DEPARTMENT OF LANDS, FORESTS AND WATER RESOURCES

FRASER RIVER FLOOD CONTROL PROGRAM

UNDER 1968 FEDERAL-PROVINCIAL AGREEMENT

THE CORPORATION OF THE DISTRICT OF SURREY

SOUTH WESTMINSTER AREA

FLOOD CONTROL WORKS

REPORT ON FINAL DESIGN - FIRST STAG

CRIPPEN

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REPORT
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FINAL DESIGN - FIRST STAGE

CRIPPEN ENGINEERING LTD.
1605 HAMILTON AVENUE
NORTH VANCOUVER, B. C.

11 April 1975

DEPARTMENT OF LANDS, FORESTS & WATER RESOURCES
FRASER RIVER FLOOD CONTROL PROGRAM
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1. GENERAL

1.1 Authorization and Terms of Reference

By concluding an agreement on 4 September 1974 with Mr. B. E. Marr, Associate Deputy Minister of the Water Resource Services, Crippen Engineering Ltd. were authorized to execute the General Engineering Design of the Flood Control Works for the South Westminster Area of the Fraser River Flood Control Program under the 1968 Federal-Provincial Agreement. The South Westminster Area is located within the boundaries of, and administered by the Corporation of the District of Surrey.

The Agreement provides for General Engineering services to the Provincial Government to complete the final engineering design for the Primary Internal Drainage System and Dyking Works within the defined area, in accordance with the flood control criteria agreed to between the Federal Government of Canada and the Provincial Government of British Columbia.

The Agreement requires that the design be developed in two stages as follows:

a. First Stage - Final Design

Advancement of the Project to the stage where layout and typical design features can be illustrated and costs estimated to within 10 - 15 per cent of the cost estimates based on the final design. A project report is to be submitted at the completion of this stage for review before proceeding with the second stage.

b. Second Stage - Final Design

Prepare construction drawings and specifications in order that tenders can be invited to complete the work.

This report is prepared in accordance with the requirements of Sub-section a. above.

1.2 Scope of Work

The scope of work covered by the Terms of Reference and presented in this Report includes the following:

a. Completion of the necessary surveys, the design, and the calculation of quantities for work required on approximately 3.6 miles of dykes to reconstruct or improve them to the extent necessary for them to meet the criteria of the minimum design standards.

b. Completion of the necessary surveys and designs for the repair or replacement of five floodboxes, and for the construction of, or improvement to, approximately 5.5 miles of primary ditches.

c. Completion of the necessary surveys and design for the repair or replacement of four pumping installations.

d. Preparation of preliminary plans and the capital cost estimates for all works, including the preparation of a proposed construction schedule.

e. Completion of legal surveys and property title searches to determine existing easements and rights-of-way in the vicinity of the works. This section of work was carried out as an addendum to the original Terms of Reference.

f. Completion of plans showing the encroachment of the proposed new works on all properties adjoining the proposed new works. These plans have been submitted to the Surrey Municipality together with an assessment of the extent of land acquisition which will be required to accomodate the new works.

In order to carry out the work listed above, it was necessary to assemble all available mapping and information concerning existing services such as sewers, water mains, stormwater drains, telephone and power cables etc. This information was made available by the respective authorities and bodies responsible for the services.

Meetings and discussions were held with all interested parties to keep them informed on progress and details of the work. Among those with whom these discussions were held were:

The Engineering Department of the District of Surrey
The Project Engineer of the Canadian National Railways
The Fraser River Harbours Commission
The Department of Fisheries
The Greater Vancouver Sewerage and Drainage District
The Burlington Great Northern Railways
B. C. Hydro and Power Authority (Property Development)
Mainland Ready Mix Limited
Weldwood of Canada Limited
B. C. Telephone Company
Professors Lipson and Russell of the University of B. C.

Appreciation is expressed for the interest and co-operation given by all the above parties.

1.3 Reference Documents and Information

The following reference documents were supplied by the Water Investigations Branch of the Provincial Government, the District of Surrey, The Greater Vancouver Water and Drainage District, and others for use in the work.

a. Reports

i 'Report on South Westminster Dykes - Investigations and Remedial Treatment.' Prepared by Ripley, Klohn and Leonoff International Ltd. dated 27 May 1970.

ii 'Report to the District of Surrey, British Columbia, on Soil Conditions of Bridgeview and Adjacent Areas.' Prepared by Golder Brawner and Associates Ltd. dated March 1974.

iii 'Report on Internal Drainage in the South Westminster Dyking District' by Water Investigations Branch dated August 1972.

iv 'The Corporation of the District of Surrey, Outline Report on the Flood Control Works for the South Westminster Area, Water Investigations Branch (E. W. D. Bonham)' dated July 1973.

v 'Barker Creek, Whalley, Preliminary Analysis of Runoff

Potential', Water Investigations Branch (D. B. Tutt), dated September 1973.

vi 'Conceptual Design Specifications for Standard Drainage Pump Stations - Drainage Pumps.' Prepared by Associated Engineering Services Ltd. dated November 1970.

vii 'Conceptual Design Specifications for Standard Drainage Pump Stations - Electrical'. Prepared by Associated Engineering Services Ltd. dated November 1970.

viii A set of descriptions of pump stations and floodboxes in the South Westminster Area including sketch drawings, prepared by the Water Investigations Branch.

ix 'District of Surrey Drainage Report' by Municipal Engineering Department dated October 1963, including Appendix B, Preliminary Study of the Manson Road Canal (June 1963), and Appendix C, Rainfall Intensities (July 1963).

x 'Study of Municipal Services for Bridgeview and South Westminster Areas, Surrey, B. C.'. Prepared by S. L. Lipson, dated November 1974.

b. Other Data

i Aerial photographs scale 1 in. = 1000 ft, monochrome, date unknown.

ii 1 in. to 500 ft enlargements of the aerial photographs Item i.

iii 1 in to 500 ft enlargements of colour aerial photography, dated 18 June 1972.

iv Monochrome prints of aerial photography, scale 1 in. = 200 ft, date unknown.

v A monochrome print of aerial photograph of Weldwood area, scale 1 in. = 100 ft, date unknown.

vi Composite mosaic from monochrome aerial photography, scale 1 in. = 500 ft compiled by the Water Investigations Branch, dated December 1974.

vii Drawings by District of Surrey covering water supply, storm drainage and sewer drainage.

viii Drawings by G. V. S. and D. D. showing as-built details of the North Surrey Interceptor Sewer.

ix Drawings by District of Surrey showing ground and secondary ditch elevations.

x Integrated Survey Area No. 1 District Municipality of Surrey, Origin of Polyconic Rectangular Co-ordinates 49-123.

xi Hourly Fraser River water levels at New Westminster, by Water Survey of Canada.

2. INTERNAL DRAINAGE AND DISPOSAL

2.1 Design Criteria and Hydrology

a. Design Criteria

The internal drainage works are designed to discharge within a period of 24 hrs, the daily volume of runoff due to precipitation with a return period of 25 years.

The main ditches are designed with a minimum hydraulic gradient of 0.0001, with the flows being calculated by Manning's formula using a friction coefficient, $n = 0.035$.

The drainage works are designed so that the groundwater levels in the low lying peat areas will not be appreciably lowered below their present summer levels.

b. Hydrology

The daily quantities of runoff following storms of 25-yr return period in summer and winter have been estimated by the Water Investigations Branch (1, 2) for the main ditches under the Program. A summary of the daily runoff formulae is given below:

Season	Catchment	Formula
Winter	Upland	$Q = 0.33\ A^{7/8}$
	Lowland - rural	$Q = 0.16\ A$
	- urban	$Q = 0.18\ A$
	- sand fill	$Q = 0.08\ A$

Season	Catchment	Formula
Summer	Upland	Q = 0.026 A
	Lowland - rural	Q = 0.022 A
	- urban	Q = 0.026 A

where Q = flow in cfs

A = drainage area in acres

These runoff values are sensibly in accordance with the results of the hydrological study on Barker Creek (3).

Calculations were made to check the daily storm flows in References 1 and 2, and to estimate the discharge at additional points along the main drains and at secondary drain inlets.

c. Tidal Cycles

The range of the ocean tide undergoes a fortnightly variation due to cyclic changes in the declination and phase of the moon. That is, high tides become continually higher and low tides continually lower for about 7-1/2 days whereafter the roles reverse.

Maximum tidal ranges each year arise when both the sun and the moon have their greatest north and south declinations at the same time, during the summer and winter solstices in June and December.

Of particular importance to floodbox operation is the characteristic diurnal inequality, the difference in height between the higher and lower successive high water levels.

When this height inequality occurs there is also a pronounced inequality in time between successive high and successive low waters.

These tidal effects on the river levels combine to produce prolonged periods of up to 10 to 12 days during the months November through January during which floodbox discharges will be restricted by successive low tides of high water elevation, and successive low tides of short duration.

These effects are illustrated by Fig. 2.1 which shows the tidal cycle at New Westminster for the period 14 November to 31 December 1973.

Since these periods of up to 10 to 12 days occur during the months of maximum rainfall, particular care has been given to calculations of floodbox discharges by selecting a daily tidal cycle (13 to 14 December 1973) for the calculations typical of the periods of high low tides.

The variation of high tide levels at various locations in the South Westminster area was investigated by monitoring a high tide on 23 November 1974. The results are shown on Fig. 2.2 and show that the peak high tide elevation is slightly lower and about 15 minutes later at Beckstrom Road pump station than at Manson Road pump station. The effect of this is not considered to be significant.

During the Fraser River freshet the river levels do not drop sufficiently at low tide for reliable discharge through the floodboxes and with the exception of Canadian Collieries Ditch, all runoff must be pumped.

NOTES

1. Elevations are to Geodetic Datum.
2. Data from gauge 8 MH-25 at New Westminster (Water Survey of Canada). Data subject to revision.

SOUTH WESTMINSTER FLOOD CONTROL WORKS

TIDE CYCLE AT NEW WESTMINSTER

WINTER MONTHS

FIGURE 2·1

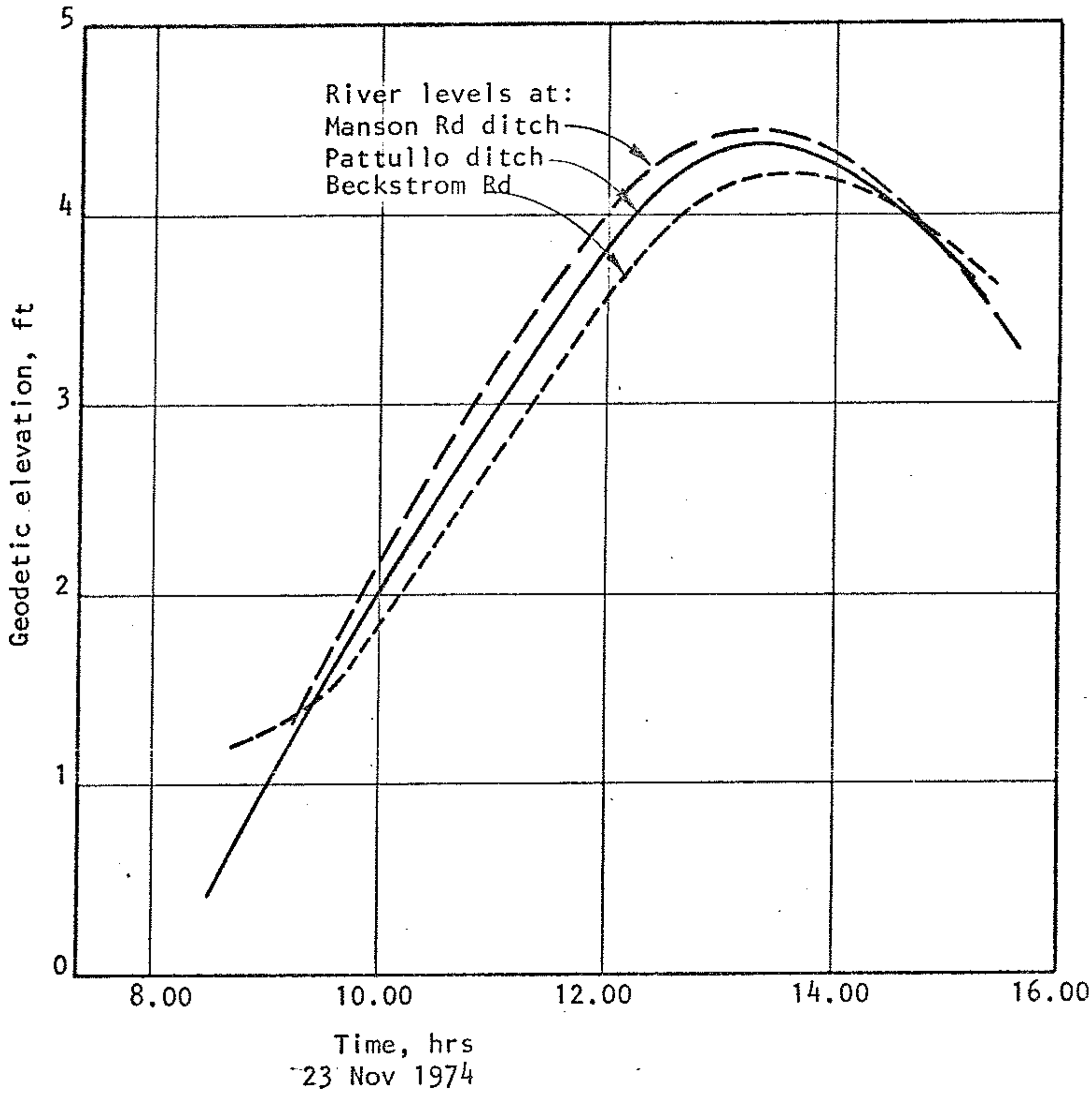


FIGURE 2.2
VARIATION OF HIGH TIDE PEAK,
SOUTH WESTMINSTER

During the winter months the ditch outlet works are designed to discharge storm runoff by combinations of floodbox operation and pumping.

2.2 Drawings

The drawings presented in Section 6.2 include 24 drawings which illustrate design concepts for the new works. The key plan (Sheet 1) shows the main ditches which are included under the Program, the existing secondary ditch layout, and additional secondary ditches which are recommended to improve the secondary drainage. The secondary ditches will not be improved or constructed under the Program.

Sheets 7 and 8 are the drawings for Manson Road Outlet Works and the details shown on these are typical for all the outlet works. The remainder of the outlet works are illustrated by general arrangement drawings only.

Ditch details are shown on the plan, profile and section drawings for each ditch. Additional hydraulic and section details are given in tabular form on Sheet 24.

2.3 Ditch Outlet Works

a. Outlet Works Design

i Canadian Collieries Ditch

Canadian Collieries Ditch has sufficient elevation to discharge by gravity during normal summer and winter river level conditions. However, a floodbox has been included to guard against flooding of the land south of the G.N.R. and C.N.R. railway tracks (approx El 9.0) when the river attains the exceptionally high design river level of El 12.0. During such high water, summer rainfall could cause the Canadian Collieries Ditch to discharge out of Inlet R2 south of the railway tracks. Extensive flooding would not occur, however, since the water would flow eastwards along the G.N. railway ditch and enter the Manson Road ditch at Inlet L4, and be pumped into the river through the Manson Road pump station.

ii <u>Manson Road Ditch</u>

Seventy-four per cent of the winter storm runoff in Manson Road ditch is from upland with sufficient elevation for it to be discharged by gravity. The remainder emanates from low lying land adjacent to Scott Road and towards the river where the ground elevations of the land adjacent to the ditch are as low as El 4.5 to 5.0. The ground elevations along parts of the ditch banks have already been raised by property owners to above the winter high tide levels, and the opportunity exists to construct low dykes along the ditch sides and form a continuous canal which will discharge the upland runoff by gravity at all times and so eliminate the necessity for winter pumping.

The Manson Road Ditch Outlet Works have therefore been designed for pumping during summer months only, when the whole of the summer storm runoff must be pumped at times of high river level.

During the winter storm of 25-yr return period, the ditch water level at the floodbox will rise to about 0.3 ft above the river high tide levels, and the estimated typical variation in ditch levels due to a storm during a period of high elevation winter tides is shown on Fig. 2.3.

