those used in the second set, and some of the consequences associated with the higher Boundary Bay flood levels were not as apparent in the results from the lower sea flood level simulations.

Two of these unexpected consequences were:

- lower rather than higher peak levels in the cells between the two rivers when river dyke breaches were introduced.

  This occurred because the river dyke breaches allowed the cells to drain during the low-tide period between the two extreme high-tide peaks. Without the river dyke breaches, water which travelled inland by overtopping roads and dykes, partly filled these cells and became trapped behind the dykes during the low-tide period, and the second peak brought additional water into these cells.

- higher peak water levels in the northeast Serpentine lowlands when Serpentine River dyke breaches were introduced.

  These breaches provided access for ponded water to travel up the river when the sea dams were closed, as the northeast cells were not filled as high as the cells downstream. During the falling tide when the sea dam gates were open, outflow from these breaches reduced the river channel's capacity to accommodate outflow from the northeast cells.

The variation in magnitude and direction of water level changes resulting from the introduction of dyke breaches into the model reflects the complex response of the model to apparently simple changes.

Variation in Peak Boundary Bay Water Levels

Comparison of the results from equivalent scenarios from both sets of final model runs which used different extreme Boundary Bay water levels for the sea floods revealed the sensitivity of peak water levels in the cells to the peak levels in Boundary Bay. The scenario selected for this sensitivity analysis was the one which involved no river dyke breaches. There were three runs which satisfied this condition. Two of the simulations (Runs 375 and 378) were part of the first set of final model runs. The third one (Run 385) was part of the second set.

The extreme Boundary Bay water levels used for each of these runs are listed in Table 15 below:

Table 15

Peak Sea Flood Levels Used in Sensitivity Tests

Location	Peak Sea Flood Level (m Geodetic)		
	Run 375	Run 378	Run 385
Along North Shore of Mud Bay	3.74	3.54	3.39
At Crescent Beach	3.38	3.18	2.98

For each of these runs, the time series data representing the storm surge water levels was scaled proportionately and added to the same astronomic tide data set, such that the peak level in the summation matched the corresponding peak listed in Table 15 above.

The effects of the different Boundary Bay flood peaks on the peak water levels in the lowlands can be seen by comparing the values listed in Table 16. As expected, peak levels in the cells near Mud Bay, such as Cell 5003, exhibited water level changes which corresponded closely to the changes in the sea flood levels. However, the effect of these changes were magnified in most of the cells east of King George Highway between the two rivers, such as Cells 5012 and 5018. Cells south of the Nicomekl, such as Cell 5019, were largely unaffected by flooding from the sea, so their peak levels remained low and generally insensitive to the peak sea flood levels.

The sensitivity test results form a rational picture of the sea flood behaviour in the lowlands, as follows:

- The cells near Mud Bay fill entirely with water under all extreme sea level and breach scenarios, so their peak levels are directly governed by the peak sea levels.

- The cells that are distant from the sea and isolated by several sufficiently-high barriers, are not affected by any of the extreme sea level and breach scenarios.

- The peak flood levels in the cells between the above two areas are most sensitive to the magnitude of the extreme Boundary Bay flood level since they may be largely unaffected by a lower extreme sea level, partly filled by a higher peak sea level, and completely filled by the highest estimates of the extreme sea level. It is noteworthy that the higher extreme sea levels would naturally correspond to a longer duration of overtopping, as well as higher discharges over the dykes and through the sea dyke breaches.

Table 16
Sensitivity Test Results for Variation in Peak Sea Flood Levels

Lowland Cell Number	Peak Water Level Run 375	Peak Water Level Run 378	Peak Water Level Run 385
5001	3.60	3.38	3.23
5002	3.59	3.39	3.24
5003	3.68	3.48	3.31
5004	3.25	3.05	2.89
5005	3.44	3.23	3.06
5006	3.44	3.20	0.59
5007	3.15	2.99	2.83
5008	3.14	2.98	2.82
5009	2.67	2.31	1.98
5010	2.73	2.60	2.28
5011	2.76	2.61	2.30
5012	2.63	2.04	1.63
5013	0.70	0.47	0.41
5014	2.62	2.04	1.24
5015	2.62	2.04	1.17
5016	2.62	2.04	1.24
5017	2.63	2.04	0.10
5018	2.63	2.04	1.50
5019	0.32	0.31	0.30
5021	2.63	2.04	1.50
5022	0.94	0.83	0.81

Lowland Cell Number	Peak Water Level Run 375	Peak Water Level Run 378	Peak Water Level Run 385
5023	2.74	2.62	2.28
5024	2.70	2.56	2.23
5025	2.62	2.54	2.19
5029	2.02	1.68	0.55
5040	0.68	0.47	0.42
5041	0.32	0.31	0.31
5042	0.94	0.83	0.81
5044	2.63	0.35	0.35
5045	1.23	1.13	1.05
Northeast Serpentine Cells	1.22	0.78	-0.46

Lowland River Location*	Peak Water Level Run 375	Peak Water Level Run 378	Peak Water Level Run 385
Serpentine River			
A	2.54	2.41	2.08
B	2.66	2.55	2.15
C	2.90	2.79	2.14
D	3.05	2.87	2.30
E	3.60	3.42	3.28
Nicomekl River			
F	2.57	2.54	2.53
G	2.55	2.46	2.46
H	2.55	2.45	2.45
I	2.58	2.46	2.45
J	3.43	3.20	3.01

* these locations shown on Figure 17

Road Grade Changes

It became apparent during the calibration process that very large differences in peak water levels could result from relatively small changes in road grade. For example, Cell 5036, located east of 176th Street in the northeast Serpentine lowlands, was estimated to have filled with water to an elevation between 0.8 m and 1.1 m during the 1968 flood. Since 1968, this part of 176th Street has been raised and the model has been changed to reflect this. The 200-year flood, much larger than the 1968 event, was estimated to fill Cell 5036 to an elevation of only 0.44 m. This is almost entirely due to the raising of 176th Street.

An investigation related to this study examined the effect that raising the grade of two roads in Surrey would have on flood levels upstream. The two roads were 192nd Street and 184th Street near their crossings of the Nicomekl River. The grade of 192nd Street was raised a maximum of 0.66 m in 1993. The grade change on 184th Street, which was a proposal undergoing review at the time of writing, pertained to the south side of the river only and featured raising the low point of the road by 0.42 m.

The impacts of raising these roads was estimated by creating detailed submodels of two ONE-D models developed for this project. The submodels covered the portion of the Nicomekl River between 176th Street and 200th Street. One submodel simulated the 200-year runoff flood, and the other attempted to replicate the 1979 flood.

The modelling results indicated that both road grade changes would cause a very small increase in the peak water levels upstream of them during a 200-year flood event. During such a large flood, the model estimated that 192nd Street after the grade change, would be inundated by more than 1.1 m of water during the peak and would be less than 0.01 m higher upstream as a result of the raised road. At 184th Street during the 200-year flood a near-zero impact was predicted by the model even though the maximum road inundation was less than 0.7 m.

Although the ONE-D model produces numerical results to the nearest millimetre, the actual precision of the model is much coarser. It is the opinion of the analysts using the ONE-D model that the predicted water level *increases* in this application of the model should be regarded as falling within a minimum confidence interval of plus or minus 0.02 m. Therefore, it was concluded that the road grade change on 192nd Street would cause water level increases of less than 0.03 m upstream, and upstream of 184th Street, less than 0.02 m.

The model indicated a larger impact of the raising of 192nd Street under a 1979 flood scenario. An upstream water level increase of 0.035 m was calculated, therefore it was concluded that the raising of 192nd Street would cause an increase in the 1979 flood level of less than 0.055 m. At 184th Street, the model predicted a near-zero increase again, therefore an increase of less than 0.02 m was concluded.

Although the impacts of raising these two roads are relatively minor, larger road grade changes here or changes in other locations in the Serpentine-Nicomekl lowlands could have significant impacts on flood levels. This study has confirmed the complexity and interrelationship of embankment barriers, such as roads, railroads and dykes, and the peak flood levels resulting from a major runoff or sea flood event.

General Observations

Three general observations pertaining to the relative sensitivity of peak water levels among the lowland cells were that:

1. the sensitivity of a partly-filled cell is approximately inversely proportional to its size.

2. the sensitivity is greater when a cell is partly filled with water than when it is completely filled.

3. flood levels in the rivers and adjacent cells are sensitive to inundation of large cells or groups of cells elsewhere along the river system.

The reason for the former trend is that an impoundment with a larger plan area requires a smaller water level difference to achieve a given volume of discharge. The discharge to and from a cell are typically a function of the length of embankment overtopped, which is approximately proportional to cell length. The plan area is usually a function of the square of the cell length, therefore large cells would respond with smaller water level changes.

When a partly-filled cell becomes "full" additional inflow is discharged to neighbouring cells which, when filled, discharge to other cells. Thus the rate of change of water level in the first cell would decrease abruptly when it initially becomes full. Once an entire group of cells is filled and the embankments between them become substantially inundated, the group of cells behaves as a single, very large cell.

Dyke breaches or other opportunities that allow flood waters to be stored in the cells during a flood event act to reduce peak flows and peak water levels elsewhere in the system. For example, if the Serpentine River dykes were raised and tied into high ground at all its upstream boundaries, the peak flood discharges and water levels along the Serpentine River would be increased to significantly greater values than those estimated for this study. If a major river dyke breach were to occur elsewhere in the system after such dyke improvements, the resulting 200-year flood levels could be considerably higher in the cells near the breach than the levels shown on the floodplain maps produced for this study.

5.8 <u>Freeboard</u>

It has been customary in floodplain delineation studies under the Floodplain Mapping Agreement, to draw the floodplain limits at elevations which are higher than the computed peak water levels. This difference, referred to as "freeboard," is included as a safety factor against possible underestimation of the flood levels due to uncertainties contained in the data, the mapping, the assumptions and the statistical analyses. The standard freeboard applied in most studies under the Agreement which used steady-state backwater analyses was the greater of 0.60 m above the mean daily maximum water level, or 0.30 m above the instantaneous peak water level. Frequently these two sums have been found to be similar to one another.

In this study, peak instantaneous water levels were computed for all the flood scenarios. Mean daily values were not meaningful for events such as flooding from the sea. However, the 200-year runoff event in the upstream reaches above the lowlands is a situation similar to that faced in a conventional floodplain mapping study. In order to determine whether mean daily peak water levels with 0.60 m of freeboard would exceed instantaneous peaks with 0.30 m of freeboard, the mean daily water levels for each of the five days of the 200-year flood event were calculated at five representative upstream locations. Table 17 lists the instantaneous and daily peaks calculated by Run 606 for the five locations.

Table 17

Instantaneous and Daily Peak Water Levels at Upland Locations

Location	Instantaneous Peak Water Level (m)	Daily Peak Water Level (m)	Difference (m)
Serpentine R. Reach 20	7.11	6.07	1.04
Nicomekl R. Reach 1000	14.32	13.87	0.45
Nicomekl R. Reach 1021	9.08	8.84	0.24
Nicomekl R. Reach 1080	7.83	7.50	0.33
Nicomekl R. Reach 1140	4.73	4.24	0.49

For those locations where the difference listed in Table 17 is greater than 0.30 m, the combination of instantaneous peak plus 0.30 m of freeboard would govern. Where it is less, the combination of daily plus 0.60 m would govern. At the one location listed above where the difference is less than 0.30 m, it is less by a small amount (0.06 m). Therefore it appears that the safety margin would not be serious compromised if, for this study, the upland channels were to be mapped using only the one criterion of instantaneous peak water level plus 0.30 m of freeboard. For simplicity and economy, the single criterion of 0.30 m of freeboard added to the instantaneous peak water level was adopted for all upland channels, with the exception of the reach of the Nicomekl River between the mouth of Anderson Creek and 203rd Street, where a freeboard of 0.50 m was added. During the recalibration of the Nicomekl River for the 1979 flood, the model consistently underpredicted the observed water level in this area.

For the lowlands, for which flood levels were determined using simulations of two flooding mechanisms and a number of scenarios, it was difficult to quantify an appropriate freeboard. Although the joint probability of occurrence of the selected scenarios might appear to be more rare than the 1 in 200 probability used for the runoff flood determination, some factors which could increase the peak flood levels, were not included in the simulations. Some of these are:

- dyke breaches may actually be larger than those assumed for this study
- sea dam openings may become plugged with debris in a major flood
- no allowance was made for waves or wind setup that may occur in the inundated cells by the same storm that creates the storm surge and Boundary Bay wind setup.

There is, however, comfort in the knowledge that when the peak levels estimated herein are achieved in any part of the lowlands, it would take a very large volume of water to increase the levels significantly higher, due to the aforementioned insensitivity of full cells or groups of full cells to further increases in water levels.

In consideration of the above factors, a freeboard varying from 0.30 m to 0.40 m was considered reasonable for the lowland cells.

5.9 Final Flood Level Determinations

The final flood level, with freeboard, was calculated for the upland channel reaches by adding the freeboard to the peak water levels calculated by Run 604, which simulated the 200-year runoff event with no dyke breaches in the system. The peak flood levels were corrected, where necessary, to remove the effect of oscillations in water level over time caused by small scale numerical instabilities in the model. These levels were transferred to the maps by plotting them on the 1:5000 scale maps at the corresponding mesh point locations along the river channels. The floodplain limits were delineated by locating the ground elevations which corresponded to the computed peak water levels plus freeboard.

In the lowlands, peak water levels plus freeboard were determined for most cells by rounding up to the nearest 0.1 m the calculated flood level, then adding 0.3 m freeboard. Cells which did not have specific flood levels calculated by the modelling of flooding from the sea, were assigned flood levels by interpolation using adjacent cell flood levels while recognizing the relative elevations of the embankments between them. It was assumed that small cells adjacent to either river could be filled to the peak river level if the dyke were to fail. In a few cases, as indicated on Table 13, manual adjustments were made to the calculated flood levels to compensate for the effect of model simplifications and assumptions. A summary of the lowland flood levels is presented in Figure 26. Peak water levels for the cells south of the Nicomekl River were taken from Run 602 (Appendix A) as the results of this run were deemed valid and showed the effect of a river dyke breach into this area.

Users of the Floodplain Maps should be warned that the position of the floodplain limit lines are based on map contours and spot heights in most areas. Due to the map accuracy, the position of the intersection of the indicated flood level and the ground may not coincide exactly with the position of the floodplain limit on the Maps. In areas near the edge of the floodplain, the exact position of the floodplain limit on the ground should be determined by level surveys tied to Geodetic bench marks. In some developed areas, such as Langley and Cloverdale, some field verification of the location of the floodplain limits shown on the Maps was carried out using level surveys tied to local spot heights.

6.0 RECOMMENDATIONS

On the basis of the findings of this study we recommend the following:

1. That the Floodplain Maps produced by this study, numbered 91-5-1 through 91-5-14, be designated under the terms of the Canada - British Columbia Floodplain Mapping Agreement.

2. That these Floodplain Maps be reviewed and updated as required on the basis of significant future flood data or information relating to major physical changes to the floodplain.

3. That Water Survey of Canada review the published flood peak data for the Nicomekl River below Murray Creek gauge for the entire period of record in view of the apparent anomalies identified in this study.

4. That a long-term tide gauge be established and maintained in Mud Bay to allow improved estimation of peak sea flood levels in the future.

5. That the District of Surrey, City of Langley, Township of Langley and the B. C. Ministry of Transportation and Highways be warned that road profile changes within the Serpentine-Nicomekl floodplain area may have significant impacts on flood levels.

6. That any proposed road, railroad or dyke profile changes within the lowland area be analyzed to assess its impact on water levels using a hydrodynamic model, unless it can be clearly shown that such an approach would not be necessary prior to approval of such profile changes.

7. That, in view of the finding that much of the Serpentine-Nicomekl lowlands are subject to a major flood threat from the sea which could result in very rapid inundation of inhabited land, the authorities having jurisdiction for emergency response in this area be notified of these findings, so that measures can be taken and response plans prepared to prevent or minimize loss of life.

8. That, at such time when reliable estimates become available of the characteristics of tsunami waves in Boundary Bay originating within the Strait of Georgia, the impact of such waves on the Serpentine-Nicomekl lowlands be analyzed if their amplitude suggests higher flood levels than those determined by this study.

9. That consideration be given to examining the major floodproofing benefits that would be realized by raising and upgrading the quality of the sea dykes protecting the Serpentine-Nicomekl lowlands.

10. That the ONE-D hydrodynamic model be upgraded to facilitate much more efficient data entry and quick access to graphical model results. Improvements to algorithms to reduce the likelihood of numerical instabilities, and to handle embankment overtopping and dyke breaches more efficiently and accurately are also recommended.

This Design Brief for the Floodplain Mapping Program for the Serpentine-Nicomekl Rivers is respectfully submitted by:

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
YAROSLAV SHUMUK

KPA ENGINEERING LTD.

Y. Shumuk, P. Eng.
Project Manager

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
C. M. D. HOLMES

C. Holmes, P. Eng.
Hydraulics Analyst

J. Czuj
Hydrology Analyst

REFERENCES

1. Surrey, District of, Engineering Department, "Drainage Report," Surrey, B. C., October 1963.

2. Nikleva, S. and D. Faulkner, Pacific Meteorology Inc., "Estimates of Short-Term Precipitation in Surrey during the Storm of January 20-25, 1935," Richmond, B. C., November 1992.

3. Won, T. K., "Meteorology in British Columbia - A Centennial Review," Vancouver, B. C., May 1971.

4. Surrey Leader newspaper article, "Damage by flood will be over million", December 22, 1982.

5. Dunbar, D. S. and D. O. Hodgins, Seaconsult Marine Research Ltd. "Extreme Water Levels in Boundary Bay," Vancouver, B. C., May 1990.

6. Dunbar D. S., D. O Hodgins and J. A. Stronach, Seaconsult Marine Research Ltd. "Extreme Water Levels in Boundary Bay Phase II," Vancouver, B. C., November 1992.

7. Hodgins, D. O., Seaconsult Marine Research Ltd., "Extreme Water Levels in Boundary Bay Phase III," Vancouver, B. C., August 1993.

8. Seaconsult Marine Research Ltd., with contributions by Hay & Company Consultants Inc., "Extreme Water Levels in Boundary Bay Phase IV," Vancouver, B. C., February 1994.

9. Agriculture, U. S. Department of, Soil Conservation Service, "National Engineering Handbook, Section 4, Hydrology," 1969.

10. McCuen, R. H., "A Guide to Hydrologic Analysis using SCS Methods," Prentice-Hall, New Jersey, 1992.

11. Agriculture, U. S. Department of, Soil Conservation Service, "Urban Hydrology for Small Watersheds," Technical Release No. 55, January 1975.

12. Environment, B. C. Ministry of, "Soils of the Langley-Vancouver Map Area," RAB Bulletin 18, Report No. 15, B. C. Soil Survey, Kelowna, 1981.

13. Agriculture, B. C. Department of, "Soil Survey of Lower Fraser Valley," 1939.

14. Ker, Priestman & Associates Ltd., "Proposed Drainage and Irrigation Improvements for the Erickson Creek Area, District of Surrey," May 1989.

