NORTH SHORE RAVINE STUDY

NOONS

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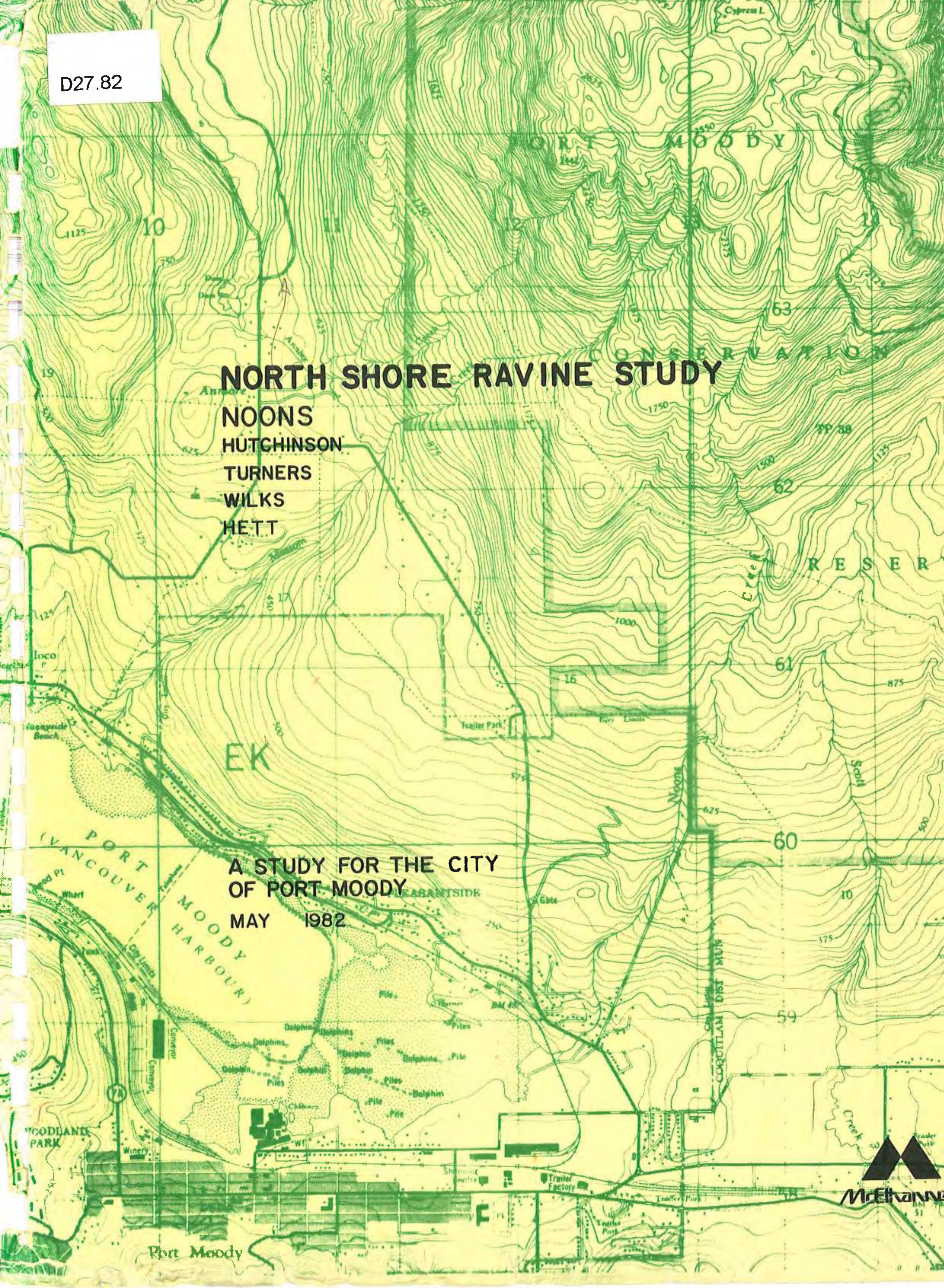
HETT

A STUDY FOR THE CITY OF PORT MOODY

MAY 1982



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McElhanney Surveying
& Engineering Ltd

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McElhanney

7 May 1982
Our File: 34269
Your File: N10-05

City of Port Moody
2425 St. Johns Street
Port Moody, British Columbia
V3H 2B2

Attention: Mr. John S. Paul, P.Eng.
North Shore Engineering Coordinator

Dear Sir:

North Shore Ravine Study

We are pleased to submit two draft copies of the North Shore Ravine Study, authorized under your letter of 17 March 1982, assignment number 86.

We are now completing our internal reviews and would appreciate your comments prior to final submission of the report.

Yours very truly

McELHANNEY SURVEYING & ENGINEERING LTD.

C. Newcomb

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HJV/jd

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ACKNOWLEDGEMENTS

We wish to acknowledge the assistance provided by the Greater Vancouver Sewerage and Drainage District and its staff in providing rainfall records for the area, and the Atmospheric Environment Service of Environment Canada. This data, along with stream discharge data furnished by Water Survey Canada and its staff, has been invaluable.

Assistance from Hardy Associates (1978) Ltd. for geotechnical data, and City staff for base plans and reports by other consultants have greatly assisted in the preparation of this report.

1. AUTHORIZATION

This study, North Shore Ravine Study, Assignment No. 86, was authorized by the City of Port Moody in their letter of 8 March 1982. The purpose and scope of the study is set out in the following section.

2. STUDY SCOPE

2.1 General

McElhanney Surveying & Engineering Ltd. was instructed to study those ravines on the North Shore which are proposed for dedication to the City of Port Moody as the North Shore develops. This study is subsequent to our study entitled North Shore Ravine Study, September 1980 (Assignment No. 34).

Due to the major shift in the senior government policy with respect to the watersheds under the previous study, a new study had to be undertaken which would assess the watersheds in terms of the current water management policies. Detailed description of both previous and current policies is summarized in Section 5.

The purpose of this study is as outlined below:

(i) Determine what steps would be necessary to ensure that any increase in storm water runoff from the developed areas of the North Shore would not cause an increased potential responsibility to the City from flooding or soil instability.

(ii) Review and evaluate improvements necessary under the previous and current water management policies, and recommend the most suitable alternative for adoption by Council.

2.2 Scope of Work

In order to provide the City with the information needed to protect downstream properties adjacent to the ravines from flooding or other associated events, the following items have been considered:

(1) Pre-development and post-development flows expected in each of the ravines during various storm return periods, up to a one in two hundred year return period.

(2) Remedial actions, if any, required to ensure that any increase in flow would not change present characteristics of the ravine and its present incidence of flooding under existing storm conditions.

(3) Remedial actions, if any, required to eliminate any existing or potential soil instability problems such as occurred on the south shore during the December 1979 storm.

The geotechnical study[1] carried out by Hardy Associates (1978) Ltd. has been used to establish the basic geotechnical parameters involved in the development concepts. The municipal services study[2] has been used to provide the basic background on servicing.

1. J.D. Madsen, A.E. Dahlman, "Geotechnical Study of North Shore, Port Moody, B.C.", (Hardy Associates (1978) Ltd.), August 1980.

2. A.E. Badke, "North Shore Report on Municipal Services, City of Port Moody", (Aplin & Martin Engineering Ltd.), April 1980.

The earlier storm drainage studies of 1974[3] and 1980[5] have been used for comparative purposes and for general background in earlier concepts.

The Neighbourhood Plan Report, Village 1,[4] has been used to provide the latest development concepts to be used in conjunction with the geotechnical study.

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3. T.Y. Miyanaga, "City of Port Moody, North Shore Area, Storm Drainage Study", (The UMA Group), August 1974.

4. R.W. Blasby, "The Villages, Port Moody, B.C., Neighbourhood Plan Report, Village 1", (Carma Developers Ltd. and IBI Group), June 1980.

5. D.F. McMaster, "North Shore Ravine Study", (McElhanney Surveying & Engineering Ltd.), September 1980.

3. WATERSHEDS AND LAND USE

The principal watersheds of the study are are defined by Mossom Creek on the westerly side and Noons Creek on the easterly side. These two major streams bracket the study area, and the four minor streams, Hutchinson, Turners, Wilks and Hett lying to the west of Noons Creek serve the minor watersheds between them. The Mossom watershed is virtually outside the City and to the west of the area under study. Drawing 1 of Appendix C illustrates the study area.

Fortunately, the major water courses originate within the Port Moody Conservation Reserve, and for the purposes of this study it is presumed that nothing will be done in the reserve that will alter the runoff characteristics within the reserve. Legal review of the conditions of the reserve should be made to ensure no future changes occur.

A geotechnical review of the Conservation Reserve was not made because the stream beds appear to have reached a condition of equilibrium over the years, but such a review should be a prerequisite if major changes are made in the use of the Conservation Reserve.

The development plans for the Port Moody North Shore call for an environmental reserve to be placed on the ravines and stream beds within the City from the top of the embankment, as defined in the geotechnical studies[1] and discussed later in this report. The environmental reserve is to be defined by legal survey plans, and development bylaws so that the conditions of equilibrium in the stream beds and banks are not changed by the ultimate development within the City limits.

It should be noted that the watersheds for the minor streams are not rigidly defined by topographical boundaries in the area between the City limits and the Conservation Reserve. The runoff from these areas, approximately 119 acres, could be diverted in whole or in part by future development.

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1 Ibid.

The concepts promulgated in previous studies[1 & 4] envisage that no expenditures or work would be undertaken within the reserve to preserve the natural environment under future runoff conditions. Similarly, no changes would be made within the reserve by filling, excavating, or clearing to change the natural conditions except for erosion and slope stability control works where necessary for both pre-development and post-development conditions. Access to the reserve for necessary utility easements would need to be avoided or at least carefully regulated to preclude unacceptable changes to natural conditions to meet these concepts.

Table number 1 summarizes the basic characteristics of the five ravines under study.

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1 Ibid.

4 Ibid.

TABLE 1

RAVINE CHARACTERISTICS

WATERSHED	WATERSHED AREA		AVERAGE SLOPE	LENGTH OF MAIN WATER COURSE		ENVIRONMENTAL RESERVE AREA	
	(Hectares)	(Acres)	(Percent)	(Metres)	(Feet)	(Hectares)	(Acres)
Noons Creek to Ioco Road	415	1,024	12.3	6,750	22,140	353	873
Tributary to Noons Creek	105	260	17.2	3,500	11,480	102	252
Hutchinson Creek to Ioco Road	69	171	14.0	1,625	5,330	12	30
Turners Creek to Ioco Road	78	193	14.4	2,125	6,970	38	93
Wilks Creek to Ioco Road	43	105	16.9	1,625	5,330	5	63
Hett Creek to Ioco Road	49	120	15.2	1,900	6,230	23	56

NOTE: The environmental reserve area includes area outside the Port Moody City Limits and area in the natural preserves in the post-development stage within the City.

4. HYDROLOGY

4.1 Rainfall

Reasonably good records are available to develop isohyets* showing the annual rainfall over the watersheds and the north shore of the City. These are shown in Figure 1, Rainfall Isohyets and Gauge Stations, with the Noons Creek drainage basin added.

The North Shore area over this basin increases in elevation from south to north by 975 metres (3,200 feet) in the relatively short distance of 6,750 metres (4.2 miles). The variation in annual rainfall from 78 to 115 inches (1,981-2,921 mm) or 37 inches (940 mm) in total is substantial. The combination of warm winter rains falling on snow in the watershed can significantly affect runoff in the locality.

The Greater Vancouver Sewerage & Drainage District (GVS & DD) has prepared intensity-duration-frequency (IDF) curves for Station 7 at the Port Moody Sewage Pump Station. Records extend over a 20 year period, 1959 to 1978, for winter intensities from October to April. Since these intensities apply at Station 7, the intensities must be adjusted to represent average intensities over Noons Creek watershed.

A spatial adjustment factor of 1.22 was calculated as the ratio of the weighted annual rainfall for the Noons Creek watershed to the annual rainfall at the gauge. It was determined to be a realistic estimate for Noons Creek. No adjustment for the smaller watersheds is needed because they are all governed by the same isohyet, close to the gauge.

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* Rainfall intensity contours.

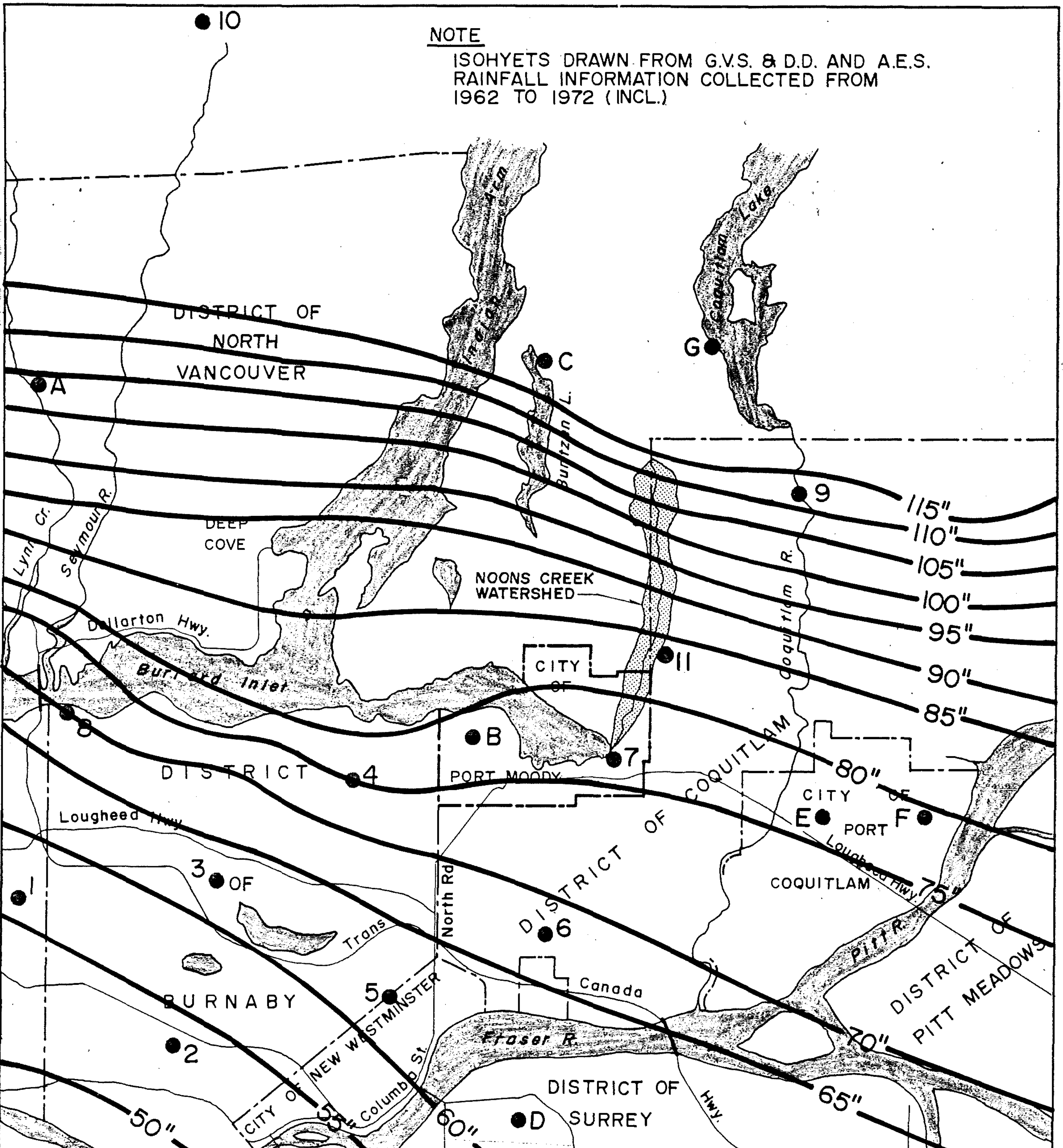
The GVS & DD curves for Station 7 provide for return periods of 2, 5, 10 and 25 years. Additional curves for 50, 100 and 200 years were extrapolated graphically using the Gumbel Extreme Probability plotting and checked using the Gumbel's mathematical formula. The graphical plotting for the extrapolated curves produced results between 0% to 7% higher than the mathematical formula for the 2 hour and the 5 minutes rainfall respectively. The graphical results were used since this data produced the higher intensities and compared more favourably with the GVS & DD curves available for the lower return periods. This data is shown in Figure 2, Rainfall Intensity-Duration-Frequency.

The foregoing statistical analysis produces an estimate of rainfall intensities for storm events of up to a 200 year return period with only 20 years of data. The results, when used for return periods in excess of 25 years, must be considered as a statistical estimate of intensity-duration-frequencies. It is believed that these curves include an adequate factor of safety for calculations of runoff flows.

All-year intensities are available for the period 1959-1971, 13 years, when a gauge was installed at the Port Moody Firehall. The all-year intensities are higher because they include the higher summer rainfall intensities. Experience suggests that the combination of the all-year intensities and corresponding lower runoff coefficients results in runoff volumes virtually identical to the combination of the winter intensities and corresponding higher runoff coefficients. For this area, the winter IDF curves used with winter runoff coefficients, are generally considered to be most suitable for design.

In general, in the study area, the winter runoff coefficients range from 0.73 to 0.91 for developed urban areas for a 10 year return period to 0.87 to 1.09 for a 200 year return period.

The higher winter runoff coefficients should be used with the winter IDF curves in Figure 2.



G.V.S. & D.D. RAIN GAUGES STATION NAME	GEODETIC ELEVATION
1. RENFREW ELEMENTARY SCHOOL	290'
2 McPHERSON PARK JUNIOR SEC.	420'
3. BURNABY CENTRAL-SPERLING P.S.	60'
4 TRANS MTN. BURNABY TERM.	700'
5. WESTBURNCO RESERVOIR	415'
6. COQUITLAM MUNICIPAL HALL	90'
7. PORT MOODY S.P.S.	26'
8. VANCOUVER HEIGHTS RESERV.	365'
9. SEYMOUR FALLS DAM	755'
10. COQUITLAM CHLORINATION HOUSE	390'
11. MERIDIAN SUB STATION	

ATMOSPHERIC ENVIRONMENT SERVICE RAIN GAUGES STATION NAME	GEODETIC ELEVATION
A. NORTH VAN.-LYNN CREEK	625'
B. PORT MOODY GULF OIL RFY.	425'
C. BUNTZEN LAKE	430'
D. SURREY KWATLEN PARK	305'
E. PORT COQUITLAM CITY YARD	22'
F. PORT COQ.-PRAIRIE ROAD	15'
G. B.C. HYDRO-COQUITLAM LAKE	530'

RAINFALL ISOHYETS
AND GAUGE STATIONS

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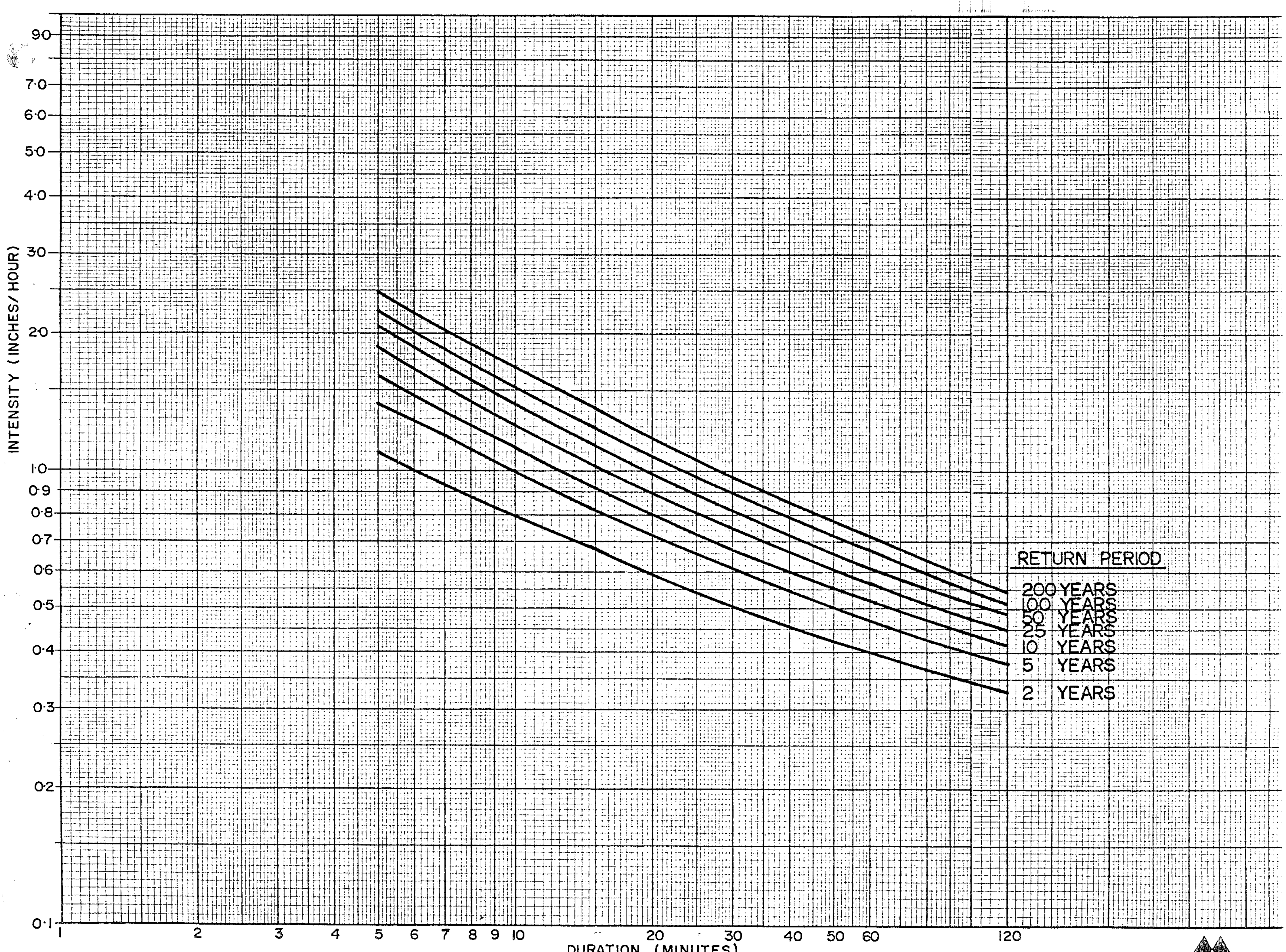
FIGURE I



RAINFALL INTENSITY-DURATION-FREQUENCY

WINTER STORMS - OCTOBER TO APRIL

STATION No. 7

PORT MOODY PUMPING STATION

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FIGURE 2

4.2 Stream Discharge

4.2.1 **Predicted Discharge**

The Water Survey of Canada (WSC) maintains a recording gauge station on Noons Creek at the Meredian Substation, Station No. 08GA065, covering an area of approximately 259 hectares (640 acres or 1 square mile). The measurements at this station were used to verify approximate stream discharges calculated from rainfall data.