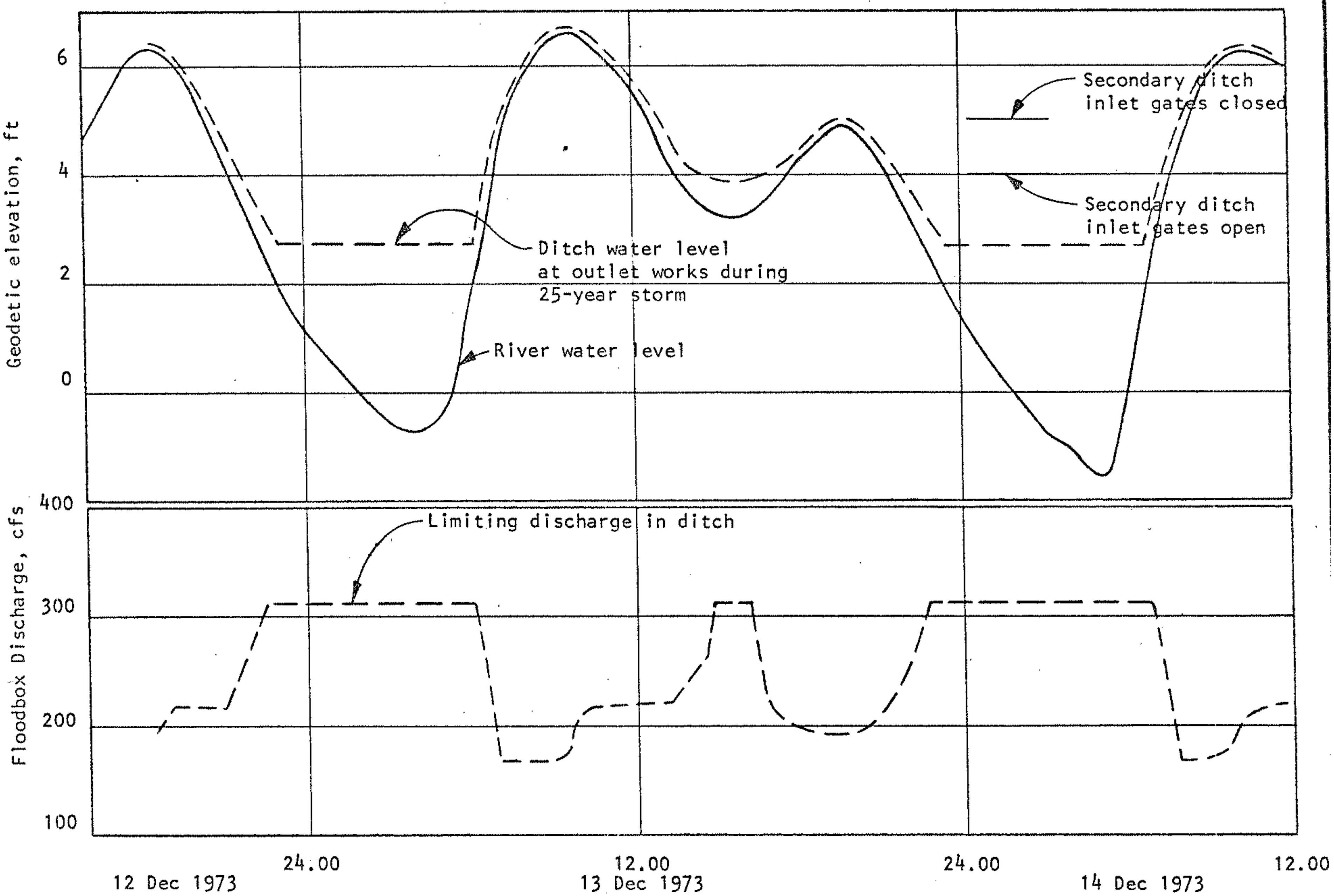
The levels and discharges shown on the figure were calculated allowing for storage in the ditch, and the effects of the floodgates on the secondary ditch inlets leading into the main ditch.

During much of the winter, when the river levels include low tides of lower elevation and longer duration than the tide shown on Fig. 2.3, the ditch water levels at high tide during the 25-yr winter storm will be lower than those shown on the figure.

iii <u>Old Yale Road Pattullo and Beckstrom Road Ditches</u>

Outlet works for Old Yale Road, Pattullo and Beckstrom Road ditches have been designed for discharge during the summer months by pumping only, and for discharge by combinations of pumping and floodbox operation during winter months.

Since the floodboxes close at high tide when the river level rises above approximately El 4.0, the discharges of main ditches and floodboxes at low tide will be greater than the steady 24-hr, 25-yr storm discharge by factors of 2.1, 1.5, and 2.2 for Old Yale Road, Pattullo and Beckstrom Road outlet works respectively.



Note - River water level shown for typical period of winter high tides

FIGURE 2.3
MANSON ROAD DITCH FLOODBOX, WINTER OPERATION

For example, the discharge during a winter storm at Beckstrom Road outlet works is given below, based on the design high tide cycle (Fig. 2.3):

Quantity required to be discharged in 24 hr	2400 cfs hr
Quantity discharged by pumping during 15.3 out of 24 hr	510 cfs hr
Quantity discharged by floodbox during 7.8 hr at full capacity (211 cfs) and 1.9 hr at part capacity	1890 cfs hr

The required capacity is 211 cfs which is 2.2 times the steady 24-hr runoff of 25-yr return period.

b. Hydraulic Design

i Pumphouses

The designs include for complete replacement of all existing pumping facilities, at Manson Road, Pattullo and Beckstrom Road pumphouses, and the installation of a new pump station of Old Yale Road. The total pumping capacity in the new pumphouses will be double the existing capacity as shown below:

Location	Existing Pumps		Proposed Pumps	
	Size (discharge)	Capacity* cfs	Approximate Size (discharge)	Capacity cfs
Manson Rd Ditch	1 x 18 in. dia 1 x 24 in. dia	13.4 22.3	1 x 16 in. dia 2 x 24 in. dia	11.2 44.6
Old Yale Rd Ditch	nil		2 x 16 in. dia	22.3
Pattullo Ditch	1 x 10 in. dia 1 x 24 in. dia	11.2 22.4	1 x 16 in. dia 2 x 24 in. dia	11.2 44.3
Bridgeview	1 x 18 in. dia	13.4	1 x 16 in. dia 1 x 24 in. dia	11.2 22.3
Total	5	82.6	10	167.5

*Estimated by Water Investigations Branch

The proposed capacities are sufficient to discharge all the summer storm runoff over a period of 24 hours. Maintenance and down time has been provided for by allowing 20 per cent additional pumping capacity.

Pump station equipment will comprise electrically driven vertical axial flow pumps set in a wet well and discharging through pipes to a bay of the outlet structure. The exits of the pipes will be fitted with flap valves, and the bay of the outlet structure will include a trashrack to prevent debris reaching the flap valves on a rising tide.

The pump station inlets will include trashracks with openings between bars of 2 in. and 6 in. for upstream and downstream trashracks respectively.

Dimensions of the pump sumps were taken from recommendations of the Hydraulic Institute (4). When the final choice of pumping equipment is made, the pump manufacturer may recommend modified dimensions to suit the pump model supplied.

Two pump sizes, approximately 18 and 24 in. diameter, were chosen to be installed in different combinations in the various pump stations. It is recommended that one spare complete pump of each size should be held so that all pump stations can remain operative while pumps are undergoing major servicing.

Preliminary hydraulic calculations were made to estimate the total pumping head. The pumping head exluding pump losses is given below:

	ft
Ditch water elevation during pumping	3.25 to 4.09
Maximum river elevation (summer flood)	12.0
Static pumping head	5 to 9
Estimated pipe and outlet losses	2
Total pumping head	7 to 11

ii Floodboxes

The typical floodbox design comprises an inlet structure with a trashrack, a conduit through the dyke, and an outlet structure with floodgates and trashrack.

A hydraulic rating curve was prepared for a single 5-ft x 5-ft floodbox and values from the curve are tabulated below:

Difference between ditch and river water levels, ft	Discharge through floodbox cfs
0.1	38
0.2	58
0.5	92
1.0	131
2.0	180
3.0	208

Trashrack openings of 4 in. and 6 in. are recommended for the upstream and downstream trashracks respectively. The downstream opening should be larger than the upstream opening so that debris will tend to pass through the structure, but logs in the river will be prevented from reaching floodgates on a rising tide.

A side-hinged floodgate design was chosen to minimize hydraulic losses.

c. Pump Station Equipment

i Mechanical Equipment

Detailed specifications for the pumping equipment are contained in Conceptual Design Specifications for Standard Drainage Pumps (5), which are suitable for the equipment for the South Westminster pump stations.

Axial flow pumps with oil filled, enclosed line shaft bearings, and grease or oil lubricated lower impellor bearings are recommended, to avoid corrosion problems with pumps which operate infrequently.

ii Electrical Equipment

The Conceptual Design Specification for Standard Drainage Pump Stations, Electrical (6) are comprehensive and will be followed.

Additional exterior lighting to illuminate the trash-racks is recommended.

The recommended pump control settings are given in 2.4a. iii. The recommended high and low water level alarm and station lockout settings which operate if a pump fails to shut-off are given below.

Pump Station	Low water level alarm and lockout Geodetic Elevation, ft	High water level alarm Geodetic Elevation, ft
Manson Road	2.80	8.0
Old Yale Road	2.55	8.0
Pattullo	2.55	8.0
Beckstrom Road	2.10	8.0

2.4 Internal Drainage

a. Ditch Design

i Hydraulic Design

In accordance with the terms of reference, calculations of flow in ditches were made using Manning's formula with the friction coefficient n=0.035. The following friction coefficients were used for calculation of head losses at structures:

concrete	n = 0.014
corrugated steel pipe	n = 0.024
wood-stave pipe	n = 0.024

Backwater curves upstream of structures were computed using a programmable calculator.

The maximum permissible water velocity in an unlined ditch was taken as 3.0 fps, and in accordance with the Terms of Reference no ditch was designed with a water surface gradient flatter than 0.0001.

Ratios of bed width to depth of flow of between 2 and 5 were chosen for ditches depending on required capacity and existing ditch dimensions. Minimum radii for bends were taken as approximately 5 times the design water surface width, unless erosion protection was provided.

ii <u>Geotechnical Design</u>

Maximum permissible ditch side slopes are 1V:2.5H for ditches in peat, and 1V:2H or 1V:1.5H in more stable soil. Exceptions are Old Yale Road, 126A and 128 Street ditches, where limited land is available for ditch reconstruction, and the existing side slopes of about 1V:2H which appear to be stable have been retained.

Where ditch dykes are included in the designs, the dykes have been designed to have a 12-ft crest width, 1V:2H side slopes, and 2-ft freeboard above the ditch water level at high tide during a winter storm. An exception is the dyking along the Manson Road ditch where the freeboard was reduced to 1.0 ft and special construction procedures are necessary (see 2.4c.i).

The above considerations are discussed in detail in Section 3.

iii <u>Groundwater Control</u>

The Terms of Reference require that the elevation of the groundwater table in the peat areas drained by the

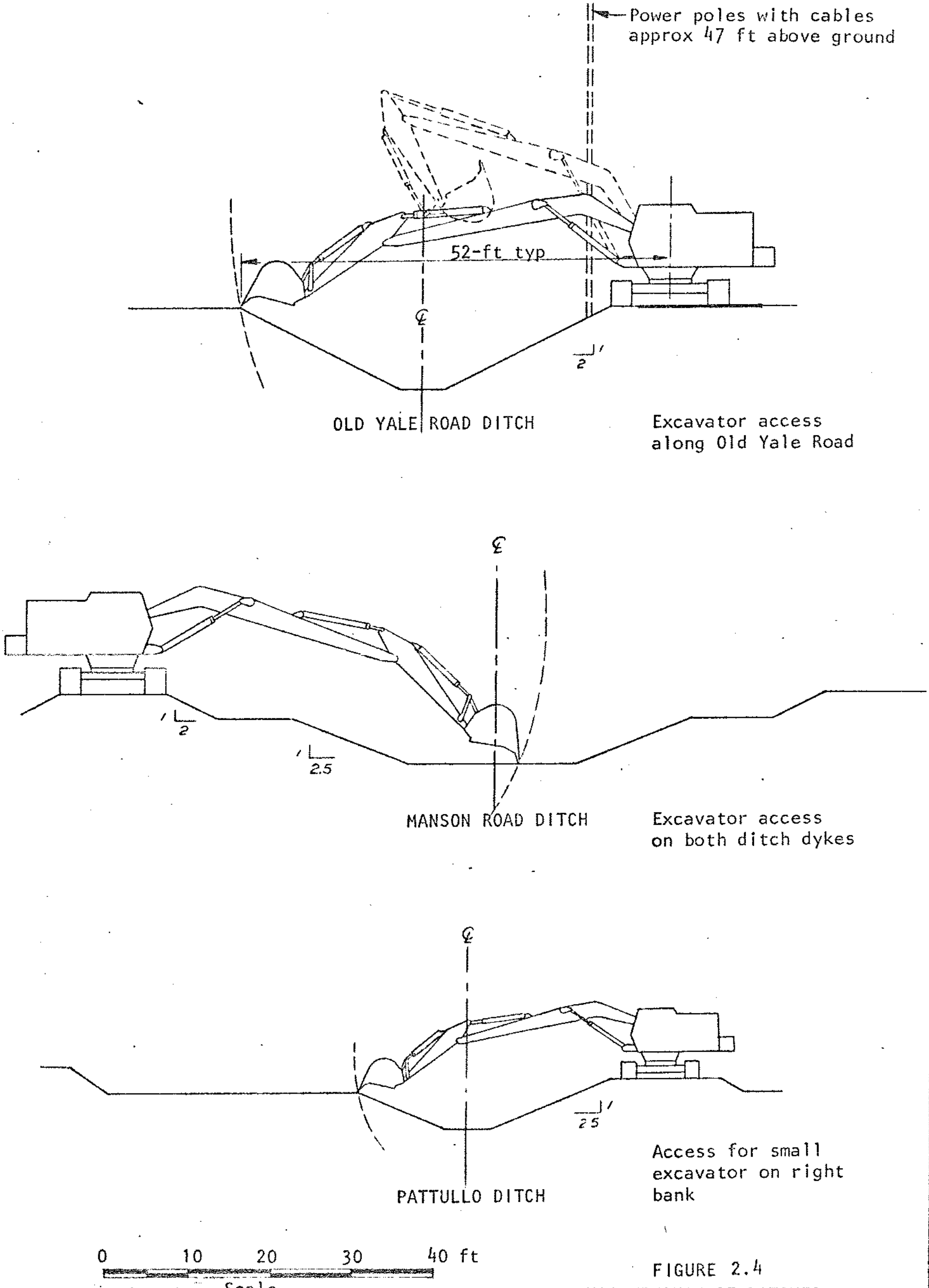


FIGURE 2.4
MAINTENANCE OF DITCHES

ditches will not be appreciably lowered below the present summer levels.

Groundwater levels in the peat area rise close to the surface during a winter of normal rainfall, and fall during the summer due to drainage into the ditches, and evapo-transpiration by vegetation. During the winter, substantial variations in ditch water elevations will occur owing to storm runoff and tidal influence. However it is not anticipated that this will cause any significant lowering of water table levels.

During the summer, the ditch water levels will be controlled by the pumps, since all discharge from the low lying areas must be pumped at times of high river levels.

Since the ditch water level influences the groundwater table, control of the groundwater level can be accomplished by adjustment to the pump control settings.

Recommended pump control settings for groundwater table control are shown below:

Pump Station	Pump Settings (Geodetic Elevation)					
	Unit 1		Unit 2		Unit 3	
	on	off	on	off	on	off
Manson Rd	4.00	3.80	4.10	3.90	4.20	4.00
Old Yale Rd	3.75	3.55	3.85	3.65		
Pattullo	3.75	3.55	3.85	3.65	3.95	3.75
Bridgeview	3.30	3.10	3.40	3.20		

Notes: 1. Unit 1 is an 11 cfs pump.

2. The pump settings may require field adjustment.

iv Maintenance

An access width of 12 ft has been provided where necessary along ditches for maintenance of side slopes and cleaning mud and weeds from ditch beds.