15. Wilson, K. W., B. C. Ministry of Environment, "Memorandum re: Curve Numbers and Base Flow Values," January 11, 1989.

16. Bruce, J. P. and R. H. Clark, "Introduction to Hydrometeorology," Pergamon Press, 1977, p. 31.

17. Morgan, P., Hay & Company Consultants Inc., facsimile transmitted to K. Wilson of B. C. Environment, 6 January, 1994.

REFERENCES (Con't)

18. Wilson, K. W., B. C. Environment letter to KPA "Re: Revision of Boundary Bay 200-Year Storm Surge Elevation," January 12, 1994.

19. Ng, M., P. H. LeBlond, and T. S. Murty, "Numerical Simulation of Tsunami Amplitudes on the Coast of British Columbia due to Local Earthquakes," Vancouver, 1990.

20. Holden, B. J. B. C. Ministry of Environment, "Coastal Environment and Coastal Construction, A Discussion Paper," Victoria, 1987.

21. Hamilton, T. S. and S. O. Wigen, "The Foreslope Hills of the Fraser Delta: Implications for Tsunamis in Georgia Strait," Science of Tsunami Hazards, Volume 5, Number 1, Hawaii, 1987.

22. Gunaratnam, D. J. and F. E. Perkins, "Numerical Solution of Unsteady Flows in Open Channels," Massachusetts Institute of Technology, Hydrodynamics Laboratory, Report No. 127, July 1970.

23. Environment Canada, Water Planning and Management Branch, "One-Dimensional Hydrodynamic Model Computer Manual," Ottawa, February 1988.

24. Environment Canada, Water Planning and Management Branch, "One-Dimensional Hydrodynamic Model, Data Preparation Programs, COORD1 & COORD2, Computer Manual," Ottawa, June 1988.

25. Environment Canada, Water Planning and Management Branch, "Documentation for the BREACH Routine Package to be used with the One-Dimensional Hydrodynamic Model," Ottawa, February 1986.

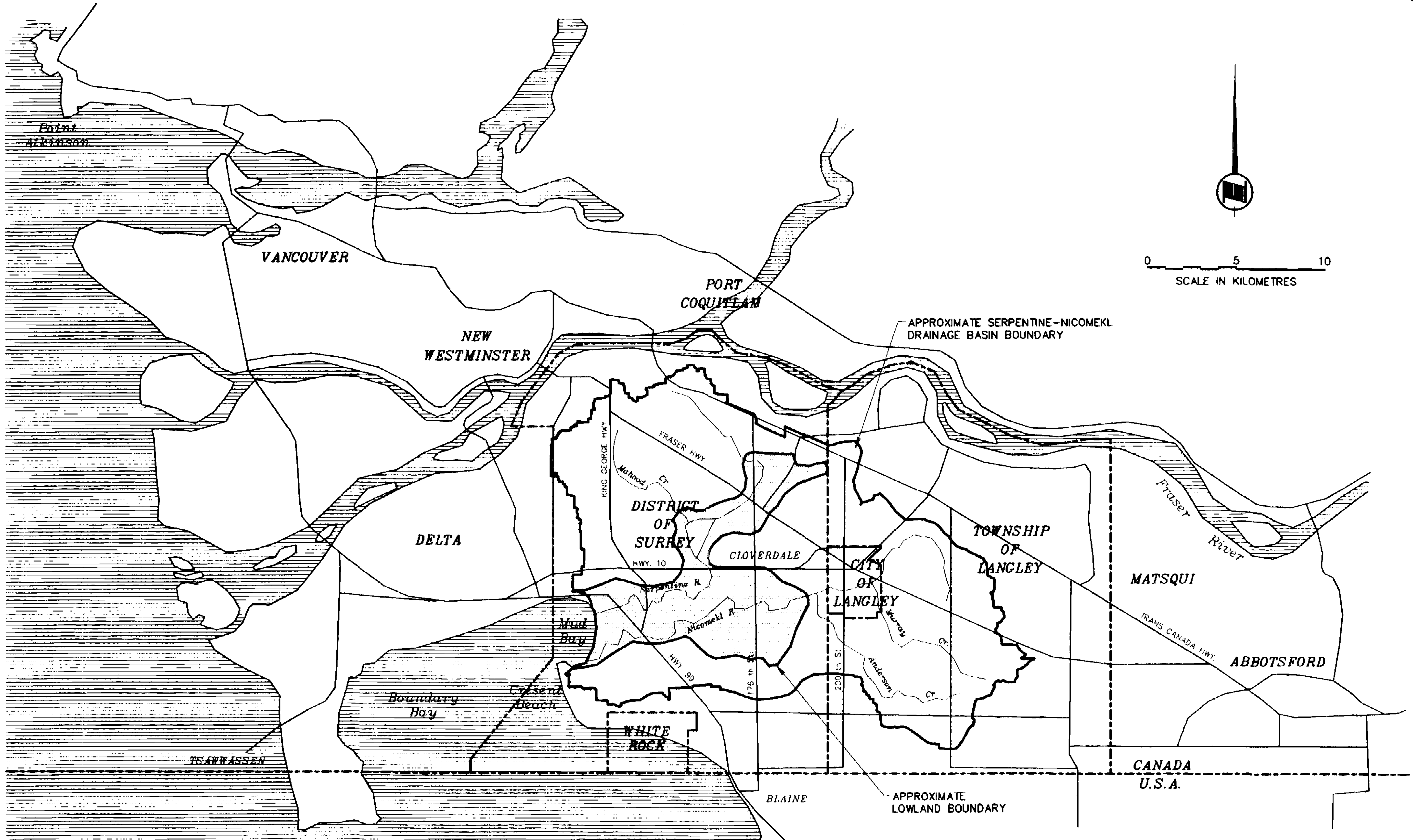
26. Environment Canada, Economics and Conservation Branch, "Serpentine-Nicomekl Floodplain Mapping Project, One-Dimensional Hydrodynamic Model, Model Modification for Sea-Dams," Ottawa, January 1993.

27. Peters, N. J., B. C. Ministry of Environment and Parks, internal memorandum, "Summary of Flood Levels Derived from Photographs in Cloverdale Area 1935, 1960 and 1968," Surrey, May 27, 1988.

28. Vancouver Sun newspaper article headlined "Pacific Highway May Close," January 24, 1935, Page 1.

29. Hay & Company Consultants Inc., "Technical Report, Floodplain Management Study," Report to the Township of Richmond, November 1989.

FIGURES



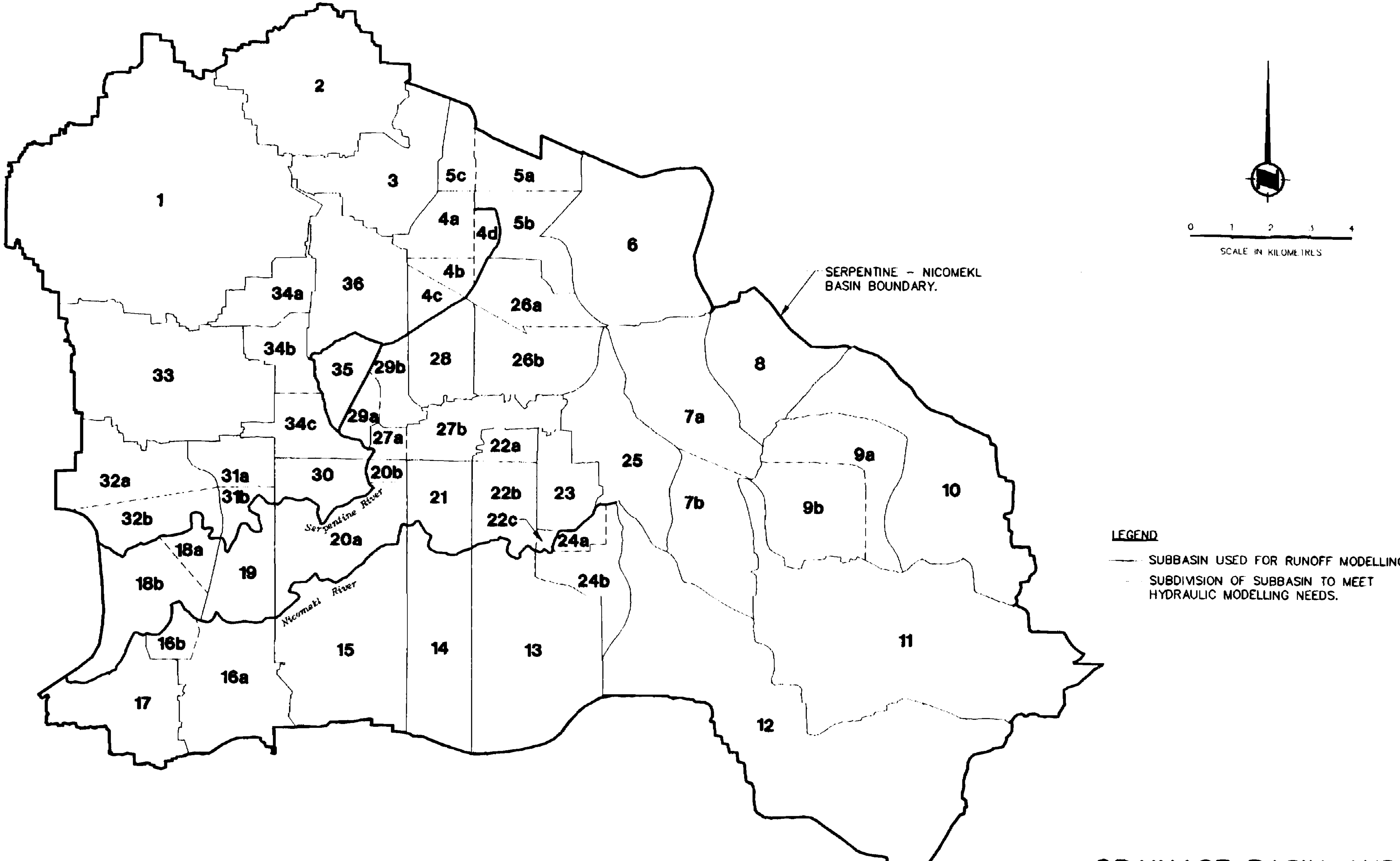
LOCATION PLAN

FIGURE 1

KPA ENGINEERING LTD.

OCTOBER 1993

2997 E03



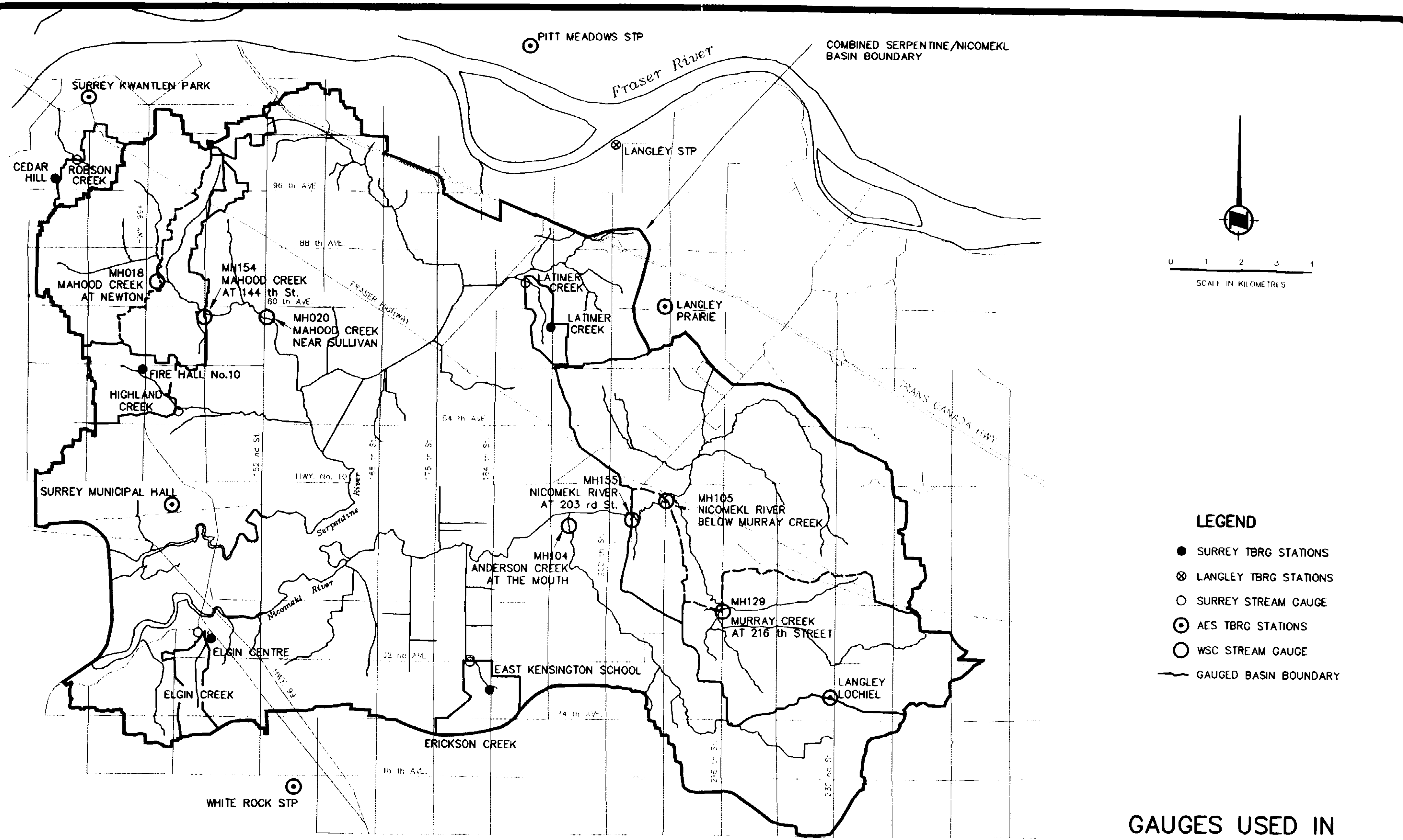
DRAINAGE BASIN AND SUBBASIN BOUNDARIES

FIGURE 2

KPA ENGINEERING LTD.

DATE: MARCH 1993

FILE NO. 2997 E03



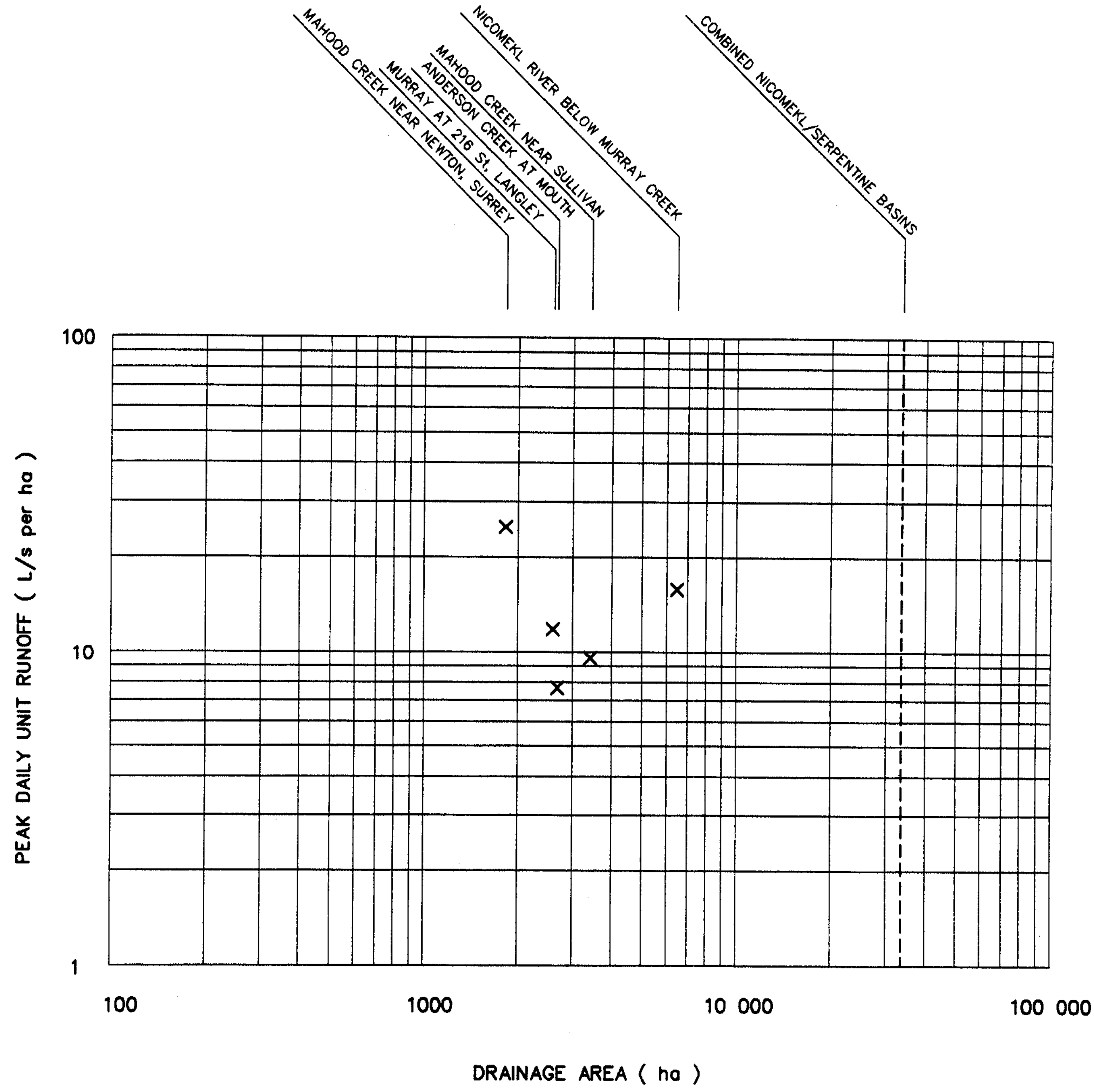
GAUGES USED IN HYDROLOGIC ANALYSIS

FIGURE 3

KPA ENGINEERING LTD.