Prior to 1977, a manual gauge, Station No. 08GA052, was in operation on Noons Creek at Ioco Road, which covered the total watershed of 725 hectares (1,792 acres or 2.8 square miles). This gauge would not show instantaneous peak flows, but would show the daily discharge at the time of reading the gauge for a maximum day.

For the purpose of this study and assessment, the measurements are with some limitations, considered to be the maximum daily discharges* or the maximum daily flows*. The allowance to be made for instantaneous peaks is discussed later.

For the 13 year continuous period of record from 1960 to 1972 at the Station No. 08GA052, the Noons Creek daily discharge was analyzed using Gumbel's extreme value distribution to determine the maximum daily discharge for various return periods.

The maximum daily discharges were computed as shown in Table 2, Maximum Daily Discharges, and plotted in Figure 3.

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* Average daily flows on maximum day in any water year.

For comparison purposes, the standard IDF curves recorded at the Port Moody Pumping Station were used to calculate the runoff using values for C equal to 0.30 and 0.60. These runoff values have been tabulated in Table 3 and plotted on Figure 3. It may be seen that the value of C should increase as the intensity of the storm increases with longer return periods. The value of C should thus vary with the intensity of the storm and the length of the return period. Using a varying value for the runoff coefficient, C_N, as tabulated in Table 3, in the Noons Creek watershed, results were obtained which correspond to actual values of the stream discharge. This analysis simplifies what is actually a much more complex relationship, but does provide a practical reconciliation of the hydrologic data available.

Instantaneous peak values in all cases will be larger than these values.

It must also be borne in mind that these storms do not include data from 1935 and 1958-59, when severe storm, following heavy snowfall, produced extremely high runoffs. Unfortunately, data for this basin in these years is not available.

Because of the large number of variables involved in the hydrologic evaluation of a watershed, and the limited data, the analysis can never be considered fully defined. However, the predicted values must be realistic so that future storm damage may be minimized.

For further comparison and verification of the predicted maximum daily discharge the data was also analyzed using Pearson Type III distribution.

The results of the various analyses are plotted on Figure 3, Predicted Maximum Daily Discharge for Noons Creek. Actual typical gauging over the period of record are shown in Table 4, Noons Creek Gauge 08GA052.

TABLE 2

MAXIMUM DAILY DISCHARGES
NOONS CREEK AT IOCO ROAD

RETURN PERIOD (Years)		2	5	10	25	50	100	200
December	m^3/s	4.47	7.88	10.14	13.0	15.1	17.3	19.4
	cfs	158	278	358	459	534	609	687
January	m^3/s	5.1	8.7	11.1	14.1	16.3	18.5	21.0
	cfs	180	307	391	497	576	654	741
October	m^3/s	7.0	11.3	14.1	17.6	20.2	22.9	25.7
to	cfs	248	398	497	623	716	808	909
April								

Calculated from measurement records of Water Survey of Canada,
Gauge 08GA052.

CREEK		AREA (ac.)	t_c (min.)	C	RAINFALL INTENSITY (in/hr)							PEAK FLOW (cfs)						
					2 YR.	5 YR.	10 YR.	25 YR.	50 YR.	100 YR.	200 YR.	2 YR.	5 YR.	10 YR.	25 YR.	50 YR.	100 YR.	200 YR.
NOONS	undev. area	1209	39	0·30	0·56	0·67	0·74	0·82	0·89	0·98	1·05	203	243	268	297	323	355	381
				0·60								406	486	537	595	646	711	762
				C_N*								251	397	501	625	710	805	914
	dev. area	75		C_N*								16	25	31	39	44	50	57
HUTCHINSON		124	15	0·30	0·67	0·82	0·90	0·99	1·10	1·17	1·27	25	31	33	37	41	44	47
				0·60								50	61	67	74	82	87	94
				C_N*								31	50	62	77	90	99	113
TURNERS		193	18	0·30	0·62	0·76	0·84	0·91	1·00	1·06	1·15	36	44	49	53	58	61	67
				0·60								72	88	97	105	116	123	133
				C_N*								44	72	91	111	127	139	160
WILKS		105	14	0·30	0·69	0·85	0·94	1·04	1·14	1·20	1·32	22	27	30	33	36	38	42
				0·60								43	54	59	66	72	76	83
				C_N*								27	44	55	69	79	86	100
HETT		120	16	0·30	0·65	0·80	0·88	0·96	1·06	1·13	1·22	23	29	32	35	38	41	44
				0·60								47	58	63	70	76	81	88
				C_N*								29	47	59	73	84	92	105

NOTE: IDF CURVES AT PORT MOODY PUMPING STATION EXCEPT FOR NOONS CREEK WHERE THE SPATIAL CORRECTION IS APPLIED TO THE IDF CURVE

* C_N — RUN-OFF COEFFICIENT FOR NATURAL AREAS (TABLE 7)

	2 YR.	5 YR.	10 YR.	25 YR.	50 YR.	100 YR.	200 YR.
*C_N	0·37	0·49	0·56	0·63	0·66	0·68	0·72

PRE-DEVELOPMENT PEAK FLOWS

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TABLE 3

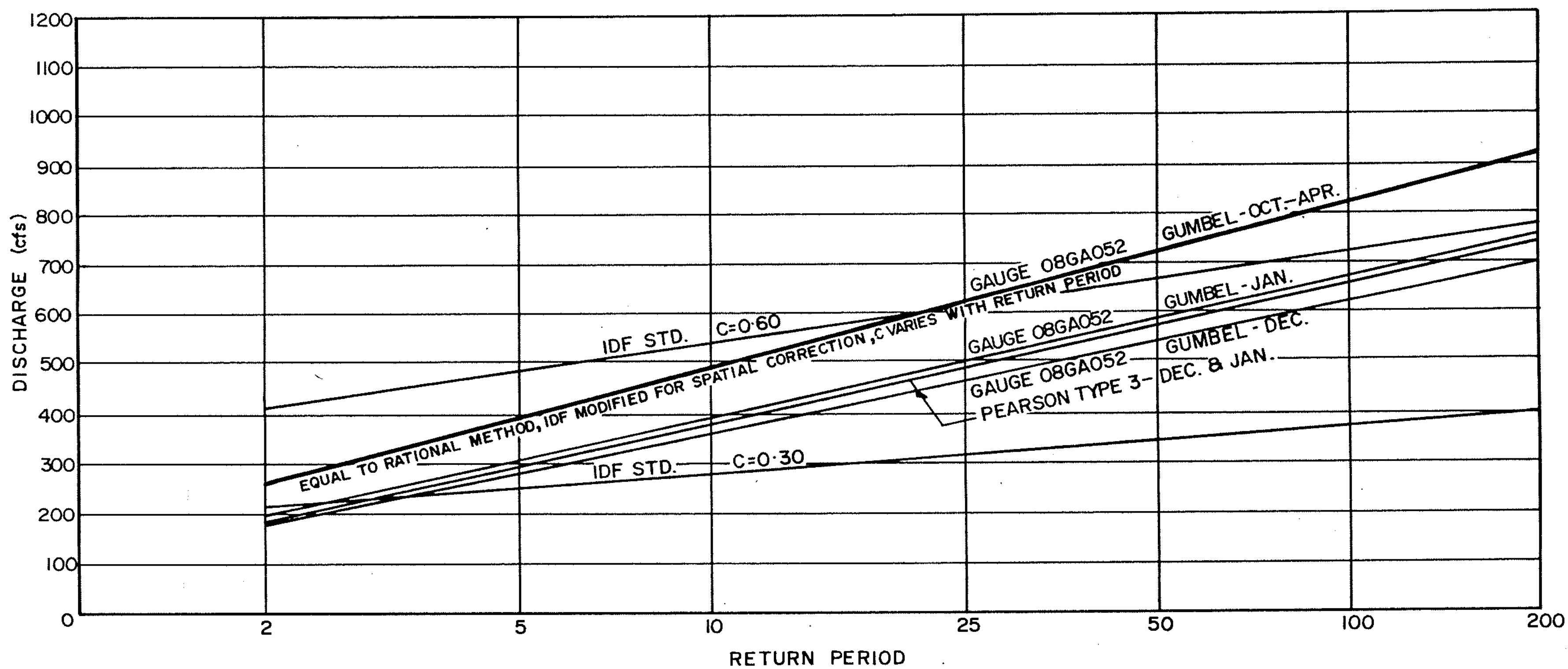
TABLE 4

NOONS CREEK GAUGE 08GA052 AT IOCO ROAD
2.8 SQUARE MILES

MAXIMUM DAILY DISCHARGE*

YEAR	CFS	M^3/S
1960	120	3.40
1961	564	15.96
1962	622	17.60
1963	430	12.17
1964	282	7.98
1965	146	4.13
1966	371	10.50
1967	173	4.90
1968	245	6.94
1969	127	3.60
1970	137	3.88
1971	186	5.27
1972	190	5.38
1973	—	—
1974	181	5.13
1975	216	6.12
1976	—	—

* Measured by manual gauge and does not represent maximum instantaneous flows.



PREDICTED MAXIMUM DAILY DISCHARGE - NOONS CREEK

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FIGURE 3

4.2.2 **Instantaneous Peaks**

The peak instantaneous discharge for the various return periods is important in that the short term peaks will occur and these values need to be used to evaluate the capacity of culverts or bridges on Noons Creek.

The available recorded data is insufficient to evaluate instantaneous peaks with relation to the maximum daily discharge established in Table 2.

Instantaneous peaks would, in most cases, have been larger than the recorded maximum daily values. The actual ratio of instantaneous peak to maximum daily discharge is unknown at this time although monitoring is now in progress by Water Survey of Canada, which should result in better knowledge of this ratio in the next few years. Since this value is unknown, we have utilized a factor of 1.25 applied to the maximum daily value to obtain instantaneous peaks. This is based on judgment from a study of the data available and cannot be taken as absolute. Following the monitoring program, this value will require reassessment. These are included in Table 5.

The rate of peak instantaneous flows to the average daily flow has been obtained by the evaluation of peak flows from the new recording gauge at Meridian Substation, and the evaluation of two major storms, 17 January 1977 and 17-18 December 1979, for the watershed using the unit hydrograph technique.

The ratio of the instantaneous peak to the average daily discharge for the two storms studied are tabled in Table 6, Ratio Instantaneous Peak to Average Daily Discharge. Note that this is average daily discharge, not maximum daily discharge as discussed elsewhere.

The instantaneous peaks have a very short duration, for example possibly 15 minutes. The storm records show that these peaks may rise above the average daily flow over a 3 hour period and fall over a 6 hour period to the

TABLE 5

INSTANTANEOUS PEAKS

RETURN PERIOD (Years)		2	5	10	25	50	100	200
Maximum Daily Discharge	m^3/s	7.0	11.3	14.1	17.6	20.2	22.9	25.7
	cfs	248	398	497	623	716	808	909
Instantaneous Peak Discharge	m^3/s	8.78	14.11	17.59	22.07	25.35	28.61	32.13
	cfs	310	498	621	779	895	1010	1136

Instantaneous peak discharge is calculated from applying a factor of 1.25 to the maximum daily discharge. This is an assumed adjustment factor requiring reassessment following monitoring now in progress.

TABLE 6

RATIO INSTANTANEOUS PEAK TO AVERAGE DAILY DISCHARGE

STORM EVENT	RATIO	PEAK INSTANTAENOUS / AVERAGE DAILY DISCHARGE
17 January 1977	298 / 134	2.22
17-18 December 1979	665 / 312	2.13
Average Value		2.2

NOTE: Average daily discharge is utilized here, not maximum daily discharge. This should be co-related in the future when a longer period of record is available from the recording gauge.

average daily value. This represents a very general characterization of an instantaneous peak for this stream and is typical of the storm of 17 December 1979.

The instantaneous peaks for the Noons Creek watershed are considered to be affected more by rainfall and basin characteristics rather than those of runoff factor.

4.2.3 **Runoff Coefficients for Natural Areas**

The basin efficiency was examined with the view to evaluating realistic runoff factors. The storm runoff for December 1971 was selected as representative of runoff conditions that may be expected to occur with a 2 year return period.

The total monthly precipitation of flow was considered to be the total runoff that could be expected, although some contribution from snow or groundwater would make this figure somewhat larger.

The total monthly discharge of 6.23 m^3/s (220.2 cfs), which is equivalent to 110 mm (4.335 inches) of rainfall, in relation to the total precipitation for the month of 273 mm (10.74 inches), yields a basin efficiency equal to $\frac{110 \text{ mm}}{273 \text{ mm}}$ or 0.403.

This figure compares favourably with the coefficients of runoff calculated in Table 7, Run-off Coefficients for Natural Areas. The coefficients calculated in the table refer to the basin efficiencies or runoff coefficients of storms with return periods of 2 to 200 years. As may be expected, storms of greater intensities, longer duration, and less frequency produce a higher runoff coefficient.

RETURN PERIOD	IDF * PRECIPITATION		SPATIAL FACTOR	CORRECTED PRECIPITATION		NOONS CREEK PEAK DAILY DISCHARGE		C_N
	mm.	in.		mm.	in.	m^3/s	cfs	
200	22	0·86	1·22	27	1·05	25·7	909	0·72
100	20	0·80	1·22	25	0·98	22·9	808	0·68
50	19	0·73	1·22	23	0·89	20·2	716	0·66
25	17	0·67	1·22	21	0·82	17·6	623	0·63
10	15	0·61	1·22	19	0·74	14·1	497	0·56
5	14	0·55	1·22	17	0·67	11·3	398	0·49
2	12	0·46	1·22	14	0·56	7·0	248	0·37

* 39 MINUTES CONCENTRATION TIME

RUN-OFF COEFFICIENTS FOR NATURAL AREAS

Based on this analysis, the runoff factor for natural areas has a coefficient of 0.37 for a design return period of 2 years, 0.56 for design return period of 10 years, and 0.68 for design return period of 100 years.

4.2.4 Runoff Coefficients for Developed Areas

The runoff coefficients currently in use in the City are tabled below and considered applicable to storms having a return period of 10 years. These are used with the City's all year IDF curve.

AREA DESCRIPTION	COEFFICIENT
Roads	.85
Multi-Family Lots	.75
Single-Family Lots	.65
Estate Lots	.55
Community Park	.35
Environmental Preserve	.30

The average runoff coefficients for the post-development conditions have been calculated for the various watersheds. Table 8 shows these coefficients for use with the all year IDF curves, along with their equivalents for use with the winter IDF curves.

These average runoff coefficients, having been derived from coefficients applicable to storms of the 10 year period, are consequently also applicable to the 10 year period.

For more severe storms, especially those combined with warm rains falling on previous snowfalls, higher runoff coefficients would result.

In the absence of hydrologic data for stream discharge generated by developed areas which could be correlated to available precipitation data, adjusted coefficients of runoff have been arrived at based on the coefficients for natural areas developed in the previous section.

Due to the effects of urbanization, mainly reduction of pervious areas and reduction of surface detention, the adjusted coefficients for developed areas vary less than those for natural areas as could be expected. The correlation analysis simplifies what actually is a much more complex relationship. The result of this analysis cannot be taken as absolute, however, it does establish values that are realistic and comparable to those of the Rawn Report[6]. These coefficients, tabulated in Table 9, have been used in subsequent calculations.

6. A.M. Rawn, "Sewage and Drainage of the Greater Vancouver Area, British Columbia", Greater Vancouver Sewerage and Drainage District, 1953.

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	A (ac)	C	AC	A (ac)	C	AC	A (ac)	C	AC	A (ac)	C	AC	A (ac)	C	AC
ROADS	9.6	.85	8.16	13.5	.85	11.48	10.7	.85	9.10	8.7	.85	7.4	6.5	.85	5.53
MULTI - FAMILY LOTS	74.8	.75	56.1	—	—	—	7.8	.75	5.85	13.6	.75	10.2	—	—	—
SINGLE FAMILY LOTS	74.1	.65	48.17	80.4	.65	52.26	45.7	.65	29.71	5.5	.65	3.58	44.7	.65	29.1
ESTATE LOTS	—	—	—	0.6	.55	.33	16.4	.55	9.02	5.0	.55	2.75	12.9	.55	7.1
COMMUNITY PARK	—	—	—	—	—	—	19.5	.35	6.83	9.1	.35	3.19	—	—	—
PRESERVES	45.3	.30	13.59	29.7	.30	8.91	26.6	.30	7.98	31.2	.30	9.36	34.7	.30	10.41
TOTAL	203.8	—	126.03	124.2	—	72.98	126.7	—	68.49	73.1	—	36.48	98.8	—	52.1
AVERAGE RUNOFF COEFFICIENT (C) ALL YEAR		0.62			0.59			0.54			0.50			0.53	
AVERAGE RUNOFF COEFFICIENT (C) WINTER		0.91			0.86			0.79			0.73			0.78	

CALCULATION OF AVERAGE RUN-OFF COEFFICIENTS FOR DEVELOPED AREAS

NOTE : VALUES OF C ABOVE FOR EACH AREA REPRESENT ALL-YEAR VALUES AND WERE TAKEN FROM THE APRIL 1980 REPORT ON MUNICIPAL SERVICES BY APLIN & MARTIN ENGINEERING LTD.

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TABLE 8

CREEK	AVERAGE RUN-OFF COEFF. (C)		ADJUSTED RUN-OFF COEFFICIENT (C_A)							
	10 YR. RET. PERIOD		25 YR. RET. PERIOD		50 YR. RET. PERIOD		100 YR. RET. PERIOD		200 YR. RET. PERIOD	
	ALL YEAR	WINTER	ALL YEAR	WINTER	ALL YEAR	WINTER	ALL YEAR	WINTER	ALL YEAR	WINTER
NOONS	0·62	0·91	0·67	0·99	0·70	1·02	0·71	1·05	0·74	1·09
HUTCHINSON	0·59	0·86	0·64	0·93	0·66	0·97	0·68	0·99	0·71	1·03
TURNERS	0·54	0·79	0·59	0·86	0·61	0·89	0·62	0·91	0·65	0·94
WILKS	0·50	0·73	0·54	0·79	0·56	0·82	0·57	0·84	0·60	0·87
HETT	0·53	0·78	0·58	0·85	0·60	0·88	0·61	0·90	0·63	0·93
AVERAGE COEFF. FOR PROPOSED DEVELOP. AREA	0·56	0·82	0·60	0·88	0·63	0·92	0·64	0·94	0·67	0·97

ADJUSTED RUN-OFF COEFFICIENTS
FOR DEVELOPED AREAS

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TABLE 9

5. ANALYSIS OF PREDEVELOPMENT AND POST-DEVELOPMENT FLOWS

The development concept for storm water discharges on the North Shore has been set out in earlier studies and reports. The preceding water management policies were based on the premise that all storm water originating from storms with a return period of 10 years or less will be discharged directly to Burrard Inlet by a system of storm sewers or designated floodpaths leading directly to the Inlet . For storms exceeding this intensity, the surplus drainage may be directed to the nearest natural watercourse.

The current water management policies were established in January 1982 and resulted in the following amendments:

(a) Minor Creeks

There is now no restriction to the flow that could be directed to these creeks, except for a minimum base flow of 1000 gallons/day on Hutchinson Creek for the duration of the water-license held on this water course. The possibility of an inter-basin flow transfer now exists, even though it is in principle discouraged by the Water Management Branch. Furthermore, base flows are to be maintained in order not to change the biological balance of natural preserves.

(b) Noons Creek

The previous policies are to remain unchanged except that an inter-basin flow transfer of runoff generated by storms with a return period of 10 years or less is now permitted.

The analysis for the various watersheds for the 10, 100 and 200 year return periods is shown in Tables 10 through 14. The analysis presumes that no changes would be made to the area outside the City, and that all development would take place within the City boundary. This may not necessarily be the case, and in fact, the City boundary could be extended northerly. In case of such boundary extension, a decrease in discharge in the Noons Creek watershed would be expected, while the watersheds of the minor creeks would experience a slight increase.

We recommend that the increase in runoff should be directed to Wilks Creek as this creek experiences the least increase of runoff under these policies. It is estimated that an improvement in culvert entrance hydraulics at a later date would be sufficient to accommodate this condition. Alternately, increasing the size of all culverts on this watercourse could be considered ahead of time.