Figure 2.4 illustrates typical maintenance operations using small (1 1/4 cu yd) hydraulic excavators for small ditch sections and larger (3 cu yd) excavators for wide ditch sections such as Manson Road ditch.

Small ditches require access for maintenance along one bank only, while wider sections need access along both banks.

Old Yale ditch has no access provided, since it can be cleaned from the existing Old Yale Road. However maintenance using an excavator will be hampered by the existing power line, since the excavators must work around the power poles and under the cables.

Spoil from maintenance operations should be loaded into trucks and disposed of.

A maintenance manual will be prepared during the final design stage of the Program.

b. Canadian Collieries Ditch

i Alternative Layouts Considered

The following alternatives were considered for the alignment of Canadian Collieries Ditch (Fig. 2.5).

1. New channel alongside G.N.R. to join Manson Road ditch, replacing the existing channel from the railway tracks to the river.

2. New culvert and ditch under railway tracks, to join existing ditch. Enlarge and straighten existing ditch between the rail tracks and the river outfall.

3. New culvert under railway tracks, and new alignment to river outfall.

93 per cent of the storm discharge in Canadian Collieries ditch results from runoff from land at sufficient elevation to discharge into the river by gravity during summer and winter high tides. Alternative 1 would necessitate pumping of all the discharge during summer months, and in winter would result in flooding behind the Manson Road ditch left dyke at high tide periods. Alternative 1 was therefore eliminated.

Hydraulic designs were prepared for Alternatives 2 and 3, and cost estimates were prepared. The alternative alignments were the subject of numerous

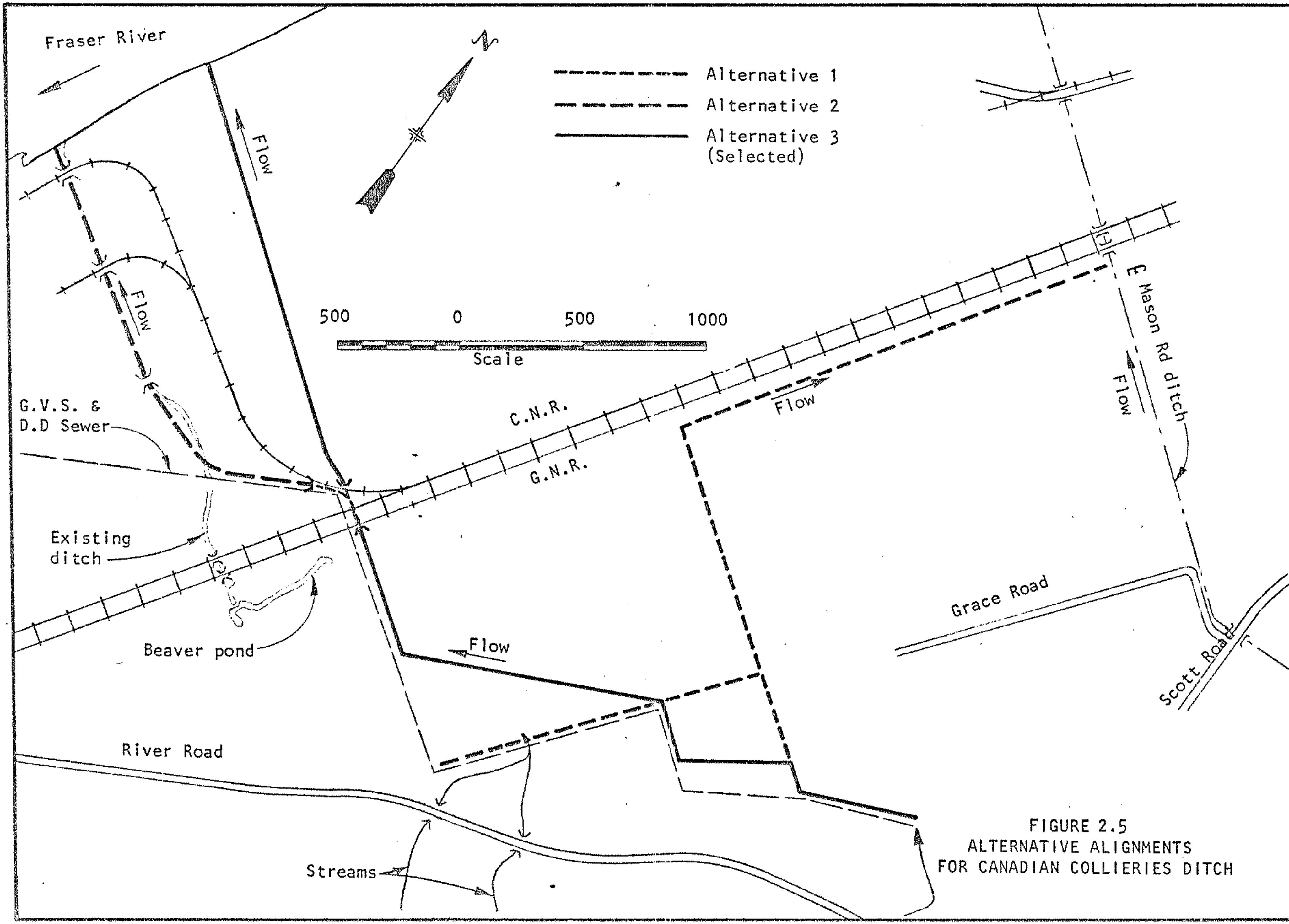


FIGURE 2.5
ALTERNATIVE ALIGNMENTS
FOR CANADIAN COLLIERIES DITCH

discussions between the Water Investigations Branch, the Fraser River Harbours Commission, B. C. Hydro and Power Authority, and Surrey Municipality, with the following results:

1. The cost study showed that Alternative 3 was the more economic solution.

2. Fraser River Harbours Commission and B. C. Hydro and Power Authority agreed to grant, free of charge, a strip of land 90-ft wide for the new ditch with its centreline on their common boundary.

3. Fraser River Harbours Commission agreed to backfill the existing ditch at their own expense.

Alternative 3 is therefore presented in the designs.

ii Hydraulic Design

A relatively flat gradient of 0.00051 has been adopted for the 1700-ft section of the ditch north of the G.N.R. rail track in order to allow for drainage of water from the beaver ponds and low lying ground south of CN and GN railway tracks. This area was surveyed showing that the ground levels are 8.0 to 14.0 ft and the top of the beaver dam is at El 7.9 ft. Drainage of the area will be possible through secondary ditch L3.

In the higher reaches, advantage has been taken of steeper ground slopes to minimize channel excavation by designing for a velocity of flow of 3 fps.

c. Manson Road Ditch

i Ditch Dykes

A previous report (7) recommended dyking Manson Road ditch, and the scheme was partly constructed. The work included installation of ditch inlets of wood-stave pipe with flap valves, and some enlargement of the channel section. However, the work was abandoned when the ditch side slopes and bed proved unstable.

The proposed dykes include special design and construction measures to overcome the problems previously encountered. These are described in detail in Section 3.

The dyke elevations shown on the drawings include 1 ft of freeboard above the computed winter storm high tide backwater curve. This reduction in the usual freeboard requirement of 2 ft is considered justifiable due to the very low probability of a 25-year return period winter storm occurring in conjunction with a high tide of above El 7.0. The dykes should be kept as low as possible to minimize soil stability problems.

ii Hydraulic Design

The Manson Road ditch drainage area includes some ground between Timberland Road and Scott Road of average El 4.5 to 5.0 ft. A maximum permissible ditch water level of El 4.0 ft at Scott Road during a winter low tide was adopted for hydraulic design of the ditch. The ditch bed level should not be lowered below the bottom of the loose mud, due to the soil stability problems as discussed in Section 3 and a wide ditch is therefore required to convey the flood discharge at low tide.

The discharge at the floodbox during a winter storm will vary between high and low tides as shown on Fig. 2.3. During high tides, stormwater will fill the secondary ditches behind the Manson ditch dykes, and be released into Manson ditch by the floodgates on the ditch inlets at low tide. Some flooding due to local runoff from land behind the dykes may occur at high tide periods during exceptionally severe storms.

Upstream of Scott Road the natural gradient of Manson ditch steepens, and drop structures have been included in the designs to avoid water velocities exceeding 3 fps.

d. Old Yale Road Ditch

The natural ground level falls from about 11 ft elevation near the floodbox to 5 ft near Scott Road. Flooding of Old Yale Road immediately upstream of Scott Road has been observed during the winter 1974/75 following minor storms, and is believed to be due to inadequate capacity of, or partial blockage in the existing culvert under Scott Road.

The designs include an enlarged culvert under Scott Road, and an enlarged ditch section to the outlet works.

Hydraulic calculations were based on a maximum permissible ditch water level at Scott Road of 4.5 ft at low tide during a winter storm.

The existing side slopes in Old Yale Road ditch appear to be stable at 1V:2H, and this slope has been retained for the new ditch.

An existing power line is located along the western edge of Old Yale Road between Scott Road and the floodbox, with the timber poles in the side slope of the existing ditch, and the lines about 47 ft above the road elevation. Care will be necessary during reconstruction of the ditch when the excavators must work around the power poles and under the lines.

The dyke ditch between Old Yale Road and Pattullo ditches will be retained and reconstructed further away from the river dyke. Since the operating levels of Old Yale Road and Pattullo ditches are similar, the dyke ditch will assist in disposing of floodwater in case of pump failure at either of the outlet works.

e. Pattullo Ditch

The Pattullo drainage network will serve the low lying land adjacent to the King George Highway by the existing secondary ditches, and will convey runoff from the high ground to the south along a new ditch to the outlet works.

The water level at the junction of the secondary ditch and the new ditch at chainage 9 + 30 ft will determine the flooding of the low lying land. Since the ground levels are 5.0 to 5.5 ft, a maximum ditch water level of 4.0 ft at low tide during a winter storm was adopted as the basis for hydraulic design.

To avoid flooding at Scott Road the gradient of the ditch between Scott Road and the outfall is relatively flat and steepens upstream of the road.

Between chainage 9 + 30 and 45 + 50 ft, where the new ditch line follows the northeast side of B. C. Hydro and Power Authority railway, a low dyke will be required where the existing ground is less than 2 ft above the ditch design water level. A dyke is not required on the left bank since the railway embankment is of sufficient elevation to contain the storm discharge in the ditch.

The upper reach of Pattullo ditch from chainage 45 + 50 ft will follow the existing GVS and DD sewer line and will be located on its northwest side. A dyke, 2 ft above the ground level on the left bank will prevent erosion of the ditch slopes by ensuring that runoff from the escarpment enters the ditch at the inlets. Where the ground is less than 2 ft above the ditch design water level, right bank dykes will contain stormwater within the ditch.

In the lower reach of the ditch, the existing ditch passes between the Pattullo Bridge approach piers, and around timber piles of the CN and B. C. Hydro railway trestles. Since the new ditch will be considerably larger than the existing ditch, culverts have been included in the designs at these crossings.

In order to provide pumping facilities in case of pump failure, the existing ditch at chainage 6 + 30 ft connecting to the Bridgeview area is to be retained, but a sluice gate structure has been added.

f. <u>Beckstrom Road Ditches</u>

During the winter 1974/75 the Bridgeview ditches were examined following storms. Although the ditches were full of water, in many places no movement of water could be detected. This is principally due to lack of maintenance and the multiplicity of small culverts in the ditches. Regardless of improvements to the main ditches and outlet works, the drainage of the area will not be effective until the secondary ditch layout has been improved.

i <u>Secondary Ditches</u>

The secondary ditches are outside the scope of this study. However, for the purpose of establishing design water levels in the main ditches, the internal drainage of the Bridgeview area was examined.

Fig. 2.6 shows the average ground levels in the area. It was found that the secondary ditch on the east side of 125A Street was the determining factor in establishing the

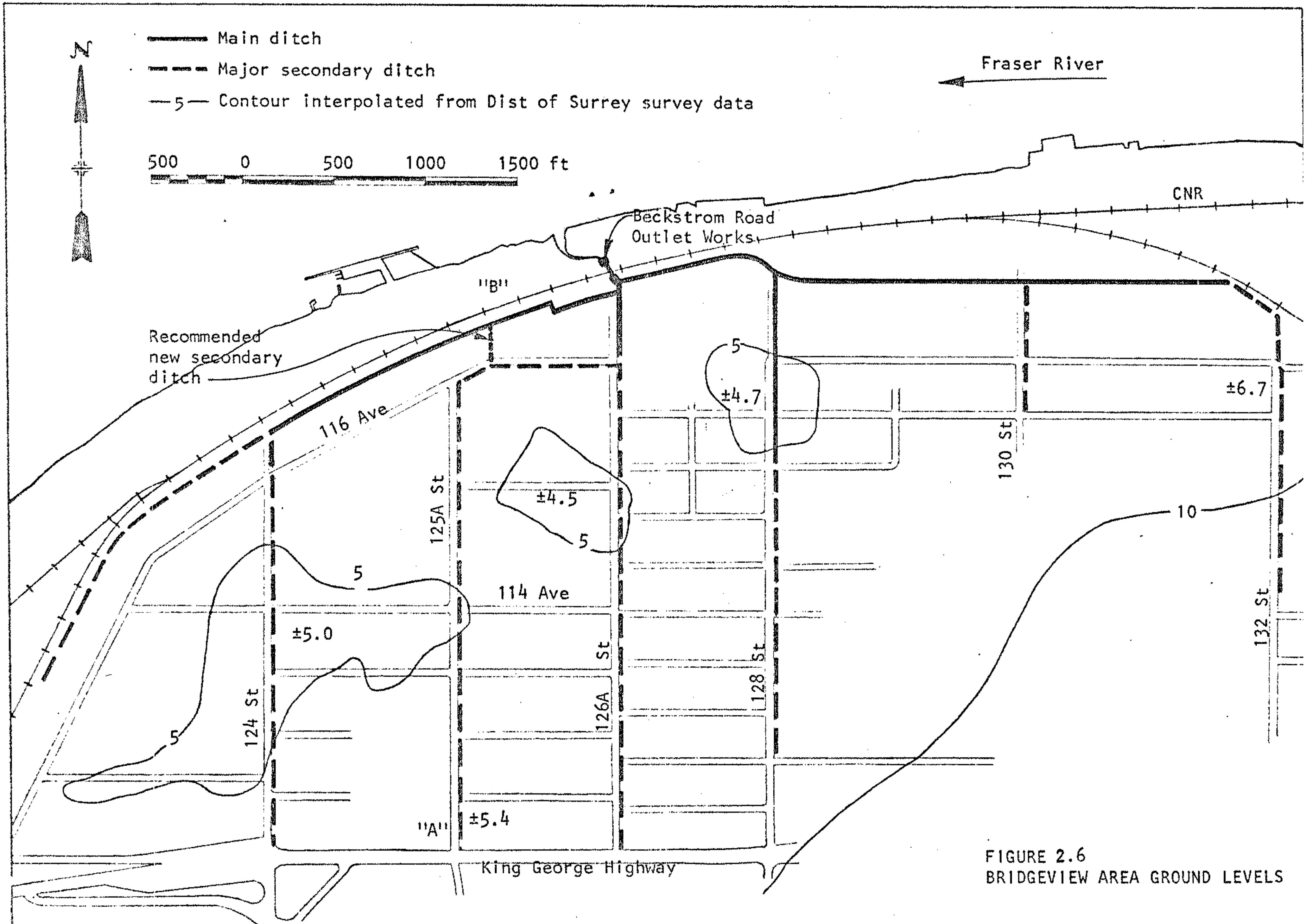


FIGURE 2.6
BRIDGEVIEW AREA GROUND LEVELS

maximum low tide water level at the outlet works as follows:

Ground level at point "A" (Fig. 2.6)	El 5.4 ft
Freeboard in ditch during storm	1.0 ft
Headloss in 125A Street secondary ditch,	
2,400 ft at slope 0.0001	.24 ft
6 culverts	.73 ft
Headloss in C.N.R. Ditch West from point "B" to outlet works	.1 ft
Water level at outlet works	3.30 ft

The 125A Street secondary ditch discharge will be improved if a connection is made between Industrial Avenue and the C.N.R. Ditch West as shown on Fig. 2.6. The hydraulic design of the main ditches was carried out assuming that this connection will be made.

ii <u>Hydraulic Design</u>

The minimum permissible gradient of 0.0001 was adopted for design of the C.N.R. Ditch West, 126A Street ditch, and C.N.R. Ditch East to the junction of 128 Street ditch. This gradient will result in a ditch water velocity of about 1 fps during major storms.