MARCH 1994

2997 E03



LEGEND

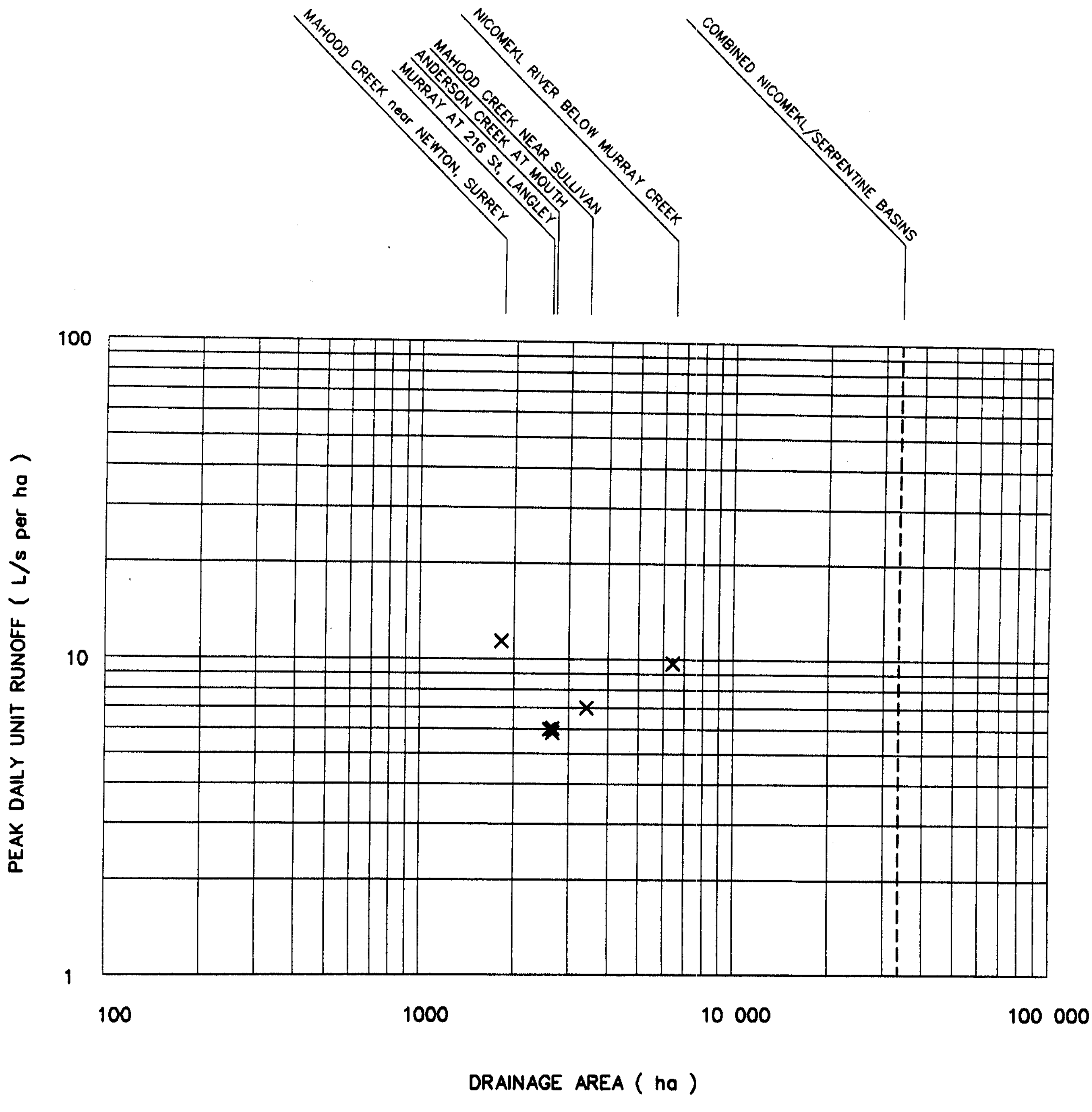
X - - - - - - 200 YEAR UNIT RUNOFF

PEAK DAY UNIT FLOWS vs DRAINAGE AREA

FIGURE 4

KPA ENGINEERING LTD.

MARCH 1994
2997 E03



LEGEND

X ------ 200 YEAR UNIT RUNOFF

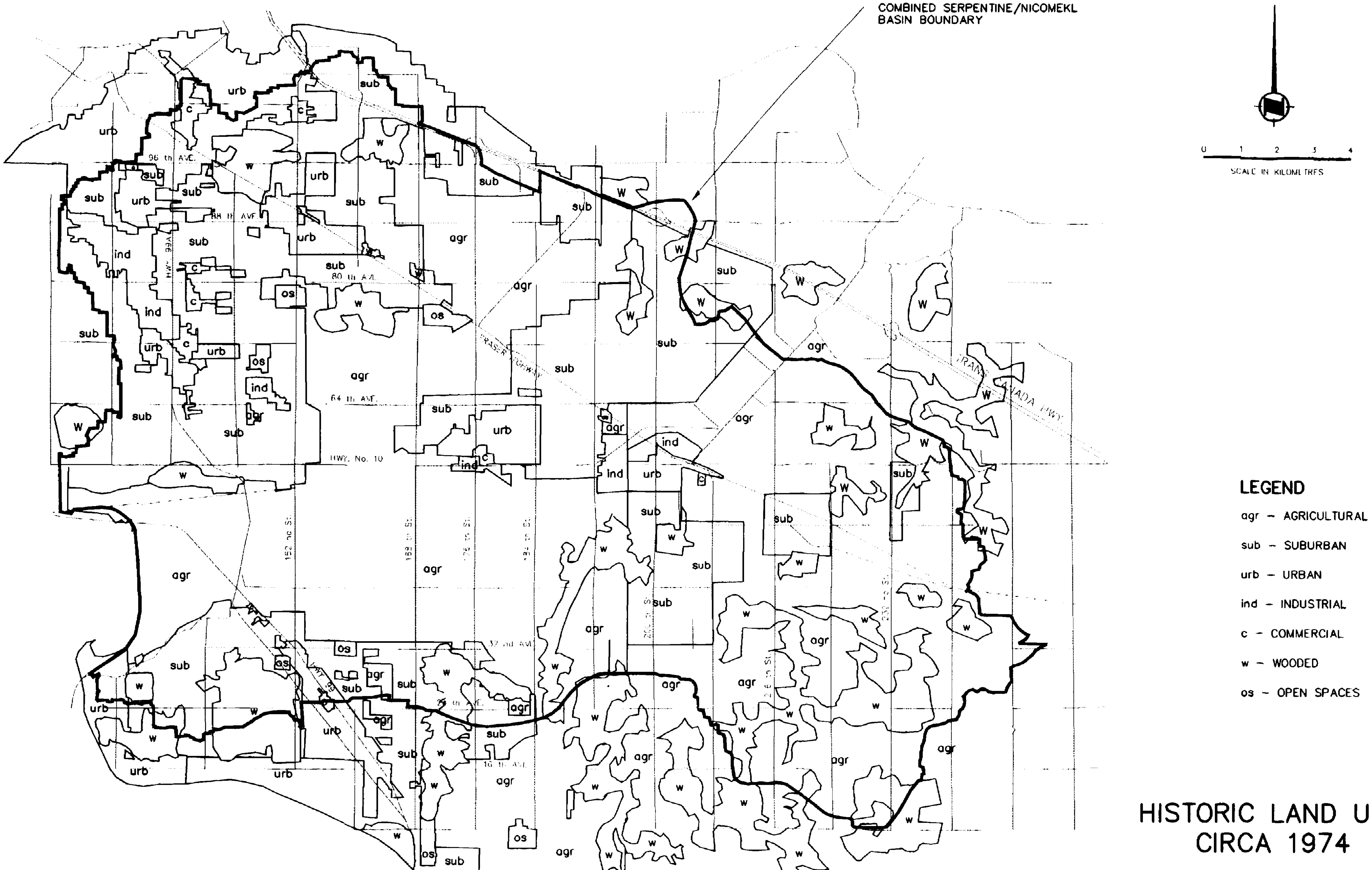
PEAK 3–DAY UNIT FLOWS vs DRAINAGE AREA

FIGURE 5

KPA ENGINEERING LTD.

MARCH 1994

2997 E03



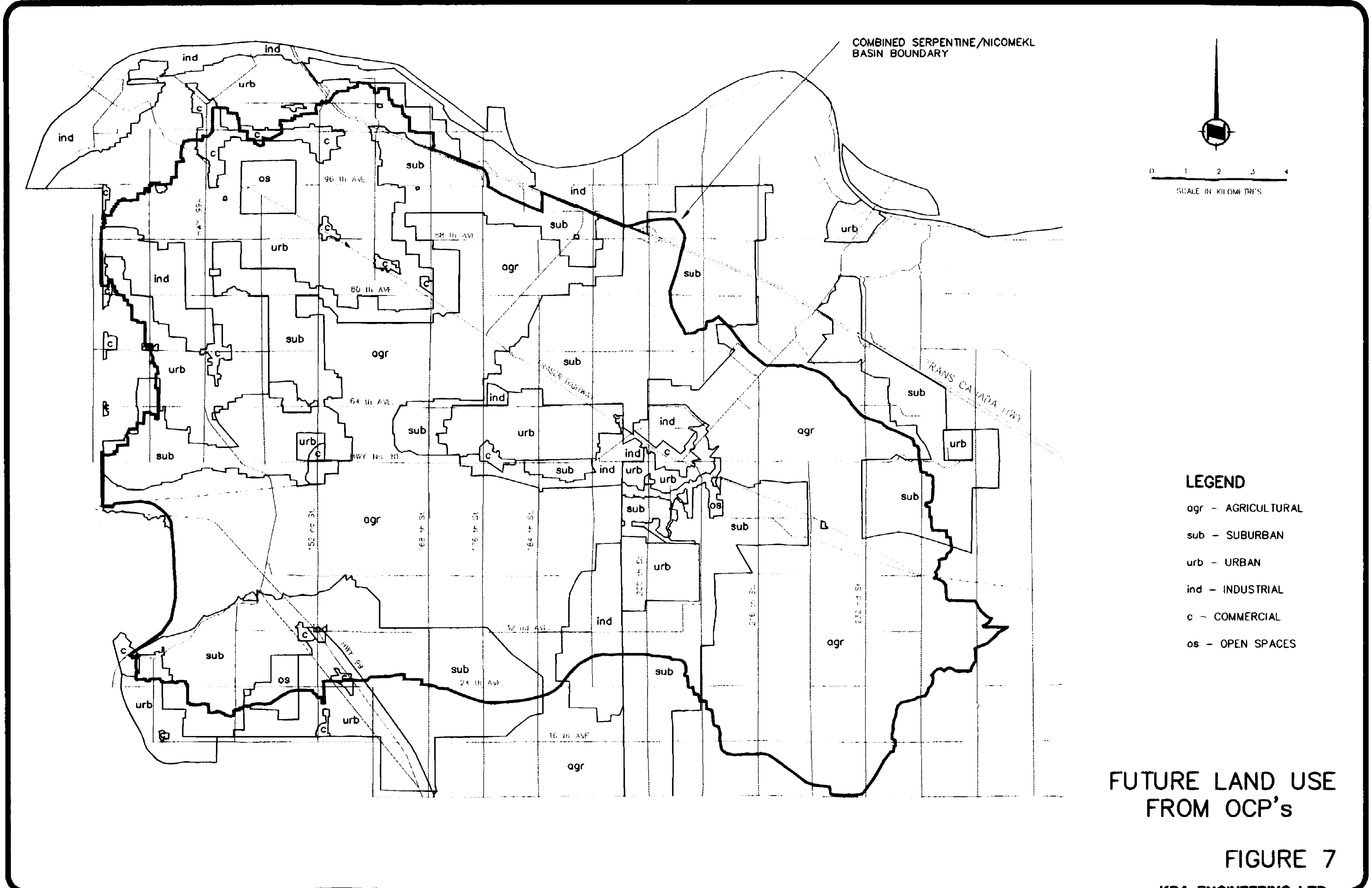
HISTORIC LAND USE
CIRCA 1974

FIGURE 6

KPA ENGINEERING LTD.

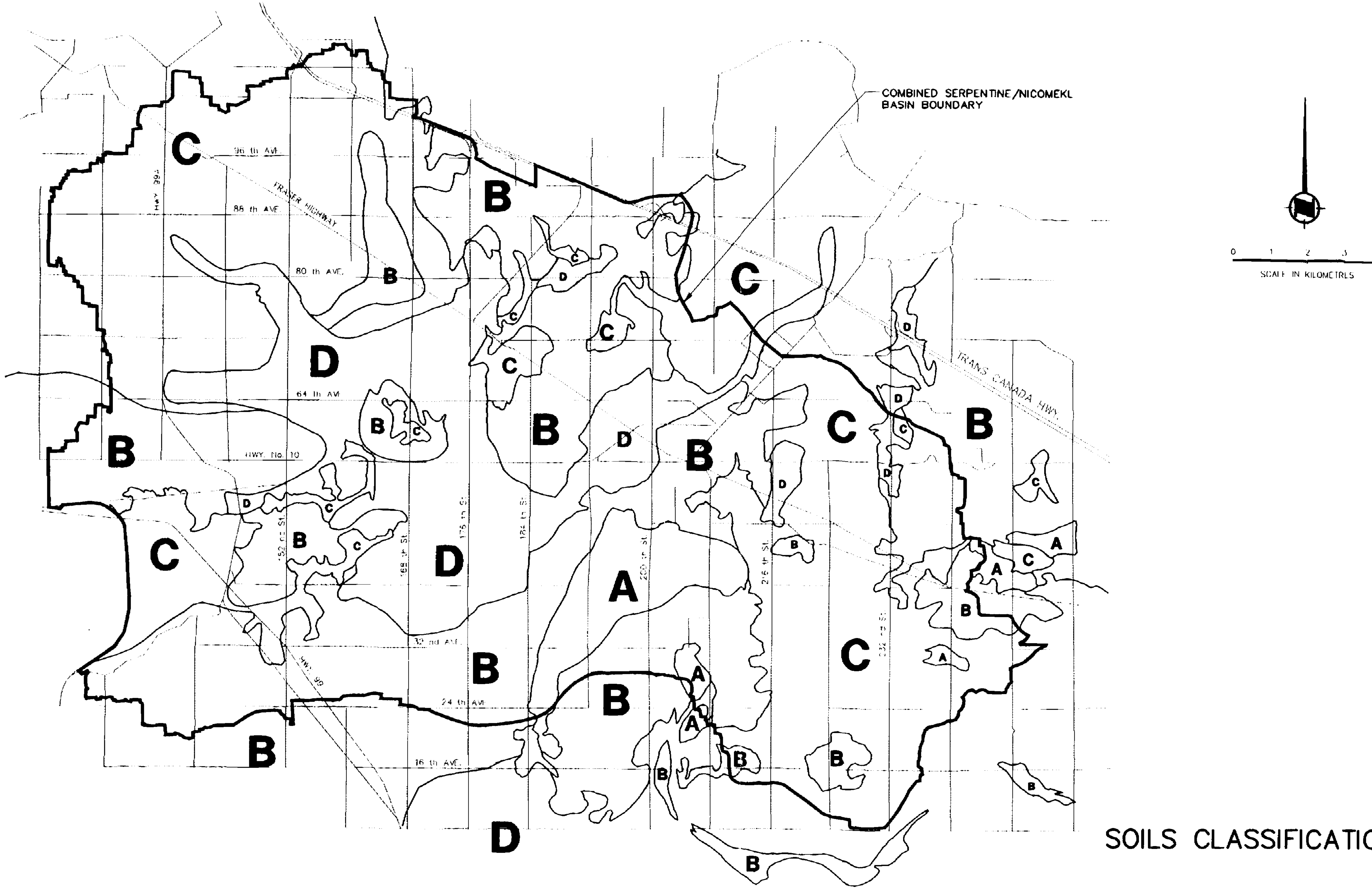
MARCH 1994

2997 E03



MARCH 1994

2997 E03



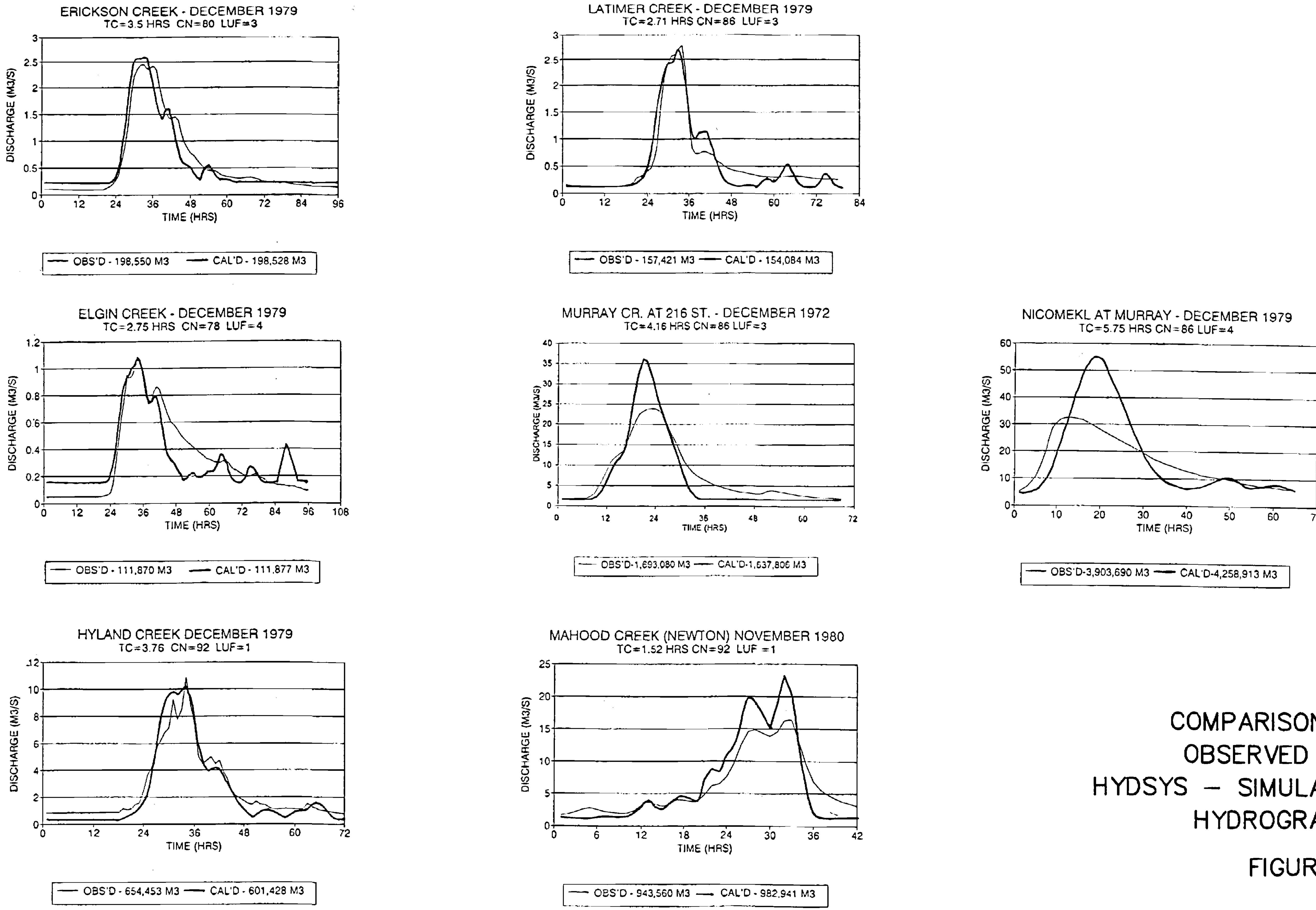
SOILS CLASSIFICATION MAP

FIGURE 8

KPA ENGINEERING LTD.