10 YR. RETURN PERIOD	I_{10}	A_o	A_i	C_N	PRE-DEVELOPMENT $Q_{10}=(A_o+A_i)I_{10}C_N$		POST-DEVELOPMENT $Q_{10}=A_oI_{10}C_N$	
	in/hr	ac.	ac.		cfs	m^3/s	cfs	m^3/s
	0·61	1080	204	0·56	439	12·4	369	10·5

A_i AND A_o AREAS REMAINING IN NATURAL STATE

100 YR. RETURN PERIOD	I_{100}	$I_{100} \times 1{\cdot}22$	A_{PRE}	C_N	PRE-DEVELOPMENT $Q_{100}=A_{PRE}IC_N$		POST-DEVELOPMENT * $Q_{100}=Q_T=Q_o+Q_i$	
	in/hr	in/hr	ac.		cfs	m^3/s	cfs	m^3/s
	0·80	0·98	1284	0·68	856	24·2	761	21·5

* SEE TABLE 11

200 YR. RETURN PERIOD	I_{200}	$I_{200} \times 1{\cdot}22$	A_{PRE}	C_N	PRE-DEVELOPMENT $Q_{200}=A_{PRE}IC_N$		POST-DEVELOPMENT * $Q_{200}=Q_T=Q_o+Q_i$	
	in/hr	in/hr	ac.		cfs	m^3/s	cfs	m^3/s
	0·86	1·05	1284	0·68	917	26·0	827	23·4

* SEE TABLE 11

NOONS CREEK — COMPARISON OF PRE-DEVELOPMENT AND POST-DEVELOPMENT DISCHARGES

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TABLE 10

10 YR. RETURN PERIOD	I_{10}	A_o	A_i	C_N	PRE-DEVELOPMENT $Q_{10}=(A_o+A_i)I_{10}C_N$		POST-DEVELOPMENT $Q_{10}=A_oI_{10}C_N$	
	in/hr	ac.	ac.		cfs	m^3/s	cfs	m^3/s
	0·61	1080	204	0·56	439	12·4	369	10·5

A_i AND A_o AREAS REMAINING IN NATURAL STATE

100 YR. RETURN PERIOD	I_{100}	$I_{100} \times 1{\cdot}22$	A_{PRE}	C_N	PRE-DEVELOPMENT $Q_{100}=A_{PRE}IC_N$		POST-DEVELOPMENT * $Q_{100}=Q_T=Q_o+Q_i$	
	in/hr	in/hr	ac.		cfs	m^3/s	cfs	m^3/s
	0·80	0·98	1284	0·68	856	24·2	761	21·5

* SEE TABLE 11

200 YR. RETURN PERIOD	I_{200}	$I_{200} \times 1{\cdot}22$	A_{PRE}	C_N	PRE-DEVELOPMENT $Q_{200}=A_{PRE}IC_N$		POST-DEVELOPMENT * $Q_{200}=Q_T=Q_o+Q_i$	
	in/hr	in/hr	ac.		cfs	m^3/s	cfs	m^3/s
	0·86	1·05	1284	0·68	917	26·0	827	23·4

* SEE TABLE 11

NOONS CREEK—COMPARISON OF PRE-DEVELOPMENT AND POST-DEVELOPMENT DISCHARGES

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TABLE 10

100 YR RETURN PERIOD	AREA		I_{100}	I_{10}	$\Delta I = I_{100} - I_{10}$	$C_{N_{100}}$	$C_{A_{100}}$	$Q_O = A_O I_{100} C_{N_{100}}$	$Q_i = A_i \Delta I C_{A_{100}}$	$Q_T = Q_O + Q_i$	
	ac.		in/hr	in/hr	in/hr			cfs	cfs	cfs	m^3/s
	OUT	1080	0·80x1·22=0·98	—	—	0·68	—	720	—	761	21·5
	IN	204	0·80	0·61	0·19	—	1·05	—	41		

200 YR RETURN PERIOD	AREA		I_{200}	I_{10}	$\Delta I = I_{200} - I_{10}$	$C_{N_{200}}$	$C_{A_{200}}$	$Q_O = A_O I_{200} C_{N_{200}}$	$Q_i = A_i \Delta I C_{A_{200}}$	$Q_T = Q_O + Q_i$	
	ac.		in/hr	in/hr	in/hr			cfs	cfs	cfs	m^3/s
	OUT	1080	0·86x1·22=1·05	—	—	0·68	—	771	—	827	23·4
	IN	204	0·86	0·61	0·25	—	1·09	—	56		

NOONS CREEK — CALCULATION OF POST-DEVELOPMENT DISCHARGES

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TABLE II

	100 YR. RETURN PERIOD						200 YR. RETURN PERIOD					
	POST-DEVELOPMENT		PRE-DEVELOPMENT		INCREASE	Q_{POST}/Q_{PRE}	POST-DEVELOPMENT		PRE-DEVELOPMENT		INCREASE	Q_{POST}/Q_{PRE}
	cfs	m^3/s	cfs	m^3/s	cfs		cfs	m^3/s	cfs	m^3/s	cfs	
HUTCHINSON	154	4·4	106	3·0	48	1·45	175	5·0	122	3·5	53	1·43
TURNERS	183	5·2	150	4·2	33	1·22	207	5·9	173	4·9	34	1·20
WILKS	108	3·1	93	2·6	15	1·16	128	3·6	111	3·2	17	1·15
HETT	125	3·5	99	2·8	26	1·26	141	4·0	113	3·2	28	1·25

MINOR CREEKS—COMPARISON OF PRE-DEVELOPMENT AND POST-DEVELOPMENT DISCHARGES

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TABLE 12

CREEK	AREA	$C_{N_{100}}$	$C_{N_{200}}$	I_{100}	I_{200}	Q_{100}		Q_{200}	
	ac.			in/hr	in/hr	cfs	m^3/s	cfs	m^3/s
HUTCHINSON	124	0·68	0·72	1·25	1·37	106	3·0	122	3·5
TURNERS	193	0·68	0·72	1·14	1·24	150	4·2	173	4·9
WILKS	105	0·68	0·72	1·30	1·47	93	2·6	111	3·2
HETT	120	0·68	0·72	1·21	1·31	99	2·8	113	3·2

MINOR CREEKS — CALCULATION OF PRE-DEVELOPMENT DISCHARGES

McElhanney

TABLE 13

CREEK		AREA	$C_{N_{100}}$	$C_{N_{200}}$	$C_{A_{100}}$	$C_{A_{200}}$	I_{100}	I_{200}	Q_{100}		Q_{200}	
		ac.					in/hr	in/hr	cfs	m^3/s	cfs	m^3/s
HUTCHINSON	OUT	–	–	–	–	–	–	–	–		–	
	IN	124	–	–	0·99	1·03	1·25	1·37	154		175	
	TOT.	124	–	–	–	–	–	–	154	4·4	175	5·0
TURNERS	OUT	66	0·68	0·72	–	–	1·14	1·24	51		59	
	IN	127	–	–	0·91	0·94	1·14	1·24	132		148	
	TOT.	193	–	–	–	–	–	–	183	5·2	207	5·9
WILKS	OUT	32	0·68	0·72	–	–	1·30	1·47	28		34	
	IN	73	–	–	0·84	0·87	1·30	1·47	80		94	
	TOT.	105	–	–	–	–	–	–	108	3·1	128	3·6
HETT	OUT	21	0·68	0·72	–	–	1·21	1·31	17		20	
	IN	99	–	–	0·90	0·93	1·21	1·31	108		121	
	TOT.	120	–	–	–	–	–	–	125	3·5	141	4·0

MINOR CREEKS — CALCULATION OF POST-DEVELOPMENT DISCHARGES

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TABLE 14

6. CONVEYANCE SYSTEMS

The capacity of the existing conveyance structures has been analyzed using the predicted discharges established in Section 5.

The results of this analysis showing the necessary pipe replacements and/or additions have been tabulated in Table 15, Summary of Capacity Analysis of Existing Conveyance Structures.

6.1 Bridges

The new bridge at Knowle Drive has been designed for the stream discharges listed below:

RETURN PERIOD	DISCHARGE		UPPER 5% CONFIDENCE LIMIT	
(years)	(m^3/s)	(cfs)	(m^3/s)	(cfs)
50	19.4	685	23.5	830
100	21.9	775	26.9	950

The capacity for this structure appears adequate to meet the predicted storm discharges determined from the data available.

Our calculations show the predicted capacity of the Ioco Road bridge at Noons Creek to be 10.8 m^3/s (380 cfs). This capacity is equivalent to the post-development 10 year discharge, but only equal to approximately one-half the post-development 100 year discharge, without allowance for instantaneous peaks. The capacity of this bridge is clearly inadequate for the long term, and more comprehensive study is required to assess the future use of the bridge, or possible remedial action. Such action appears to have several alternatives, including:

CREEK	CULV. No.	EXISTING CULV./STRUCTURE DESCRIPTION	CAPACITY LIMITATIONS: PIPE FLOW** m³/sec	CAPACITY LIMITATIONS: INLET CONTROL m³/sec	CAPACITY LIMITATIONS: OUTLET CONTROL m³/sec	MAX. PIPE CAPACITY m³/sec	100 YR. DEVELOPMENT FLOWS: PRE	100 YR. DEVELOPMENT FLOWS: POST	10 YR. DEVELOPMENT FLOWS: PRE	10 YR. DEVELOPMENT FLOWS: POST	RECOMMENDED PIPE SIZE	COMMENTS (SEE NOTES i TO vii)
NOONS	N1	2x2400mmØ C.S.P.	38	23·8	20·4	20·4	24·2	21·5	12·4	10·5	—	
	N2	BRIDGE	—	—	—	—					—	
	N3	BRIDGE	—	10·8	—	10·8					—	
	N4	BRIDGE	—	—	—	—					—	
HUTCHINSON	Hu1	1050mmØ CONC.	7·1	2·6	4·4	2·6	3·0	4·4	—	—	900mmØ CONC.	(ADD'L.)
	Hu2	1200mmØ C.S.P.	9·9	1·5 *	—	1·5					1400mmØ C.S.P.	(REPL.)(i)
	Hu3	1350mmØ C.S.P.	13·2	3·8	5·1	3·8					—	(ii)
	Hu4	750mmØ C.S.P.	1·4	1·1	0·9	0·9					1600mmØ C.S.P.	(REPL.)
	Hu5	900mmØ CONC.& 750mmØ C.S.P.	4·8	1·9	4·8	1·9					900mmØ C.S.P.	(ADD'L.)
TURNERS	T1	900mmØ C.S.P.	2·4	1·4	—	1·4	4·3	5·2	—	—	1600mmØ C.S.P.	(REPL.)(iii)
	T2	750mmØ CONC.	3·7	—	—	3·7					1200mmØ C.S.P.	(REPL.)
	T3	750mmØ CONC.	2·8	—	—	2·8					1200mmØ C.S.P.	(REPL.)
	T4	900mmØ C.S.P.	3·1	—	—	3·1					1200mmØ C.S.P.	(REPL.)
	T5	750mmØ CONC.	3·1	1·1	1·7	1·1					1400mmØ C.S.P.	(ADD'L.)
	T6	900mmØ C.S.P.	2·0	1·4	1·8	1·4					1200mmØ CONC.	(ADD'L.)(iv)
WILKS	W1	900mmØ C.S.P.	2·9	1·1 *	2·2	1·1	2·6	3·1	—	—	1200mmØ C.S.P.	(REPL.)
	W2	900mmØ C.S.P.	—	1·4	—	1·4					1200mmØ C.S.P.	(REPL.)(v)
	W3	900mmØ C.S.P.(INL.)	3·6	1·4	—	1·4					1200mmØ C.S.P.	(REPL.)(vi)
		1050mm CONC.(OUT.)	2·0								—	
HETT	He1	750mmØ CONC.	1·2	1·1	2·1	1·1	2·8	3·5			1350mmØ CONC.	(REPL.)
	He2	750mmØ CONC.	4·5	1·1	2·1	1·1					1200mmØ CONC.	(REPL.)
	He3	900mmØ C.S.P.	2·9	1·4	2·1	1·4					1200mmØ CONC.	(ADD'L.)
	He4	900mmØ C.S.P.	—	1·4	—	1·4					1200mmØ CONC.	(DIVERSION)(vii)

* HW/D RATIO LESS THAN 1·5

** PIPEFLOW $Q = \frac{R^{2/3} S^{1/2}}{n}$

SUMMARY OF CAPACITY ANALYSIS OF EXISTING CONVEYANCE STRUCTURES

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TABLE 15

(i) Widening the channel at the vicinity of the bridge and constructing additional span of cross-sectional area approximately equal to the existing.

(ii) Deepening the channel of the existing bridge, constructing additional span of cross-sectional area somewhat smaller than the existing.

(iii) Removal of the existing bridge and replacement with a new four lane, multiple cell culvert structure.

(iv) Removal of the existing bridge and constructing a new four lane single span bridge structure.

The railway bridge structure is deemed to be the responsibility of the railway company but does not appear to have any problems with respect to its capacity for the maximum stream discharges predicted.

6.2 Culverts

The culverts from Ioco Road to the inlet are varied in design and construction. These culverts do not appear to have been designed to an overall drainage plan, nor has the level of service been established by Council policy or bylaw for these particular structures.

6.2.1 Level of Service

The suggested level of service is discussed and reviewed here so that suitable standards may be adopted.

Three different levels of service may be considered on a basis of current design practices adjusted to suit the area of the study:

(i) Capacity equal to or exceeding runoff under post-development conditions for the 100 year return period.

(ii) Capacity equal to or exceeding runoff under post-development conditions for the 200 year return period.

(iii) Capacity equal to or exceeding runoff generated by existing conditions for the 10 year return period and enchanced under post-development improvements.

Level (i) is suggested as the level of service for new installations where the headwater pool does not cause unacceptable flooding at the 200 year storm event.

Level (ii) is suggested as the level of service for new installations where unacceptable flooding would result under the level of service (i) at the 200 year storm event.

Level (iii) is suggested as the level of service for existing installations to provide for improvements of an interim character. Records of satisfactory service to date should be evaluated and the planning and timing of the various phases of the development of upstream areas should be considered.

The existing systems do not, for the most part, meet levels (i) and (ii) and improvements are required.

The capacity of the culverts is determined using both inlet and outlet control and in some instances pipe flow conditions.

It is suggested that the allowable headwater be limited to provide for a practical balance between the hydraulic efficiency on one hand and the risk of damage to embankments on the other, as outlined below:

(a) For new installations - the ratio of $\frac{HW*}{D}$ = 1.5 max. for the 100 year return period, with the possibility of flooding checked for the 200 year return period.

(b) For existing installations - the ratio of $\frac{HW*}{D}$ not limited and consideration should be given to all site specific features, allowing more flexibility for culvert improvements at a relatively low cost.

In both cases the maximum elevation of the headwater pool should be at least 0.50 metres below the top of embankment.

6.2.2 Culvert Floodproofing

All creeks studied are high energy mountainous creeks running through steep terrain with a heavy tree cover which is to remain in its natural state.

This type of a watershed is relatively likely to wash branches, tree trunks and other debris down to the culvert entrances, resulting in possible blockage and considerable damage.

Two methods of protection, discussed and reviewed below, have been considered in this report and could possibly provide basis for design standards:

(a) Floodproofing by means of controlled overland flow.

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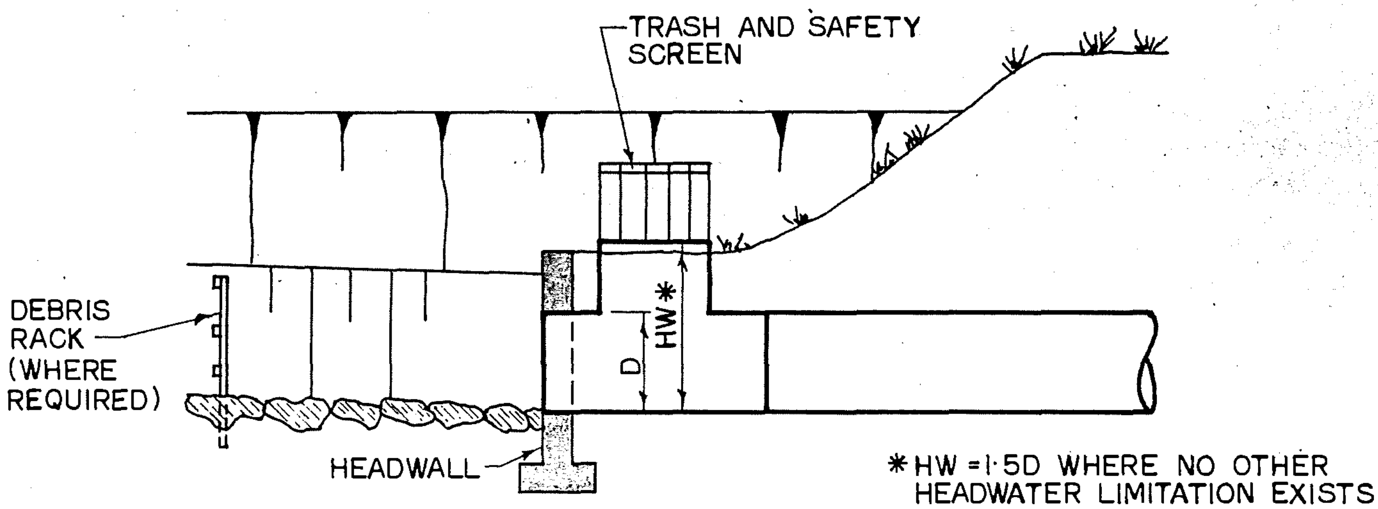
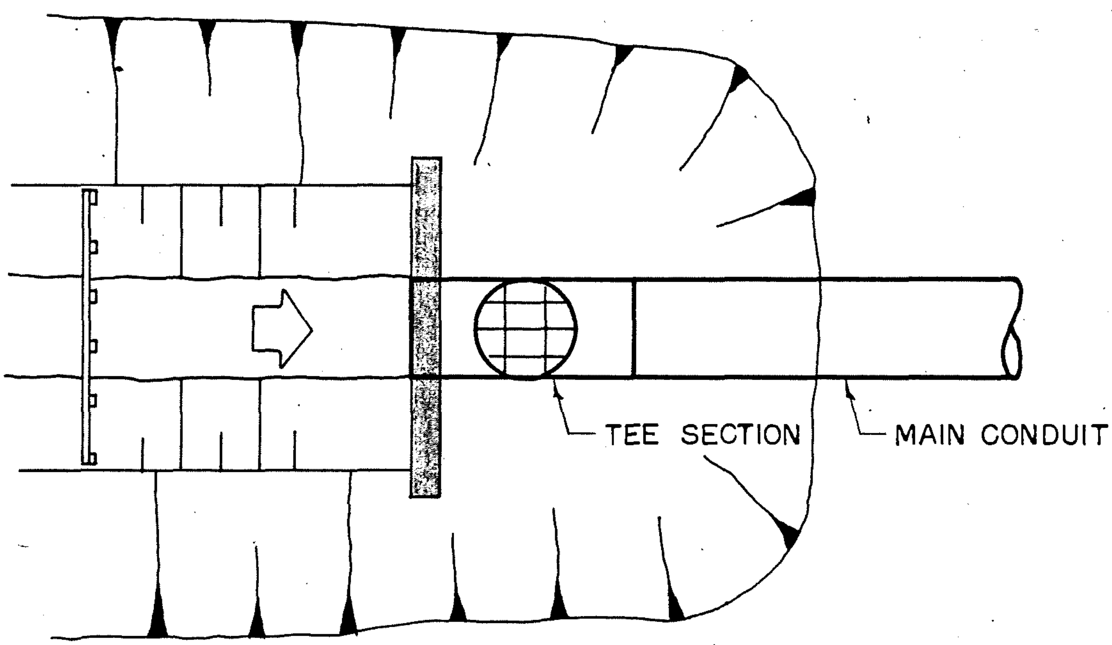
* $\frac{\text{Headwater depth}}{\text{Diameter (Rise)}}$



PROFILE



PLAN

CULVERT FLOODPROOFING - SPILLWAY ENTRANCE TYPE

N.T.S.

McElhanney

FIGURE 4

(b) Floodproofing at the conduit entrance.

The method (a), while often an inexpensive solution, has proved to be impractical or economically unfeasible for culverts evaluated in this report.

The method (b), in general terms, provides for an additional, separate culvert entrance capable of conveying the flow when the main entrance is obstructed. This method has been used in this report. A typical floodproofing arrangement is shown in Figure 4 and has been used in this report.

It is suggested that suitable debris racks be installed ahead of entrances to culverts located in natural preserves, generally at and north of Ioco Road.

Drawings No. 2 through 5 of the Appendix C show the Flood Envelopes before and after the culverts improvements and floodproofing.

6.2.3 Culvert Improvements

The improvement works required to upgrade the culverts and associated structures-headwalls, culvert floodproofing, debris rocks, etc. are described and summarized in Table 16, Summary or Culvert Improvements and Estimated Costs (Current Policy) in Section 9. With reference to this table, some comments pertaining to the details of these improvements are included herein:

(i) The existing 1200 mm Ø C.S.P. culvert should be replaced by extending the downstream culvert.

(ii) The existing 1350 mm Ø C.S.P. culvert should be extended to replace the upstream culvert.

(iii) The proposed 1600 mm Ø C.S.P. culvert is of a larger size than that proposed for the storm sewers immediately downstream because it operates under the inlet control.

(iv) It is assumed that floodproofing is not required. This is in a park area where with suitable grading flooding could be tolerated.

(v) Floodproofing deemed not necessary as flooding would occur at a low point in an asphalt driveway.

(vi) Only the upstream C.S.P. section of this structure requires replacement. The connection between the proposed 1200 mm Ø and the existing 1050 mm Ø concrete pipes should be obtained by using an eccentric reducer.