Advantage was taken of slightly higher ground elevations to increase the gradient of the C.N.R. Ditch East to 0.00067 upstream of its confluence with 128 Street ditch.

2.5 Structural Design

The structural designs shown on the drawings are preliminary, and are intended to be refined during the final design, second stage, of the program.

a. Ditch Outlet Works

i Foundation Conditions

The ditch outlet works are to be constructed on various thicknesses of peat and organic silt and measures necessary to deal with these unstable soils are described in Section 3.

ii Floodboxes

The floodbox inlets and outlets include provision for stoplogs for dewatering the structure. Dewatering may be necessary for cleaning out silt, or for removing trashracks.

The main supporting steel for the trashracks is to be bolted to holding down studs in the concrete, and will not normally require removal. The trashrack sections will be bolted to the supporting steelwork and individual sections may be removed for repair or replacement.

Trashracks may be cleaned during floodbox operation using a long handled rake with tines spaced to mesh with the trashrack bars.

A reinforced concrete conduit is envisaged for the section through the dyke, cast-in-situ in 15-ft articulated bays. Where pile support is necessary, the piles and pile cap will support the conduit at joints between bays.

A side hinged floodgate design was chosen with the hinges placed on a slope of 1H:20V to provide a gravity closing force. A gate stop will prevent excessive opening, and ensure that the gate will always have a tendency to close.

iii <u>Pumphouses</u>

Design of the pump station generally followed the conceptual standards (5) with modifications to suit the smaller pump sizes adopted for South Westminster, and to facilitate cleaning of the trashracks.

The pump station structure includes provision for stoplogs for dewatering which may be necessary for cleaning the sump, and maintenance of the trashrack. Similar provision is made in the pump outlet bay of the outlet structure.

Trashrack design will be similar to that described for the floodboxes.

Since all pump stations are accessible from adjacent roads, access from the top of the dyke is not necessary. A boom truck may be used to install and remove pumps as complete units through the pump station hatches and a monorail is not required. Double access doors have however been included in the wall of the pump station at operating level.

b. Inlets

Ditch inlets comprise corrugated steel pipe sizes up to 48 in. in diameter through the ditch banks to convey water from secondary ditches to the main ditches. To minimize scour in the ditch bed the pipes should slope so that their outlets are submerged by the ditch low water levels.

Where the inlet design includes a flap valve (inlets on Manson Road ditch) the corrugated steel pipes should be laid on natural ground, and the inlet excavation should be backfilled with compacted fine-grained material to minimize seepage along the pipe.

c. Culverts for Road and Rail Crossings

Culverts comprise corrugated steel pipes 72 in. or 84 in. in diameter. The culverts are generally set to flow submerged during the winter low tide design condition. This setting will ensure good hydraulic conveyance at lower ditch water levels.

Where the minimum cover of 3 ft is not available, such as at Old Yale Road ditch culvert at Scott Road, a reinforced concrete slab over the culvert is required to support traffic loads.

Pipe arch culverts have been included in the design for Pattullo ditch between the King George Highway piers since the culvert does not bear a traffic load, and the minimum cover of about 5 ft need not be provided.

d. Bridges

Bridges will be required on Old Yale Road ditch where the existing timber bridges have insufficient span for the reconstructed ditch. The bridge design comprises round log timbers, pressure-treated with preservative for abutments, and sawn, pressure-treated timbers for the bridge beams and deck.

e. Drop Structures

Drop structures are required to control water velocities in the upper reaches of Canadian Collieries and Manson Road ditches.

A timber crib design suitable for weak foundation material was chosen to allow flexibility when constructed on peat soils.

2.6 References

(1) Water Investigations Branch — Outline Report on Flood Control Works for the South Westminster Area, July 1973.

(2) Water Investigations Branch — Report on Internal Drainage in the South Westminster Dyking District, August 1972.

(3) Water Investigations Branch — Barker Creek, Whalley. Preliminary Analysis of Runoff Potential, September 1973.

(4)	Hydraulic Institute	Standards of the Hydraulic Institute, New York, May 1965.
(5)	AESL	Conceptual Design Specification for Standard Drainage Pump Stations, Drainage Pumps, November 1970
(6)	AESL	Conceptual Design Specification for Standard Drainage Pump Stations, Electrical, November 1970
(7)	District of Surrey Engineering Department	Drainage Report, Appendix B Preliminary Study of the Manson Road Canal, June 1963

3. GEOTECHNICAL

3.1 General

This section of the report discusses the geotechnical aspects of the design of the dykes, outlet works and ditches.

The design of the dykes has been carried out in accordance with the Terms of Reference and basically in accordance with the Ripley, Klohn and Leonoff report referred to in Section 1.

The basic parameters used for the dykes are as follows:

a. Design grades as specified in Appendix A of Terms of Reference.

b. Minimum crest width of 12 ft, to allow for vehicular traffic.

c. Side slopes of 1 vertical on 3 horizontal on the riverside of the dyke and 1 vertical on 2.5 horizontal on the landside. These slopes have been adopted assuming the dyke fill materials will, for the most part, consist of sand as discussed below. The upstream slope design considers the effect of drawdown under extreme tidal fluctuations and flood flows.

The only exceptions to the criteria outlined in the Ripley, Klohn and Leonoff report for the dyke and ditch design are in regard to the provision of dyke toe drains and ditch filter and drainage layers. If the recommendations of the report had been strictly adopted, toe drains and ditch protection would have been required at all locations regardless of the height of the dyke or the likely hydraulic gradient into the ditches. However, considering the location of the dykes and ditches, relative to the worst seepage gradients which will likely occur under normal high flood conditions, and the short time over which the maximum design flood levels would occur, such drains and filters are not required at many locations. Dyke toe drains and ditch protection have therefore been provided only at locations where they are considered necessary under the normal high flood conditions following a careful review of the existing topography on the riverside of the dyke, the height of the dyke, the location of the ditch, and the subsoil conditions in the area as they are known at present.

3.2 Field Inspection

Visual field inspections of the existing dykes and ditches were carried out to determine to the extent possible the characteristics of the dyke fill and foundation materials, and the materials exposed in the ditch excavations. During these inspections, any areas of dyke or ditch slope instability were noted.

3.3 Sub-surface Exploration

A program of sub-surface exploration was carried out in early 1975 and the results are given in detail in Section 6.1

The exploratory program had the objective of providing subsoil information along the dyke and ditch alignments and at the locations of the proposed outlet works. At a few locations, the holes were drilled through the existing dyke fill to determine the nature of the fill materials.

The 1975 subsoil investigation was carried out to supplement the information given in the reports by Ripley, Klohn and Leonoff and Golder, Brawner Associates, referred to in Section 1.

Although the sub-surface information available at present is adequate for this stage of design, some further investigation will be required before final design, particularly at the proposed outlet works.

3.4 Materials Search

A search was made to determine the most economical sources of construction materials for improvement of existing dykes, construction of new dykes, for backfilling existing ditches and other excavations, and for protecting the slopes of existing and new ditches.

The materials search indicated that the most economical bulk fill and backfill material is dredged Fraser River sand. A representative sample obtained from the dredged sand fill at the north end of Tannery Road was tested for gradation with the result shown on Fig. 6.1-5 of Section 6.1. It will be noted from this figure that the material is a clean, uniformly graded, fine sand.

The closest developed source of sand and gravel for drainage and filter zones and for dyke surfacing which was located is about 5 miles southwest of the south end of Pattullo Bridge. The material in the pit is generally a well graded sand and gravel with particle sizes up to 6 in. maximum, although the percentage of material coarser than 2 in. is small. Samples of the pit-run material from two corners of the pit were tested for gradation and the results are shown on Fig. 6.1-6 of Section 6.1.

Clean drainage and filter material could also be obtained from Ready Mix concrete plants in the area.

The limited amount of the well graded mixture of silt, sand and gravel required as fill in the vicinity of the floodbox conduits could probably be obtained from selected stripping of the pit described above since there is a capping of till-like material on top of the sand and gravel.

Discussion of the utilization of the materials described above is contained in Sections 3.5 through 3.7.

3.5 Dyke Design

a. General

Except for a surface running course, and as otherwise commented upon herein, the dykes should be constructed entirely of dredged Fraser River fine sand. The surface course should consist of a 6-in. thick layer of sand and gravel. All dyke fill material except the surface course should be compacted to at least 90 per cent Modified Proctor density. The surface course should be compacted to 95 per cent Modified Proctor density.

Dyke foundations should be stripped of any trees, bushes or other sizeable plant matter. However, considering the fact that over much of their lengths the dykes will be resting on peat and organic silt, topsoil or other organic soil need not be removed.

It will be noted from the following discussion that in many areas considerable settlement of the dyke is anticipated. An allowance will have to be made during construction for the anticipated settlement to ensure that the final crest of the dyke is at, or above, the design grade.

At selected locations, settlement plates and simple piezometers will have to be installed to record settlements and to monitor construction since in some areas multi-stage construction will be required in order to avoid dyke foundation failures. Where more than one stage of construction is required, the settlement and piezometric data will be used to determine the time required between stages.

Unless otherwise stated, where ditches have to be backfilled, the Fraser River sand should be used as fill.

b. Bolivar Creek Section

The proposed new dyke alignment, profile and typical cross-sections for this approximately 1200-ft long section of the project are shown on Sheet 26.

A road immediately west of the Bolivar Creek acts as a dyke at present. However the elevation of the road is about 3 to 5 ft below the dyke design grade.

The centreline of the new dyke is located about 24 ft west of the road centreline. This places the dyke almost entirely separate from the existing road embankment, except over a

300-ft long section where the new dyke slightly overlaps the side slope of the existing road. This new dyke alignment was chosen in order to provide a reasonably wide berm between the dyke toe and the edge of the creek, and also to minimize differential settlement, which could have been quite severe had part of the dyke been located on the road and part on the low natural ground surface further west.

The average height of the new dyke over this section is about 3.5 ft, with the maximum height being about 7 ft.

The subsoil conditions at this section of the project are discussed in Sub-section e.i of Section 6.1. Broadly, the foundation soil consists of 10 to 20-ft thick deposits of peat underlain by about a 25 to 40-ft thick layer of organic silt and soft grey silt.

On the basis of settlement calculations and a study of settlement data for similar subsoil conditions provided by the C.N.R., as described below, the settlements in the middle of the section, where the dyke height is maximum, are expected to be about 4.5 ft.

With such high anticipated settlements, an initial layer of hogfuel, 1 to 2-ft thick, should be placed over some of the length in order to reduce the fill load as much as possible.

Over much of the length of the dyke the fill can be placed in a single continuous operation. However 2-stage construction may have to be resorted to over the middle position, where the fill is highest, with a waiting period of a few weeks between stages.

c. C.N.R. Section

The proposed new dyke alignment, profile and typical cross-sections for this approximately 4300-ft long section of the project are shown on Sheets 27 and 28.

The existing C.N.R. embankment (main line relocation) between Bolivar Creek and 130th Street, which was constructed in stages for the purposes of preloading the foundation, will function as dyke between Sta 0+00 and Sta 21+00 of this section. The present crest elevation of the embankment varies between 14.0 ft and 16.0 ft. This embankment will be cut back over most of the section to the design grade of the dykes as shown on the profile and cross-sections on Sheets 27 and 28.

Over the western half of the section, between Sta 21+00 and Sta 43+00, it is proposed to construct a new dyke on the top of sand and gravel fill, immediately to the south of the C.N.R. tracks. The average height of the dyke is about 4 ft with the maximum height being about 5 ft. There is an existing ditch to the south of the new dyke and between Sta 35+00 and Sta 43+00. This ditch is too close to the downstream toe of the proposed dyke. Over this stretch, therefore, the ditch will have to be backfilled and a new ditch constructed further to the south, as shown on Sheet 21.

The subsoil conditions at this part of the project are discussed in Sub-section e.i of Section 6.1. There is a significant variation in subsoil conditions over the area Between Sta 0+00 and about Sta 25+00 the subsoil consists of a 10 to 20-ft thick deposit of peat underlain by organic silt and soft inorganic silt to a depth ranging between 40 and 50 ft.

The subsoil conditions improve considerably to the west of about Sta 25+00 and the combined thickness of peat and organic silt is only about 5 ft at the end of the section, Sta 43+00.

As discussed above, the section of the dyke between Bolivar Creek and 130th Street is a part of the C.N.R. embankment. This embankment was constructed in 4 to 5-ft thick lifts between May 1974 and February 1975 in order to preload the foundations since large settlements were anticipated. Information obtained from the C.N.R. indicates that settlements of up to 7 ft under a fill height above the original surface of 11 ft have occurred in some areas. On the average, settlements were about 60 to 70 per cent of the fill height remaining above the original ground after the settlement was sensibly completed. Observations to date indicate that the settlements at 8 of the 10 gauges, the exceptions being gauges at Sta 8+00 and Sta 15+00, are complete with no additional settlement having taken place over the last 2 months of observation. At Sta 8+00 and Sta 15+00 the rate of settlement as observed over the last 2 months is quite small, and it would appear that over the next 2 to 3 months the settlements at these locations will also be complete. On the basis of these observations, there should be no further settlement in this section of the proposed dyke.

Over the western half of the section, Sta 21+00 to Sta 43+00, where, as stated above, a new dyke has to be constructed to the south of the railway tracks, the material beneath the dykes consists of a 4 to 6-ft thick layer of sand and gravel fill. This layer will tend to distribute the load from the dyke fill and consequently reduce the stress on the underlying peat and organic silt. However, the settlements may still be significant over the eastern portion to about Sta 30+00, owing to the presence of considerable depths of peat and organic silt. Computations indicate that these settlements could be about 1-1/2 to 2 ft between Sta 22+00 and Sta 30+00, decreasing to about 1 ft about Sta 35+00, and about 6 in. at the western end of the C.N.R. section.

Considering the foundation conditions, even the maximum height of fill can probably be placed in one continuous operation.

It should be noted that, because of the proximity of the new dyke to the existing railway embankment, some settlement of the tracks may occur. However, it is understood from discussions with the C.N.R. that there is a substantial depth of granular fill under the tracks and ballast has been placed at regular intervals to compensate for continuing settlement. It is believed, therefore, that this relatively competent foundation material will limit the effect of the new dyke on the tracks and that the resulting settlement will be small and within limits which can be controlled by the existing maintenance program.

d. Beckstrom Road Section

The proposed new dyke alignment, profile and typical sections for this approximately 5800-ft long section of the project are shown on Sheets 29 and 30.

The existing C.N.R. embankment from the beginning of the section to about Sta 40+00 serves as the dyke at present.

However the elevation of this railway embankment is at or above the design dyke grade only to the west of Sta 32+50. Over the length between Sta 0+00 and Sta 32+50, the embankment is about 2 ft below the dyke grade. Consequently, a new dyke located immediately to the south of the railway embankment will be required. The average maximum heights of this dyke are about 4.5 and 7 ft, respectively.

Beyond Sta 32+50 the new dyke swings to the north and then runs east-west again south of a spur line as shown on Sheet 30. In parts of the stretch between Sta 46+00 and the end of the Beckstrom Road section there is a dyke of loose sand fill. This dyke will have to be removed and a new dyke built as shown. The average and maximum heights of the dyke are about 3.5 ft and 7 ft.

The subsoil conditions at this section of the project are described in Sub-section e.i of Section 6.1. The subsoil at the eastern end of the section consists of about 7 ft of fill underlain by clay, peat and organic silt with a combined thickness of some 8 ft. This is in turn underlain by silty fine sand. Towards the west the foundation conditions improve with the peat no longer being present and the clay and organic silt decreasing in thickness.

Computations indicate that between Sta 0+00 and Sta 15+00, where the fill height is maximum, up to about 1 ft of settlement can be expected. Over the remainder of the dyke to Sta 32+50 only about 6 in. of settlement is anticipated. Beyond Sta 32+50, where the dyke swings to the north of the main railway line and where the foundation materials are somewhat better, less than 6 in. of settlement should occur under the maximum height of fill.