MARCH 1994

2997 E03



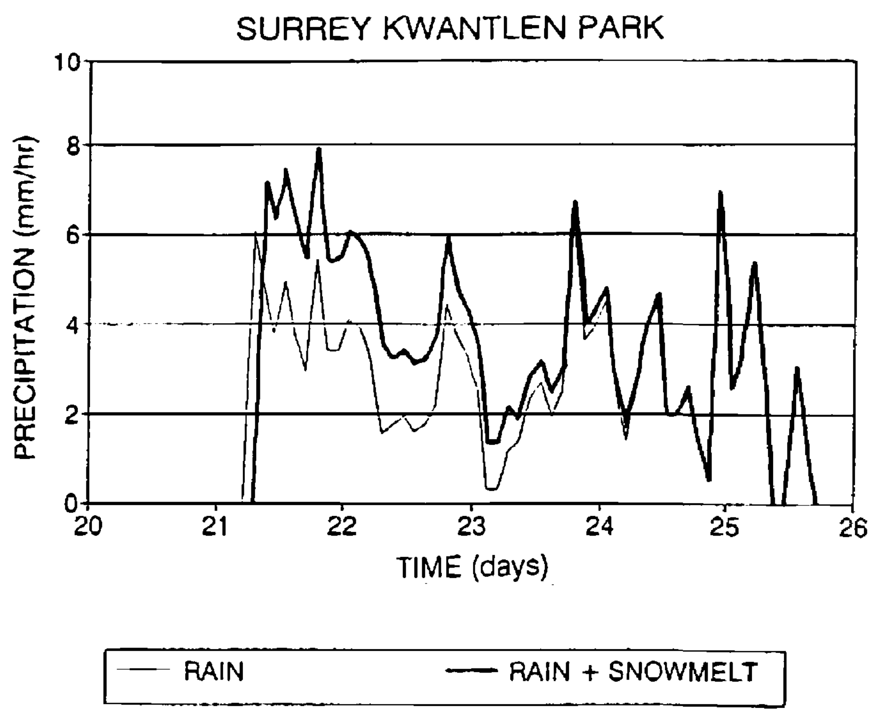
COMPARISON OF OBSERVED AND HYDSYS – SIMULATED HYDROGRAPHS

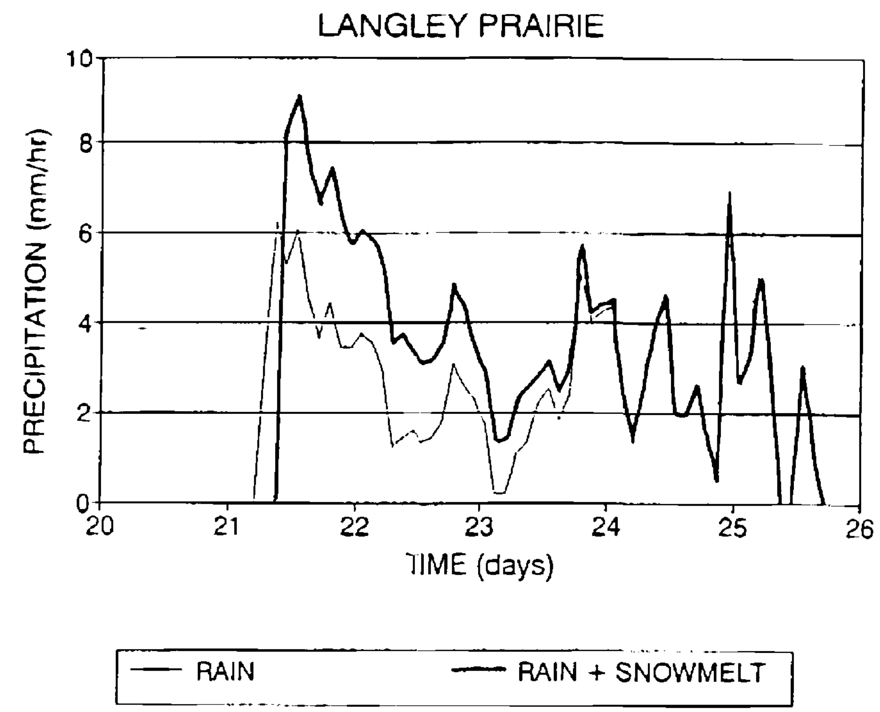
FIGURE 9

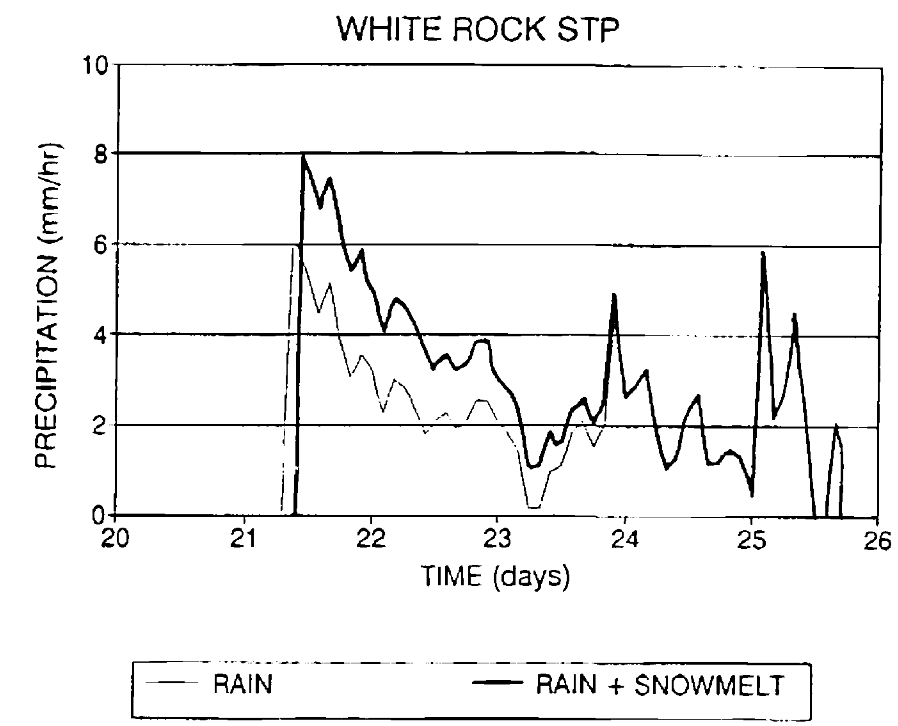
KPA ENGINEERING LTD.

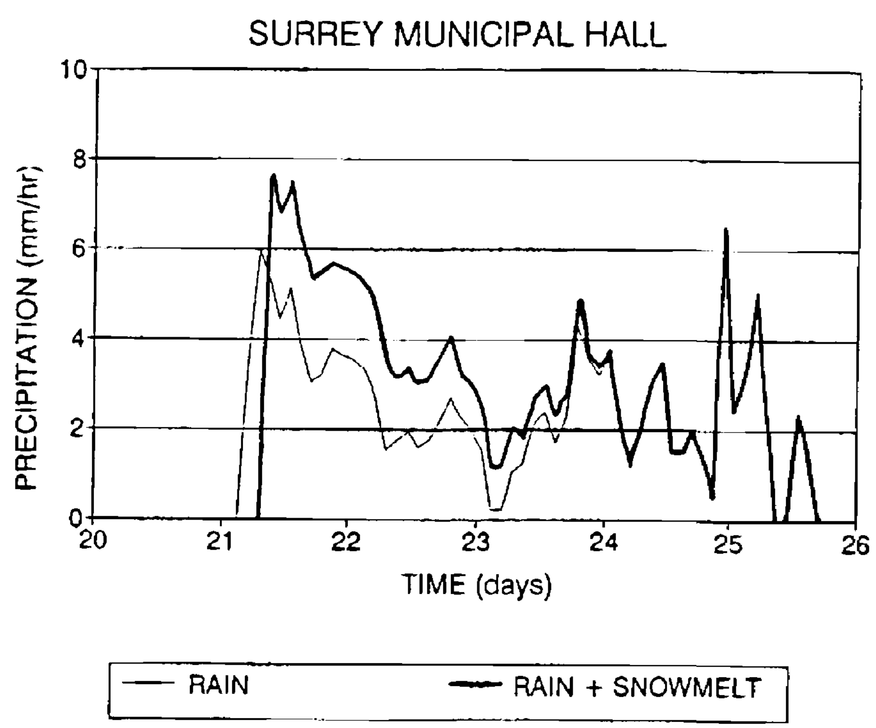
OCTOBER 1993

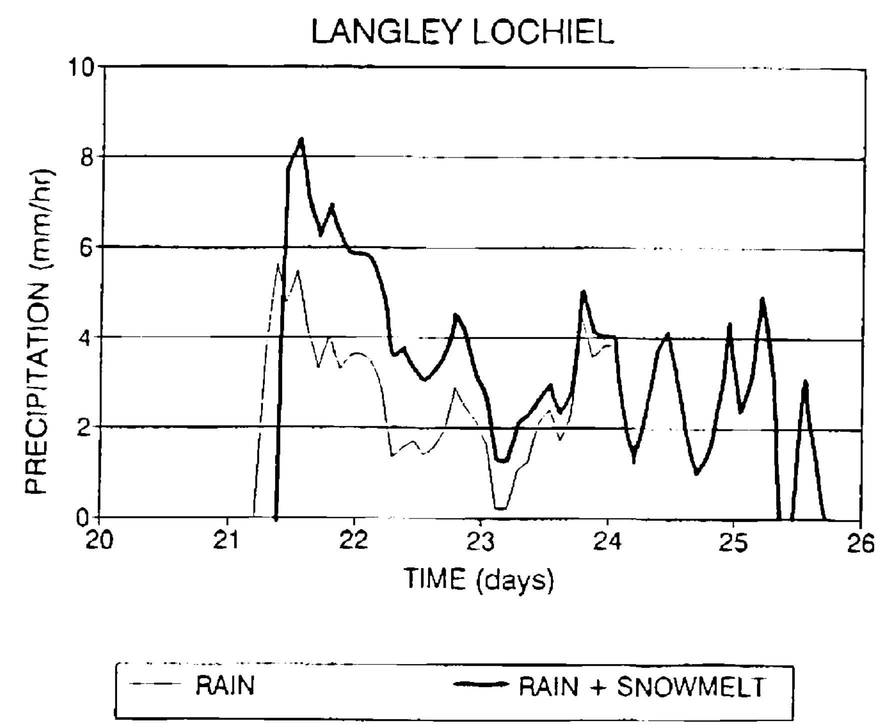
2997 E03









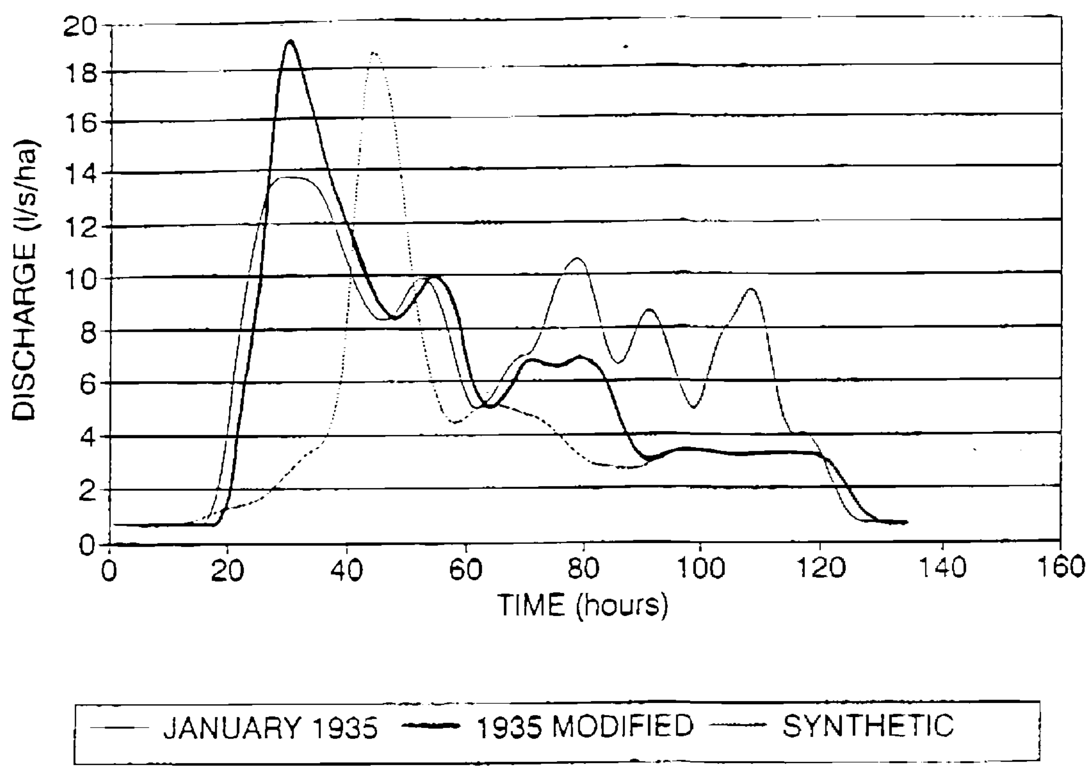


NOTES:
1. PRECIPITATION DETERMINED BY THE ISOHYETAL METHOD BY PACIFIC METEOROLOGY INC.
2. NAMES REFER TO LOCATIONS ONLY. GAUGES DID NOT EXIST IN 1935.

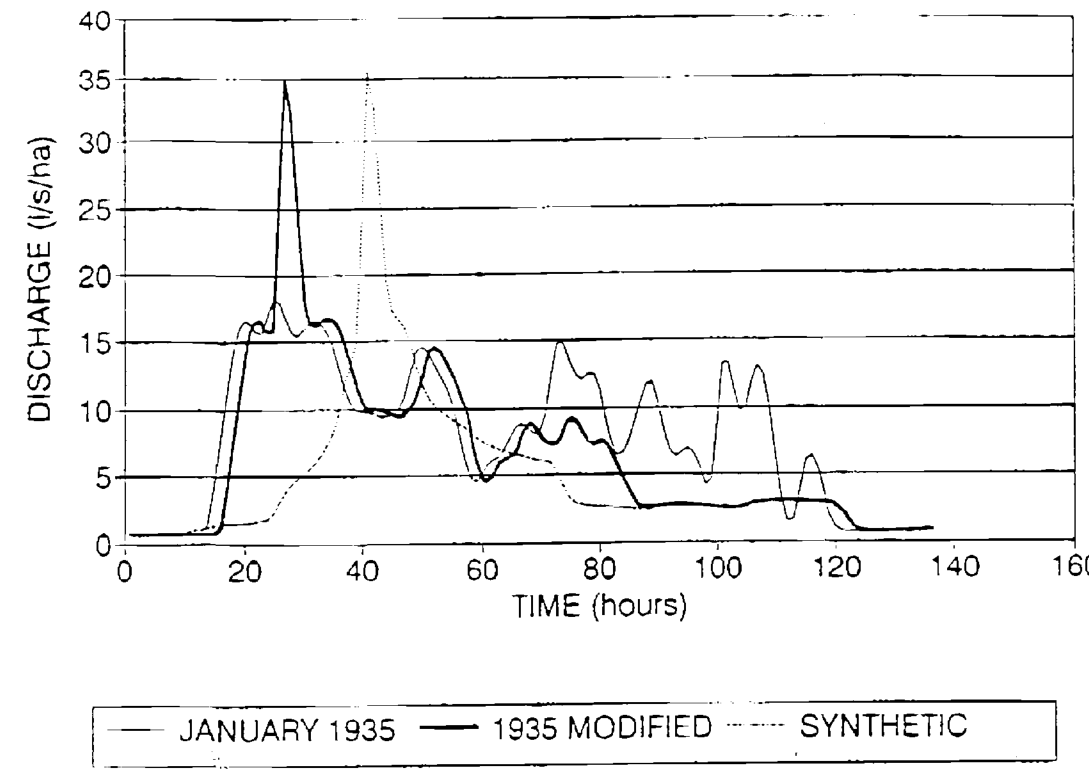
ESTIMATED
RAIN PLUS SNOWMELT
HYETOGRAPHS FOR THE
1935 STORM

FIGURE 10

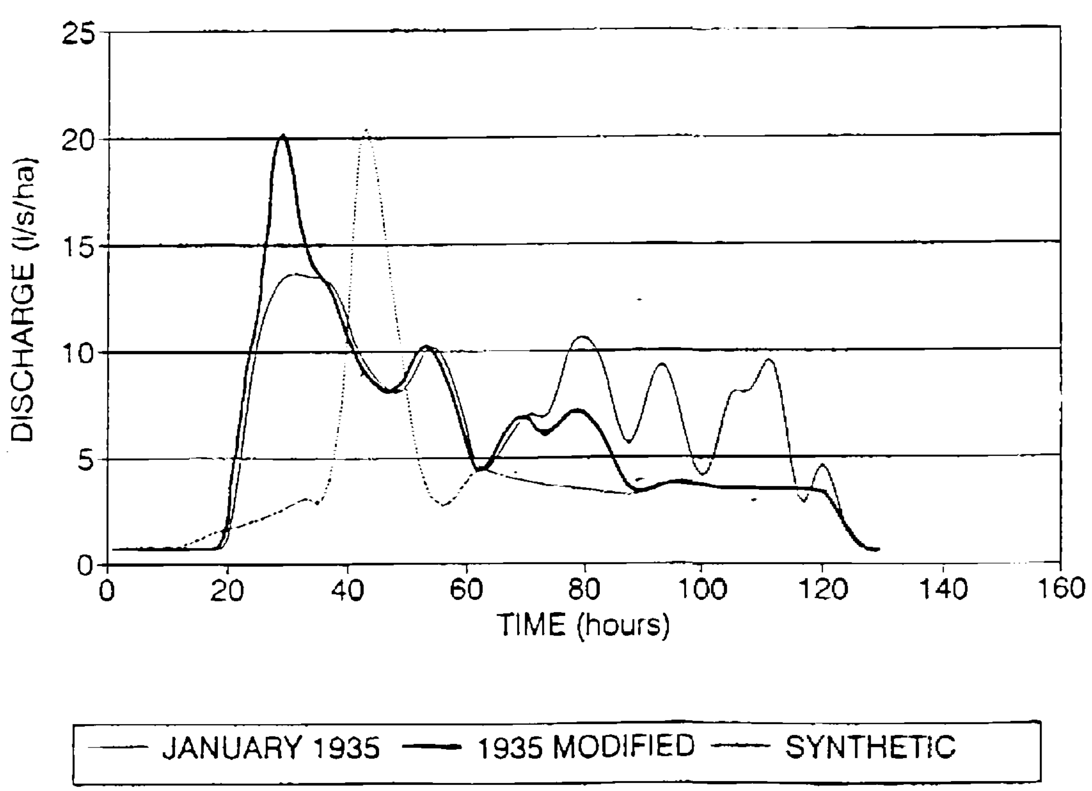
NICOMEKL RIVER BELOW MURRAY CR(08MH105)
200 YEAR HYDROGRAPHS



MAHOOD CREEK AT NEWTON (08MH018)
200 YEAR HYDROGRAPHS



MURRAY CREEK AT 216 ST. (08MH129)
200 YEAR HYDROGRAPHS



NOTE: THE 1935 MODIFIED CURVES HAVE BEEN OFFSET SLIGHTLY FOR CLARITY IN PRESENTATION.