(vii) The existing structure should be used only for the base flows in order not to alter its appearance amenities. All other flows should be diverted along the Alderside Road west and south into Burrard Inlet.

Because of the complexity of providing additional capacity and considering numerous design alternatives available, more extensive site specific studies would be required to ascertain the most economical design for each individual installation.

6.3 Channel Improvements and Bank Protection

An inspection of the lower reaches of ravines in developed areas has indicated several locations of potential flooding. Most of these locations are adjacent to existing buildings within the flood envelopes where possible bank erosion and flooding damage could have serious consequences.

These areas require mitigative measures both in terms of pre-development and post-development conditions, the difference being only the magnitude of these measures.

The location of these works is shown in Drawing No. 4 and 5 of the Appendix C and the type of these improvements is described below:

(i) Location I1

Extensive bank protection exists on the west side and some widening and bank protection is required on the opposite side.

(ii) Location I2

Bank protection at the north-west corner of the existing house is required as the creek channel is narrow and changes direction at this location.

(iii) Location I3

Bank protection is required along the west side of the existing house as the creek bank appears soft and succeptible to erosion.

(iv) Location I4

New channel with bank revetments is required to replace existing collapsed wooden culvert.

(v) Location I5

Widening, deepening and bank protection is required to prevent flooding of low lying lands on the west side of the creek.

(vi) Location I6

Continuous berm revetment is required along the east side of the creek to improve the channel capacity at higher flood stages and to prevent overflowing creek banks at points where available channel area reduced or constricted. A shorter section of berm is required on the west side of the creek, just north of Ioco Road to prevent flooding of an existing house.

7. GEOTECHNICAL ASSESSMENT OF RAVINES

The proposed development of the North Shore will result in an increase of flow in all creeks, with the exception of Noons Creek. The impact of this flow increase on creek beds and ravine slopes is discussed here to provide a basis for feasibility evaluation. It should be recognized that this assessment is of a rather qualitative nature as the location of the point discharges, the respective magnitude of flow contribution and other important factors are not yet defined.

The basic geotechnical parameters are based on geotechnical study and further assessment carried out by Hardy Associates (1978) Ltd.[1].

The increase in flows is tabulated in Tables 10 and 12 for the 10, 100 and 200 year return periods. Similar percentile increases may be expected for the mean annual flood and for events of any other probability of recurrence.

The creek beds are generally of a typical mountainous character, consisting of boulders, gravels and coarse sands - all highly erosion resistant materials. The ravine bottoms are with perhaps some localized exceptions wider than the presently utilized flow chnnels and meandering relief channels often exist.

It is expected that under this type of flow regime the principle change in hydraulic properties would result in the channel and meander belt width increase. The depth and the velocity would remain essentially unchanged. The magnitude of the impact under these conditions would not be measurable in both the short or the long term.

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1. Ibid.

Field inspection of the ravines has revealed a few locations where the ravine bottoms were relatively narrower. A greater tendency to erode may be expected in these areas. The change in hydraulic properties at these locations would be more evenly distributed, the increase in depth, width and velocity of flow fluctuating in the order of 10 to 20 percent. The toes of slopes generally consist of less erosion resistant materials than the creek beds and consequently, some erosion may be expected where the stream flow impinges on the channel banks. This process, in our opinion, would be very granual and hardly measurable, unless a storm event of an unusually low frequency of recurrence occurred early after the development of the area is completed.

The possible damage in these areas would likely be of a minor, self-correcting character as the creeks would strive to return into a regime of a natural equilibrium. This type of possible localized instability is not expected to appreciably aggravate the stability conditions of the ravines which are considered to be resistive to any deep seated movements.

Hutchinson creek will experience the greatest percentile increase of flow. Fortunately, the ravine appears to be in quite good condition in terms of absorbing the additional runoff.

An examination of the ravines in natural, undeveloped areas did not reveal any requirements for major works to protect the ravine banks for both the pre-development and post-development conditions. Conditions were generally observed to be stable and in equilibrium. A recent small slide was discovered in the Noons Creek ravine, quite representative of an on-going process of attaining an equilibrium without a requirement of any corrective action.

Development in the ravines should be restricted to nature paths. Care must be taken with any utilities crossing the environmental reserve.

It should be noted carefully that loading the top of the ravine banks with fill from builders or home owners can be extremely dangerous procedure leading to potential collapse of the upper ravine sides, and subsequent downstream damage and blockage.

Introduction of water into the upper ravine banks from such aggravating factors of development as broken watermains, leaky swimming pools, concentrated runoff without adequate protection at discharge points in the bank or at the crest of the bank should be controlled and minimized.

8. ENVIRONMENTAL RESERVES, BUFFER STRIPS AND SETBACKS

Several considerations were examined in determining the most acceptable criteria for building setbacks and buffer strips of preserved natural vegetation at the edges of ravines and are discussed below:

(i) The stope stability criterium remains as established in the geotechnical study[1] at a minimum of 6 metres for building setbacks from the edge of the ravine.

(ii) The Water Management Branch of the Ministry of the Environment requires a minimum of 15 metre building setback from the edge of the ravine for these watersheds.

(iii) Both the Fish and Wildlife Branch of the Ministry of Environment, and the Fisheries and Oceans Department of the Federal Government request a buffer strip of undisturbed natural vegetation of a minimum 9 metre width from the edge of the ravine.

The edge of the ravine in this study is defined as the point along the ravine's top of bank where the cross-sectional slope becomes 20% (approximately 11^{o}).

The City of Port Moody representatives have met with the concerned authorities on January 13, 1982 and negotiated a mutual agreement with respect to these issues as follows:

> City regulations should be amended to specify that the rear lot lines can be no closer to the ravine edge than 8 metres and that the side lot lines can be no closer to the ravine edge than 13.5 metres. A city regulation stipulating that the rear yard dimension can not be less than 7 metres is presently in effect and requires no amendment.

---

1. Ibid.

These recommendations are to be presented to Council for review and adoption. The environmental reserve is then to be established by field surveys and defined by legal survey plans so that non-disturbance of its natural character is assured.

9. COSTS AND CONCLUSIONS

9.1 Costs

The main purpose of this study is to compare the improvements required under the current water management policies* with those required under the previous water management policies*. The feasibility of these alternatives is to be assessed in terms of economic, engineering and environmental considerations. The engineering and environmental considerations can be resolved rather routinely and therefore do not appear to be a decisive factor. For the purpose of an economic evaluation cost estimates have been prepared and tabulated in Tables 16 through 20.

The following comments with respect to these cost estimates should be noted:

(i) The estimates are based on 1982 construction costs and are not adjusted for future inflation.

(ii) The quantities and therefore the estimates are approximate as they were derived only from aerial mapping supplemented by a limited field survey.

(iii) The land acquisition costs are excluded since they were also excluded in the previous reports.[2&5]

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* Refer to Section 5 for a detail description

2. Ibid.

5. Ibid.

(iv) There is a possibility of cost sharing with the Ministry of Highways and Transportation and the Canadian Pacific Railway Company based on the premise that some of their structures are underdesigned for the present conditions. However, the cost sharing is likely to be difficult to obtain since they may be prepared to accept a higher level of risk. As this is a comparative feasibility study it is considered unrealistic to include an allowance for these effects.

(v) Under the current water management policies there is no need for trunk storm sewers to convey storm discharges into Burrard Inlet provided that gravity drainage into the minor creeks or existing storm sewers is possible. This is possible in all areas with the exception of a small area tributary to Noons Creek. This is however overcome by other means, detailed in Section 10.

(vi) We understand that the cost of the trunk storm sewer system, based on the Report on Municipal Services, City of Port Moody[2] -drawing No. 80321-3-0, has been adjusted for inflation by the City staff and is now estimated at $1,800,000.

The total costs for both alternatives are summarized and compared in Table 20, and recapitulated herein.

Total Cost of Improvements under Current Water Management Policies	$ 595,000
Total Cost of Improvements under Previous Water Management Policies	$ 2,218,000
Difference in Costs	$ 1,623,000

2. Ibid.

9.2 Conclusions

It is obvious that the current water management policies result in substantial savings. It should be noted that improvements to the existing conveyance systems in the creeks are necessary under both alternatives and that the difference in their corresponding costs is relatively small.

The cost estimates provided herein, we trust, will enable you to fully realize the extent of the work involved and evaluate the benefits derived from it.

BRIDGE LOCATION	BRIDGE REF. No.	IMPROVEMENT ALTERNATIVE *	DESCRIPTION OF IMPROVEMENT *	SUBTOTAL	ENGINEERING & CONTINGENCIES (APPROX. 25%)	TOTAL
KNOWLE DRIVE	N2	—	NONE	—	—	NIL
IOCO ROAD	N3	(i)	CHANNEL WIDENING, ADDITIONAL SPAN (2 LANE WIDTH)	$ 135,000	$ 35,000	$ 170,000
		(ii)	CHANNEL WIDENING AND DEEPENING, ADD. SPAN (2 LANE WIDTH)	$ 130,000	$ 35,000	$ 165,000
		(iii)	NEW MULTIPLE CELL CULVERT STRUCTURE (4 LANE WIDTH)	$ 160,000	$ 40,000	$ 200,000 **
		(iv)	NEW SINGLE SPAN BRIDGE STRUCTURE (4 LANE WIDTH)	$ 245,000	$ 60,000	$ 305,000
C.P.R. TRACKS	N4	—	NONE	—	—	NIL

* FOR FULL DESCRIPTION SEE SECTION 6.1

** ALTERNATIVE ASSUMED IN THIS STUDY

SUMMARY OF BRIDGE IMPROVEMENTS AND ESTIMATED COSTS (CURRENT AND PREVIOUS POLICIES)

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TABLE 16

CREEK	CULV. No.	PROPOSED ADDITIONAL PIPE	PROPOSED REPLACEMENT PIPE	PROP. LENGTH (m)	HEAD WALLS REQ'D.	DEBRIS RACKS REQ'D.	FLOOD PROOFING REQ'D.	ASPH. PAVEM'T RESTOR. REQ'D.	APPROX. COST	TOTAL COST	COMMENTS *
NOONS	N1	—	—	—	—	—	—	—	—	—	EX. ADEQUATE
HUTCHINSON	Hu 1	900mm Ø CONC.	—	23	1	YES	YES	NO	$ 12,800		
	Hu 2	—	1400mm Ø C.S.P.	12	1	YES	YES	YES	$ 9,400		(i)
	Hu 3	—	—	—	—	—	—	—	—	$ 50,700	(ii)
	Hu 4	—	1600mm Ø C.S.P.	13	2	NO	YES	YES	$ 12,500		
	Hu 5	900mm Ø C.S.P.	—	30	2	NO	YES	NO	$ 16,000		
TURNERS	T1	—	1600mm Ø C.S.P.	28	1	YES	YES	YES	$ 17,800		(iii)
	T2	—	1200mm Ø C.S.P.	40	NO	NO	NO	NO	$ 14,500		
	T3	—	1200mm Ø C.S.P.	19	NO	NO	NO	YES	$ 9,100	$ 78,000	
	T4	—	1200mm Ø C.S.P.	17	NO	NO	NO	NO	$ 7,600		
	T5	1400mm Ø C.S.P.	—	10	1	NO	YES	NO	$ 7,800		
	T6	1200mm Ø CONC.	—	45	1	NO	NO	NO	$ 21,200		(iv)
WILKS	W1	—	1200mm Ø C.S.P.	30	2	YES	YES	YES	$ 16,600		
	W2	—	1200mm Ø C.S.P.	12	2	NO	NO	YES	$ 7,900	$ 37,100	(v)
	W3	—	1200mm Ø CONC.	20	1	NO	YES	NO	$ 12,600		(vi)
HETT	He 1	—	1350mm Ø CONC.	30	1	YES	YES	YES	$ 23,000		
	He2	—	1200mm Ø CONC.	8	1	NO	YES	NO	$ 7,500	$ 111,200	
	He3	1200mm Ø CONC.	—	40	1	NO	YES	YES	$ 10,500		(vii)
	He4	1200mm Ø CONC.	—	145	NO	NO	NO	YES	$ 70,400		(viii)

* REFER TO SECTION 6.2.3 FOR COMMENTS

SUBTOTAL		$277,000
ENGINEERING & CONTINGENCIES	25 %	±$ 70,000
TOTAL		$347,000

SUMMARY OF CULVERT IMPROVEMENTS
AND ESTIMATED COSTS
(CURRENT POLICY)

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TABLE 17

CREEK	CULV. No.	PROPOSED PIPE	PROP. LENGTH (m)	HEAD WALLS REQ'D.	DEBRIS RACKS REQ'D.	FLOOD PROOFING REQ'D.	ASPH. PAVEM'T RESTOR. REQ'D.	APPROX. COST	TOTAL COST
HUTCHINSON	Hu 1	—	—	—	YES	YES	—	$ 3,700	$ 24,900
	Hu 2	1400mmØ C.S.P.	12	1	YES	YES	YES	$ 9,800	
	Hu 4	1400mmØ C.S.P.	13	2	NO	YES	YES	$ 11,400	
TURNERS	T1	1050mmØ CONC.	28	1	YES	YES	YES	$ 16,500	$ 43,100
	T5	1050mmØ CONC.	10	2	NO	YES	NO	$ 9,100	
	T6	1200mmØ C.S.P.	45	2	NO	NO	NO	$ 17,500	
WILKS	W1	1200mmØ C.S.P.	30	2	YES	YES	YES	$ 13,900	$ 40,300
	W2	1200mmØ C.S.P.	12	2	NO	NO	YES	$ 7,900	
	W3	1200mmØ C.S.P.	20	1	NO	YES	YES	$ 18,500	
HETT	He 1	900mmØ CONC.	30	2	YES	YES	YES	$ 16,300	$ 40,300
	He 2	900 mmØ CONC.	8	2	NO	YES	NO	$ 7,600	
	He 3	1000mmØ C.S.P.	40	2	NO	YES	NO	$ 16,400	

SUBTOTAL		$ 148,600
ENGINEERING & CONTINGENCIES	25%	±$ 37,400
TOTAL		$ 186,000

SUMMARY OF CULVERT IMPROVEMENTS
AND ESTIMATED COSTS
(PREVIOUS POLICY)

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TABLE 18

CREEK		CREEK WIDENING	BANK PROTECTION	BERMS	APPROX. COST PREVIOUS POLICY	APPROX. COST CURRENT POLICY
HUTCHINSON	I1	YES	YES	NO	$ 3,000	$ 4,000
	I2	NO	YES	NO	$ 1,000	$ 1,500
	I3	NO	YES	NO	$ 1,600	$ 2,500
WILKS	I4	YES	YES	NO	$ 4,000	$ 6,000
	I5	YES	YES	NO	$ 4,400	$ 6,500
NOONS	I6	NO	NO	YES	$ 12,000	$ 18,000
			SUB TOTAL		$ 26,000	$ 38,500
			ENGINEERING CONTINGENCIES ±25%		$ 6,000	$ 9,500
			TOTAL COST		$ 32,000	$ 48,000

SUMMARY OF CHANNEL IMPROVEMENTS
AND ESTIMATED COST
(CURRENT AND PREVIOUS POLICIES)

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TABLE 19

COST ESTIMATE CURRENT WATER MANAGEMENT POLICIES		
DESCRIPTION	REFERENCE	COST
1. BRIDGE IMPROVEMENTS	TABLE 16	$ 200,000
2. CULVERT IMPROVEMENTS	TABLE 17	$ 347,000
3. CHANNEL IMPROVEMENTS	TABLE 19	$ 48,000
4. TRUNK SEWER SYSTEM ($Q \leq Q_{10}$)	SECTION 9	NIL
	TOTAL COST	$ 595,000

COST ESTIMATE PREVIOUS WATER MANAGEMENT POLICIES		
DESCRIPTION	REFERENCE	COST
1. BRIDGE IMPROVEMENTS	TABLE 16	$ 200,000
2. CULVERT IMPROVEMENTS	TABLE 18	$ 186,000
3. CHANNEL IMPROVEMENTS	TABLE 19	$ 32,000
4. TRUNK SEWER SYSTEM ($Q \leq Q_{10}$)	SECTION 9	$ 1,800,000
	TOTAL COST	$ 2,218,000

COMPARSION OF TOTAL COST FOR CURRENT AND PREVIOUS WATER MANAGEMENT POLICIES *

* REFER TO SECTION 5 FOR A DETAILED DESCRIPTION

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TABLE 20

10. RECOMMENDED POLICIES

(a) Environmental reserves should be established on all ravines embodying the concepts summarized in Section 8, Environmental Reserves, Buffer Strips and Setbacks.

(b) In the Noons Creek watershed, all storm water from developed areas for return periods up to and including 10 years should be diverted away from the creek and discharged directly into Burrard Inlet. Excess flows may be discharged to the natural drainage channel. The 10 year storm water runoff from an area bounded by Noons Creek, its tributary and the City limits appears to have topographic difficulties in conveying this flow away from both Noons Creek and its tributary. It is recognized that the development potential of this area is quite limited due to the ravine banks that render a large portion of the area undevelopable. In view of this, it is suggested that all storm water be discharged into the ravines. The impact of this limited development should be minimized by implementing runoff detention measures.

(c) In all other watersheds all storm water from developed areas for all return periods should be discharged directly into the creeks serving the watersheds.

(d) Levels of service should be established for the sizing of culverts. Levels (i) and (ii) or an alternative level (iii) are levels of service that may be technically justified.

(e) Standards for culvert floodproofing and perhaps debris racks should be adopted to standardize and ensure acceptable performance of these items.

(f) Policies for possible future boundary expansion should be established now, if at all possible, to delineate future drainage boundaries and to make adjustments to the conveyance systems affected by this change.

(g) Run-off coefficients and IDF curves should be standardized for all North Shore drainage.

(h) Internal drainage systems should provide flood paths for controlled flows to natural reserves to protect both private property and reserves.

APPENDIX A

SYMBOLS

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SYMBOLS

Q_{10}, Q_{100}, Q_{200} = STORM WATER DISCHARGE FOR 10, 100, & 200 YEAR RETURN PERIOD

Q_{PRE} = PRE-DEVELOPMENT STORM WATER DISCHARGE

Q_{POST} = POST-DEVELOPMENT STORM WATER DISCHARGE

A = AREA

I_{10}, I_{100}, I_{200} = INTENSITY FOR 10, 100, & 200 YEAR RETURN

C = COEFFICIENT OF RUN-OFF

C_N = RUN-OFF COEFFICIENT FOR NATURAL AREAS

C_A = ADJUSTED RUN-OFF COEFFICIENTS FOR DEVELOPED AREAS

o = OUTSIDE CITY LIMITS (eg. A_o = AREA OUTSIDE CITY LIMITS)

i = INSIDE CITY LIMITS (eg. A_i = AREA INSIDE CITY LIMITS)

T = TOTAL (eg. Q_T = FLOW FROM OUTSIDE & INSIDE CITY LIMITS)

t_c = TIME OF CONCENTRATION

HW/D = HEADWATER ÷ DIAMETER OF PIPE

R = PIPE AREA ÷ WETTED PERIMETER

S = SLOPE OF CULVERT PIPE

n = MANNING'S COEFFICIENT OF ROUGHNESS

APPENDIX B

DETAILS OF EXISTING CULVERTS

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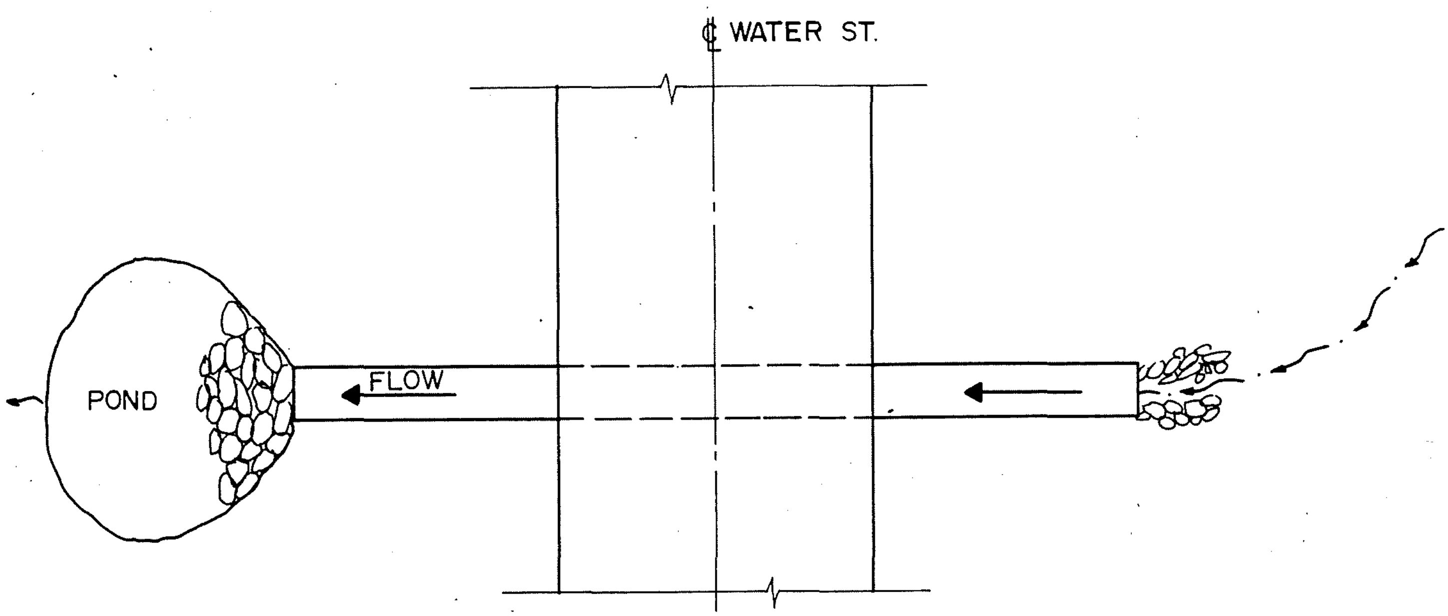