Considering the foundation conditions even the maximum height of fill can probably be placed in one continuous operation.

Between Sta 0+00 and Sta 32+50 the new dyke is close to the C.N.R. tracks. However, the effect of the dyke on settlement of the tracks is likely to be even less than at the C.N.R. Section, and consequently should be readily handled by the railway's regular maintenance program.

Over the stretch between Sta 5+00 and Sta 20+00 the existing dyke ditch runs too close to the toe of the new dyke. Between these stations, therefore, the ditch will have to be backfilled and a new ditch excavated at the location given on Sheet 20.

e. Pattullo Bridge Section

The proposed new dyke alignment, profile and cross-sections for this approximately 1000-ft long section of the project are shown on Sheet 31.

There is an existing dyke to the design grade with the minimum required crest width over nearly the entire length of this section. However, the side slopes of the dyke are much steeper than those required by the design standards, particularly on the landside, where there is an existing ditch about 10 ft deep. The average landside slope of the dyke from the dyke crest to the ditch bottom is about 1 on 1.25. Furthermore, owing to the presence of the ditch at the toe of the dyke and the steep downstream slope of the dyke, the exit seepage gradients under high flood levels may be too high for ensure the long-term stability of the dyke against piping. Reduction of the exit gradient can be achieved by increasing the seepage path by either backfilling the existing ditch and relocating it at a sufficient distance beyond the dyke toe, or by relocating the dyke sufficiently far away from the ditch. The latter alternative appears to be the more economical solution considering the existing ground topography on either side of the existing dyke, and this solution has therefore been adopted. As shown on Sheet 31, the new dyke is approximately 20-ft north of the existing dyke. It will also be noted from this sheet that the existing ditch is proposed to be deepened to El 0.0 and cut back to flatter side slopes, with a 12-ft wide berm being provided at El 12.0 between the ditch and the dyke. A ditch filter and drainage zone has also been provided.

The subsoil conditions at this section of the project are described in Sub-section e.i of Section 6.1. The subsoil beneath the entire length of the dyke consists mostly of silty fine sand and silt. As a result, and considering that the area has already been essentially preloaded by the existing dyke, no significant settlements are anticipated under any part of the new dyke.

Non-organic material from the excavation of the new ditch and existing dyke can be used for constructing the new dyke providing it is at a moisture content suitable for placement. Any additional fill required can be dredged Fraser River fine sand.

f. Old Yale Road Section

i General

The proposed new dyke alignment profile and cross-sections for this approximately 3400-ft long section of the project are shown on Sheets 32 and 33.

Between Sta 1+00 and Sta 4+00 and between Sta 15+00 and Sta 17+00 there are buildings located immediately north of the dyke and in fact the existing dyke in general rests against the buildings.

A ditch immediately south of the dyke runs along almost the entire length. The existing dyke to Sta 19+00 is narrow and has steep side slopes and the crest is 1 to 3 ft below the design grade.

No dyke exists across Old Yale Road itself, and between Old Yale Road and the access road into a lumber yard at about Sta 1+00, the dyke exists only as loose sand fill and sand bags. Between Sta 19+50 and Tannery Road (about Sta 24+00) an existing road to a lumber works with a crest elevation about 11.0 acts as a dyke. Over the section west of Tannery Road, the present dyke has generally sufficient crest width but its crest elevation is about 2 to 3 ft below the design grade.

The subsoil conditions at this section of the project are described in Sub-section e.i of Section 6.1.

ii Sta 0+00 to Sta 1+00

As will be noted from Sheet 32, a new dyke, about 3 to 4-ft high, is proposed from the end of the Pattullo Bridge Section to Sta 1+00 of the Old Yale Road Section, and the existing portion of the dyke in this section is to be removed. The foundation soils consist of about 6 ft of fill underlain by an approximately 20-ft thick deposit of silty fine sand with traces of organic matter. This in turn is underlain by organic silt. Considering the presence of the fill and the thick sand layer, these foundation conditions are relatively good. Furthermore, this area has been subjected to heavy traffic loads which has resulted in consolidation of the foundations. Consequently, no significant settlement is expected to occur under the weight of the new dyke fill.

iii Sta 1+00 to Sta 13+00

In the stretch between Sta 1+00 and Sta 13+00, the existing dyke is to be raised by a maximum of about 3 ft and it is to be relocated slightly to the south of the existing dyke. As will be noted from Sheet 32, this results in the entire existing ditch having to be backfilled. The existing dyke at a few locations along this section contains lumber and other waste materials near the surface. This material will have to be removed prior to the addition of the new fill, which means at least 2 ft of stripping will be required. A dyke toe drain will also have to be provided.

The subsoil investigation in this area indicates the presence of a 7 to 9-ft thickness of fill, including some hogfuel, under the riverside toe of the existing dyke. The natural foundation material below the fill consists of a 12 to 18-ft thick layer of silt with silty fine sand layers which is underlain by a deposit of organic silt to a depth of 45 to 50 ft. It is anticipated that the presence of the fill beneath the dyke will tend to reduce the additional stress from the new dyke fill on the underlying compressible soils. As a result, the settlements below that portion of the new dyke overlying the existing dyke are not likely to be very large, probably about 6 in. at the western end of the section and somewhat more at Sta 12+00. However, the portion of the new dyke which will overlie the existing ditch will not be on fill, and will thus settle more. Consequently, the ditch should be backfilled first and the backfill allowed to settle before the rest of the dyke fill is added.

iv <u>Sta 13+00 to Sta 19+00</u>

In this stretch the existing dyke has to be raised by a maximum of 3 ft. The upper part of the dyke appears to contain a considerable amount of lumber, scrap metal and other waste and it will be necessary to strip the top 3 to 4 ft of the dyke until competent dyke fill material is encountered.

The foundation material under the existing dyke along this stretch consists of about 5 ft of fill underlain by silt and silty fine sand to a depth of about 20 ft. This, in turn, is underlain by about 30 ft of organic silt. Settlement under the weight of the new dyke fill is expected to be about 1.0 ft.

A similar sequence of operations of backfilling the ditch and then raising the dyke, as discussed for the section between Sta 1+00 and Sta 13+00 should also apply to this section.

It will be noted from Sheet 32 and Section 1, Sheet 33, that in order to preserve the existing buildings to the north, the dyke has been located as far south as is practicable. This has the effect of causing the ditch backfill to rest against a lumber company's retaining wall at the south side of the ditch. The load from the ditch backfill could cause movements of the wall and the ground behind. As a result, the ditch backfill has been shown as consisting mainly of lightweight fill probably hogfuel. However, before the final design stage this has to be examined in more detail with the lumber company.

v <u>Sta 19+00 to Sta 24+00</u>

The present dyke between Sta 19+00 and Sta 24+00 is an existing road and this must be raised by about 3 ft in order to meet the design grade requirements.

The foundation material in this stretch consists of about 3 ft of road fill underlain by about 15 ft of silt with some fine sand and traces of organic matter. This, in turn, is underlain by some 30 ft of organic silt. It is considered that the road fill material will tend to distribute the load from the new dyke on the compressible layers of the foundation. This effect, combined with the fact that the road foundation has been well consolidated under traffic, should cause the settlement from the added dyke height to be relatively small, probably in the order of 6 in.

The sequence of operations for filling the ditch and constructing the new dyke for this section should be as described above for the section between Sta1+00 and Sta 13+00.

vi Sta 24+00 to Sta 30+00

Between Sta 24+00 and Sta 30+00 as shown on Sheet 33, the new dyke has been relocated slightly to the riverside of the existing dyke since the ditch to the south of the dyke has to be maintained and a berm is required between the dyke and the ditch to guard against foundation piping at high flood levels.

The subsoil beneath the dyke between Sta 24+00 and Sta 30+00 consists of about a 9-ft thick layer of peat underlain by some 32 ft of organic silt.

The maximum height of the new dyke between Sta 24+00 and 30+00 is about 4 ft and this height of the dyke is expected to settle some 1.5 to 2.0 ft.

viii Sta 30+00 to Sta 34+00

The section of dyke west of about Sta 30+00 has been located along a straight line extension of Dyke Road (original dyke Section 24+00 to 30+00).

The average and maximum heights of dyke throughout this stretch are 5 ft and 7 ft, respectively.

The subsoil in this area consists of a 10 to 12-ft thick deposit of peat underlain by 30 to 35 ft of organic silt.

Under the maximum height of dyke fill some 4.5 ft of settlement is expected.

Although most of the length of the dyke in this stretch can be constructed in one continuous operation, some of it will probably require 2-stage construction.

It would be desirable for this entire section to be constructed at the same time that the Manson Road outlet works are preloaded (see Sub-section 3.6 e.), and allowed to settle.

g. Manson Road Section

The plan, profile and typical cross-sections for this approximately 750 ft long section of the project are shown on Sheet 34.

For the first 100 ft of its length, the new dyke runs normal to the Manson Road ditch. It then swings to the north and runs along the west side of the ditch to Sta 3+00. Thereafter, it is aligned parallel to the river.

Except for the first 100 ft length, the entire area below the proposed dyke has been filled by Weldwood to an elevation ranging between 10.0 and 12.0 and the height of the new dyke therefore varies from 2 to 4 ft.

The subsoil conditions in the area are described in Sub-section

5.e.i of Section 6.1. The subsoil near the beginning of the dyke section, at the location of the proposed Manson Road pump station, consists of about a 7-ft high thick layer of peat underlying about 8 ft of fill. The peat is, in turn, underlain by a 33-ft thick deposit of organic silt. Over the rest of the area the subsoil comprises a 10-ft thick deposit of organic silt underlying 8 ft of granular fill mixed with hogfuel. About 16 ft of silty fine sand with traces of organic matter, was encountered under the organic silt, and this in turn is underlain by another 13-ft thick deposit of organic silt. The depth to clean fine sand throughout the section is about 48 ft.

With the above mentioned subsoil conditions, about 1.0 to 2.0 ft of settlement is likely to occur under the new dyke except in the vicinity of the Manson Road pump station. In this vicinity, larger settlements will occur but as will be discussed in detail under Section 3.6 e., it is expected that the dyke section above the conduit and adjacent to the Manson Road ditch will be constructed as part of the preloading required for the outlet works.

Considering the foundation and height of the dyke, it can be constructed in one continuous operation, except for the first 300-ft length as discussed above.

h. Weldwood Plant Area

The general arrangement and details of the flood protection works in the area of the Weldwood plant are shown on Sheet 35.

Owing to the limited space available in this section of the project for conventional dyke protection, a timber crib wall supported on piles and backfilled with impervious or semi-pervious soil has been provided along a significant stretch upstream of the existing retaining wall, which is about 2-ft below the design grade. Through the existing buildings, a new concrete retaining wall is proposed. Where sufficient space is available for the dyke construction, i.e., between the sawmill building and the end of Weldwood plant buildings, a conventional dyke has been provided around the retaining wall.

Owing to access difficulties, no subsoil investigation has as yet been carried out in this area. However, some investigation should be carried out before final design.

3.6 Outlet Works

a. General

It will be noted from the following discussion that several of the outlet works structures have to be founded on piles. In all such cases the piles should be nominal 12-in. dia timber piles driven at 4 to 5-ft centres to the approximate elevations indicated.

In order to prevent any problems arising due to seepage along the outside of the floodbox conduits, the fill in the vicinity of the conduits will be a "self-healing" well graded mixture of silt, sand and gravel, rather than the Fraser River sand which is to be used elsewhere in the dykes. In addition the conduits will be provided with seepage collars.

It will be noted from the discussion below that none of the conduits are to be placed on piles. As a result, some differential settlement can be expected at all conduit locations even where the area has already been, or will be, preloaded. So that a reasonable amount of differential settlement can be accommodated, flexible joints will be provided between adjacent sections of conduit.

Graded filter zones for the protection against internal erosion will be provided on the ditch slopes in the vicinity of all outlet works.

b. Beckstrom Road Outlet Works

The general arrangement of the pumphouse and floodbox is shown on Sheet 23.

It will be noted that the existing pumphouse is to be removed and a new pumphouse constructed about 50 ft to the south.

The pumphouse floor slab is to be founded at El -7.4, the floodbox inlet slab at El -3.2 and the floodbox outlet slab at El -4.7.

The subsoil conditions are described in Sub-section e.ii of Section 6.1.

The soil encountered in Hole 75-10, drilled adjacent to the existing pumphouse, indicates a 3-ft thick layer of peat between El 0.0 and El -3.0 underlain by a 1.5-ft thick layer of organic silt, which in turn is underlain by fine sand.

The sand is loose for the upper 10 ft whereafter it becomes medium dense. It contains some roots in the upper few feet.

Although no drilling was carried out to the north of the existing pumphouse, there is evidence from previous exploration in the area that the peat and organic silt strata decrease in thickness towards the river. On this basis, improved foundation conditions, over those encountered at the pumphouse location, can be expected at the floodbox outlet structure.

The subsoil conditions discussed above would indicate that the pumphouse and floodbox inlet structure should be founded on short piles driven into the sand. As indicated above, the upper layers of the sand are loose and contain roots. Therefore, to avoid possible detrimental settlements of the structures, it is considered prudent to the resort to the use of piles, rather than to excavation of peat and organic silt, which would otherwise exist, at least under the floodbox inlet structure. The piles should be driven some 20 ft into the sand to approximate El -25.0.

Regarding the floodbox conduit, it is likely that over some of its length, particularly towards the south end, it would rest on either peat or organic silt if placed on the natural ground. Over such length, excavation should be carried out to the level of the underlying sands and this excavation backfilled to the design grade with well compacted fill. The conduit can then be founded directly on this fill.

The floodbox outlet foundation would likely rest directly on natural silty sand. However, since the upper layers of the sand are likely to be loose, it has been assumed at this stage that the outlet structure will be founded on piles. Before final design, further exploratory drilling should be carried out to confirm, or otherwise the necessity for piles.

c. Pattullo Bridge Outlet Works

The general arrangement of the pumphouse and floodbox is shown on Sheet 18.

It will be noted that the new pumphouse and floodbox are to be located slightly north of the existing works, which are to be removed.

The pumphouse floor slab is to be founded at El -5.4, the floodbox inlet slab at El -2.7 and the floodbox outlet slab at El -3.2.

The subsoil conditions are described in Sub-section e.ii of Section 6.1. Hole 75-11, drilled adjacent to the existing pumphouse encountered a 6-ft layer of organic silt from El -2.1 to El -8.6 below sand and gravel fill. Fine sand was encountered below the silt. About the top 10 ft of the sand is loose but it becomes compact below. Drill Hole C.N.R. 2, described in the Golder Brawner Associates' report mentioned in Section 3.3 above, located on the river bank approximately 500-ft north of Hole 75-11 encountered fine sand with silt below the sand and gravel fill. It was reported that the top 10 to 15 ft is loose but that it becomes compact with depth.

On the basis of these subsoil conditions, the pumphouse and floodbox inlet foundations can be expected to be on organic silt immediately underlain by loose fine sand. Consequently, both structures should be founded on piles driven some 20 ft into the sand to approximate El -30.0.

Towards the pumphouse end, the conduit, if placed on natural ground, would likely rest on organic silt at the design foundation grade. At this location, therefore, excavation should be carried out to the level of underlying silty sand and the excavation backfilled to the design grade with well compacted fill. The northern section of the conduit can be founded directly on the natural subsoil which should be fine sand.

The subsoil immediately below the floodbox outlet structure is expected to be loose sand. As a result, it has been assumed at this stage that the structure will be supported on piles to the same depths as for the pumphouse and floodbox inlet structure. However, prior to final design further exploration should be carried out to determine whether or not this is necessary.

d. Old Yale Road Outlet Works

The general arrangement of the pumphouse and floodbox is shown on Sheet 13.

It will be noted that there is only an existing floodbox at this location and that it is to be removed and replaced with the new pumphouse and floodbox as shown.

The pumphouse and floodbox inlet slabs are to be founded at El -3.1, and the floodbox outlet structure slab at El -4.6.