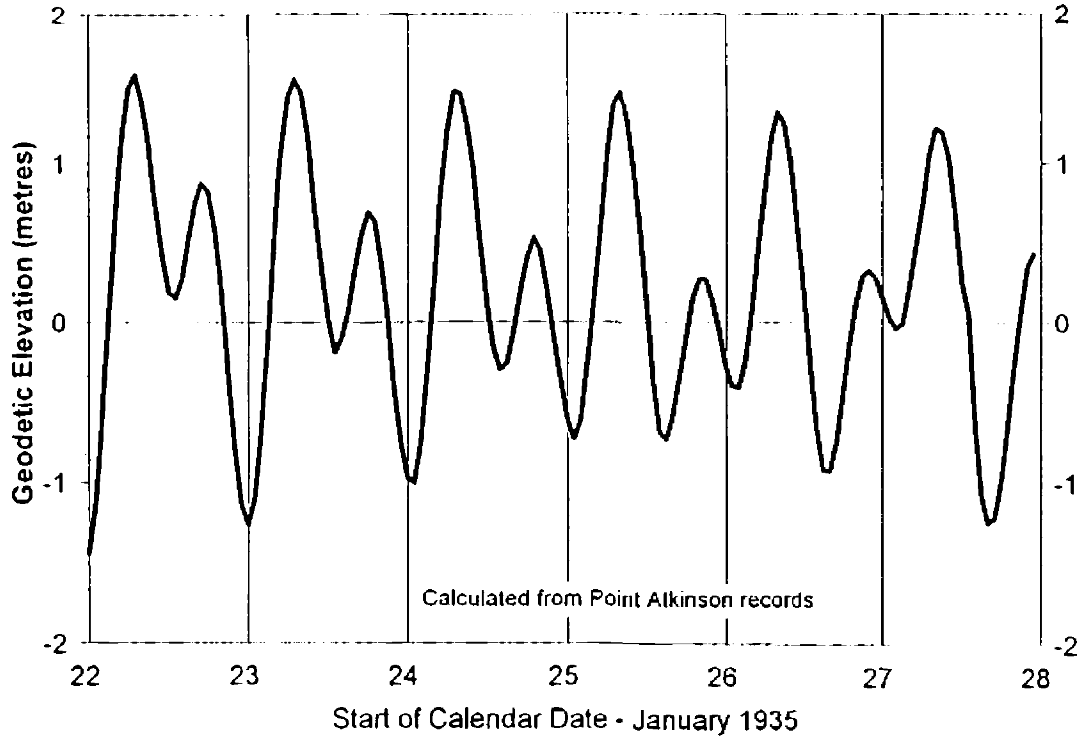
MARCH 1994

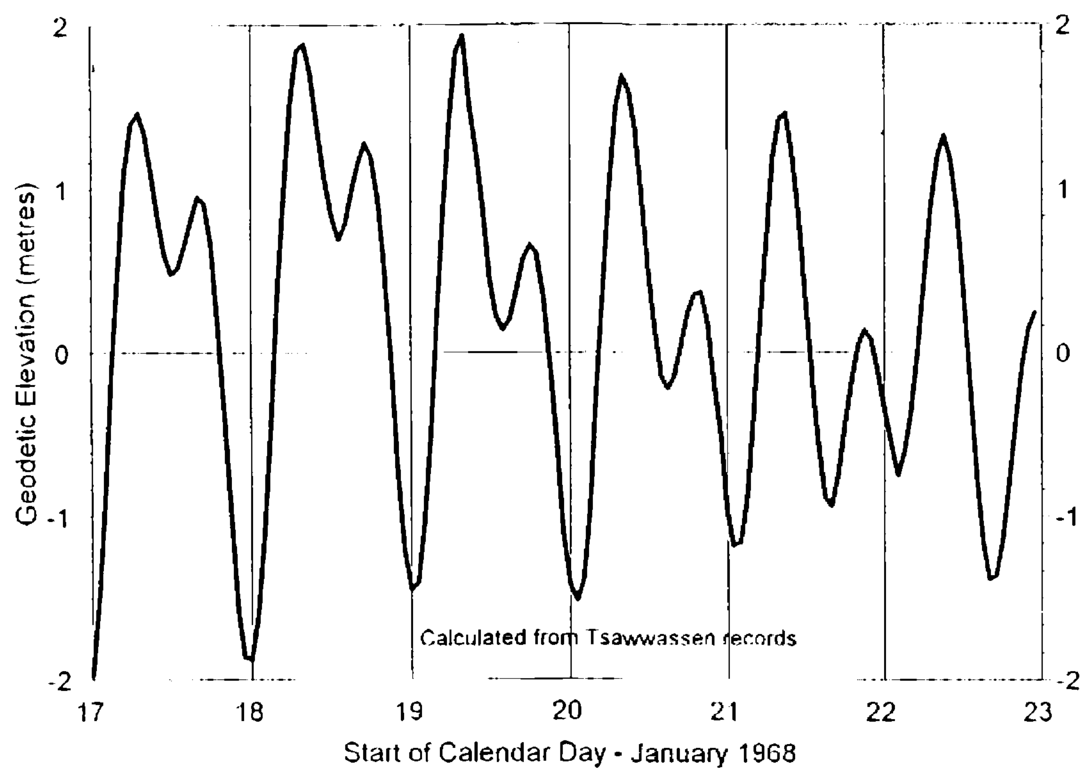
2997 E03

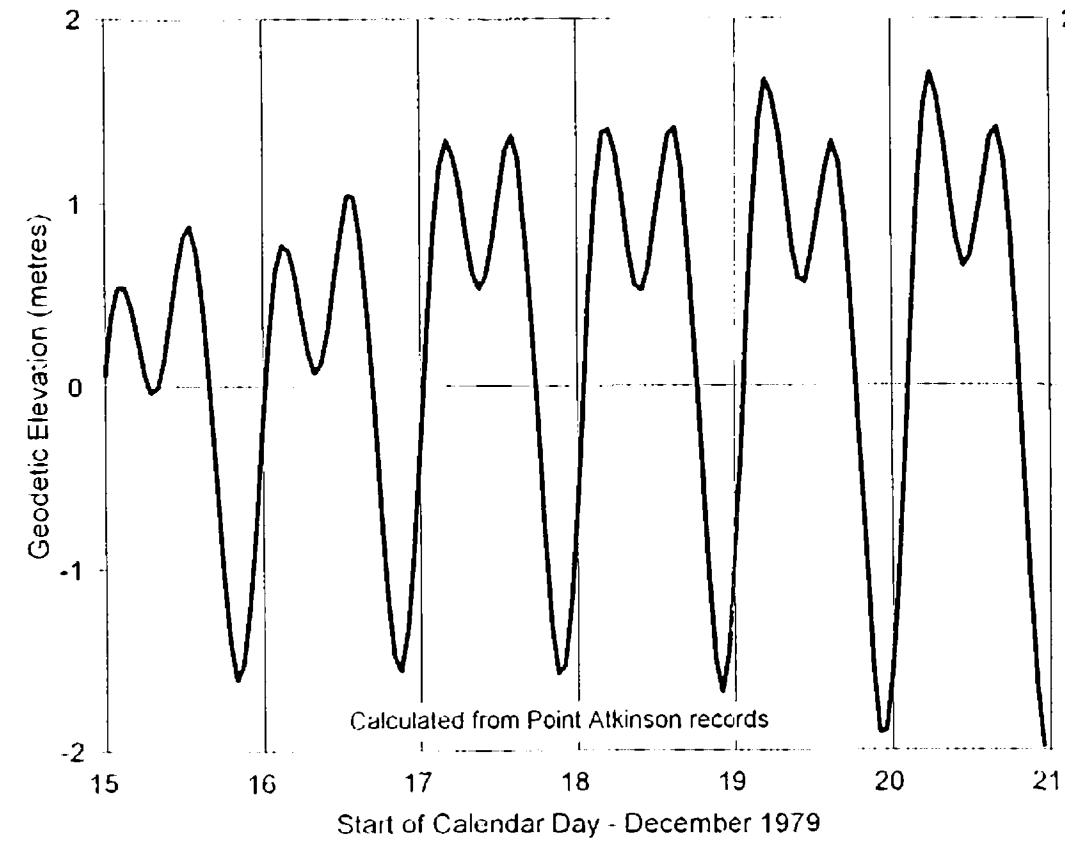
EXAMPLES OF 200-YEAR HYDROGRAPH DERIVATION

FIGURE 11

KPA ENGINEERING LTD.







ESTIMATED TIDE LEVELS
FOR BOUNDARY BAY
DURING THREE STORM EVENTS

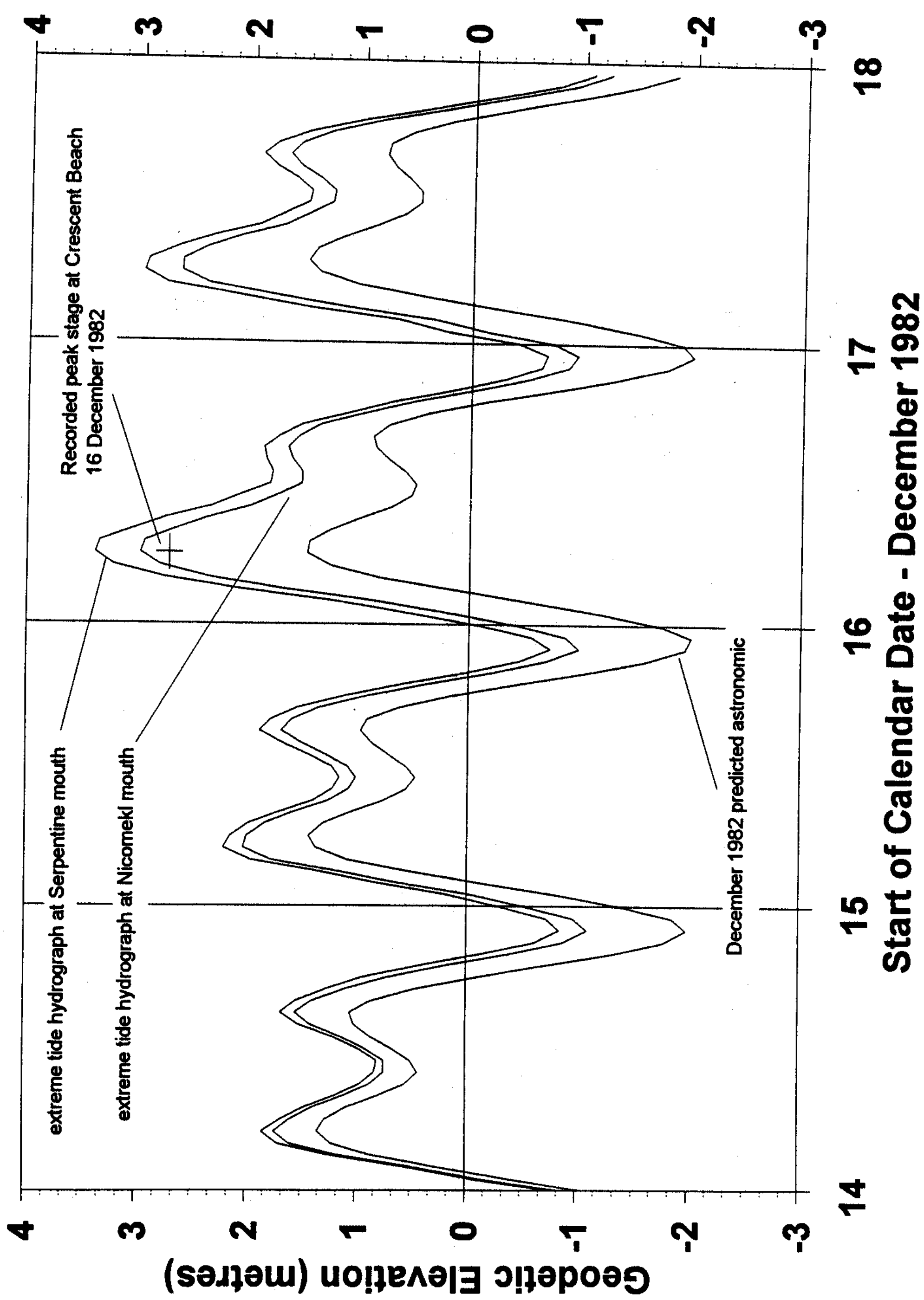
FIGURE 12

OCTOBER 1993

2997 E03

KPA ENGINEERING LTD.

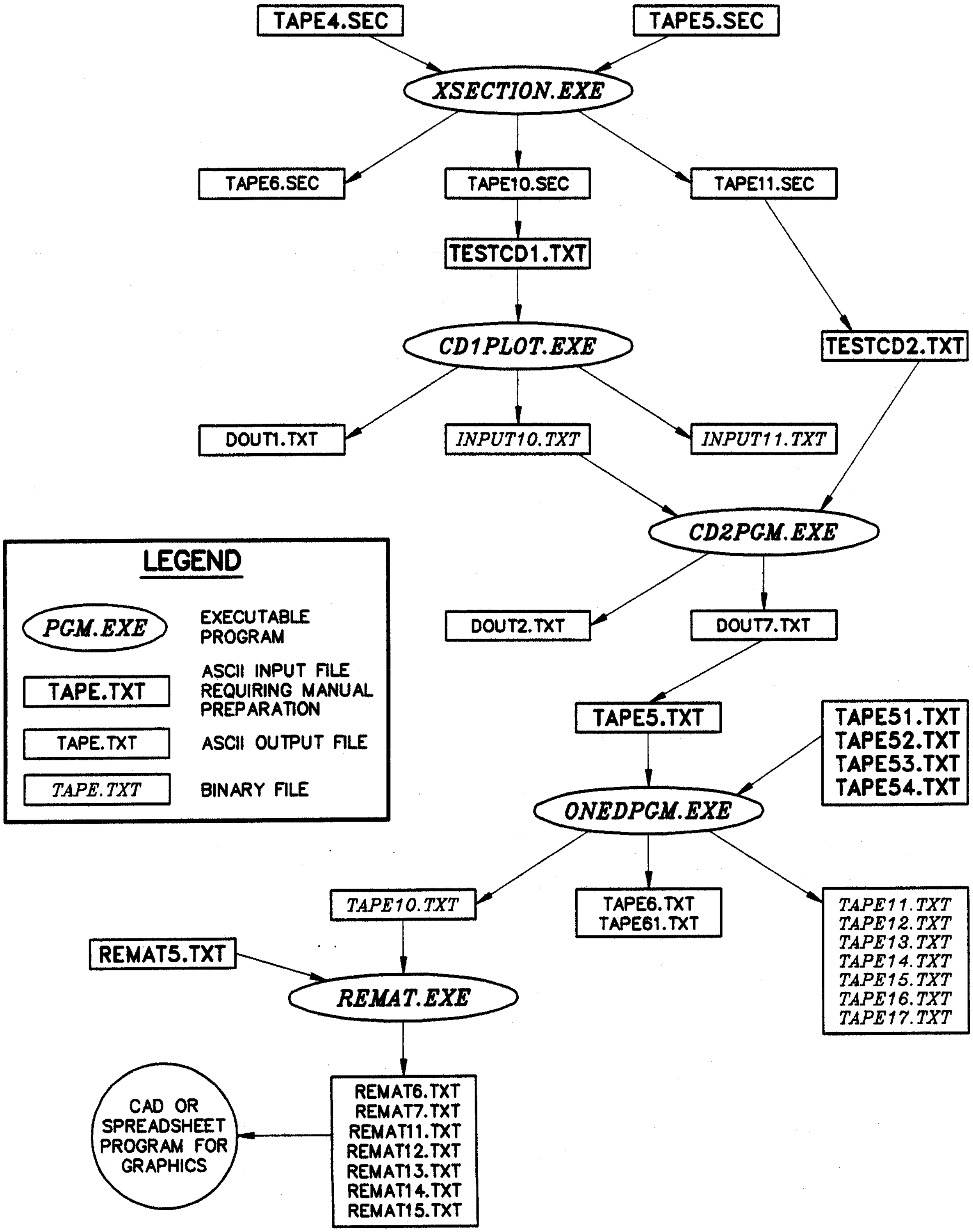
FILE NO. 2997 DATE: FEBUARY 1994



DERIVATION OF TIDE HYDROGRAPHS CONTAINING EXTREME EVENT PEAKS

FIGURE 13

KPA ENGINEERING LTD.

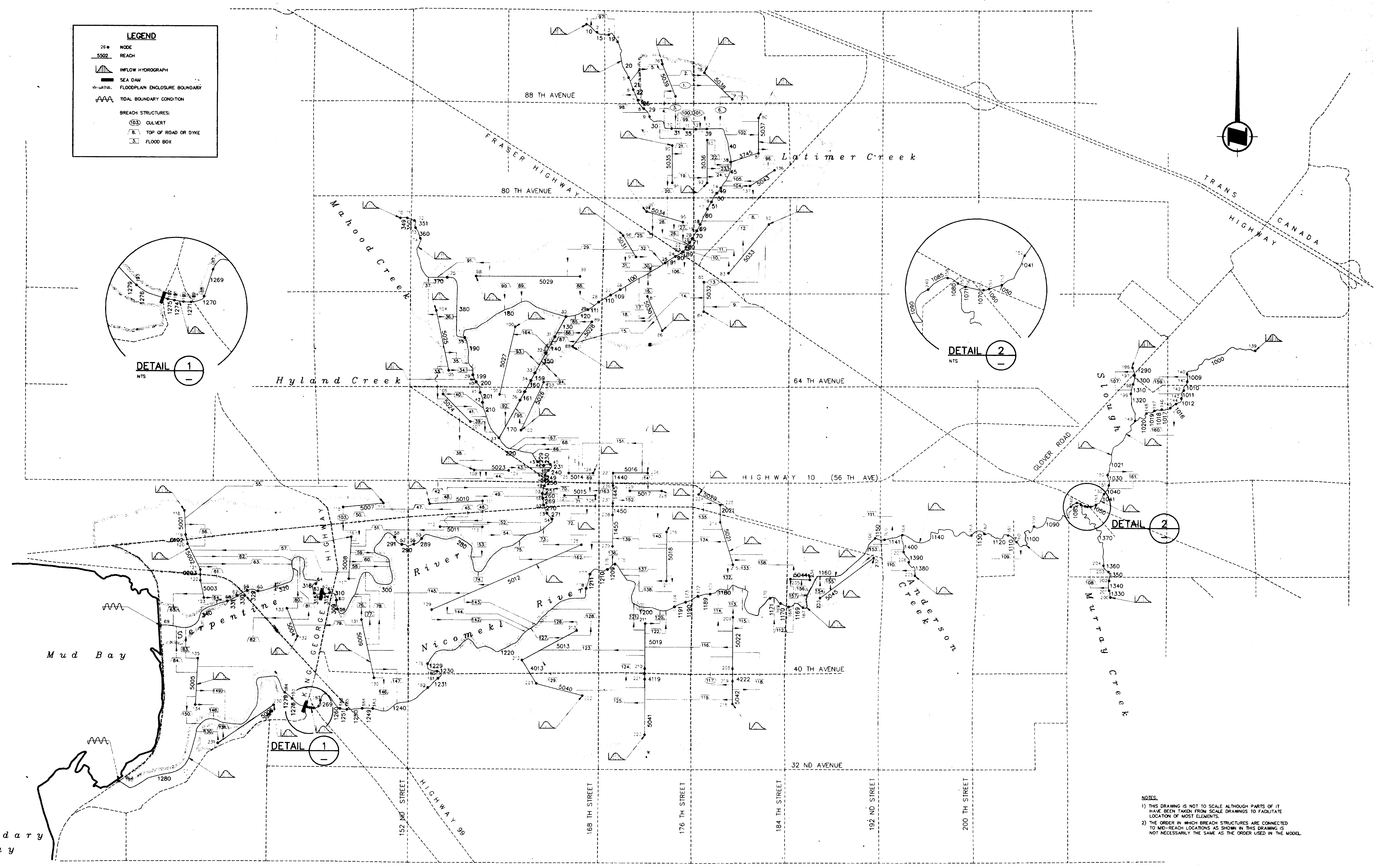


FILE NO. 2997 MARCH 1994

ONE-D HYDRODYNAMIC MODEL
FLOW CHART OF PROGRAMS AND FILES

FIGURE 14

KPA ENGINEERING LTD.