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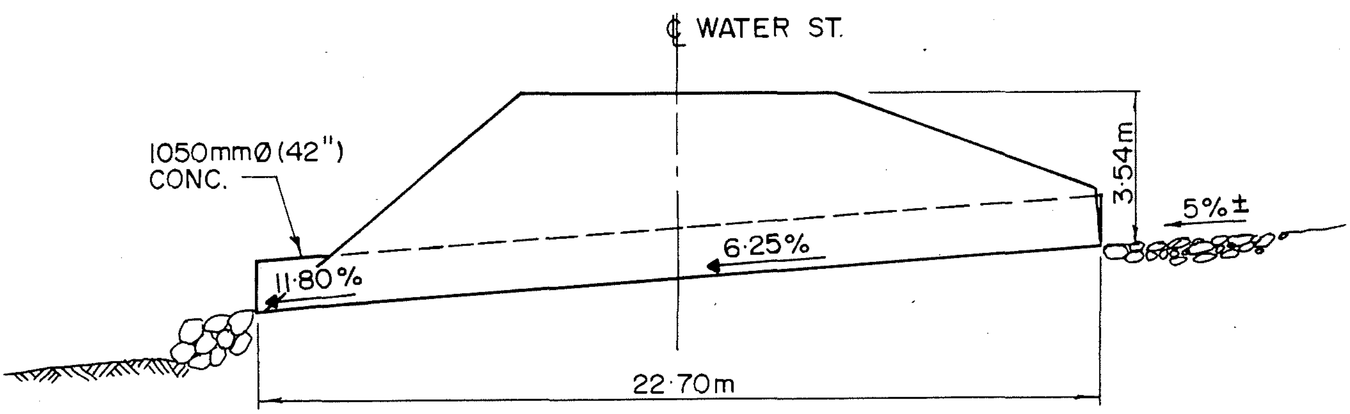
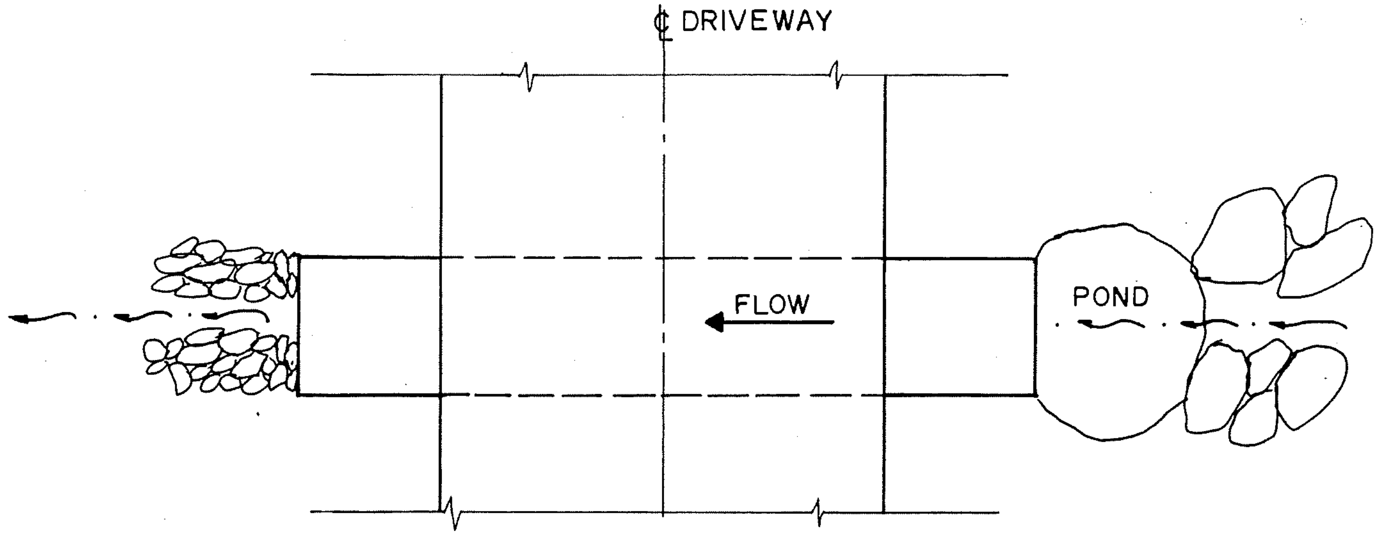


PROFILE

CULVERT No. Hu1



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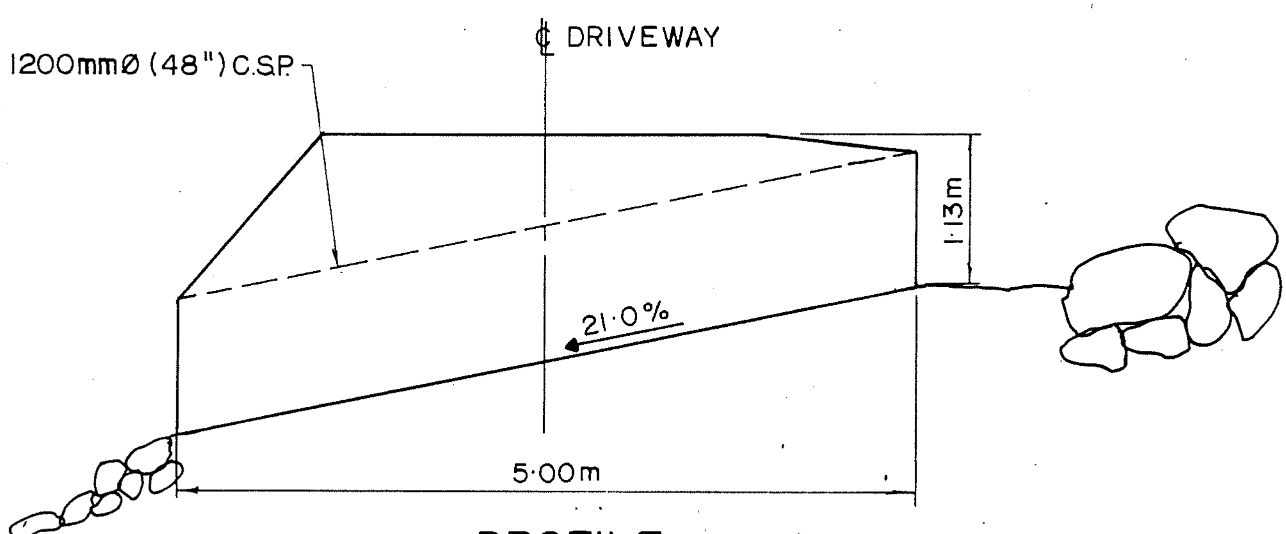

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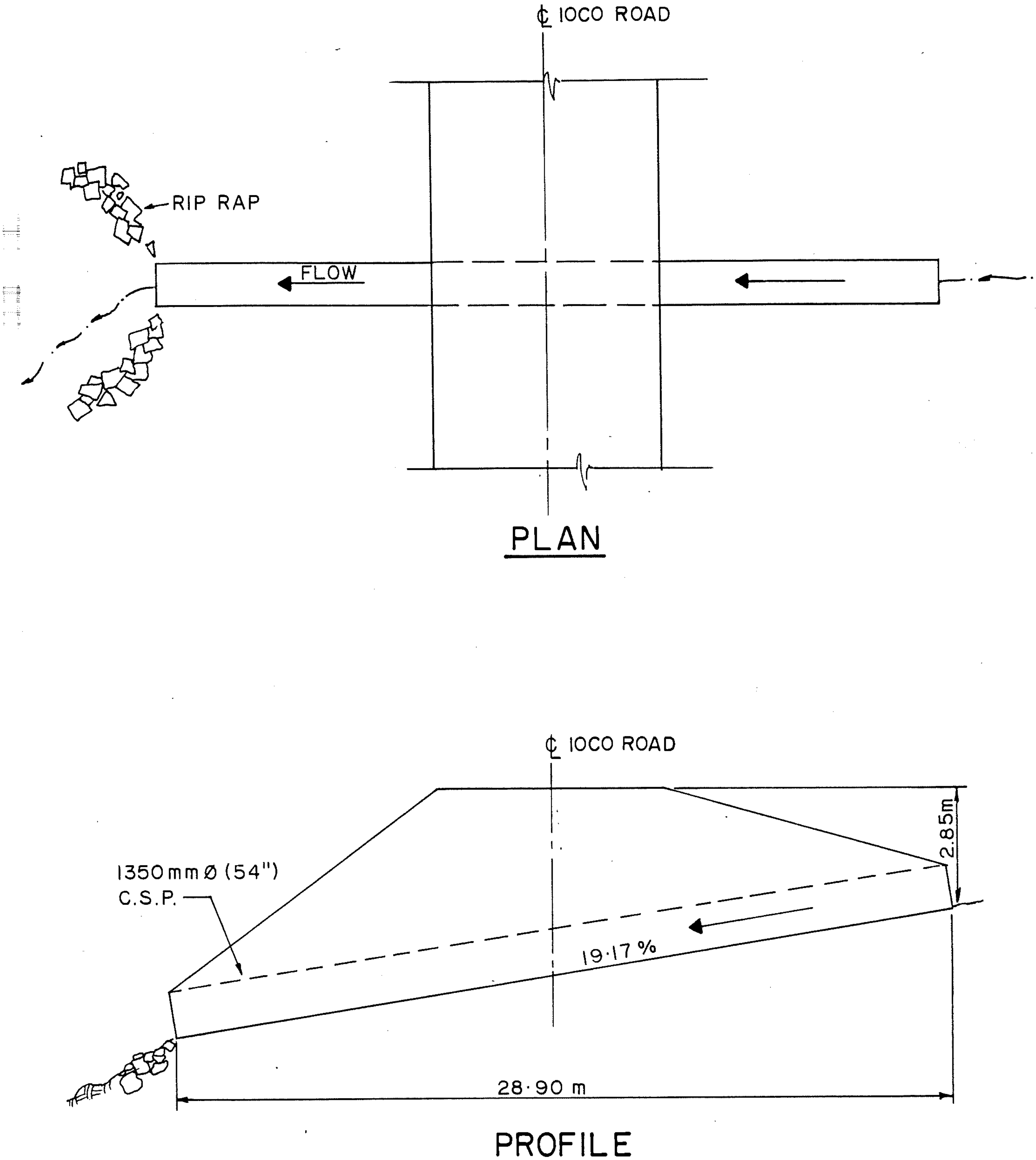
CULVERT No. Hu2

McElhanney



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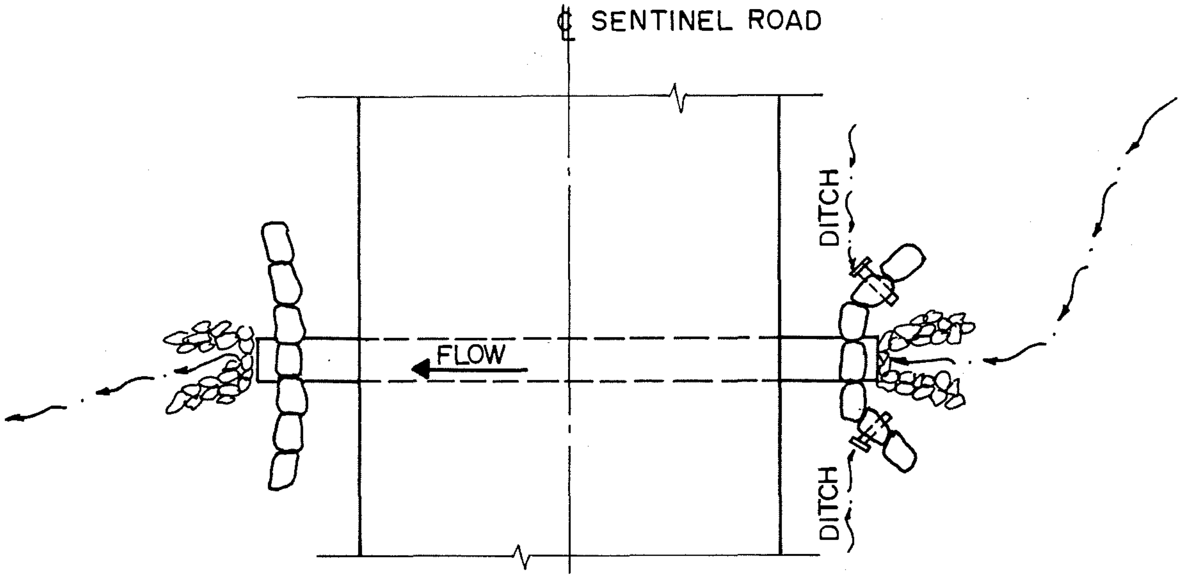

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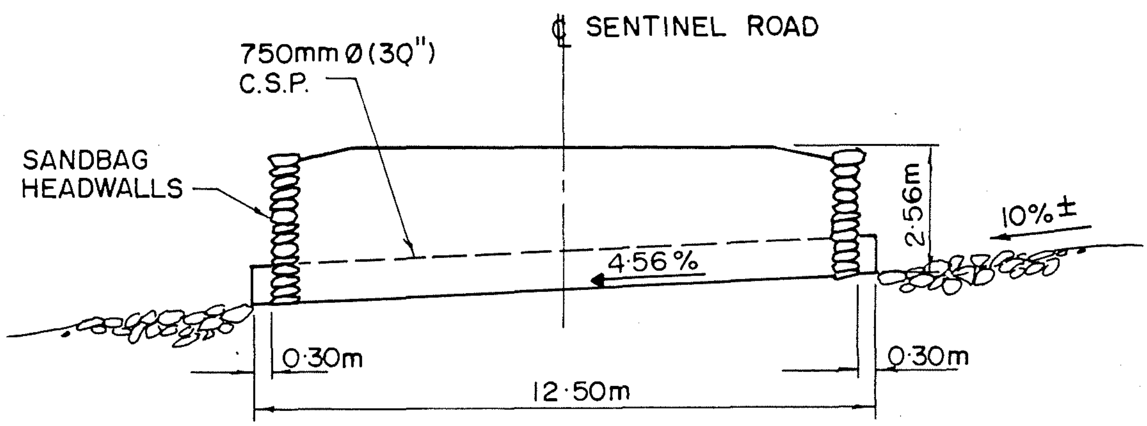

PROFILE

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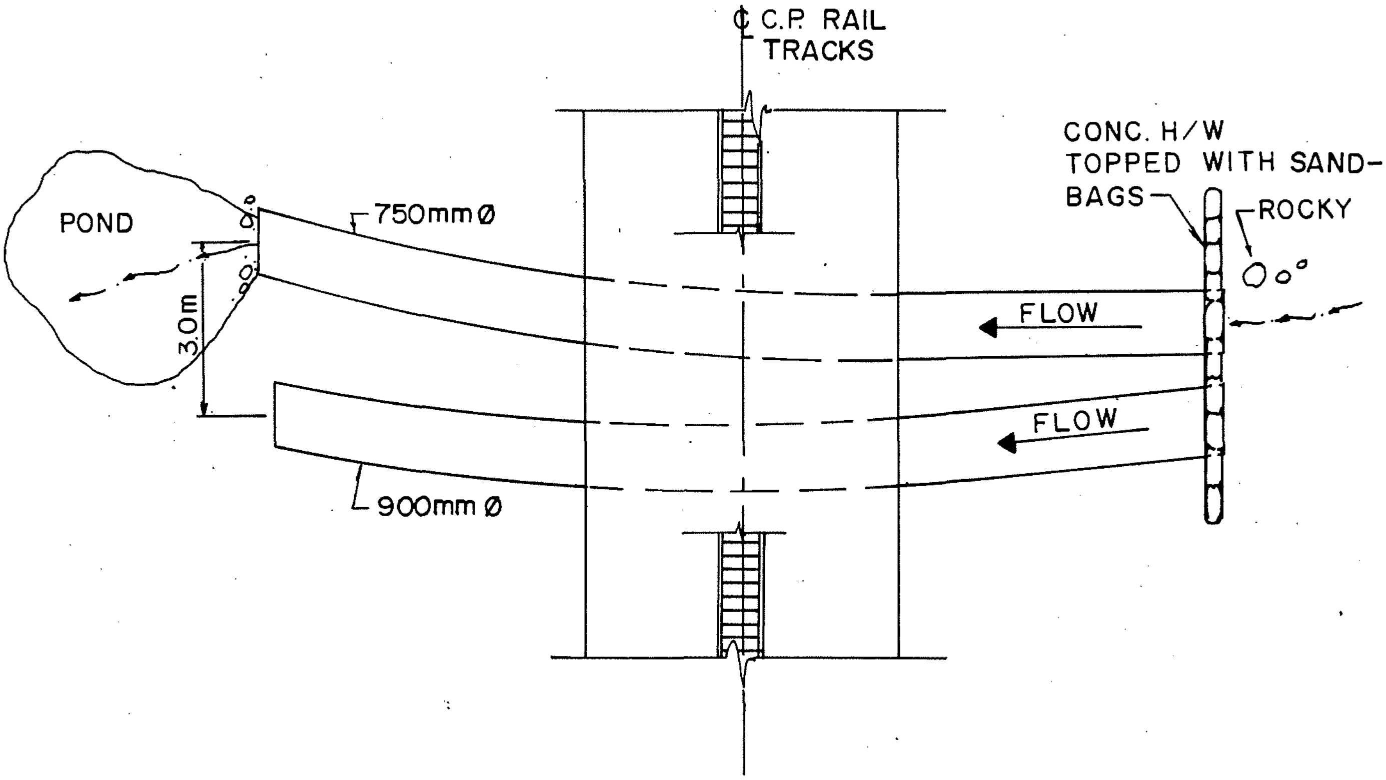

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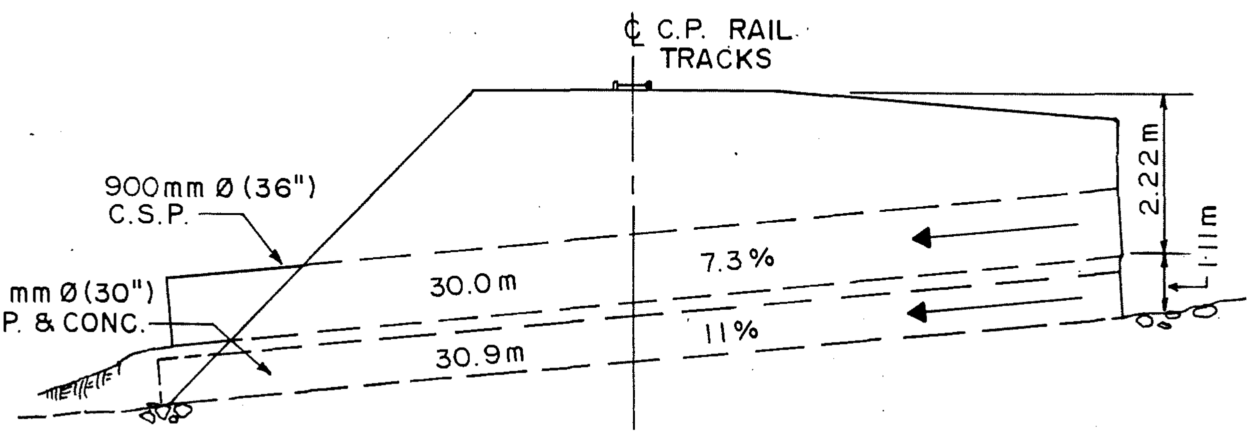
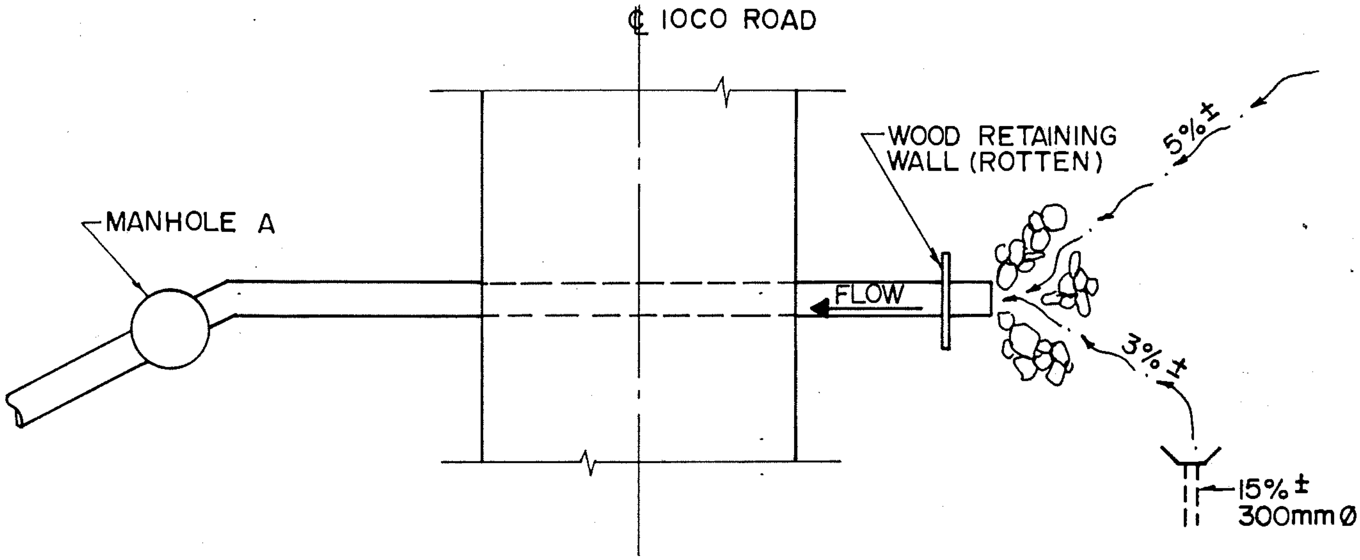