The subsoil conditions are described in Sub-section e.ii of Section 6.1. Hole 75-20 drilled adjacent to the proposed pumphouse indicates that the foundation subsoil consists of fine silty sand with some organic matter to El -18.5 and that this is underlain by a 16-ft thick deposit of organic silt. Compact to dense silty fine sand was encountered below the organic silt. Another hole, 75-12, was drilled about 50 ft east of the location of the existing floodbox. This hole encountered organic silt with fine sand layers to El -5.5 underlain by silty sand. The sand encountered in this hole is very loose to loose in the upper 15 ft but it becomes compact thereafter.

On the basis of these subsoil conditions, the pumphouse and floodbox inlet foundations will be on the upper stratum of silty sand. However, considering that the sand is loose and that it is underlain by a thick layer of organic silt, the structures should be supported on piles driven through the upper silty sand and organic silt deposits and some 10 ft into the underlying sands, i.e., to about El -45.0.

Over much of the length of the conduit, silty fine sand is expected to be encountered at the foundation level and even although this is underlain by organic silt the area has in effect been preloaded by existing fill and heavy traffic and the conduit can be placed directly on this stratum. However, should organic silt be encountered at foundation grade, it should be excavated to the level of the underlying sand and replaced by fill as discussed above for the other outlet works.

The subsoils at the level of the floodbox outlet structure foundation should be either organic silt or loose silty sand. This structure should therefore be supported on piles. Because the top of the lower sand stratum increases in elevation towards the north, the piles for the floodbox outlet structure are expected to be shorter than those for the pumphouse. They should be driven some 20 ft into sand to approximate El -25.0.

e. Manson Road Outlet Works

The general arrangement of the pumphouse and floodbox is shown on Sheet 7.

It will be noted that the existing pumphouse is to be removed and replaced by a new structure located about 170 ft to the south.

The pumphouse floor is to be founded at El -6.8, the floodbox inlet floor at El -4.5 and the floodbox outlet structure at El -6.5.

The subsoil conditions are described in Sub-section e.ii of Section 6.1. The subsoil in the area of the outlet works comprises thick deposits of peat and organic silt. In Hole 75-13, drilled near the location of the proposed new pumphouse, a peat layer was found, below fill, to El -7.0 to -8.0, and this was underlain by a thick deposit of organic silt to El -40.0

The subsoil conditions clearly indicate that none of the structures can be founded directly on the natural subsoil since excessive settlements would occur. Both the total and differential settlements under the pumphouse, floodbox inlet and outlet structures and the conduit could be effectively controlled by placing all these structures on piles. However, this would result in considerable differential movements between these structures and the dyke in the vicinity of the conduit since the section of the dyke immediately above the conduit would undergo very little settlement while the immediately adjacent section of the dyke will undergo very large settlements, as discussed in Section 3.5 f. above. This would cause cracks within the dyke in the vicinity of the conduit and additional loads to come onto the piles. The differential settlement problem can be greatly reduced by omitting the piles from under the conduit, and by preloading the area as described below.

The area to be preloaded would cover the locations of the pumphouse, floodbox inlet and outlet structures, the conduit and the adjacent dyke. Any areas near the structures where future filling may take place should also be filled at the same time. Under the design height of the dyke, about 9 ft, the maximum settlement of the foundation is expected to be in the range of 5 to 6 ft. It should be noted that this settlement is the ultimate value, i.e., the value which will occur over a long period of time. In order to cause the foundation to settle by this amount within a relatively short period of time, say 6 to 9 months, it will be necessary to apply a greater load than that which would occur under the design height of dyke. This surcharge above the design elevation of the dyke should be about 5 ft. Owing to the considerable height of fill which will have to be placed in order to surcharge the area to this height, filling will have to be carried out in stages, with sufficient time allowed between the stages for the settlement under a given stage to be about 75 per cent complete.

After the water from the Manson Road ditch is diverted around the area, preloading should be carried out as follows:

First, the existing Manson Road ditch should be filled to the adjacent ground surface. Secondly, the area should be filled to about El 12.0. Thirdly, filling should be carried out to about El 16.0, and finally to about El 20.0. After settlement is nearly complete, the surcharge should be removed and the preload fill cut back to the design dyke section plus a small settlement allowance. At this point, the foundations for the pumphouse, floodbox inlet and outlet, and conduit should be on sand fill which had displaced peat during the preloading process. Consequently, most of the excavation for the structures will be in sand.

Although much of the settlement should be complete during the preloading period, a limited amount of settlement at a slow rate could continue over a long period of time. This long-term 'secondary' settlement is a characteristic of peat. It is desirable, therefore, to support the pumphouse, floodbox inlet and outlet structures on piles. The conduit can rest directly on the foundation subsoil.

The piles for the structures should be driven some 15 ft into the sand underlying the organic silt. At the location of the pumphouse and floodbox inlet the piles would thus be driven to approximate El -55.0. At the floodbox outlet structure the piles should be shorter since the depth to the stratum should be less than that at the pumphouse location.

f. Collieries Ditch Outlet Works

The general arrangement of the floodbox is shown on Sheet 4.

The floodbox inlet and outlet floor slabs are to be founded at El 1.0 and El -0.5, respectively.

The subsoil conditions are described in Sub-section eii of Section 6.1. The subsoil as encountered in Hole 75-24 drilled adjacent to the floodbox inlet location, consists of a 4-ft thick layer of sand fill underlain by 10 ft of organic silt, extending to El -1.7. Silty fine sand with traces of organic matter was encountered under the silt. This sand is very loose in the upper 10 to 15 ft but it becomes compact below.

It would appear from the above that the floodbox inlet and outlet and the conduit, if placed on natural ground, would likely rest on a thin layer of organic silt. However, this material could be readily excavated and replaced with well compacted fill as described above for some of the other floodbox conduits.

The existing ground in the vicinity of the floodbox is approximately at the design grade. However, it will be noted from Sheet 4 that it does decrease in elevation towards the floodbox outlet structure. For this reason, and since the fill in the vicinity of the structure is to be a well graded mixture of silt, sand and gravel which has a greater unit weight than the existing snad fill, it would be desirable to build-up the fill near the floodbox works to about 3-ft above the design grade at least 2 months before the works are to be constructed. This small amount of preloading will enable the underlying silty fine sand with traces of organic material to settle prior to construction. If this is done, considering the fact that the loading from the Collieries Ditch floodbox is very light, the inlet and outlet structures and the conduit need not be placed on piles. They should be placed on a layer of fill after the organic silt has been excavated, as discussed above.

3.7 Ditch Design

a. General

The hydraulic design of the ditches is discussed in Section 2.3. The geotechnical considerations are discussed below.

b.. Canadian Collieries Ditch

The ditch plan, profile and typical cross-sections are shown on Sheets 2 and 3. It will be noted that the maximum depth of excavation is about 10 ft.

The soil conditions along the ditch are described in Sub-section e.iii of Section 6.1. Broadly, the subsoil to the south of the C.N.R. and G.N.R. railway tracks consists of deep peat and organic silt strata and the ditch invert is likely to be in peat in some areas, particularly in the vicinity of Grace Road.

At some locations, loose fill, consisting of lumber waste, hogfuel, and sand and gravel, overlies the peat and organic silt.

Ditches in such soil conditions must have relatively flat side slopes and a value of 1 on 2.5 has been used along the entire length. Should the ditch invert be in hogfuel, or other unsuitable fill, excavation should be continued down to the level of the natural subsoil, and dredged sand backfill placed.

It will be noted that at locations near Grace Road and between Sta 44+00 and Sta 50+00, small dykes about 2 ft high are required on either bank in order to achieve the required hydraulic cross-section. Owing to the very poor subsoil conditions, proportionately large settlements of these dykes will occur. Furthermore the presence of these dykes decreases somewhat the stability of the ditch side slopes. As a result, in order to reduce the fill load as much as possible, an initial layer of hogfuel, about 1-ft thick, should be placed. In addition, the dykes should be located some distance back from the ditch, as shown.

The area along the ditch alignment to the north of the C.N.R. and G.N.R. railway tracks has been filled with dredged sand which varies in depth from about 4 ft at the north end of the ditch to about 9 ft near the tracks. The subsoil below the sand fill consists of about a 10-ft thick layer of organic silt underlain by silty fine sand. Over some of the area, a layer of hogfuel underlies the dredged sand.

In this area, the ditch invert over much of the length is expected to be in organic silt while most of the ditch excavation will be through sand fill. Such conditions also call for relatively flat side slopes and although they could be made slightly steeper than those to the south of the railway tracks, they have also been fixed at 1 on 2.5 as the most ready way of attaining the required ditch cross-sectional area. Should hogfuel be encountered at the ditch invert level, overexcavation and backfill as described above should be carried out.

c. Manson Road Ditch

The alignment, profile and typical cross-sections of the ditch are shown on Sheets 5 and 6.

As discussed in Section 2.4 c. of the report, the capacity of the existing ditch is insufficient at some locations and the ditch will have to be widened or deepened.

The subsoil conditions along the ditch are described in Subsection e.iii of Section 6.1. The subsoil along both banks of the ditch consists of 8 to 12 ft of highly compressible peat underlain by about a 32-ft thick deposit of organic silt. The invert of the existing ditch is in peat along almost its entire length.

With these soil conditions, any significant deepening of the ditch would not be practicable since very flat side slopes would be required. It was decided therefore, to attain the required additional capacity by constructing dykes on both the banks to El 8.0, 1-ft above the maximum design water level in the ditch. Since the new ditch will thus be widened rather than deepened, the side slopes can be made 1 on 2.5.

As can be seen from Sheets 5 and 6, the maximum height of the ditch dykes will be about 4-ft above the existing ground surface. Owing to the very poor subsoil conditions, proportionately settlements, in order of about 80 to 90 per cent of the fill height can be expected to occur. As a result, as described above for the Collieries Ditch dykes, an initial layer of hogfuel up to 2-ft thick should be placed, and the dykes should be kept about 12 ft back from the top of the ditch slope, as shown.

d. Other Ditches

Other main drainage ditches required are the C.N.R. east and west ditches, the 126A St. and 128 St. ditches, the Pattullo ditch and the Old Yale Road ditch.

In addition, dyke ditches are also required along the Pattullo Bridge and the western part of Old Yale Road sections of the dykes.

No investigations along the alignment of these ditches was carried out other than what was considered necessary for dyke and the outlet works design purposes.

The results of the dyke and outlet works investigations and previous investigations by others in the area, indicate that the subsoil within the depth of the ditches in general, comprises peat and organic silt. Although the subsoil conditions along the other ditches are not usually as severe as along the Manson Road ditch and the southern part of Collieries Ditch, there are no sizeable stretches where the material within the ditch depth is competent enough to warrant relaxation of the design side slopes of 1 in 2.5 adopted for the Manson Road and Collieries ditches. It was considered prudent, therefore, to adopt these side slopes for all other ditches, with the exception of the Old Yale Road ditch and a part of Pattullo ditch. In these cases, side slopes of 1 in 2 have been adopted owing to property line limitations and consideration of the fact that there is no evidence of any instability of the existing ditch slopes, which are steeper than the design slopes.

At a few locations, along the Pattullo and Old Yale Road ditches, very small dykes, with a maximum height of 2 ft are required on one or both banks for hydraulic reasons. Considering the small height of these dykes, no berm has been provided between the ditch and the dyke.

4. CONSTRUCTION PROGRAM AND COST ESTIMATE

4.1 Scope of Work

The project covers five concrete outfall structures, six main drainage ditches, totalling 22,600 ft in length, and dyking works from Weldwood to 130th Street, totalling 14,400 ft of river bank protection.

The scope of this work can be divided into four main categories wherein the work is repetitive or labour orientated, such that by so grouping the work, cheaper costs could be anticipated both from labour familiarization and re-use of material.

This division would be on the following basis:

a. Outfall structures.
b. Dykes.
c. Ditches.
d. Timber piling and sheeting, sheet piled walls and foreshore dredging.

Due to the fact that much of Group d. is tied with Group a., though some savings could be expected in separating some of this work, it is recommended that at this stage these two groups be considered as one main contract as shown in Section 4.5b.

The dykes and ditches could also be split into two separate groups, as shown in Section 4.6d, however it is again recommended that this work be let as one contract to allow more flexible use of equipment and to provide incentive to use suitable excavated material in the dykes, rather than trucking it to waste fill.

This grouping and the proposed construction schedules Section 4.6a and 4.6b will also permit better control in ensuring that the group with the higher labour content, Group a. and d., will be completed by the end of May 1976.

The construction industry labour agreements expire 30 April 1976 and the estimate of the cost of this portion of the work has allowed for sufficient equipment and material, so that the work, as shown on the schedule, can be expedited at no extra cost, to ensure substantial completion by this date.

Allowance has been made for the dykes and ditching to be carried out after the current labour agreements have expired, based on a 10 per cent wage increase.

By bringing a limited section of dyking into certain of the outfall contracts a savings will be made in purchasing and handling fill material. This has not been taken into account.

4.2 Outfall Structures

a. These pump stations and floodboxes are located as follows:

i At the new Collieries Ditch; there is a twin floodbox only.

ii At Manson ditch, the new pump station will have two 10,000 gpm and one 5,000 gpm pump. There will also be three floodbox outlets.

iii At Old Yale Road, the new pump station will have two 5,000 gpm pumps and one floodbox outlet.

iv At Pattullo Bridge, the new pump station will have two 10,000 gpm and one 5,000 gpm pump and a double floodbox.

v At Beckstrom Road, the new pump station will have one 10,000 gpm and one 5,000 gpm pump and a double floodbox.

b. The five pumphouse and outfall structures are to be founded on timber piles and in the special case of the Manson Road outfall, this site will be preloaded with sand for a period of 6 months.

c. The construction of the outlet structures will be carried out behind a sand bermed site fully enclosed with a single line Well Point Dewatering System. This system will operate from the time pile driving has been completed until the back fill has been placed to conduit roof level. It is not anticipated a double line of well points will be required. This dewatering system in each case is large enough to include the timber crib and timber sheet and piled walls that are required at some outfalls.

d. At three of the sites; Old Yale Road, Pattullo Bridge and

Beckstrom Road, rail service will be affected by construction. To ensure minimum interruption to these services, steel girder spans carried on timber pile bents, will be installed over weekends, prior to commencing excavation.

The cost estimates include provision to leave these steel spans in position, should this be considered necessary.

Where road access will be affected diversions around the work sites have been provided.

e. The concrete work has been priced on the assumption that these are utilitarian structures requiring good quality concrete, but no special finish or additives.

f. Sufficient material, equipment and labour have been considered so that two outfalls will be under construction simultaneously, and allowance has been made both from supervision and material so that the final work schedule can be accelerated at no additional cost, to complete all work, except Manson outfall, by 1 May 1975.

It will be necessary to place the preload on the Manson Road outfall in late July 1975 to allow completion of this structure by the same date. A separate contract would be arranged to acheive this.

g. Due to the fact that a 12 month delivery period is presently quoted for the pumps, a Provisional Sum has been allowed to reinstall the old pumps in the new structures as a temporary measure. No allowance has been made for operating costs.

h. The cost estimate provides for one set of stoplogs and it is assumed that these will serve all outlet works.

i. The culverts under the C.N.R. and G.N.R. tracks at Collieries Ditch have been included with the outfall structures.

j. It is assumed the existing transformers and wiring to the poles adjacent to the old pump stations are of sufficient capacity.

4.3 Dykes and Ditches

a. General

It has been recommended that this work be let as one contract, as shown in Section 4.6d and it is not anticipated that either the value of the contract or scope of work, will provide a problem to the average sized contractor.

The proposed construction schedule, Section 4.5b, calls for work to be progressing on two ditches and a section of dyke at the same time. This will give the Contractor scope to deploy his equipment more efficiently.

b. Dykes

As previously stated the dykes run from the Weldwood Plant, adjacent to the Manson Road pump station, to a point some 100 ft east of 130th Street, where the dyke ties in to the new C.N.R. main line embankment.

Access is not considered to be a major problem in the execution of the work.

In general the foreshore dykes and ditch berms will be constructed from river sand.

It is expected that a considerable part of the ditch excavation on Collieries Ditch will be suitable fill material. This has been taken into account for use between Manson Road and Old Yale Road outfalls.

Similarily where the old dykes are removed, in the Pattullo Bridge and Beckstrom Road area a percentage of this material has been considered suitable for re-use. It is for this reason that it was recommended this work be combined with the ditching.