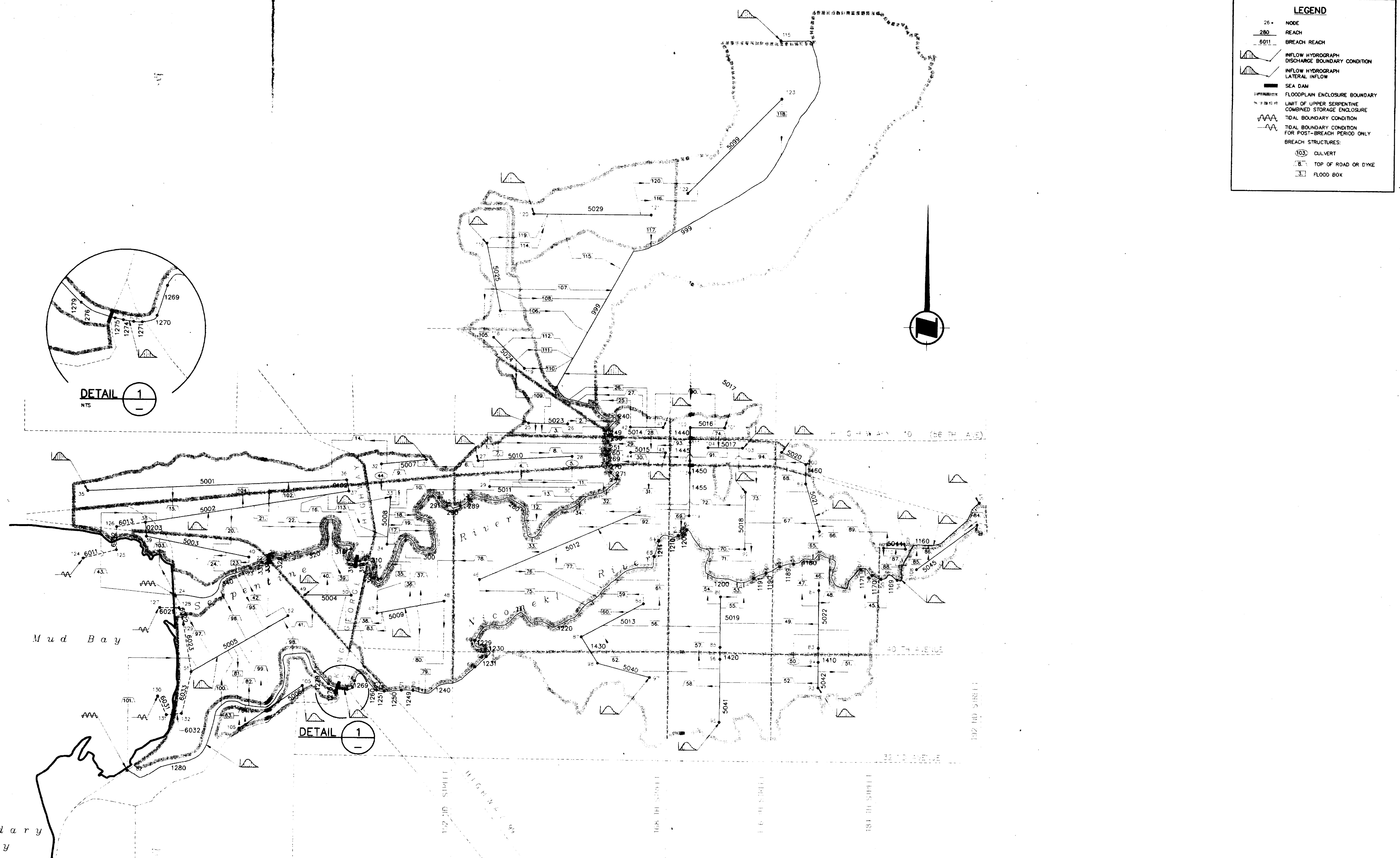


NOTES:
1) THIS DRAWING IS NOT TO SCALE ALTHOUGH PARTS OF IT HAVE BEEN TAKEN FROM SCALE DRAWINGS TO FACILITATE LOCATION OF MOST ELEMENTS.
2) THE ORDER IN WHICH BREACH STRUCTURES ARE CONNECTED TO MID-REACH LOCATIONS AS SHOWN IN THIS DRAWING IS NOT NECESSARILY THE SAME AS THE ORDER USED IN THE MODEL.

SCHEMATIC DIAGRAM OF THE HYDRODYNAMIC MODEL FOR RUNOFF FLOODS

FIGURE 15

KPA ENGINEERING LTD.



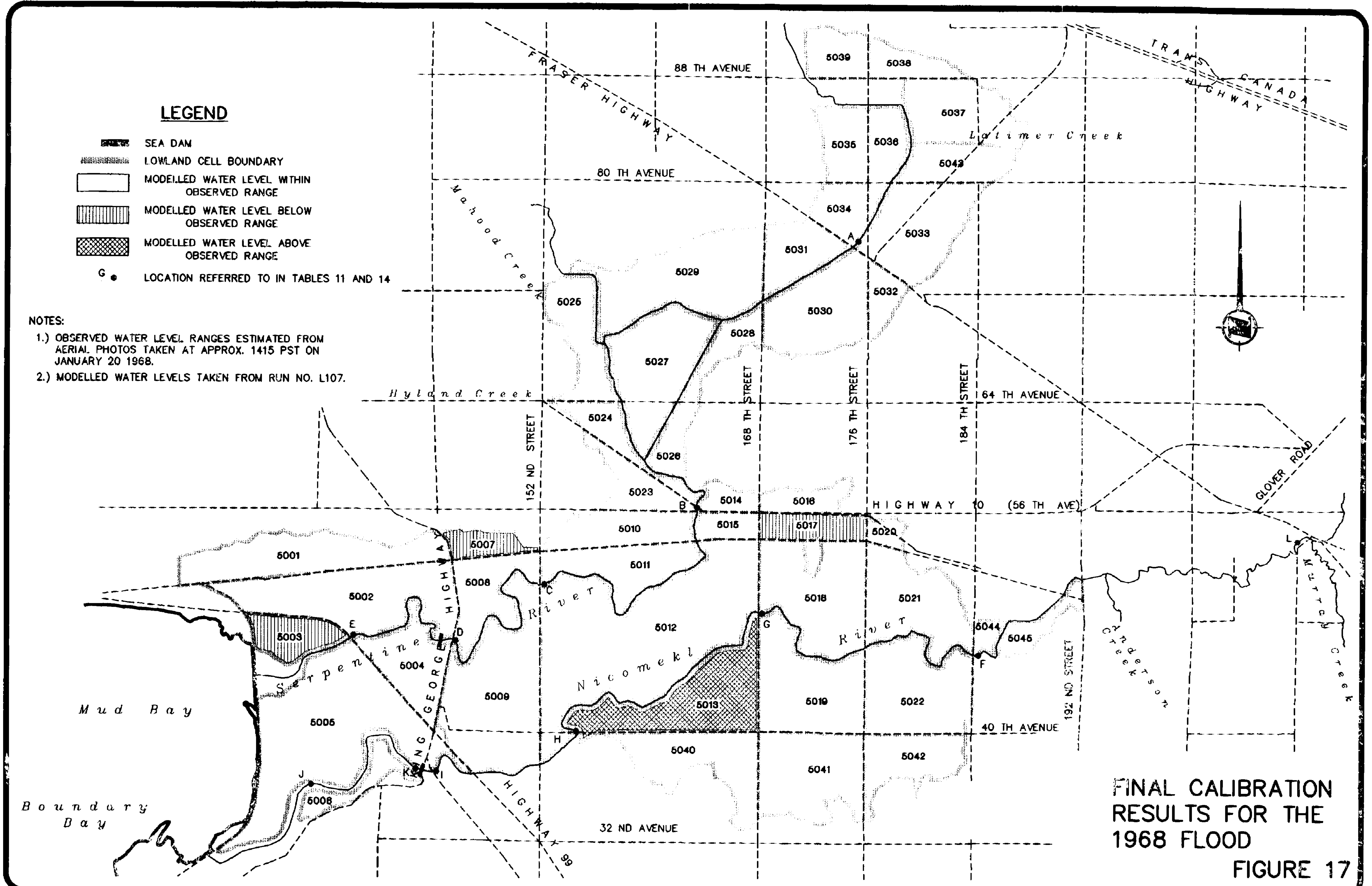
NOTES:

1) THIS DRAWING IS NOT TO SCALE ALTHOUGH PARTS OF IT HAVE BEEN TAKEN FROM SCALE DRAWINGS TO FACILITATE LOCATION OF MOST ELEMENTS.
2) THE ORDER IN WHICH BREACH STRUCTURES ARE CONNECTED TO MID-REACH LOCATIONS AS SHOWN IN THIS DRAWING IS NOT NECESSARILY THE SAME AS THE ORDER USED IN THE MODEL.

SCHEMATIC DIAGRAM OF THE HYDRODYNAMIC MODEL FOR FLOODS FROM THE SEA

FIGURE 16

KPA ENGINEERING LTD.



FINAL CALIBRATION RESULTS FOR THE 1968 FLOOD

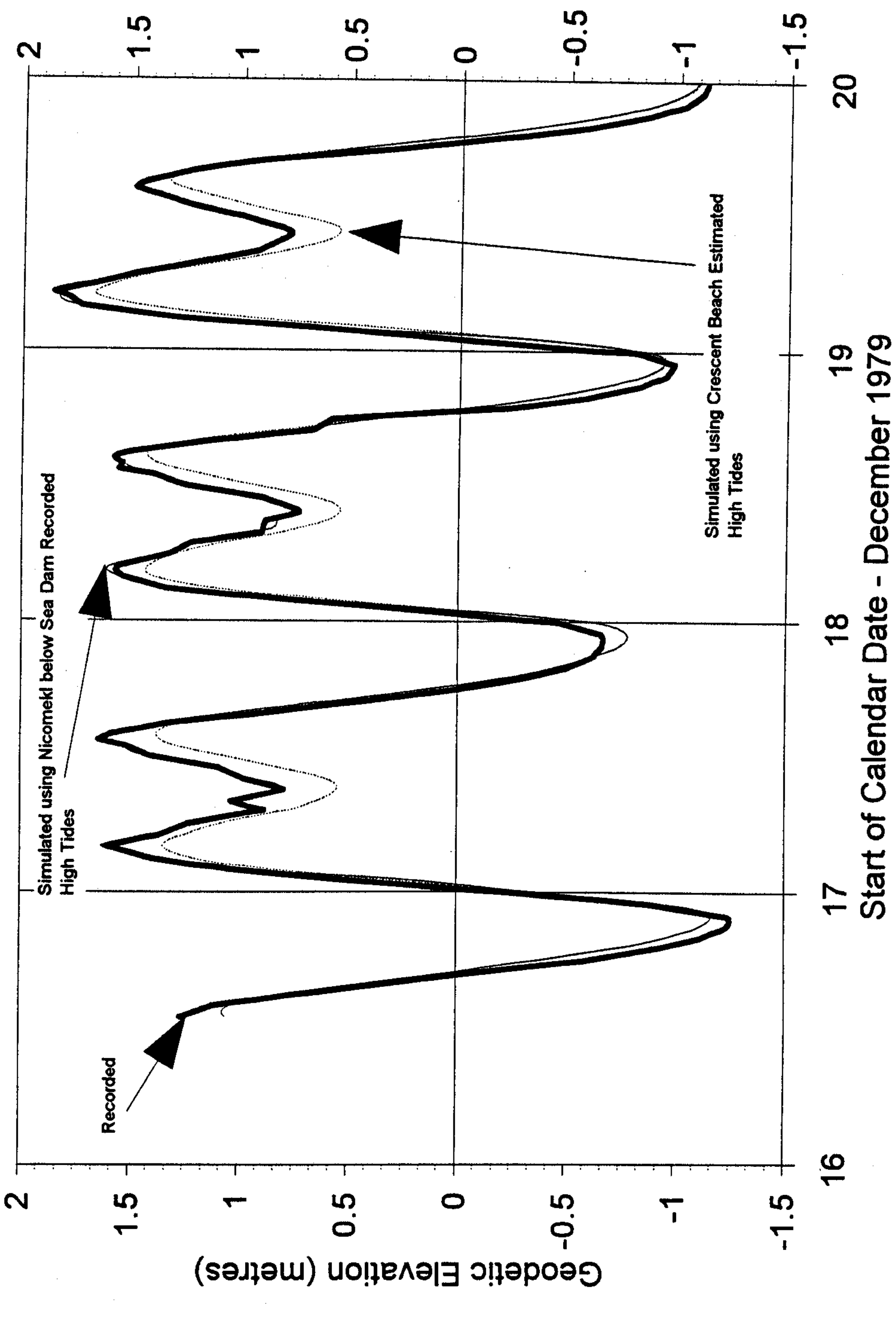
FIGURE 17

KPA ENGINEERING LTD.

MARCH 1994

FILE NO. 2997

FILE NO. 2997 DATE: OCT 1993



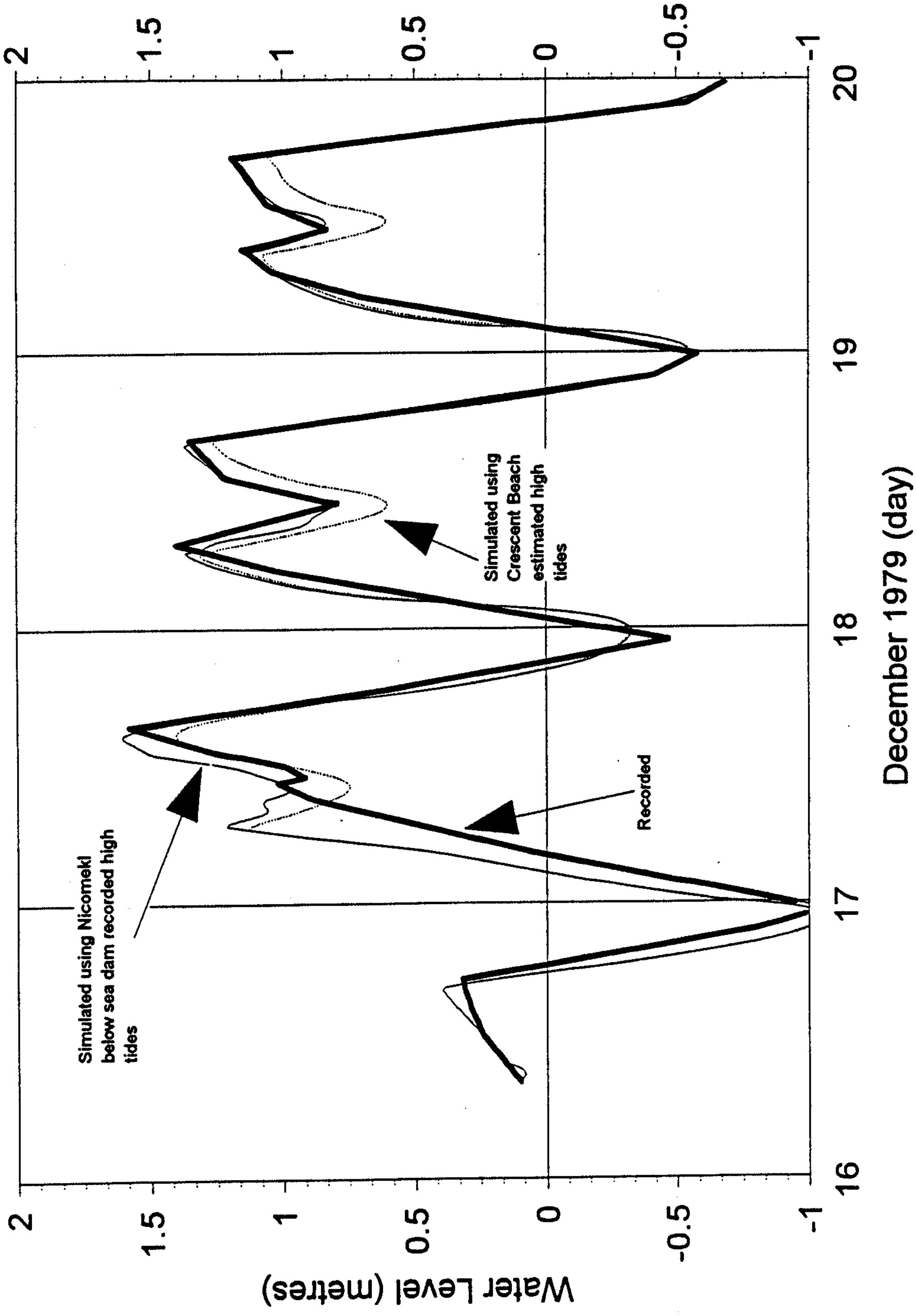
1979 CALIBRATION HYDROGRAPHS FOR NICOMEKL RIVER BELOW SEA DAM

FIGURE 18

KPA ENGINEERING LTD.