PROFILE

CULVERT No.Hu5

McElhanney



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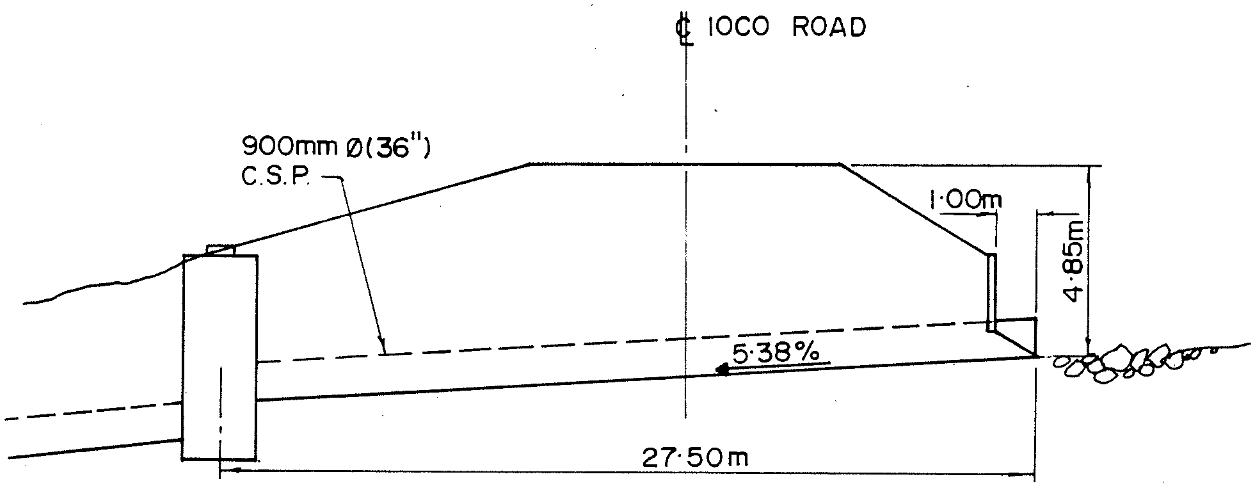
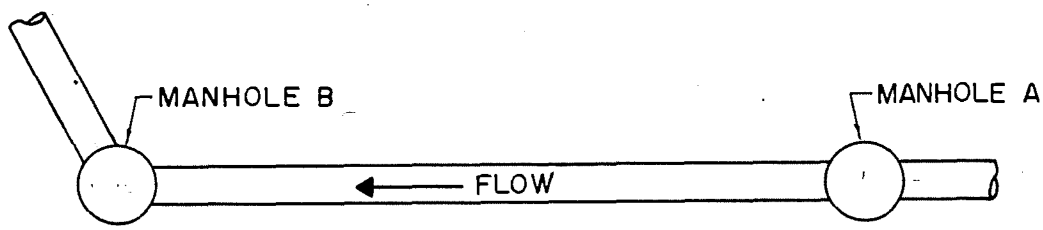


PROFILE

CULVERT No. T1

McElhanney



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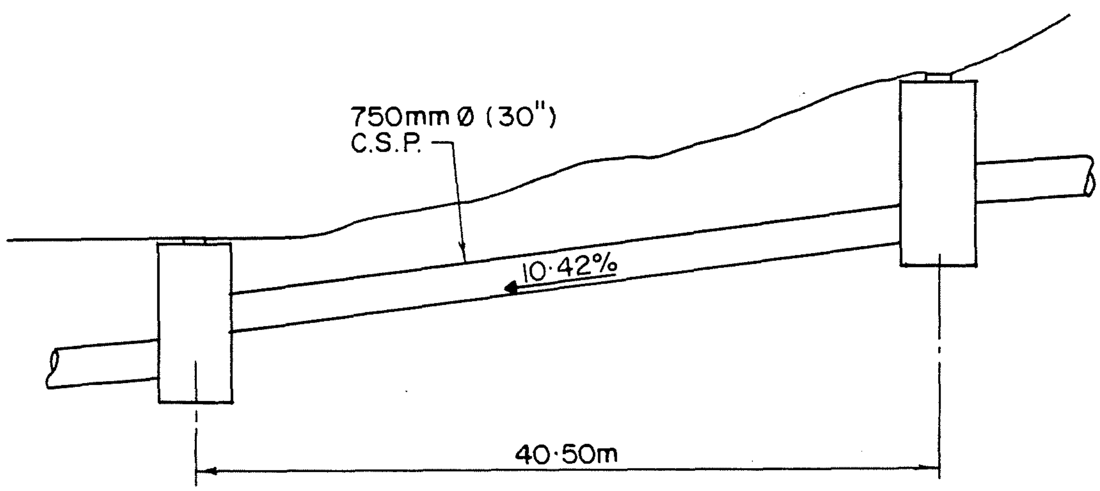

PROFILE

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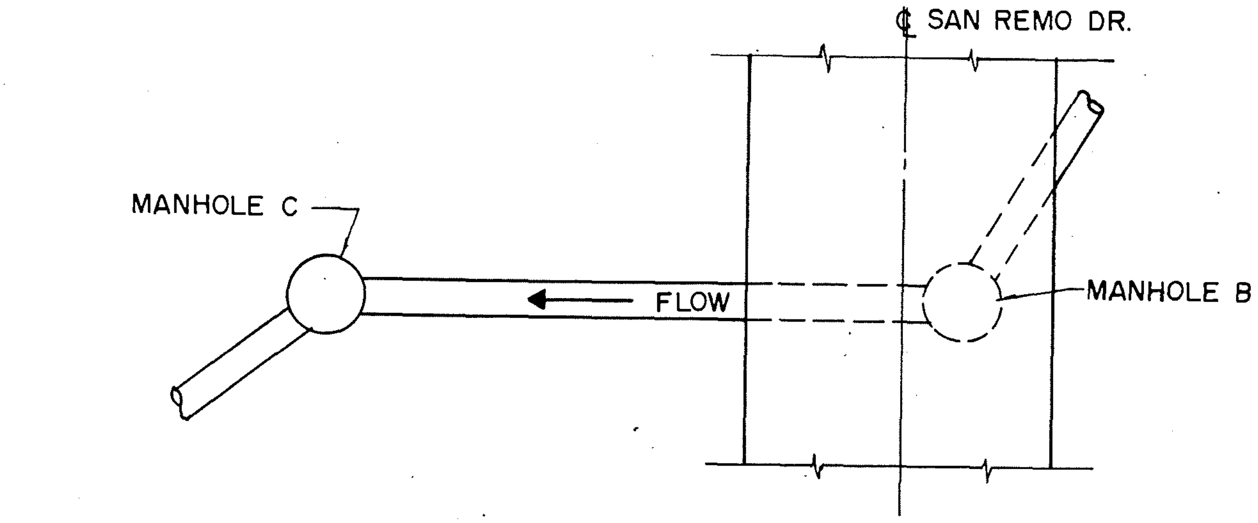

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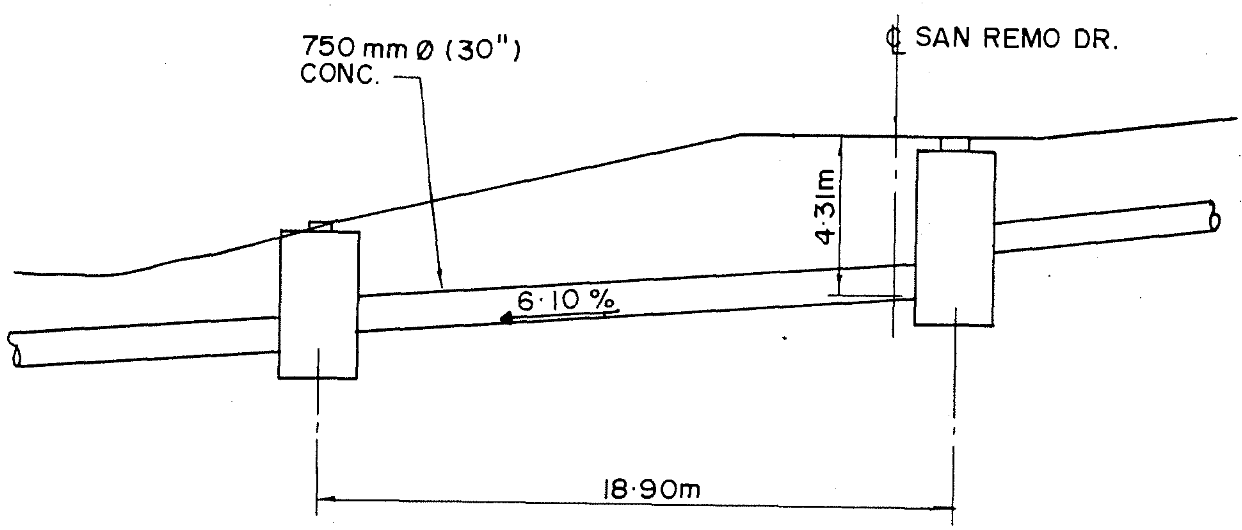

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CULVERT No. T3

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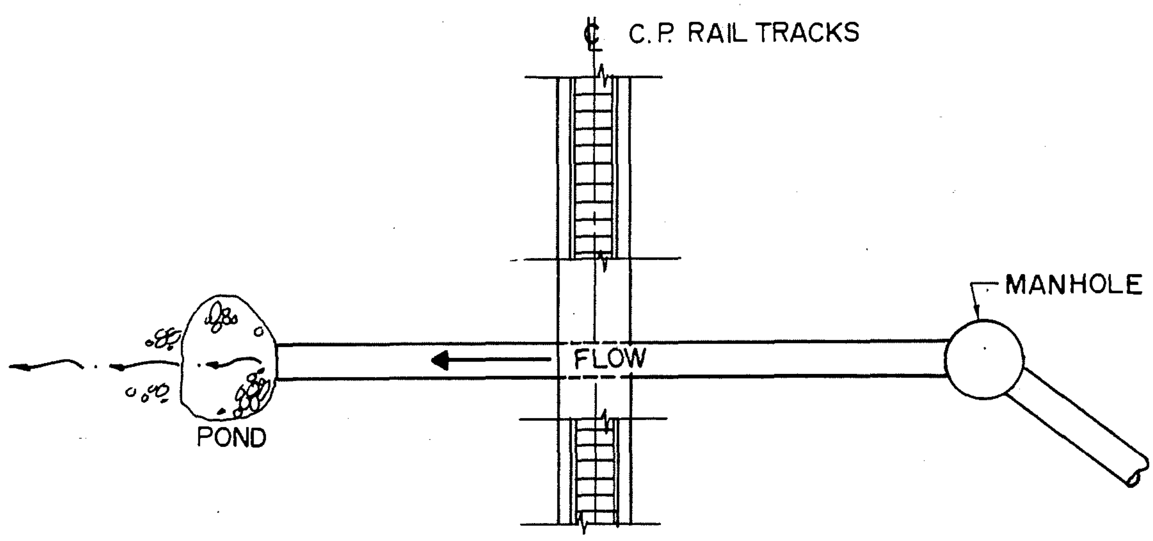

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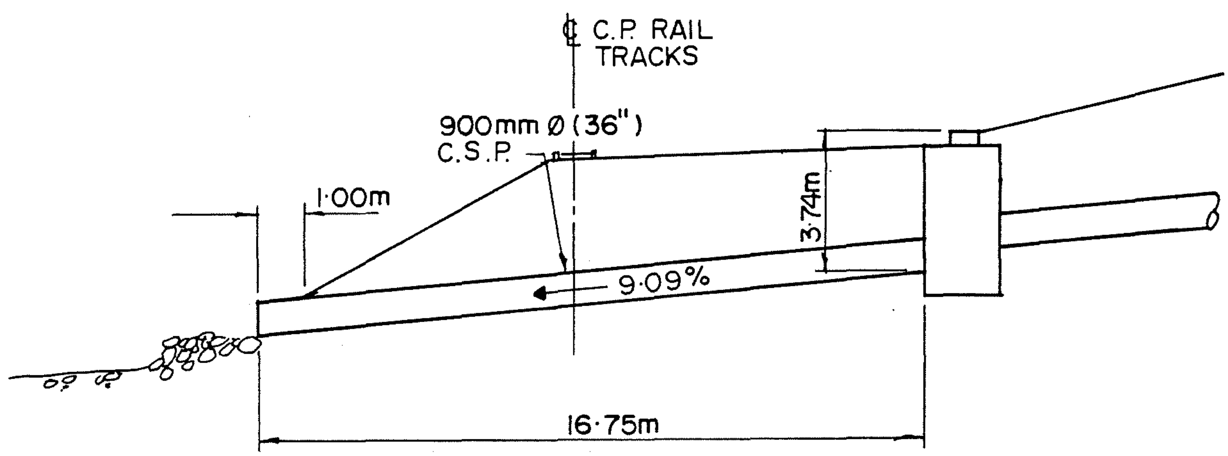

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CULVERT No. T4

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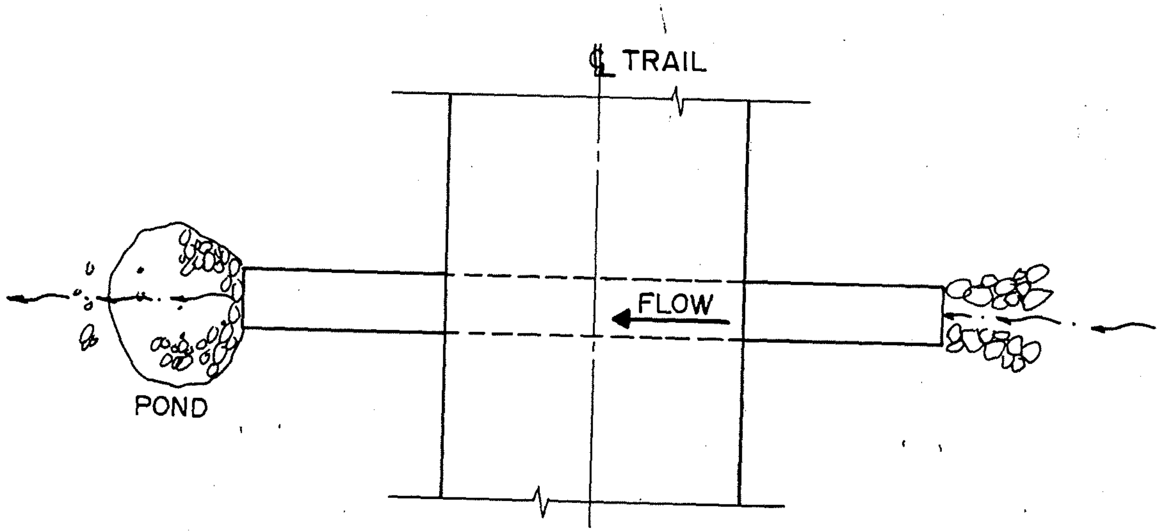

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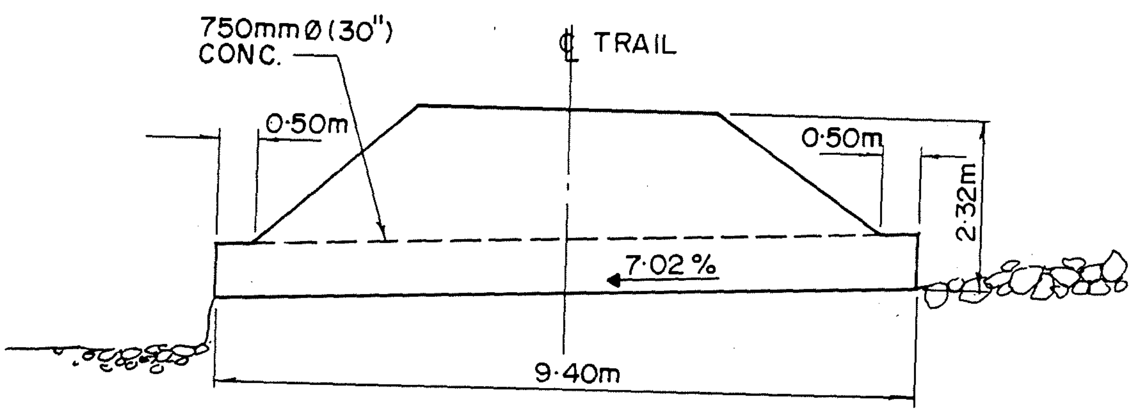
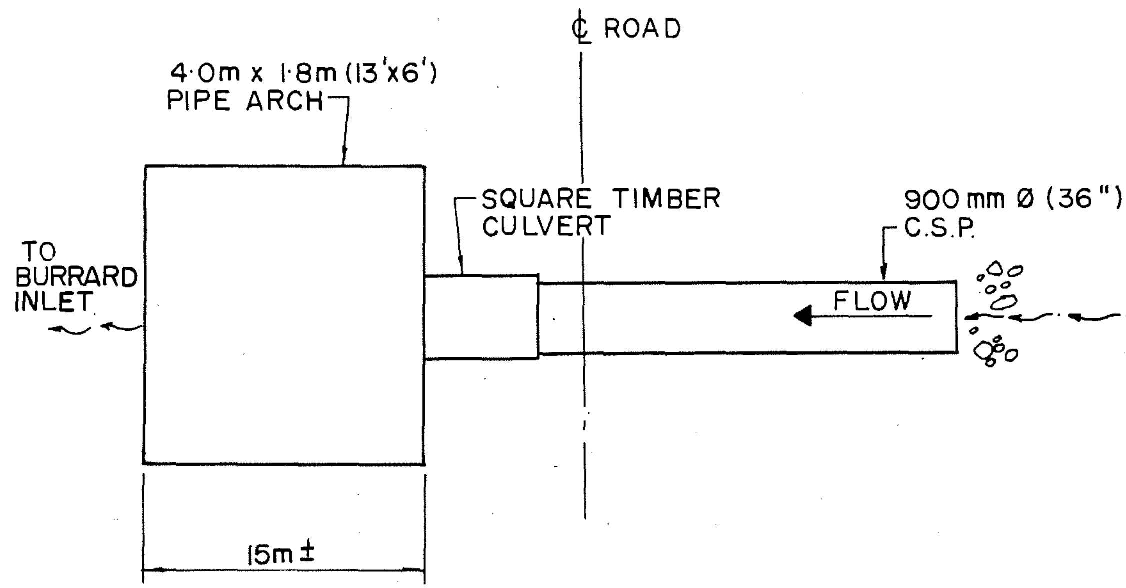


PROFILE

CULVERT No. T5

McElhanney



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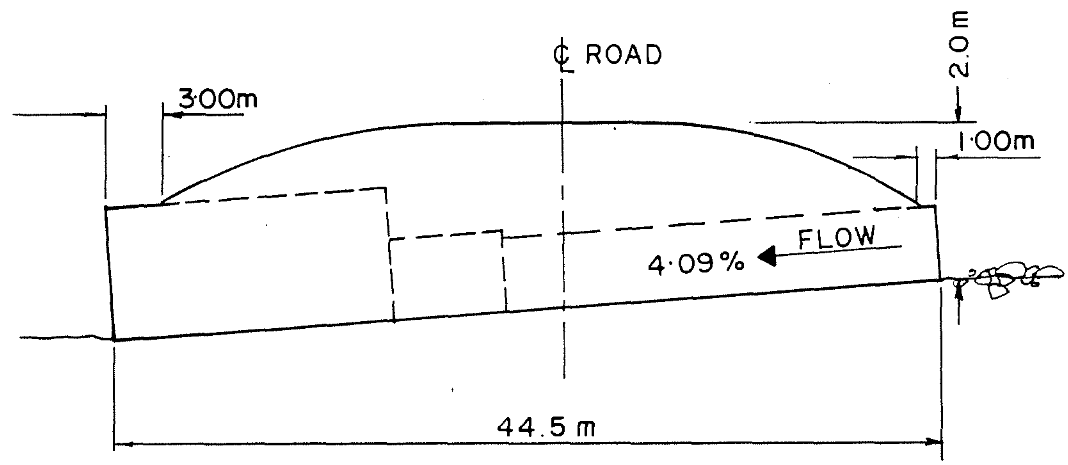
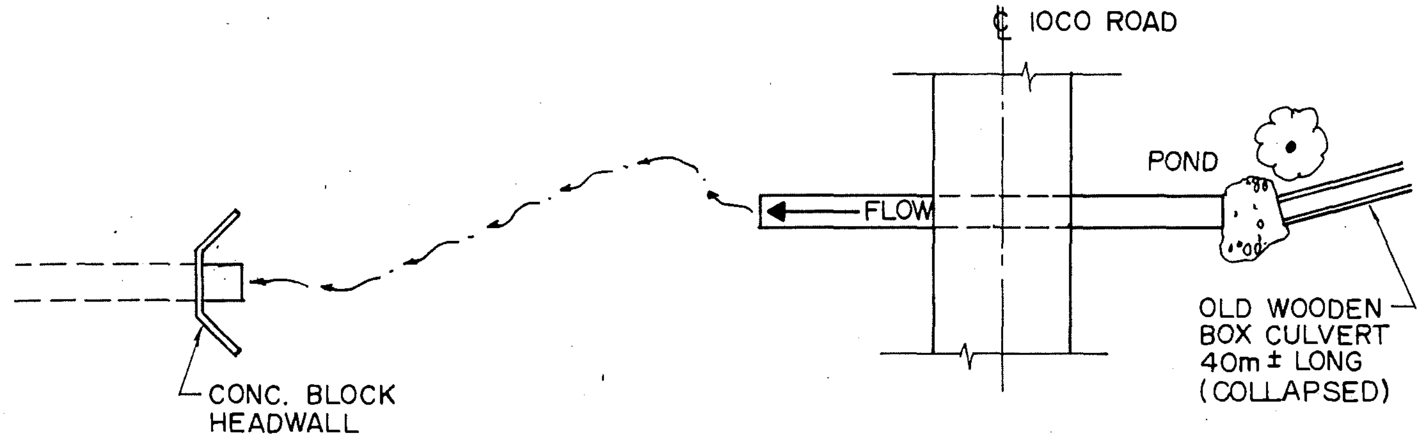