All permanent access roads upon the dykes and berms have a 6-in. surface dressing of road mulch included for surfacing.

c. Ditches

Of the six main ditches involved, four have major road or rail line crossings. These are as follows:

i Collieries Ditch crosses the main C.N.R. and G.N.R. tracks.

This crossing is included with the Pump Station Contracts as it involves steel girder bridges and well point dewatering.

The twin culverts are scheduled to be installed before the ditch commences.

ii Manson Road ditch crosses both Scott Road and Timberland Road, where there is also a spur rail line.

Allowance has been made for diversions of Scott Road across a culvert in Manson ditch to Grace Road and hence back to Scott Road. The local Authority will have to ensure that the right-of-way is cleared.

At Timberland Road, the road will be relocated to give continuous access, however it has been considered that the rail line can be closed for 72 hrs to avoid the cost of bridging.

iii Old Yale Road ditch crosses both Scott Road and Timberland Road, where there is also a spur rail line.

The road is diverted north and south of Scott Road maintaining single line traffic each way.

While at Timberland Road the road can be diverted to give continuous access and again it is been assumed the rail line can be closed for 72 hrs.

iv At Pattullo Road ditch the Scott Road intersection is handled similarly to Old Yale Road.

The intersection at Bridge Street is handled by road diversion.

Allowance has been made for full dewatering systems at each crossing. It is proposed that test pits will be put down in the vicinity of each crossing to investigate the possibility of omitting these systems as this would considerably reduce costs.

Berms and dewatering systems have been allowed for in the considerable culvert work in the area of Pattullo Bridge. Again single line well point dewatering systems have been considered adequate to keep the excavations dry.

No other problems are foreseen in the construction of this work.

Allowance has been made for heavy traffic access roads to all dykes and ditches to allow removal of excavated material with the exception of the top sections of Collieries and Pattullo Road ditch.

4.4 Timber Piling and Sheeting, Sheet Piled Walls, etc

Besides the timber piles for the foundations, at several of the outfalls and at culvert entrance and exits there are timber crib, timber piled or sheeted retaining walls. It is more practical to construct these at the same time the dewatering systems are in operation and it is for this reason that this work is included with the Pumphouse Contract.

No problems are foreseen with this work providing it is carried out as outlined above.

For reasons of access, the foreshore work with the exception of Beckstrom Road outfall, has been considered best done from the water. Due to the relatively small value of this work it is not recommended that this be made a separate contract.

4.5 Construction Schedule

a. Pumphouse and Outfall Works

The proposed schedule shown in Fig. 4.1 shows the duration of the contract based on economical use of labour, equipment and material.

Allowance for material etc has been made so that the schedule can be accelerated by working at three sites rather than two at any one time in order to achieve substantial completion by 30 April 1976.

The schedule indicates that the preloading at Manson Road outfall should be commenced in July 1975 in order to achieve substantial completion by 30 April 1976. A separate contract would be placed for this work.

b. Ditching and Dyking

A tentative schedule for this work is shown in Fig. 4.2.

a. Pump Stations and Ancillary Work

Collieries Ditch Main Railway Crossing	$ 116,100
Collieries Ditch Outfall	108,670
Manson Road Ditch Outfall	369,900
Old Yale Road Ditch Outfall	257,115
Pattullo Ditch Outfall	318,275
Beckstrom Road Outfall	337,475
Beckstrom Road Outfall Sheet Piling	112,500
Weldwood Retaining Wall	69,100
Dredging to Outfall Channels	53,000
	$1,742,135
Allow for additional electrical requirements	22,000
Allow for 3 sets of stoplogs	15,000
Allow to reinstall old pumps - 4 stations	20,000
	$1,799,135
Contingency 15 per cent	269,870
	$2,069,005
Overhead at 12 per cent	248,280
	$2,317,285
Profit 9 per cent	185,382
	$2,502,670

b. Ditches and Dykes

	Quantity	Unit	Amount	Total
Collieries Ditch	4700	lin ft	$ 62,250	
Manson Road Ditch	5233	lin ft	363,200	
Old Yale Road Ditch	2927	lin ft	209,500	
Pattullo Road Ditch	4280	lin ft	379,900	
C.N.R. Ditch (west & east)	4100	lin ft	103,255	
126th and 128th St. Ditch	1340	lin ft	60,730	$1,178,835
Manson Road & Weldwood Dyke	1040	lin ft	25,110	
Old Yale Road Dyke	3400	lin ft	50,300	
Pattullo Road Dyke	1050	lin ft	40,050	
Beckstrom Road Dyke	5400	lin ft	116,100	
C.N.R. (east) Dyke	2250	lin ft	62,272	
Bolivar Creek Dyke	1300	lin ft	21,690	$ 315,522
				$1,494.357
Labour Escalation - 5 per cent				77,300
Contingency - 15 per cent				224,154
				$1,795,811
Overhead - 12 per cent				215,497
				$2,011,308
Profit - 8 per cent				160,902
				$2,172,210

Total Direct Construction Costs:

Pump Stations & Ancillary Works	$2,502,670
Dykes & Ditches	1,172,210
	$4,674.880

PROPOSED CONSTRUCTION SCHEDULE - PUMPHOUSE AND OUTFALLS

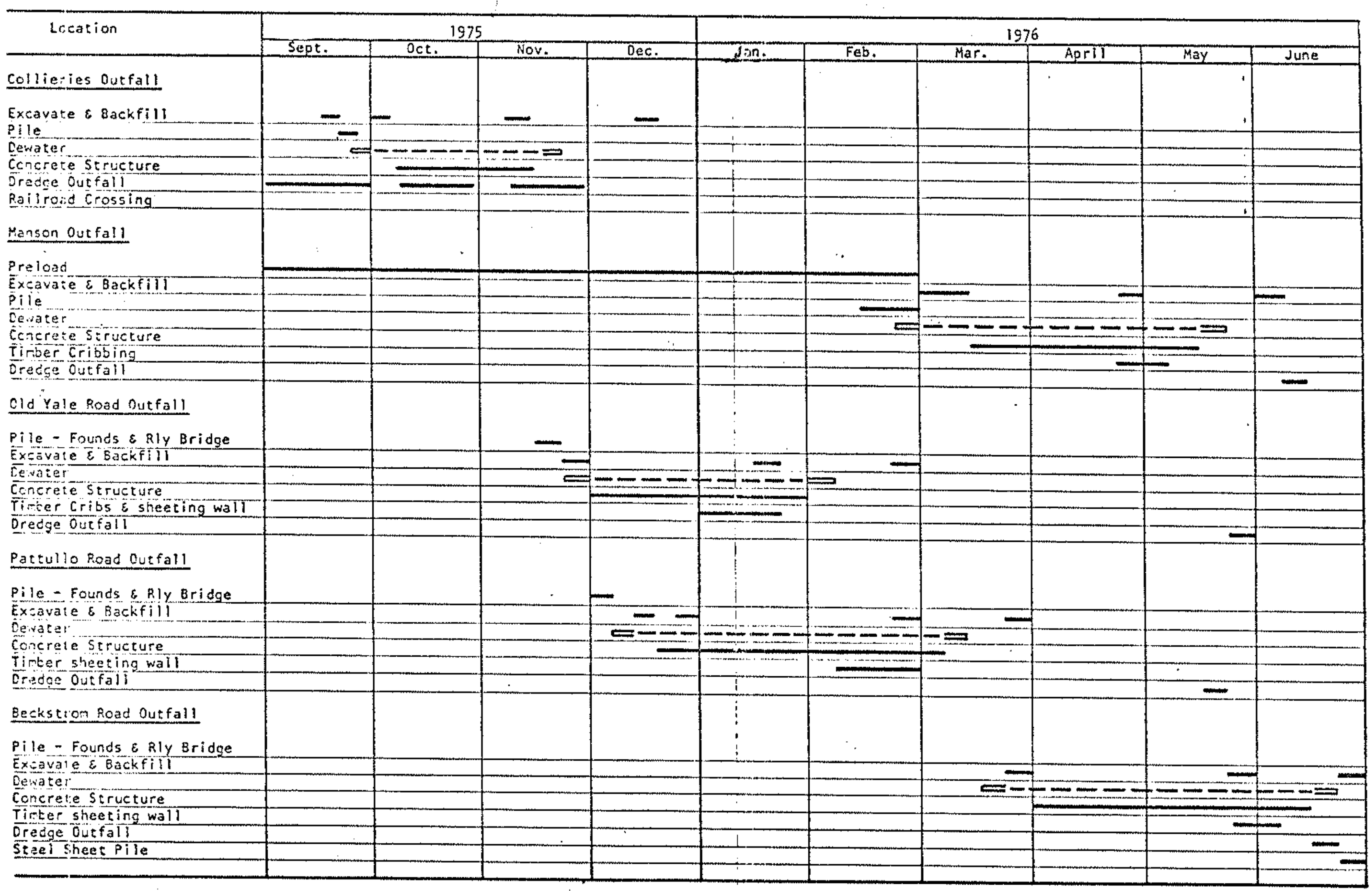


Fig. 4.1

CONSTRUCTION SCHEDULE FOR DYKES & DITCHES

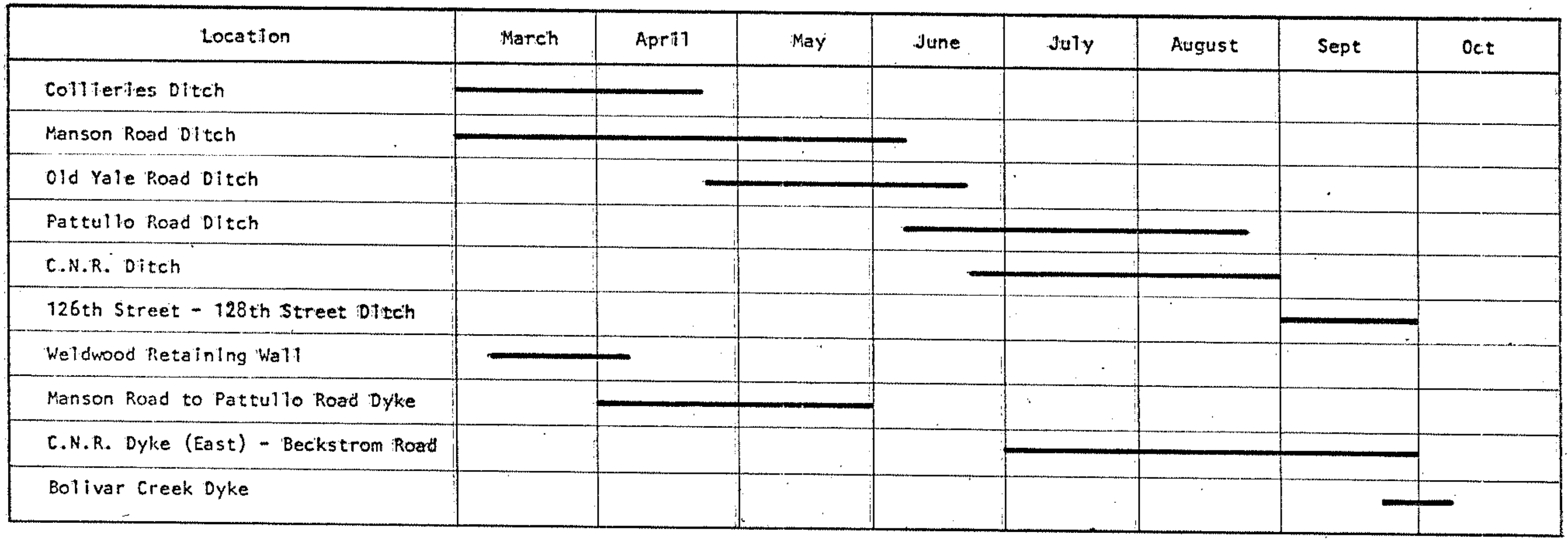


Fig. 4.2

5. SURVEYS AND PROPERTY ACQUISITION

5.1 Engineering Survey

It was found that there was no reliable information available on the existing locations of the dykes and ditches and it was necessary to carry out a complete survey before any work could be commenced on the design.

Details of the engineering survey carried out are as follows:

a. A control traverse, based on the integrated co-ordinate system, was run along the length of the dykes. This was tied into existing monuments and property corners wherever these could be located and identified.

b. Cross-sections of the dyke and the adjacent ground surface, approximately 50 ft either side of the dyke, were observed at intervals of 100 ft along the dyke and plotted at a scale of 1 in. to 10 ft on section paper.

c. Cross-sections of the primary drainage ditches and the adjacent ground surface were observed at intervals of 200 ft and plotted on section paper at a scale of 1 in. to 10 ft.

d. Detailed cadastral surveys, using tacheometric methods, were made at all pump station and floodbox sites, all main ditch and rail crossings, all main ditch and street intersections, across the Weldwood Plant area, and at any other location deemed necessary to enable the design layout drawings to be completed. These surveys were plotted at scales ranging from 1 in. to 10 ft to 1 in. to 100 ft depending on the particular details and requirements at each location. In addition all available details of underground services such as sewers, drains, pipelines, cables, etc, obtained from the relevant authorities was plotted on these detailed drawings.

e. Spot ground elevations, to supplement those available in the reference reports, were observed over the low areas of the entire lowland drainage area. These were used in the hydraulic studies to ensure that the drainage system would meet the criteria relating to control of the groundwater table.

f. Locations and elevations were established for all drill hole collars completed for the soils investigation program.

In general, the engineering survey completed to date has been carried out in sufficient detail to enable the final stage of the final design to be completed.

5.2 LEGAL SURVEY

In order to facilitate early commencement of negotiations for property acquisition by the Surrey Municipality drawings were prepared to show the additional right-of-way requirements for the ditches, outlet works and dykes.

Messrs. Aplin and Murray were commissioned to prepare plans at a scale of 1 in. to 100 ft showing all property boundaries, liens, easements and existing right-of-way reserves in the areas occupied by and adjoining the existing and proposed dykes and ditches. This survey was tied into the control and traverse points used for the engineering survey.

The final layout of all ditches and dykes was superimposed on the plans prepared by Aplin and Murray. All right-of-way requirements which are additional to the existing were clearly shown by cross hatching. These plans were handed to the Surrey Municipality.

It has been estimated that the additional right-of-way reserves which will have to be acquired for the ditches, auxiliary structures, and dykes amounts to approximately 22 acres.

6. APPENDIX

6.1 Sub-surface Investigations, 1975

a. Introduction

The sub-surface investigations described herein were carried out in the period January through early March 1975. A total of 22 holes were drilled for the following purposes.

i to determine the subsoil conditions beneath the existing dykes, particularly at the locations where the dykes have to be raised by a significant amount;

ii to determine the nature of the fill materials in the existing dykes;

iii to evaluate the subsoil conditions at the location of proposed new dykes, outlet works, and selected drainage ditches.

The information available from the sub-surface investigations carried out in the area by others (see Section 1) was reviewed prior to finalizing the drilling program, and as a result, holes were drilled only at locations previously not investigated at all, or where additional information was definitely needed in order to arrive at the basic conclusions necessary at this stage of the design.

b. Location and Depth of Drill Holes

The locations of the drill holes are shown on Sheet 36. A total of 22 holes were drilled as follows:

Drill Hole No.	Depth ft	Location	Basic Purpose
75-1	61.5	C.N.R. Section	Dyke Design
75-2 -3	31.5 36.5	Beckstrom Road Section	Dyke Design
75-4 -5 -6 -7	61.5 62.0 56.5 61.5	Old Yale Rd Section	Dyke Design
75-10	36.5	Beckstrom Rd Section	Outlet Works Foundation Design
75-11	41.5	Pattullo Bridge Section	Outlet Works Foundation Design
75-12 -20	41.5 66.5	Old Yale Rd Section	Outlet Works Foundtion Design
75-13	71.5	Manson Rd Section	Outlet Works Foundation Design
75-14 -15	56.5 56.5	Manson Rd Canal	Ditch and Ditch Dyke Design
75-17 -18 -19 -23	61.5 31.5 31.5 46.5	Collieries Ditch	Ditch and Ditch Dyke Design
75-21 -22	40.0 14.0	Pattullo Bridge Section	Dyke Design
75-24	41.5	Collieries Ditch	Outlet Works Foundation Design
75-25	51.5	Manson Rd	Dyke Design

The locations of all holes were surveyed and the collar elevation of the holes were determined with respect to Geodetic Datum.

c. Drilling and Sampling Procedures

All holes were 4-1/2-in. dia and were drilled by rotary drill using drilling mud.