FILE NO. 2997

DATE: OCT 1993

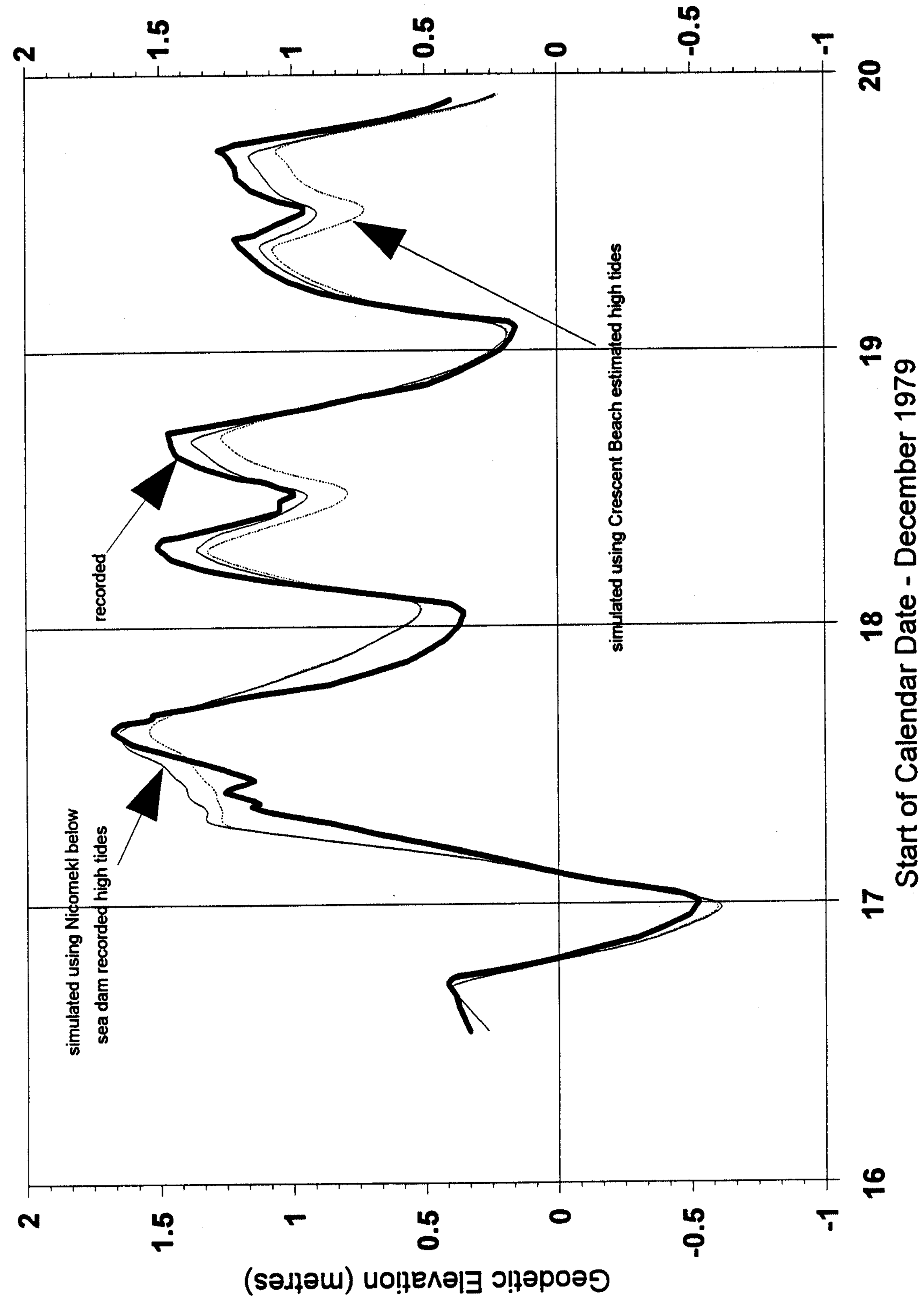


1979 CALIBRATION HYDROGRAPHS FOR SERPENTINE RIVER ABOVE SEA DAM

FIGURE 19

FILE NO. 2997

DATE: OCT 1993

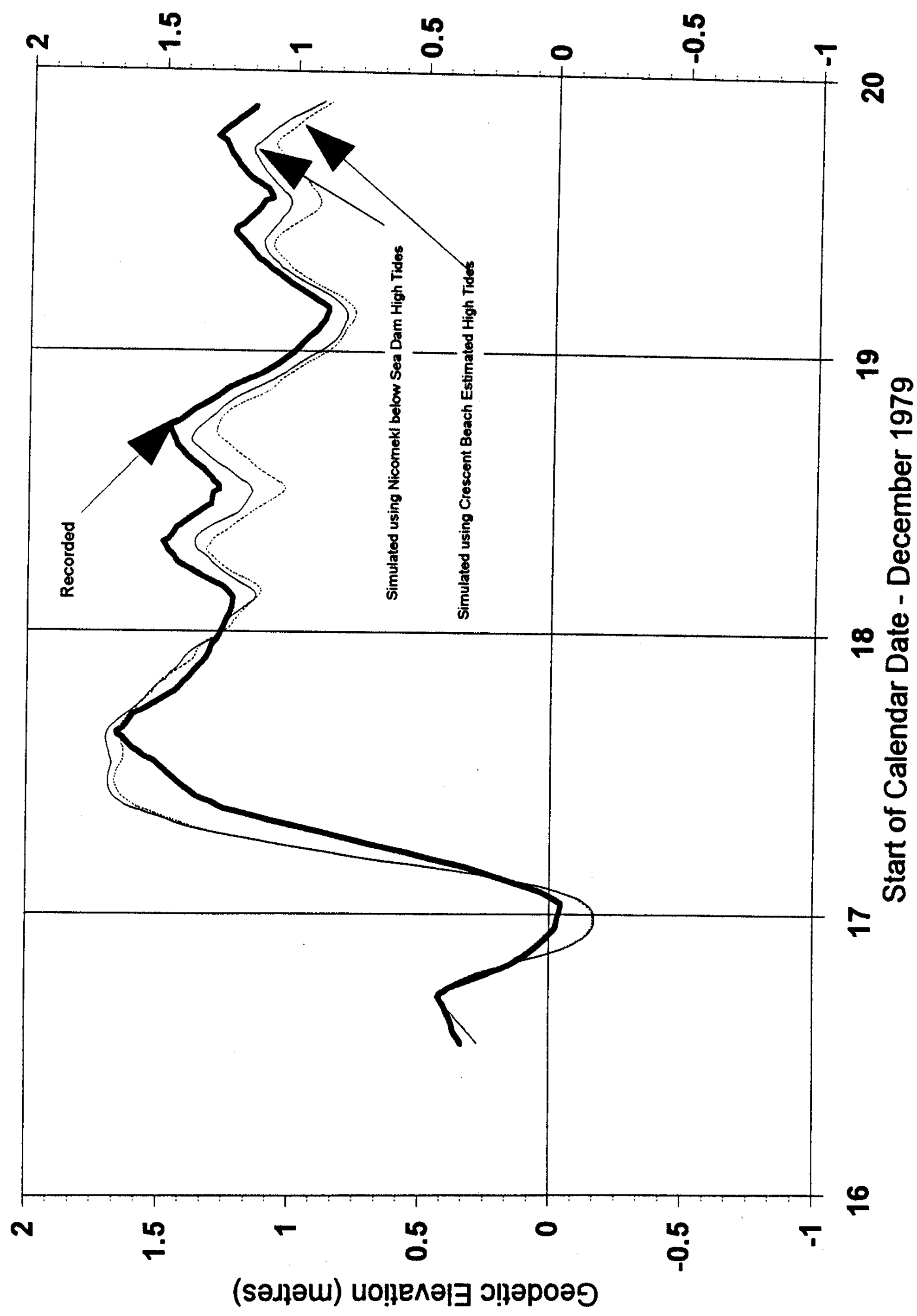


1979 CALIBRATION HYDROGRAPHS FOR SERPENTINE RIVER AT HIGHWAY 10

FIGURE 20

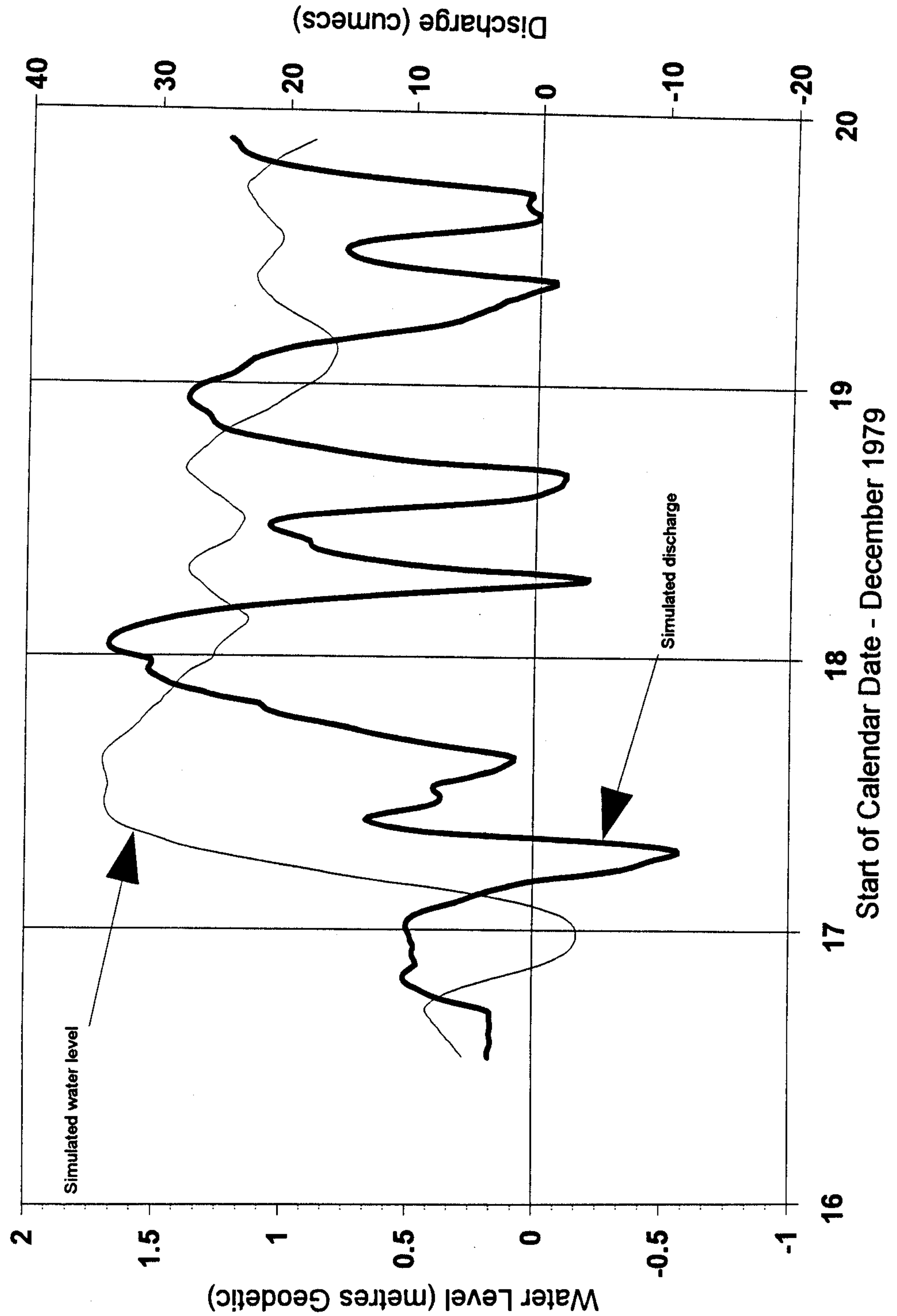
KPA ENGINEERING LTD.

FILE NO. 2997 DATE: OCT 1993



1979 CALIBRATION HYDROGRAPHS FOR SERPENTINE RIVER AT FRASER HIGHWAY

FIGURE 21



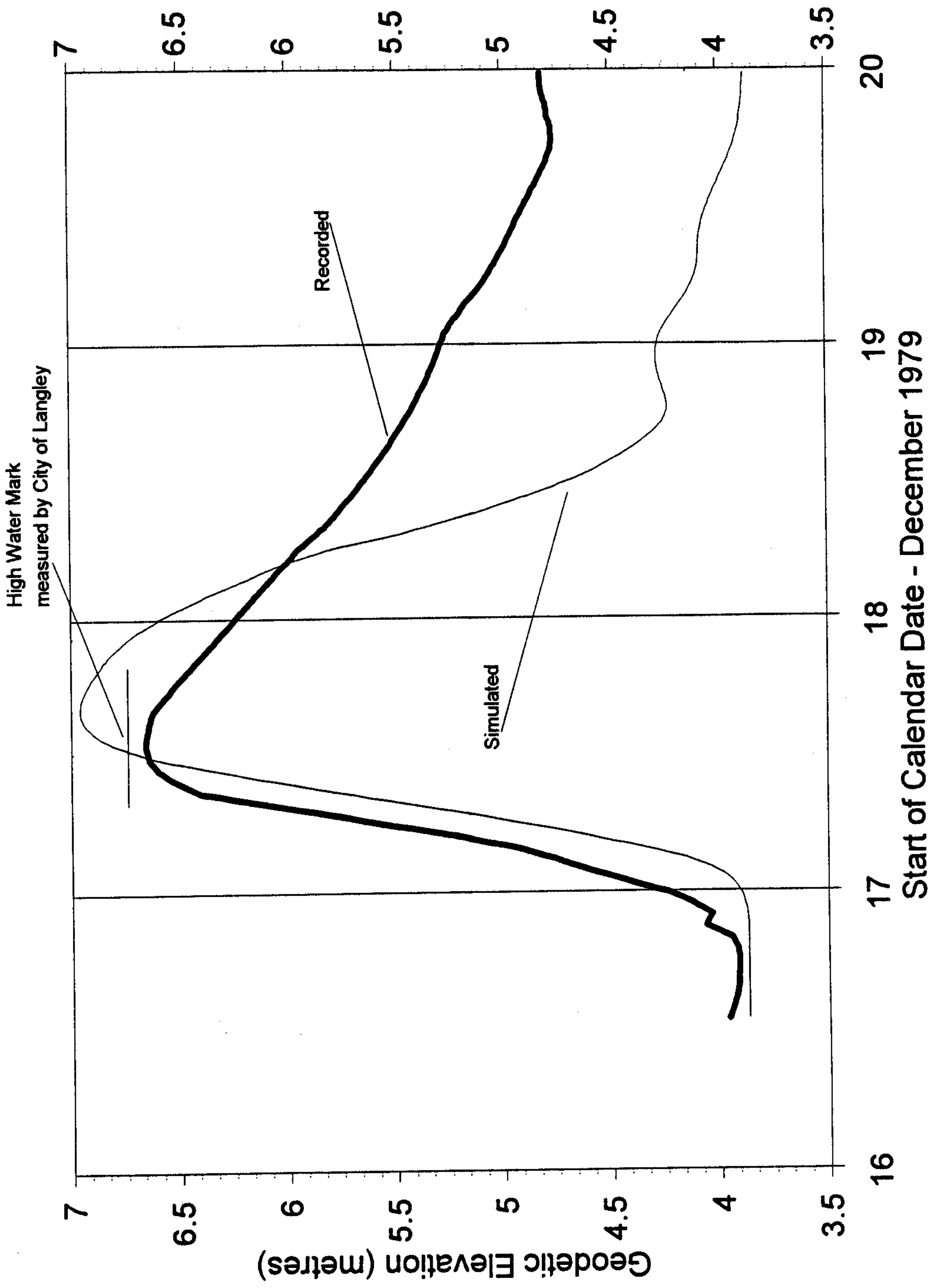
COMPARISON OF 1979 STAGE AND DISCHARGE HYDROGRAPHS FOR SERPENTINE RIVER AT FRASER HIGHWAY

FIGURE 22

FILE NO. 2997

DATE: OCT 1993

KPA ENGINEERING LTD.



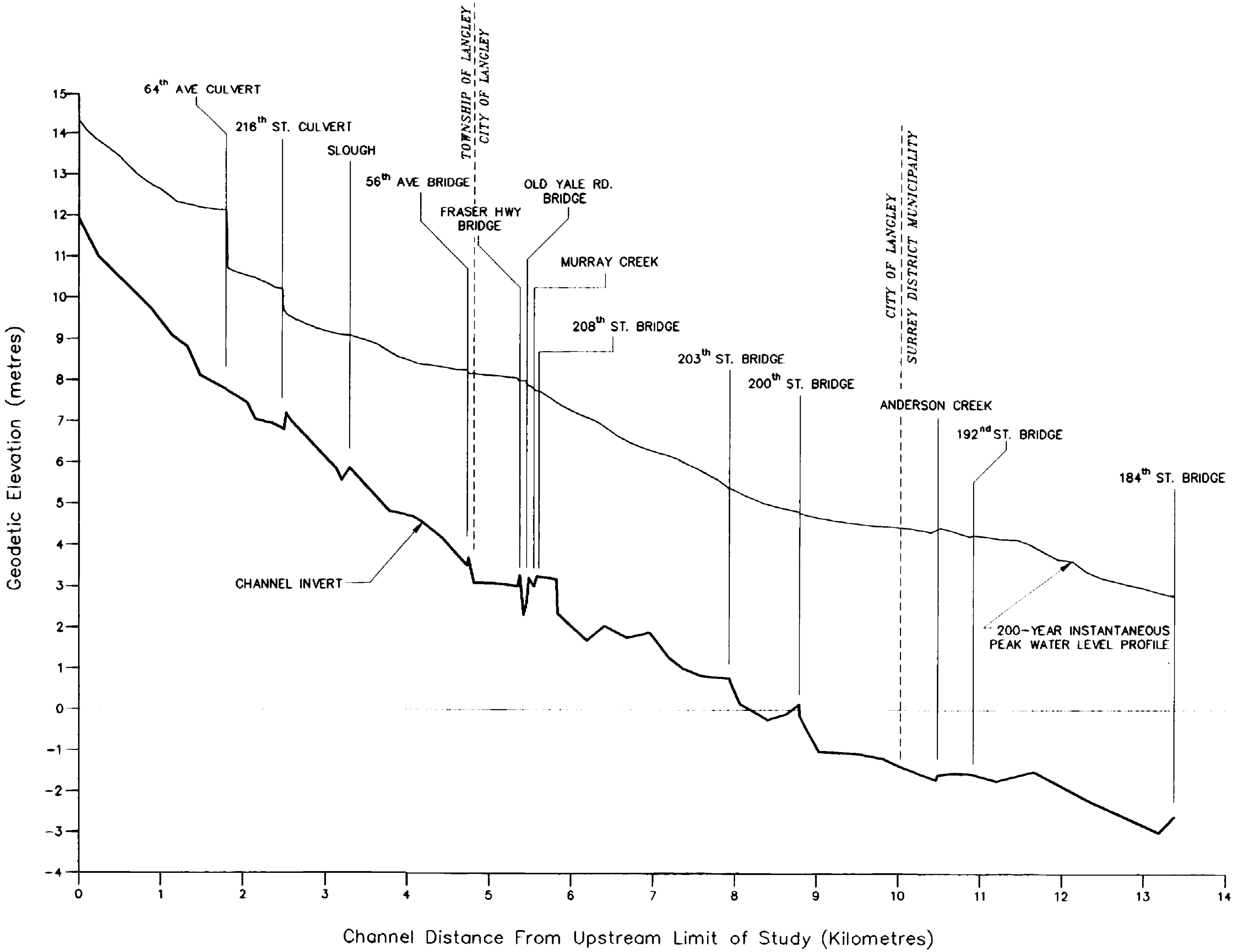
FILE NO. 2997

DATE: MARCH 1994

1979 CALIBRATION HYDROGRAPHS FOR NICOMEKL RIVER BELOW MURRAY CREEK

FIGURE 23

KPA ENGINEERING LTD.