PROFILE

CULVERT No. T6

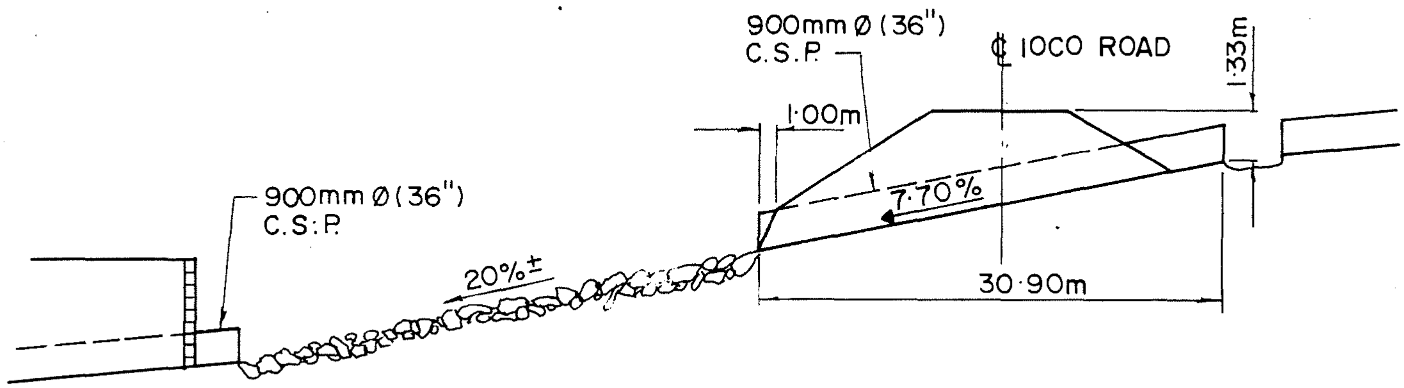
McElhanney



W2

W1

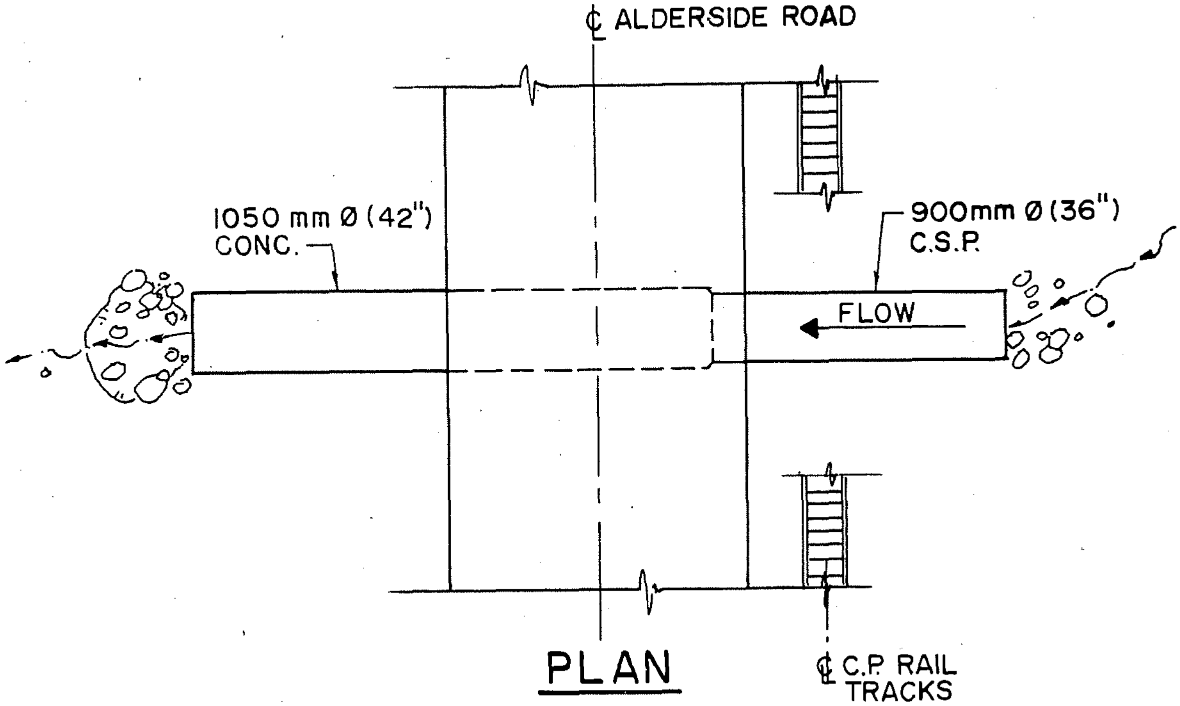
PLAN



PROFILE

CULVERT Nos. W1 & W2

McElhanney



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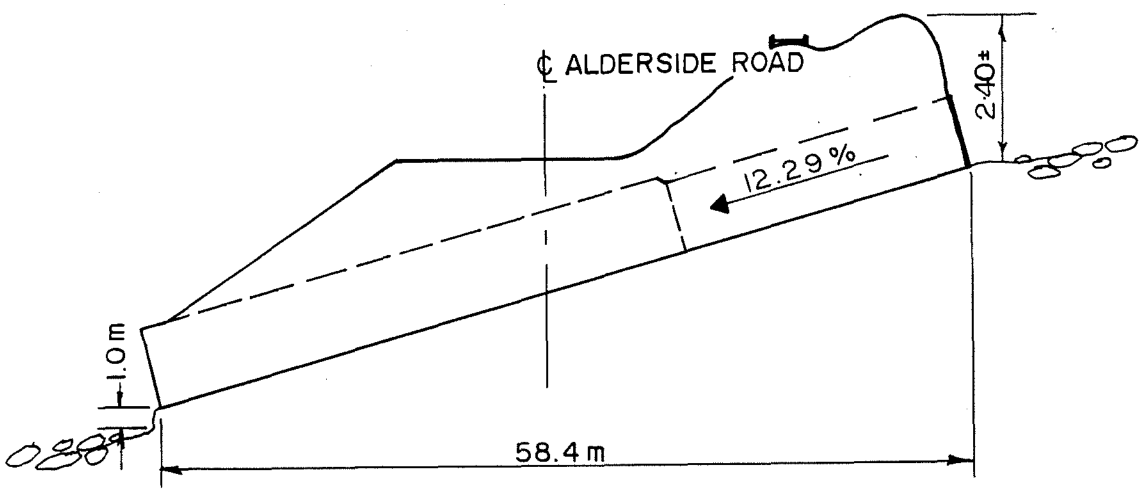

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CULVERT No. W3

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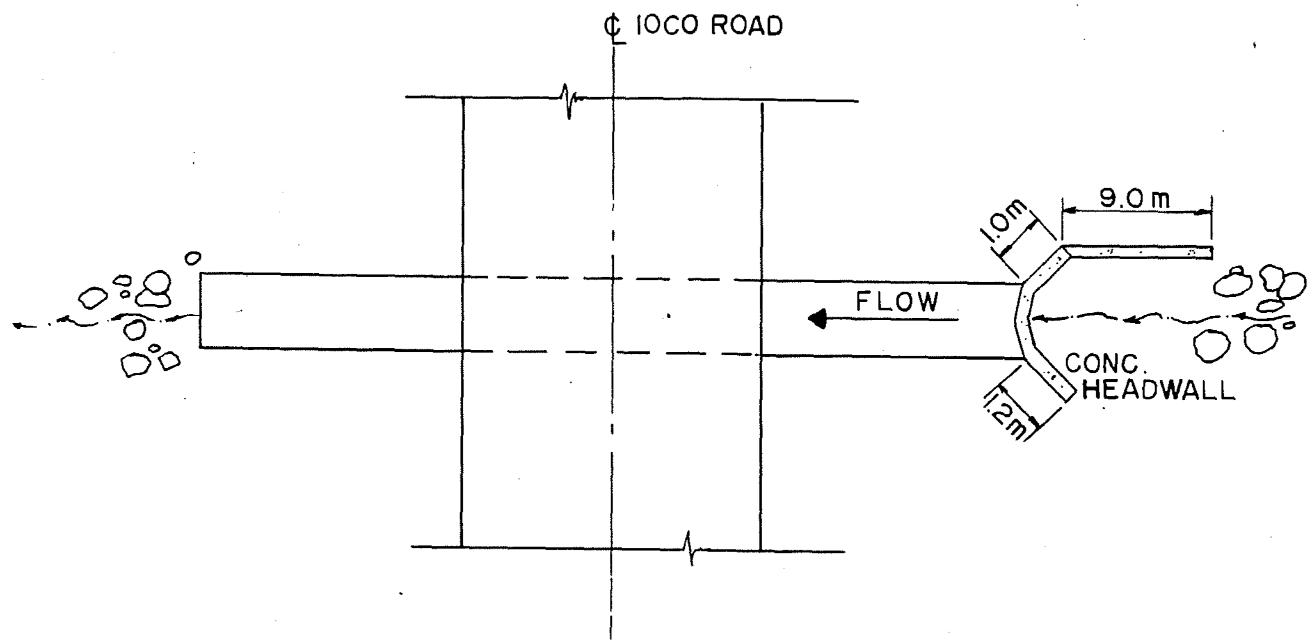

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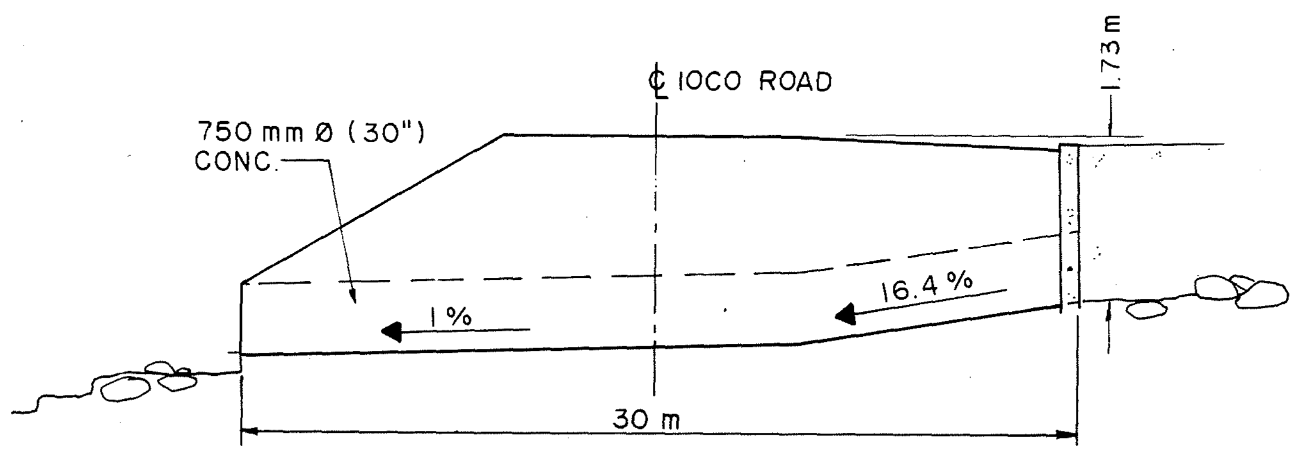
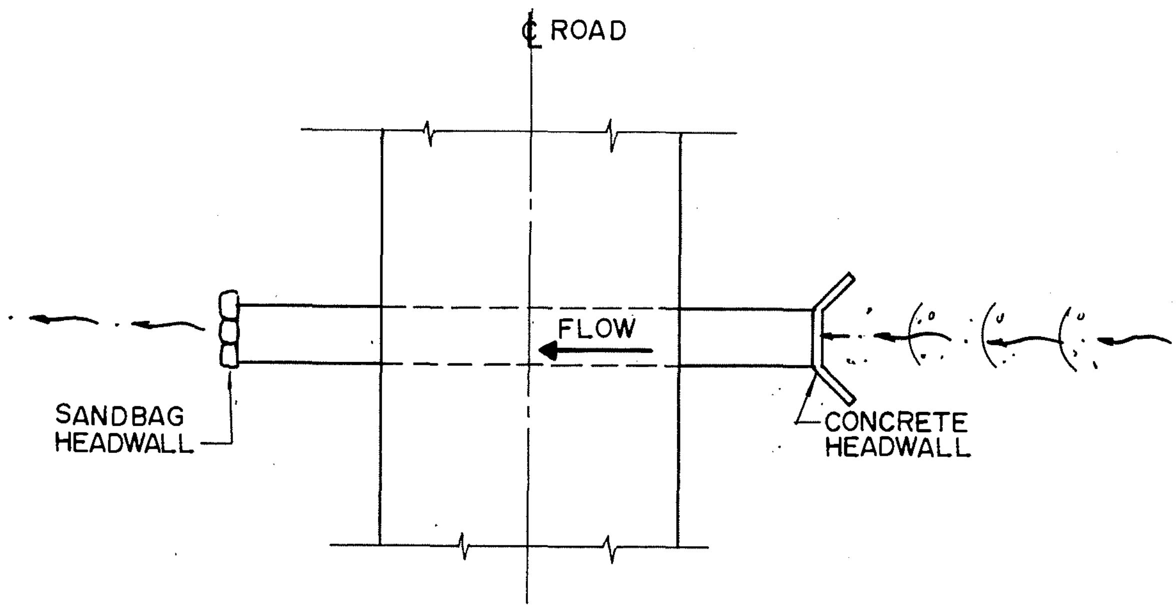


PROFILE

CULVERT No. He1

McElhanney



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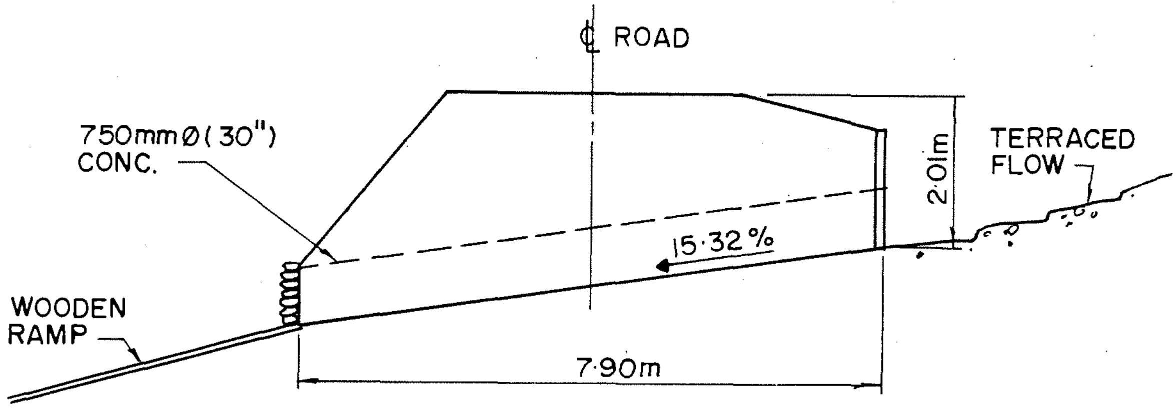
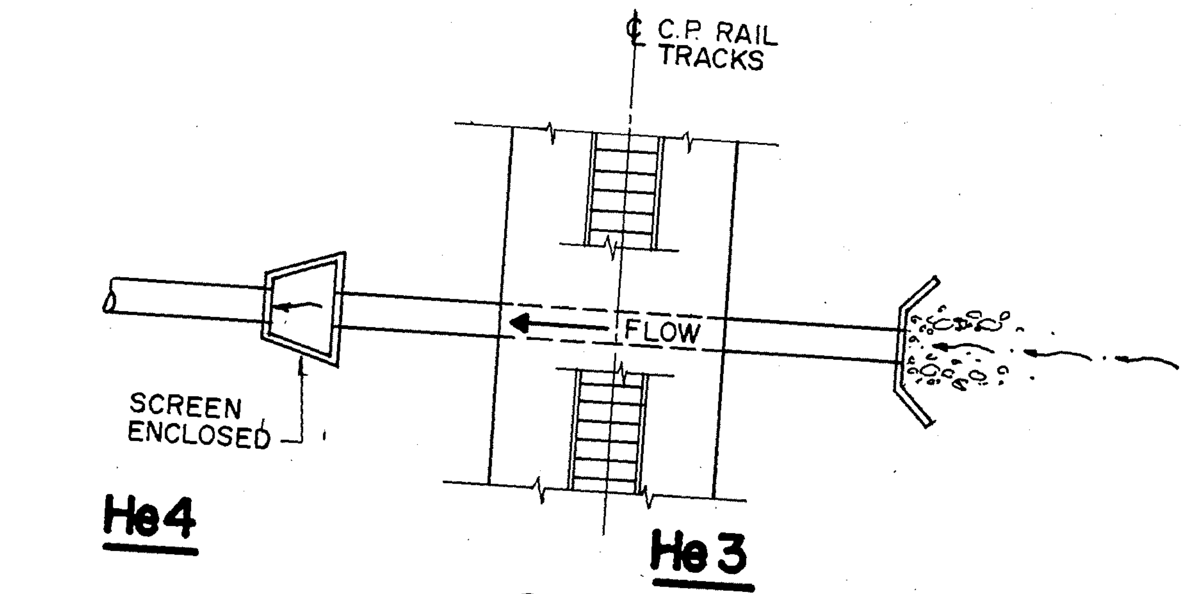


PROFILE

CULVERT No. He2

McElhanney



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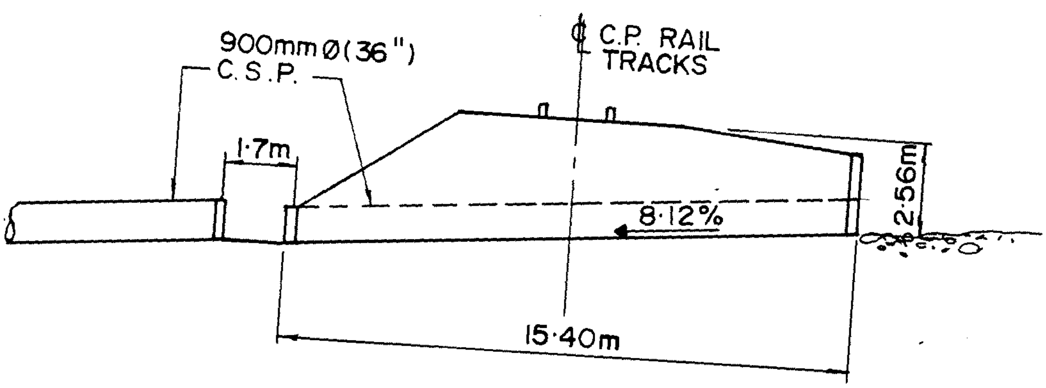


PROFILE

CULVERT Nos. He3 & He4

McElhanney

APPENDIX C

DRAWINGS

DRAWING 1: WATERSHED CATCHMENT AREAS

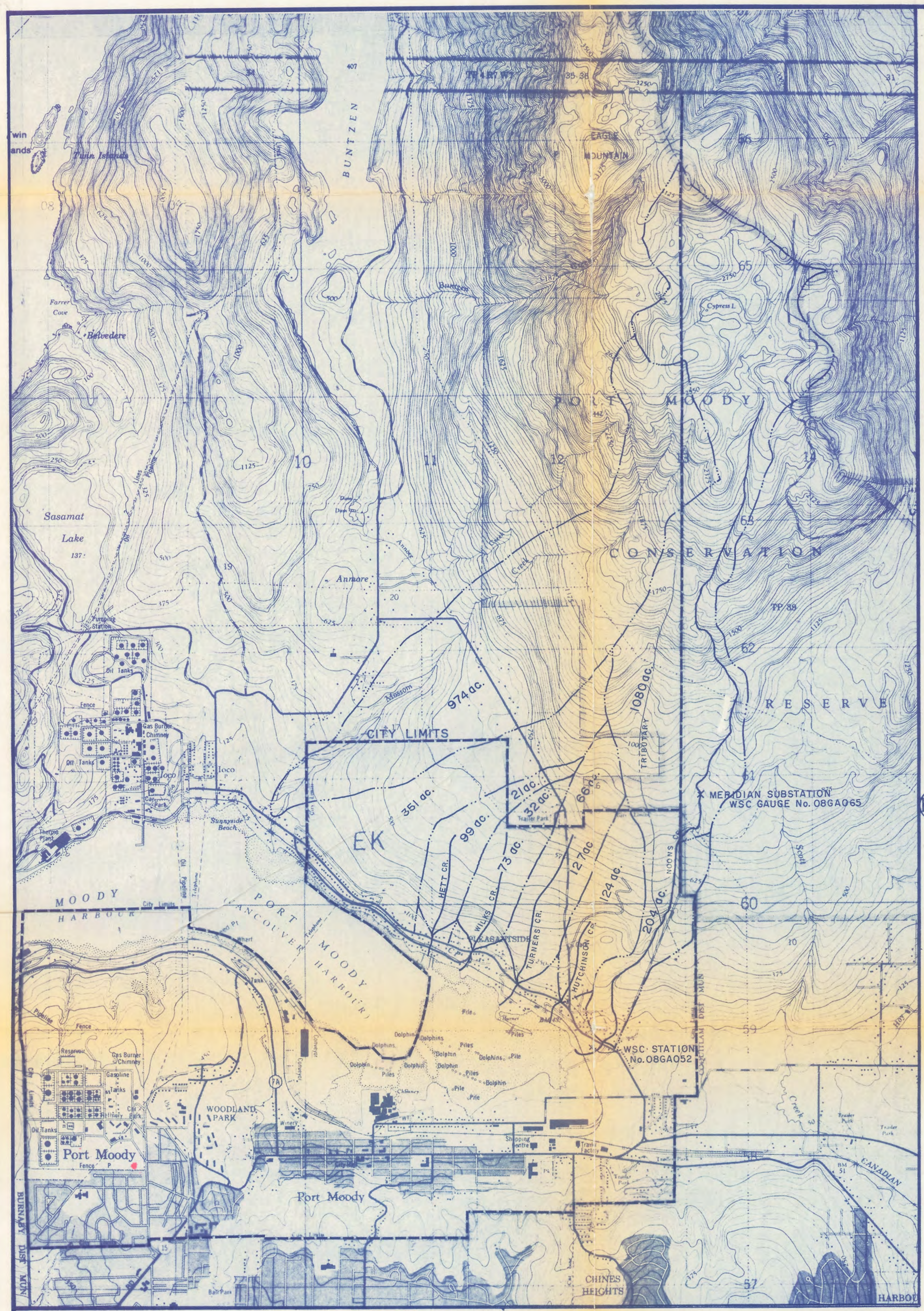
DRAWING 2: FLOOD ENVELOPES, POST-DEVELOPMENT FLOWS, EXISTING CONVEYANCE STRUCTURES