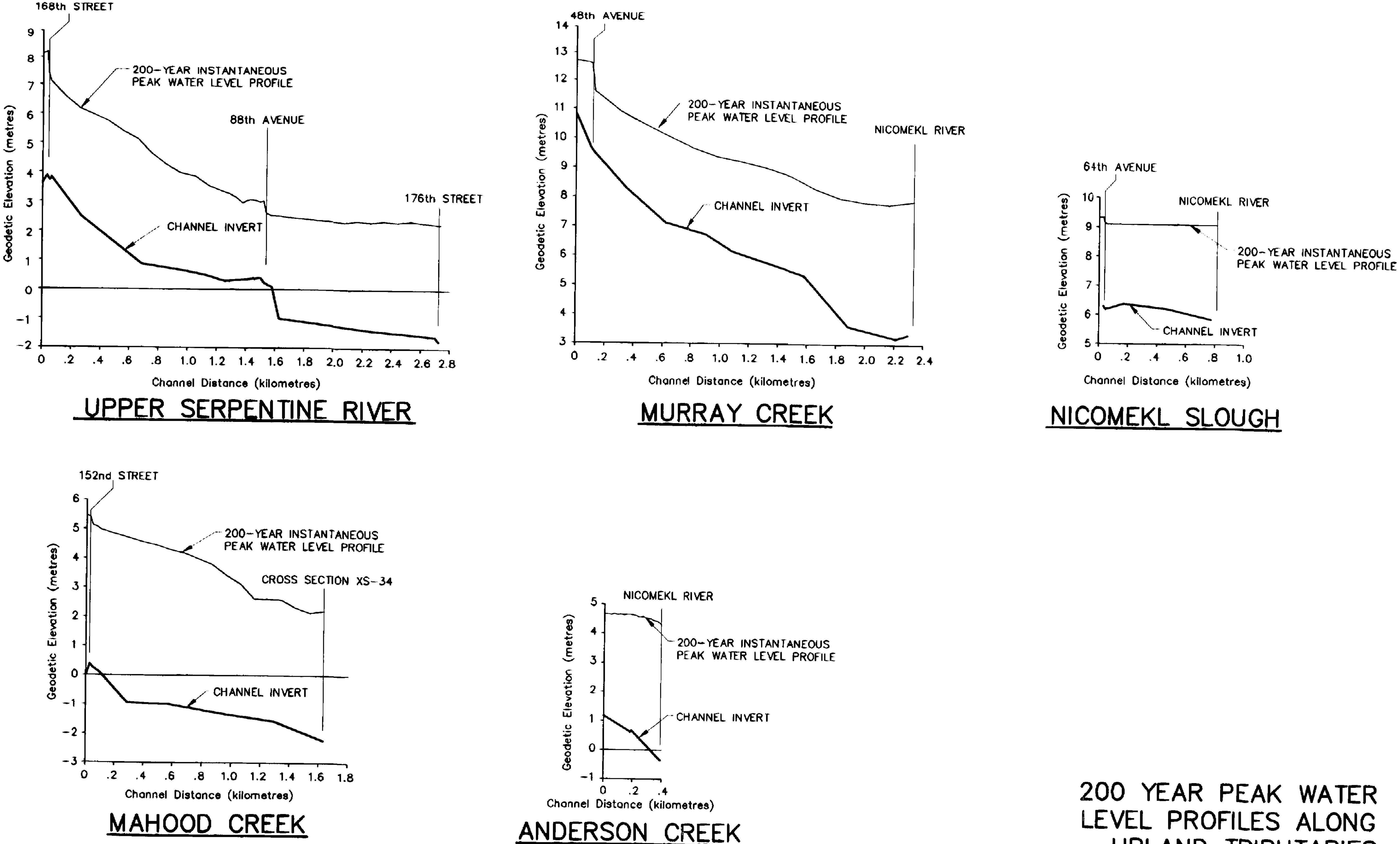
MARCH 1994

2937 E03

200 YEAR PEAK WATER LEVEL PROFILE ALONG UPPER NICOMEKL RIVER

FIGURE 24

KPA ENGINEERING LTD.



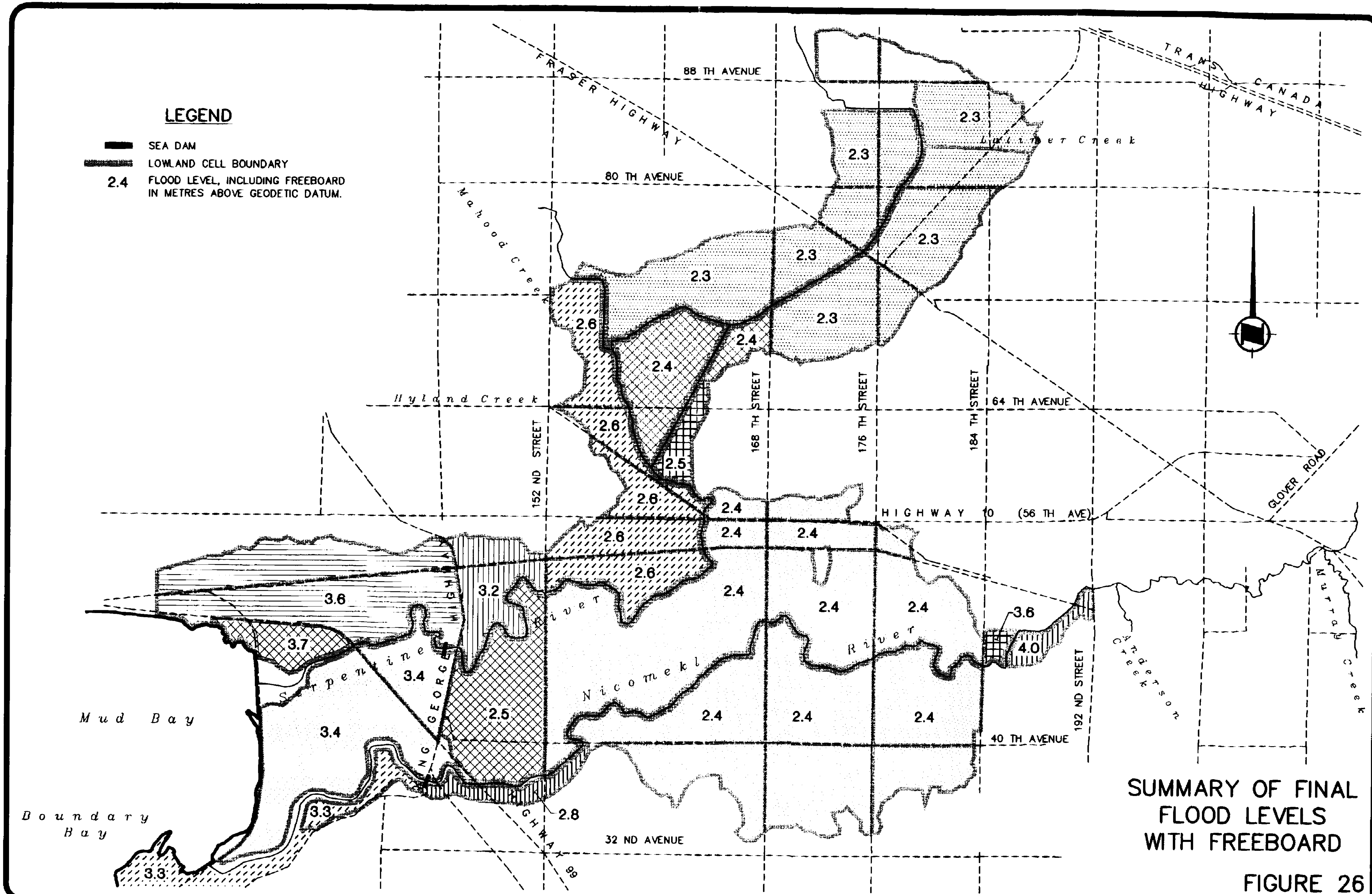
200 YEAR PEAK WATER LEVEL PROFILES ALONG UPLAND TRIBUTARIES

FIGURE 25

KPA ENGINEERING LTD.

MARCH 1994

2957



SUMMARY OF FINAL FLOOD LEVELS WITH FREEBOARD

FIGURE 26

KPA ENGINEERING LTD.

DATE : MARCH '994

FILE VC 2997

APPENDIX 1

HISTORICAL FLOOD PHOTOGRAPHS

January 1935 Flood Photo 1
Looking West from 176th Street South of Cloverdale to Ryan House

January 1935 Flood Photo 2
R. J. Livingston's Barn near Fry's Corner

January 1935 Flood Photo 3

Truck on 176th Street

January 1935 Flood Photo 4

Ed Hamre's Truck on 176th Street

December 1979 Flood Photo 5
Looking SE to Serpentine R. and 168th St.

24 February, 1983 Photo 6
Looking West along 83rd Avenue

December 1979 Flood Photo 7
Looking East along Highway 10 from 168th Street

January 1968 Flood

Looking west along the Nicomekl River in Langley.
Fraser Highway Bridge appears at bottom of photo

Photo 8

January 1968 Flood Looking Southeast to the Serpentine River Crossings with 176th Street. Nicomekl Lowlands are visible in the distance on right side of photo Photo 9

January 1968 Flood

Looking Northeast towards Cloverdale over Farmland between the Serpentine and Nicomekl Rivers

Photo 10

January 1968 Flood

Looking West along Serpentine River
toward Boundary Bay from 160th Street

Photo 11

ACKNOWLEDGEMENTS

Photos 1, 2, 3 and 4	Courtesy Surrey Museum Archives
Photos 5 and 7	Courtesy K. Wilson, B. C. Environment
Photo 6	Courtesy J. Brown, Surrey Dyking District
Photos 8, 9, 10 and 11	George Allen Aerial Photos

APPENDIX 2

SUPERSEDED MODEL SIMULATIONS

APPENDIX 2

As described in Section 5.3 of the Design Brief, two sets of "final" ONE-D model runs were completed during the course of the study. The first set was superseded by the second after adjusted extreme Boundary Bay water levels and additional Nicomekl high water data were received and subsequent model revisions made. Despite the decision to disregard the results from the first set of model runs, they retain some value as sensitivity runs. For this reason, these earlier results are presented here.

The results are summarized in three tables. Table A2-1 contains descriptions of each of the eight runs, which include two 200-year runoff flood scenarios and six sea flood scenarios. Table A2-2 summarizes the peak water levels attained in the lowland cells. Table A2-3 lists the peak levels reached at several locations along the lowland river channels.

Table A2-1
Summary Descriptions of the First Set of Final Model Runs

Run Number	Description
200-Year Runoff Flood Event Scenarios	
601	• simulation of the entire system within the study area • 200-year flood inflow hydrographs • tidal data for period 17 - 21 January, 1968 • no dyke breaches
602	• same as 601, but with a 60-m long river dyke breach added on south side of Nicomekl River downstream of 184th Street for entire duration of run • objective was to maximize flood levels south of Nicomekl River
Sea Dyke Breach Event Scenarios	
374	• simulation of the lowland area channels and storage cells excluding northeast Serpentine low-lands • tidal data containing extreme Boundary Bay water levels • 1979 flood inflows into the lowland area • three 100-m long breaches of the sea dykes • no river dyke breaches
375	• same as 374, but with - a 600 ha cell introduced to represent alternative portions of the northeast Serpentine low-lands - 30-m long dyke breaches added on both sides of Mahood Creek - 20-m long dyke breaches added on both sides of the north linear pond adjacent to 168th St. • objective was to maximize flood levels in northeast Serpentine lowlands
376	• same as 375, but with 50-m long breaches added on both sides of the Serpentine River near 160th Street • objective was to maximize flood levels in the lowlands between the two rivers
377	• same as 376, but with 30-m long breaches added on both sides of Nicomekl River due south from the Serpentine breaches • objective was to maximize flood levels south of the Nicomekl River
379	• same as 377, but with: - elimination of breaches across Serpentine River - enlargement of existing breaches across Nicomekl River to 60 m - addition of 60-m long breaches on both sides of Nicomekl River downstream of 184th St. • objective was to maximize flood levels south of Nicomekl River.
381	• same as 375, but with: - the 600 ha cell replaced with a 300 ha cell to represent smaller alternative portions of the northeast Serpentine lowlands - addition of a 60-m long breach into this 300 ha cell - removal of the breach and overflow element across the north linear pond adjacent to 168th St. - reapportionment of upland runoff to the 300 ha cell from the upstream end of the Serpentine River to maximize prefilling of cells by runoff prior to the sea dyke breach • objective was to maximize flood levels in the northeast Serpentine lowlands

Table A2-2

Summary of Simulated Peak Water Levels in
Lowland Cells for Eight Flood Scenarios
(First Set of Final Model Runs)

Lowland Cell Number	200-Year Flood Event		Extreme Ocean Water Levels and Sea Dyke Breach Event					
	Run 601	Run 602	Run 374	Run 375	Run 376	Run 377	Run 379	Run 381
5001	0.96	0.96	**3.60**	**3.60**	**3.60**	**3.60**	**3.60**	**3.60**
5002	0.68	0.68	**3.59**	**3.59**	**3.59**	**3.59**	**3.59**	**3.59**
5003	0.65	0.65	**3.68**	**3.68**	3.67	3.67	3.67	**3.68**
5004	1.08	1.08	**3.25**	**3.25**	**3.25**	**3.25**	**3.25**	**3.25**
5005	1.06	1.06	**3.44**	**3.44**	**3.44**	**3.44**	**3.44**	**3.44**
5006	1.07	1.07	**3.44**	**3.44**	**3.44**	**3.44**	**3.44**	**3.44**
5007	0.98	0.98	**3.15**	**3.15**	**3.15**	**3.15**	**3.15**	**3.15**
5008	0.79	0.79	**3.14**	**3.14**	3.13	3.13	**3.14**	**3.14**
5009	0.90	0.91	**2.67**	**2.67**	**2.67**	2.66	2.66	**2.67**
5010	1.31	1.31	**2.77**	2.73	2.66	2.67	2.74	2.72
5011	1.07	1.07	**2.80**	2.76	2.68	2.68	2.76	2.75
5012	0.70	0.74	**2.64**	2.63	2.58	2.46	2.37	**2.64**
5013	1.20	1.75	0.76	0.70	0.66	1.71	**1.98**	0.72
5014	1.15	1.15	**2.65**	2.62	2.57	2.33	2.05	2.63
5015	0.82	0.85	**2.64**	2.62	2.57	2.33	2.05	2.63
5016	1.28	1.28	**2.64**	2.62	2.57	2.33	2.05	2.63
5017	0.32	0.32	**2.64**	2.63	2.57	2.33	2.05	2.63
5018	0.25	0.24	**2.64**	2.63	2.57	2.33	2.14	2.63
5019	1.20	1.76	0.32	0.32	0.32	1.70	**1.98**	0.32
5020	2.18	2.18	**2.64**	2.63	2.57	2.32	2.05	2.63
5021	1.00	0.91	**2.64**	2.63	2.57	2.32	2.05	2.63
5022	1.74	**2.05**	0.96	0.94	0.90	1.71	1.98	0.95
5023	1.32	1.32	**2.76**	2.74	2.67	2.66	2.73	2.72
5024	1.67	1.67	**2.74**	2.70	2.64	2.64	2.70	2.69
5025	2.00	2.00	**2.68**	2.62	2.59	2.59	2.62	2.62
5026	0.89	0.89	**[2.62]**	**[2.62]**	[2.59]	[2.59]	[2.61]	[2.60]

Table A2-2

Summary of Simulated Peak Water Levels in
Lowland Cells for Eight Flood Scenarios
(First Set of Final Model Runs)

Lowland Cell Number	200-Year Flood Event		Extreme Ocean Water Levels and Sea Dyke Breach Event					
	Run 601	Run 602	Run 374	Run 375	Run 376	Run 377	Run 379	Run 381
5027	0.46	0.46	**[2.62]**	**[2.62]**	[2.59]	[2.59]	[2.61]	[2.60]
5028	1.61	1.61						
5029	1.81	1.80	2.10	2.02	2.09	1.93	2.02	**2.49***
5030	1.82	1.78		(1.22)	(1.38)	(1.14)	(1.19)	**(2.17)**
5031	0.36	0.36	1.07	(1.22)	(1.38)	(1.14)	(1.19)	**(2.17)**
5032	1.98	1.96		(1.22)	(1.38)	(1.14)	(1.19)	**(2.17)**
5033	1.99	1.98		(1.22)	(1.38)	(1.14)	(1.19)	**(2.17)**
5034	1.86	1.83		(1.22)	(1.38)	(1.14)	(1.19)	**(2.17)**
5035	1.85	1.82		(1.22)	(1.38)	(1.14)	(1.19)	**(2.17)**
5036	0.44	0.44		(1.22)	(1.38)	(1.14)	(1.19)	**(2.17)**
5037	1.98	1.98		(1.22)	(1.38)	(1.14)	(1.19)	**(2.17)**
5038	2.00	2.02		(1.22)	(1.38)	(1.14)	(1.19)	**(2.17)**
5039	**2.74**	**2.74**		(1.22)	(1.38)	(1.14)	(1.19)	(2.17)
5040	1.20	1.76	0.71	0.68	0.60	1.71	**1.98**	0.69
5041	1.20	1.76	0.33	0.32	0.32	1.71	**1.98**	0.32
5042	1.85	**2.04**	0.96	0.94	0.91	1.71	1.98	0.95
5043	2.07	2.03		(1.22)	(1.38)	(1.14)	(1.19)	**(2.17)**
5044	2.52	0.90	**2.64**	2.63	2.57	2.01	0.35	2.63
5045	**3.51**	3.39	1.23	1.23	1.23	1.23	1.25	1.23

* This peak water level is high because no overflow element over 168th Street dykes was included in this run.

[ ] Numbers in square brackets are based on the assumption that the cell is filled to the adjacent river level through a dyke breach.

() Numbers in parentheses are estimated peak water levels based on simulations using a large combined cell in the northeastern Serpentine lowlands.

Table A-3

Summary of Simulated Peak Water Levels
in Lowland River Channels for Eight Flood Scenarios
(First Set of Final Model Runs)

Lowland River Location	200-Year Flood Event		Extreme Ocean Water Levels and Sea Dyke Breach Event					
	Run 601	Run 602	Run 374	Run 375	Run 376	Run 377	Run 379	Run 381
Serpentine River								
A	1.98	1.97	**2.56**	2.54	2.53	2.49	2.54	2.37
B	1.97	1.97	**2.67**	2.66	2.62	2.62	2.66	2.65
C	1.97	1.97	**2.90**	**2.90**	2.80	2.80	**2.90**	**2.90**
D	1.97	1.97	3.05	3.05	3.05	3.05	3.05	**3.06**
E	1.96	1.96	3.60	3.60	3.60	3.60	3.60	**3.62**
Nicomekl River								
F	**2.82**	2.45	2.58	2.57	2.56	2.54	2.29	2.58
G	2.24	2.02	**2.56**	2.55	2.51	2.49	2.21	2.55
H	2.13	2.00	**2.57**	2.55	2.55	2.50	2.22	2.56
I	2.07	1.99	**2.58**	**2.58**	**2.58**	2.52	2.23	**2.58**
J	1.97	1.97	3.43	3.43	3.43	**3.44**	3.43	3.43

Description of River Locations

Serpentine River:

A Upstream side of Fraser Highway Bridge
B Upstream side of Highway 10 Bridge
C Upstream side of 152nd Street Bridge
D Upstream side of King George Highway Bridge
E Upstream side of Highway 99 Bridge

Nicomekl River:

F Upstream side of 184th Street Bridge
G Upstream side of 168th Street Bridge
H Upstream side of 40th Avenue Bridge
I Upstream side of King George Highway Bridge
J Midway between King George Highway and the Mouth.

These locations are also shown on Figure 17.