DRAWING 3: FLOOD ENVELOPES, POST-DEVELOPMENT FLOWS, EXISTING CONVEYANCE STRUCTURES

DRAWING 4: FLOOD ENVELOPES, POST-DEVELOPMENT FLOWS, PROPOSED CONVEYANCE STRUCTURES

DRAWING 5: FLOOD ENVELOPES, POST-DEVELOPMENT FLOWS, PROPOSED CONVEYANCE STRUCTURES

McElhanney



KEY PLAN
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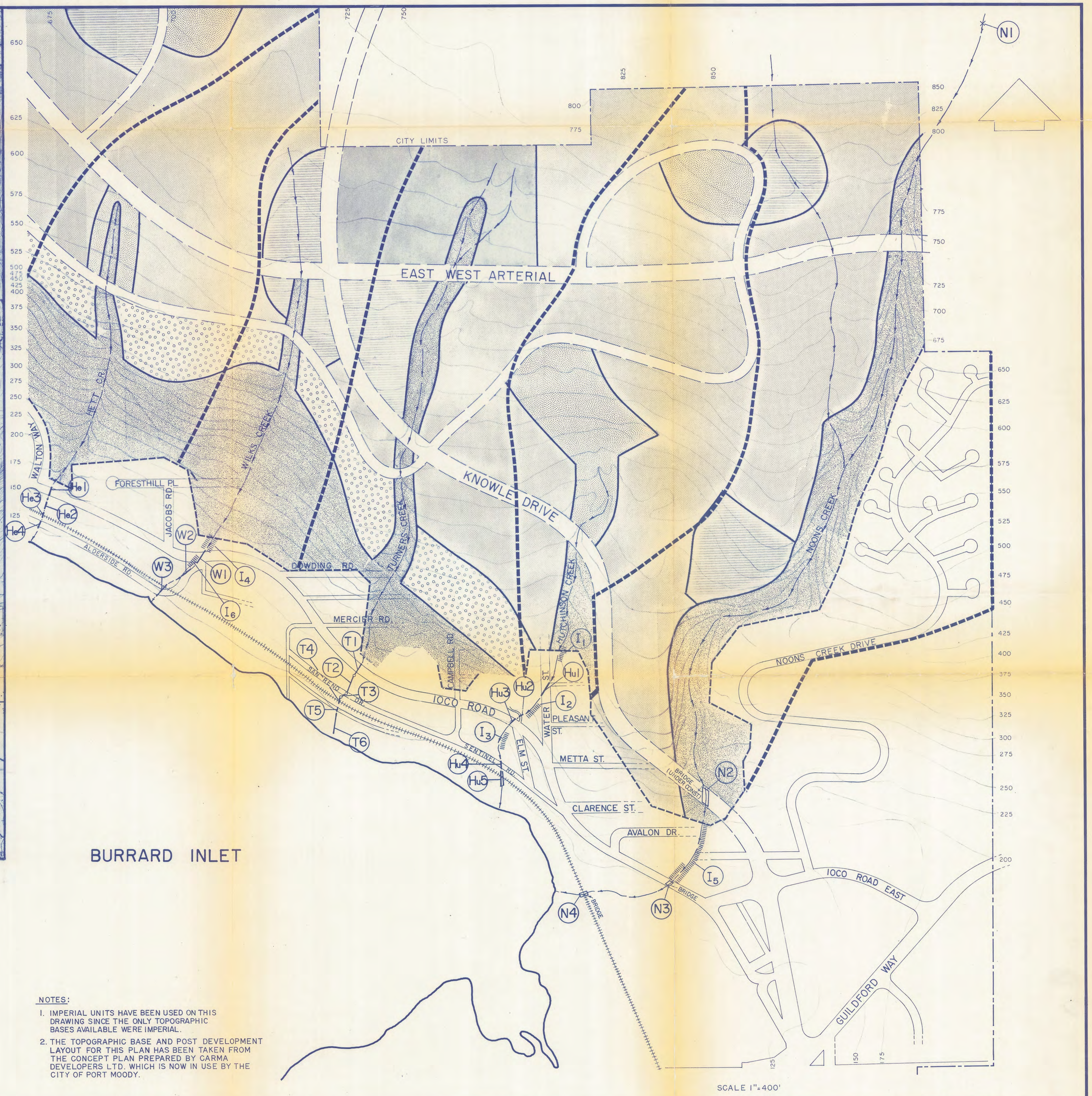
MILES
METRES 1000 500 0



LEGEND

NEIGHBOURHOOD PARK	ENVIRONMENTAL PRESERVE	SINGLE FAMILY LOTS	CREEK	RAILWAY TRACKS	CREEK CATCHMENT AREA
ESTATE LOTS	COMMUNITY PARK	MULTI FAMILY DWELLING	LIMIT OF EXISTING DEVELOPMENT	PROPOSED ARTERIALS	T2 LOCATION OF IMPROVEMENT WORKS

NOTES:

1. IMPERIAL UNITS HAVE BEEN USED ON THIS DRAWING SINCE THE ONLY TOPOGRAPHIC BASES AVAILABLE WERE IMPERIAL.
2. THE TOPOGRAPHIC BASE AND POST DEVELOPMENT LAYOUT FOR THIS PLAN HAS BEEN TAKEN FROM THE CONCEPT PLAN PREPARED BY CARMA DEVELOPERS LTD. WHICH IS NOW IN USE BY THE CITY OF PORT MOODY.

Job No. 34269-1-1/5

No.	Date	Revision	Dr.	Ch.
1	08.05.82	IMPROVEMENT WORKS LOCATIONS ADDED		

VANCAL 2287

McElhanney Surveying & Engineering Ltd.
13160 - 88 Ave., Surrey, B.C., Canada V3W 3K3 Telephone (604) 596-0391

McElhanney

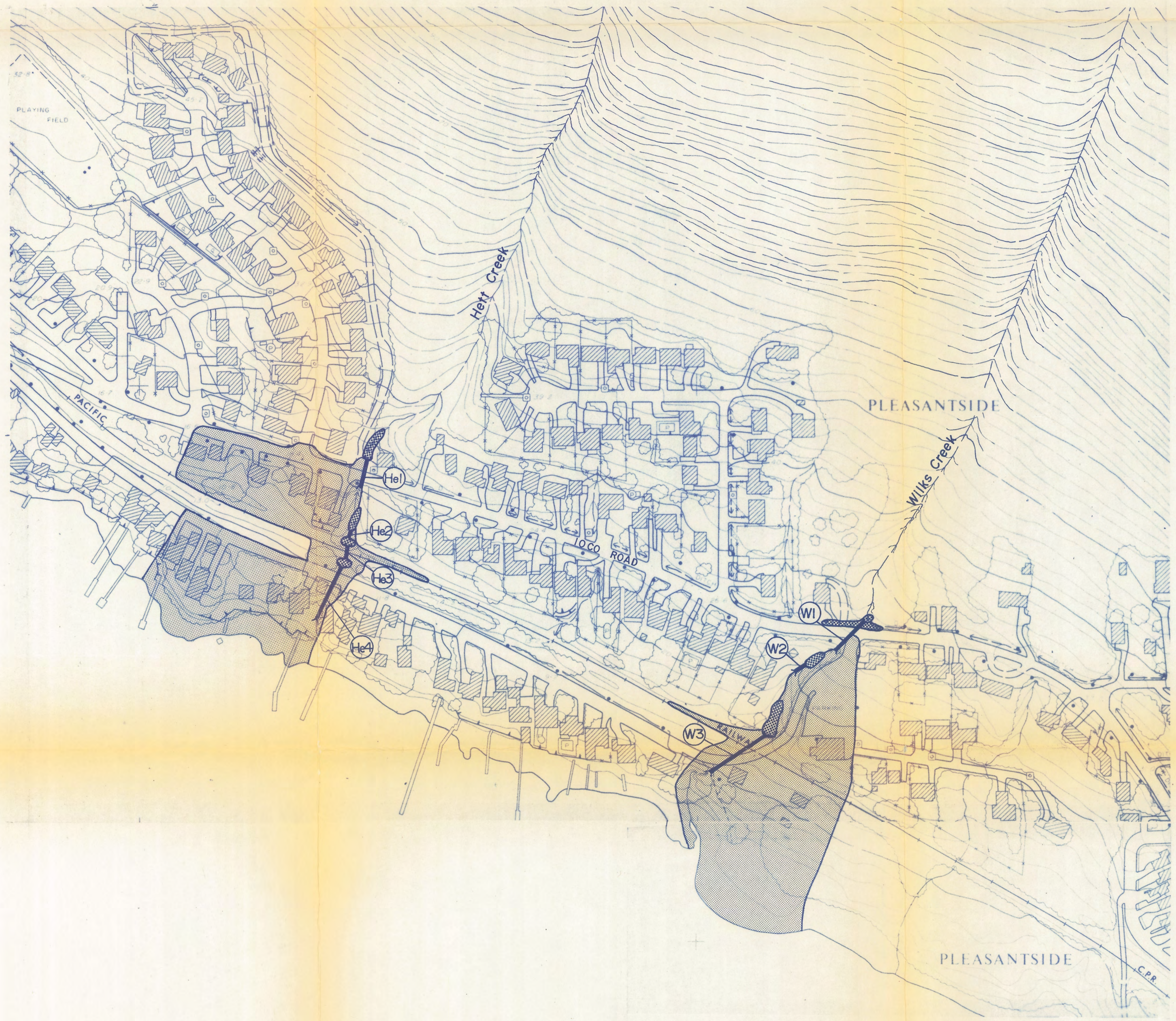
CITY OF PORT MOODY

NORTH SHORE RAVINE STUDY
WATERSHED CATCHMENT AREAS

PORT MOODY, B.C.

Designed DAH.	Job No. 34269-1	Drawing No. 1
Drawn JJ.	Scale AS SHOWN	
Checked DAH.	Date 23·9·80	
Approved	Revision 1	of 5

Destroy all prints bearing previous number ▲



LEGEND

- CULVERT
- STORM SEWER
- DEEP PONDING
- SURFACE FLOW
- T2 CULVERT REFERENCE NUMBER

Job No. 34269-1-2/5

No.	Date	Revision	Dr.	Ch.

McElhanney Surveying & Engineering Ltd.
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THE CITY OF PORT MOODY
NORTH SHORE RAVINE STUDY
FLOOD ENVELOPES – POST-DEVELOPMENT FLOWS
EXISTING CONVEYANCE STRUCTURES

Designed	Job No. 34269-1	Drawing No. 2
Drawn	Scale 1:2000	
Checked	Date MAY 1982	
Approved	Revision	of 5

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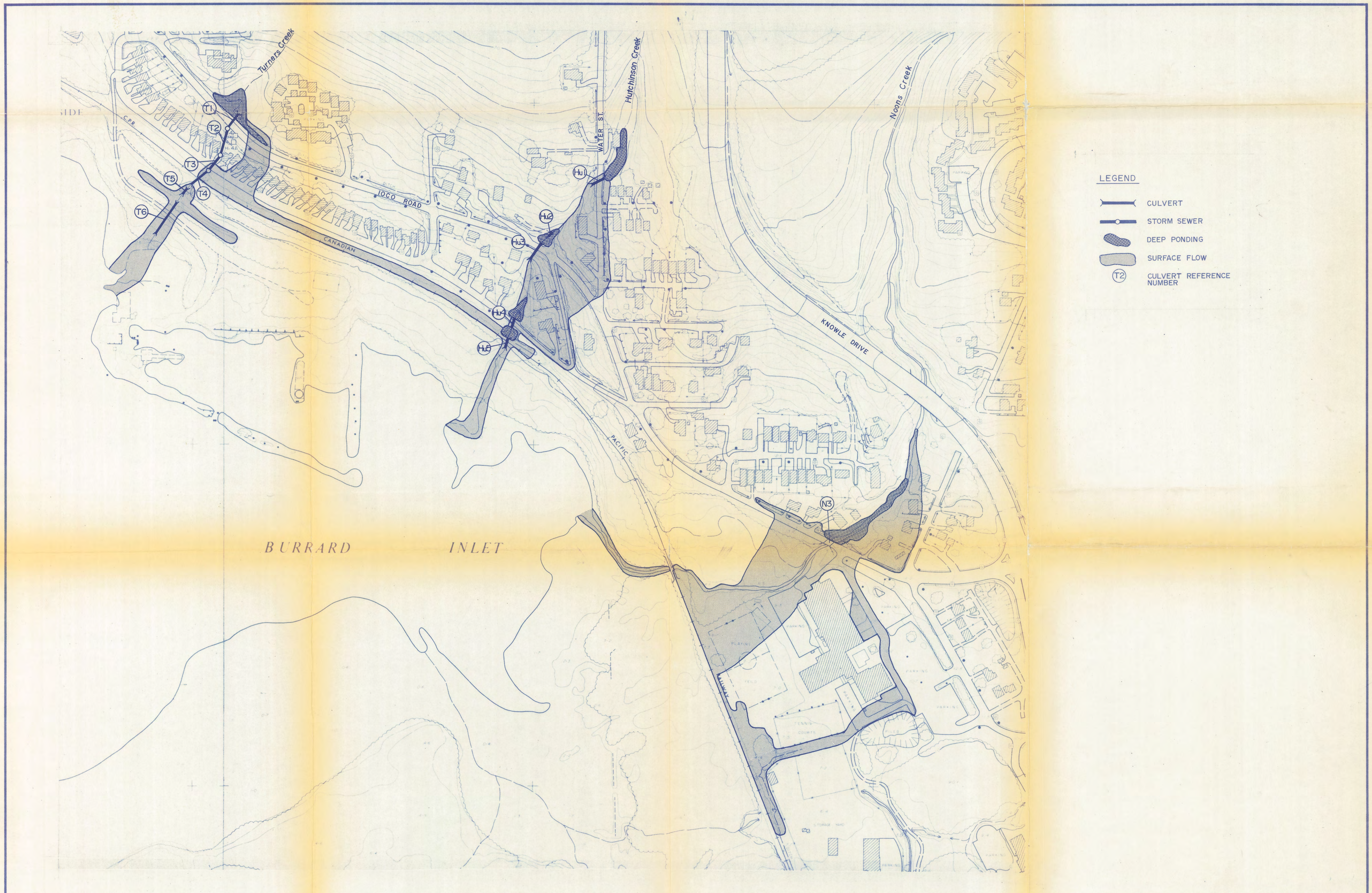

Job No. 34269-1-3/5

No.	Date	Revision	Dr.	Ch.

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THE CITY OF PORT MOODY

NORTH SHORE RAVINE STUDY

FLOOD ENVELOPES – POST-DEVELOPMENT FLOWS
EXISTING CONVEYANCE STRUCTURES

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA J. VYTASEK

Designed	Job No. 34269-1	Drawing No.
Drawn	Scale 1 : 2000	3
Checked	Date MAY 1982	
Approved	Revision	of 5

VANCAL 3297

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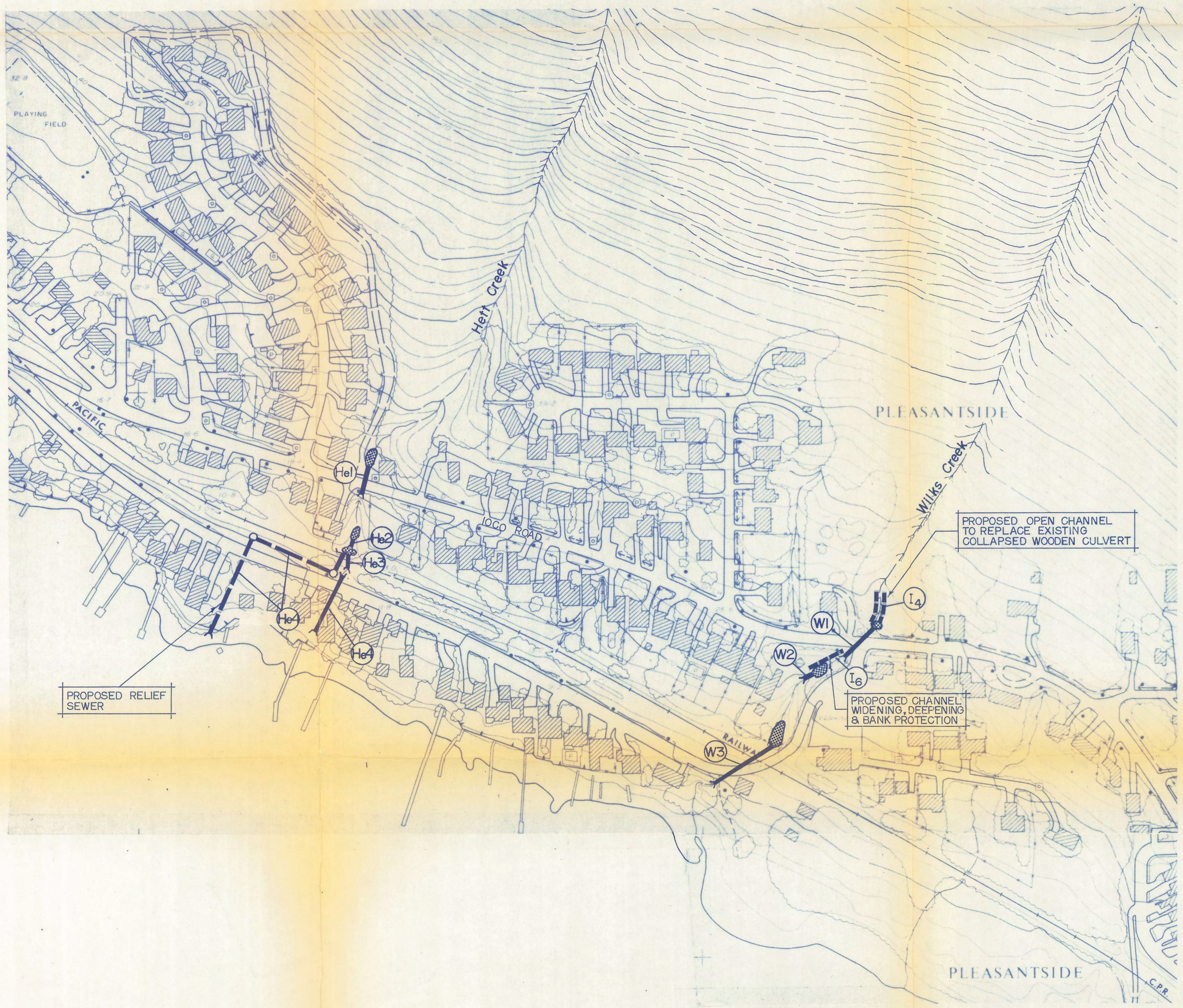

LEGEND

- CULVERT
- STORM SEWER
- DEEP PONDING
- SURFACE FLOW
- T2 CULVERT REFERENCE NUMBER

Job No. 34269-I-4/5

No.	Date	Revision	Dr.	Ch.

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THE CITY OF PORT MOODY
NORTH SHORE RAVINE STUDY
FLOOD ENVELOPES – POST-DEVELOPMENT FLOWS
PROPOSED CONVEYANCE STRUCTURES

Designed	Job No. 34269-I	Drawing No.	
Drawn	Scale 1:2000	4	
Checked	Date MAY 1982		
Approved	Revision	of 5	

VANCAL 3397

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Job No. 34269-1-5/5

No.	Date	Revision	Dr.	Ch.

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THE CITY OF PORT MOODY

NORTH SHORE RAVINE STUDY

FLOOD ENVELOPES – POST-DEVELOPMENT FLOWS
PROPOSED CONVEYANCE STRUCTURES

Designed	Job No. 34269-1	Drawing No.
Drawn	Scale 1:2000	5
Checked	Date MAY 1982	
Approved	Revision	of 5

VANCAL 3297

Destroy all prints bearing previous number