The sampling procedure consisted in general of alternate undisturbed and disturbed samples, the latter in conjunction with standard penetration tests, at 5-ft intervals or change of stratum. In Drill Holes 75-21 and 75-22, continuous undisturbed samples of the dyke fill material were obtained. Where the foundation material consisted of alluvial sands, only disturbed samples were obtained.

d. Laboratory Testing

All disturbed and undisturbed samples recovered from the drill holes were visually inspected. All undisturbed samples were tested in the laboratory for natural moisture content, and, where applicable, laboratory vane shear strength. In addition, Atterberg limits were carried out on a few representative samples of organic silt. A total of four consolidation tests, two each on peat and organic silt, were performed. A few representative disturbed samples were also tested in the laboratory for natural water content and Atterberg limits.

e. Results of Investigation

The results of the investigation are shown on the attached drill hole logs. Only the results of the 1975 investigation are discussed in detail below. However, in arriving at the sub-surface stratigraphy, the results of the previous investigations by others were, of course, considered.

In general, a surficial layer of peat covers the site along the dyke alignment in the areas east of 126A Street and west of Tannery Road. This peat layer is underlain by a deposit of organic silt which varies in thickness from a nominal amount to greater than 30 ft. In the Beckstrom Road and Pattullo Bridge Sections, and in the eastern part of the Old Yale Road Section, organic silt becomes the surficial layer. The thickness of organic silt is a minimum in the Pattullo Bridge Section and it increases towards both the east and the west of this section, with the maximum thickness being at locations where the thickness of peat is also a maximum. The organic silts are underlain by essentially inorganic silts which gradually change with depth to silty fine sands and clean alluvial sands. The depth to the alluvial sands is a minimum in the Pattullo Bridge Section of the dykes and a maximum where the thickness of peat and organic silt strata is also a maximum.

A detailed description of the subsoil conditions in the various areas is given below:

i <u>Dyke Foundations</u>

<u>Bolivar Creek Section</u>

No additional drilling was carried out in this section of the dykes during the 1975 investigation.

Drilling carried out by others in this area encountered a 2 to 3-ft thick sand and gravel fill underlain by a peat deposit varying in thickness from about 10 ft near the north end of the section (near the junction of Industrial Avenue and 133rd Street) to 19 ft near the south end. The peat is underlain by a deposit of organic silt whose thickness increases to the south and varies between 13 and 25 ft. To the north, soft inorganic silt was encountered below the deposit of organic silt. Silty fine sand was encountered at a depth of about 48-ft over the entire section.

C.N.R. Section

All holes drilled during the previous investigations by others and during the 1975 investigation were located on the top of 6 to 8 ft of granular fill adjacent to the railway tracks.

The subsoil below the granular fill consists of peat varying in thickness from a maximum of about 20 ft in the middle of the section (near the intersection with 130th Street) to about 3 ft near the western end and about 6 ft near the eastern end. The peat is underlain by organic silt whose thickness is also a maximum in the middle of the section and decreases towards both ends, particularly towards the west. Organic silts are underlain by firm grey silts which are in turn underlain by silty sands. The depth from the ground surface to the silty sand stratum decreases gradually to the east and rapidly to the west from a maximum of 53 ft in the middle of the section. The depth near Bolivar Creek is 48 ft and near the Beckstrom Road outlet works about 15 ft.

Beckstrom Road Section

All holes drilled in this section encountered 4 to 6 ft of granular fill which to the east of about STA 10+00 is underlain by a small amount of peat which in turn is underlain by organic silt with occasional peat layers. The maximum combined thickness of peat and organic silt as determined from the 1975 investigations is about 8 ft. However, Hole 4A drilled in 1970 not far from Hole 75-2 encountered about 15 ft of peat and organic silt. The smaller depths of peat and organic silt encountered in 1975 are possibly due in part to the settlement of these compressible deposits under filling operations.

Pattullo Bridge Section

Two of the holes drilled in this section were located on the crest of the dyke.

The dyke fill material consists of a 2 to 4 ft dense mixture of clayey silt, sand and gravel. Loose brown sand, 1 to 3-ft thick, was encountered below, and this in turn is underlain by compact clayey silt and some fine sand.

The natural subsoil below the dyke is silt with a trace of organic matter and layers of silty fine sand. This material exists to a depth of 20 to 25 ft below the dyke crest and it is underlain by clean sand. The foundation stratigraphy appears to be consistent over the section.

Old Yale Road Section

This section of the dykes was investigated by drilling at relatively close intervals owing to the considerable variation in subsoil conditions.

In general, the existing dyke fill material in the section between Old Yale Road and Tannery Road, as revealed by Hole 75-5 is clayey silt with a trace fine sand. However, in the section between Sta 1+00 and Sta 19+00, a loose mixture of fill, wood and scrap metal was found in the dyke.

On the basis of Hole 75-7, the existing dyke fill west of Tannery Road would appear to consist of organic silt with some peat and hogfuel. The hogfuel had presumably been used as the base layer between the foundation and the dyke fill, and the dyke fill itself was probably obtained from adjacent ditch excavation. The foundation material under the dykes, between Old Yale Road and approximate Sta 19+00 comprises silt and silty fine sand with some organic matter. This material exists to a depth varying between 22 and 27-ft below the ground surface (approximate El -10.0 to -18.0) and it is underlain by organic silt which extends to a depth of between 44 and 50-ft below the ground surface (El -34.5 to -38.0). Silty fine sand which grades with depth to clean sand, underlies this organic silt.

Hole 75-6 drilled at Sta 19+50 indicates a transition in foundation stratigraphy between the above section and the section west of Tannery Road. At the location of this hole, silt with fine sand extends to a depth of about 17 ft below the ground surface (El -6.0) and this is underlain by a thick deposit of organic silt, extending to a depth of 48-ft below ground surface (El -37.0). Silty fine sand was encountered below this depth.

In the section west of Tannery Road, a peat deposit 7 to 12-ft thick was encountered below the dyke fill and this is underlain by about a 30-ft thick deposit of organic silt. There is a marked decrease in organic content in the lower half of the organic silt layer.

Manson Road Section

Most of this section has been brought close to the required grade by filling operations carried out by Weldwood. Consequently, no extensive investigation was undertaken.

One hole was drilled in the eastern part of the section through the fill in the Weldwood Plant. The foundation below the 8 to 9-ft thick fill consists of organic silt to a depth of about 18 ft. A deposit of sandy silt and silty fine sand extending to a depth of 34-ft below the top of the fill was then encountered. This deposit is underlain by more organic silt, extending to a depth of about 48 ft. Silty fine sand was then encountered and the hole was terminated after drilling about 4 ft in this material.

West of the Weldwood plant about 4 ft of dredged sand fill covers the area. The foundation under the fill consists of about a 10-ft thick organic silt layer which is underlain by fine sand with a trace of organic matter. The sand becomes clean and coarser with depth.

ii Outlet Works Foundations

Beckstrom Road

A drill hole located near the proposed pumphouse encountered thin layers of peat and organic silt under granular fill. Silty sand which changes to clean sand with depth, was found at a depth of about 15 ft (El -4.5). The standard penetration resistance of the sand below a 25-ft depth was about 20 blows per ft.

Old Yale Road

In Hole 75-20 drilled at the location of the proposed pumphouse, a relatively deep deposit of silty sand with organic matter was encountered below 6 ft of fill. At a depth of 28 ft (El -18.5) organic silt was encountered and this material extends to a depth of 44 ft (El -34.5). Dense silty sand was encountered below the organic silt with the standard penetration value being in the range of 40 to 60 blows per ft over the 20-ft length drilled into the sand. The stratigraphy varies considerably towards the riverside, as revealed by Drill Hole 75-12 which encountered compact clean sand at a depth of about 30 ft (El -20.0).

Manson Road

The foundation at the location of the pumphouse consists of compressible deposits of peat and organic silts to a depth of about 48 ft (El -40.0). Those deposits are underlain by sand with standard penetration values of about 25 blows per ft to a depth of about 60 ft (El -52.0) and 50 to 55 blows per ft between 60 and 70 ft.

Collieries

A 4-ft high dredged sand fill covers the area and this is underlain by a 10-ft thick layer of organic silt. Loose fine silty sand with a trace of organic matter was encountered below the organic silt to a depth of about 25 ft. The sand below ths depth becomes clean and compact with standard penetration value in the range of about 25 to 35 blows per ft.

iii Ditches

Collieries Ditch

The subsoil conditions along the Collieries ditch alignment are variable, with significant thicknesses of peat and organic silt over the section south of the C.N.R. and G.N.R. railway tracks. A 12-ft thick deposit of peat was encountered below the access road fill, which consists of lightweight material, logs, hogfuel mixed with topsoil at the location of Hole 75-17, about 700 ft north of River Road. Underlying the peat is a deposit of organic silt extending to a depth of about 56-ft below the ground surface. Near the railway tracks, (Hole 75-23) a 20-ft thick deposit of organic silt was encountered below the fill material. This organic silt is underlain by fine silty sand which grades to clean sand with depth. The area north of the tracks is covered with a 4 to 9-ft thick deposit of dredged sand. The subsoil below the sand fill consists of organic silt to a depth of 15 to 20 ft and this is underlain by silty fine sand with a minor organic content near the top of the layer. Over the area around Hole 75-18, a layer of hogfuel appears to have been placed below the dredged sand fill.

Manson Road Canal

In all the holes drilled along the banks of the Manson Road canal, a surficial layer of peat, about 10 to 12-ft thick, was encountered. This is underlain by organic silt to a depth of about 43 to 48-ft below ground surface. The upper part of the silt is very soft with a high organic content. Below a depth of 30 to 35 ft, the silt becomes firmer and the organic content becomes relatively low. The organic silt is underlain by fine silty sand.

A considerable variation in moisture content of the peat encountered in the holes was noted. The moisture content of a representative sample of peat from Hole 75-13 was found to be about 260 per cent, while that of the peat from Holes 75-14 and 75-15 was found to vary from about 700 to 960 per cent. Hole 75-13 was drilled from the top of 8 ft of fill and the lower moisture content is probably due to consolidation of the peat under the fill load. At holes 75-14 and 75-15, the height of fill above the peat is relatively small, and the higher moisture contents probably represent the virtually unconsolidated state of the peat.

f. Results of Laboratory Testing

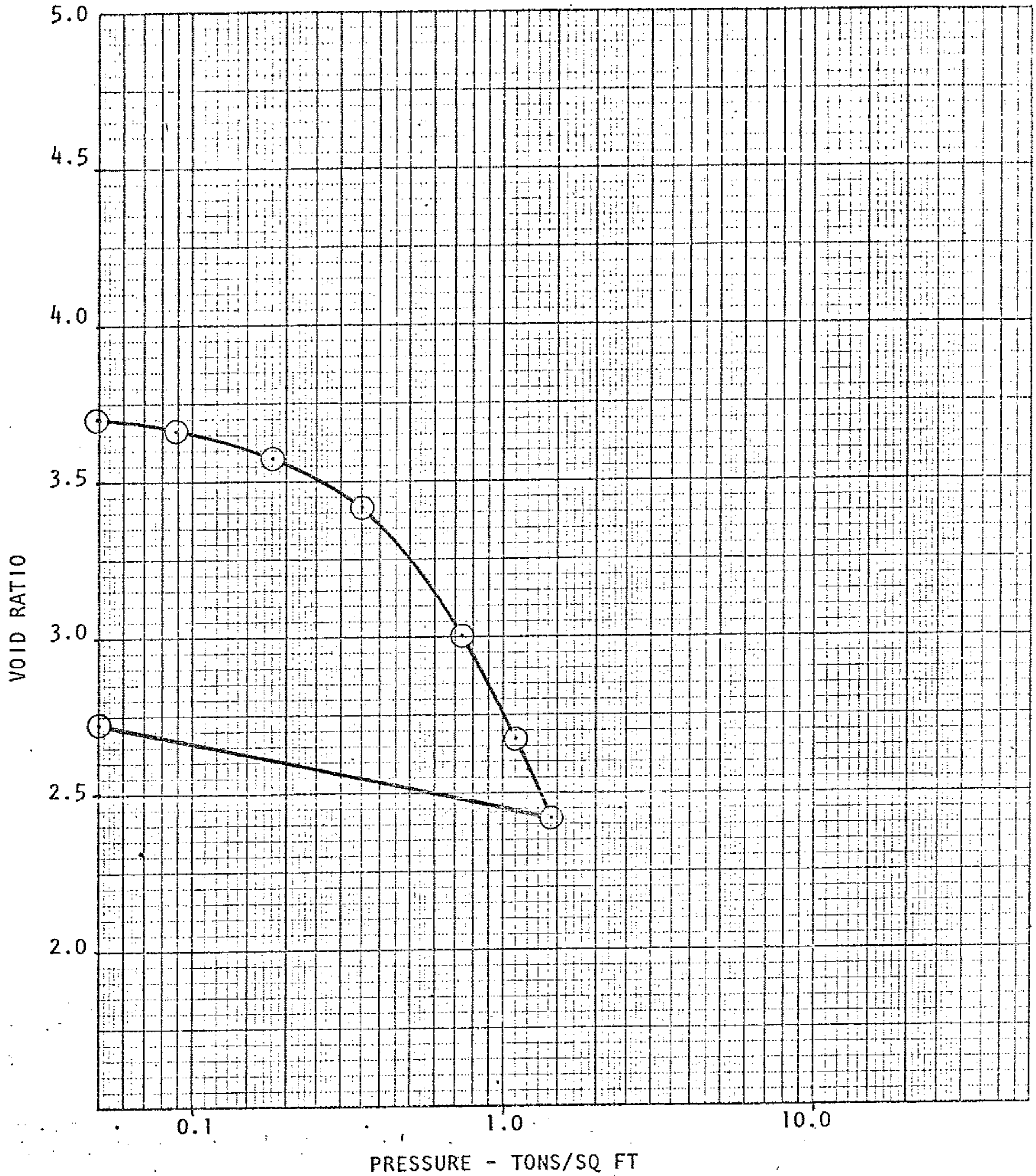
All the laboratory test results on samples from the exploratory drilling, with the exception of the consolidation test results, are summarized on the attached drill hole logs. The consolidation test results are given in the following table, and the pressure-void ratio curves are shown on Figs. 6.1-1 to 6.1-4 inclusive.

The test results indicate that in general, the organic silt is a soft, compressible material with its moisture content close to the liquid limit. It is not, of course, as compressible as the peat.

In addition to the laboratory testing carried out on samples from the exploratory drilling, gradation tests were carried out on samples of Fraser River dredged sand, and on sand and gravel from a potential borrow pit, as discussed in Sub-section 3.4. The results of these tests are shown on Figs. 6.1-5 and 6.1-6 respectively.

SUMMARY OF CONSOLIDATION TEST RESULTS

Hole No.	Sample Depth	Description	Moisture Content %	Atterberg Limits L.L.	Atterberg Limits P.L.	Atterberg Limits P.I.	Dry Density lb/cft	Wet Density lb/cft	Initial Void Ratio	Compression Index Cc	Preconsolidation Pressure Tons/sq ft
75-1	12'-14'	Peat, amorphous	178.0				27.8	77.5	3.72	2.15	0.50
75-13	10'-12'	Peat, fibrous	236.0	280.0	154.0	126.0	21.8	67.0	4.44	2.30	0.50
75-7	30'-32'	Organic silt	82.6	120.9	55.1	65.8	50.5	92.4	2.19	0.92	0.70
75-13	22'-24'	Organic silt	82.1	81.4	40.7	40.7	52.5	95.5	2.15	0.74	0.25



Drill Hole No. - 75-1
Sample Depth - 12.0'-14.0'
Description - Peat, dark brown, amorphous, with roots

Initial Moisture Content - 178.0%
Liquid Limit -) Not Tested
Plastic Limit -)

CONSOLIDATION TEST
e-log p CURVE

Fig. 6.1-1

Drill Hole No. - 75-13

Sample Depth - 10.0'-12.0'

Description - Peat, dark brown, fibrous with roots

Initial Moisture Content - 236.0%

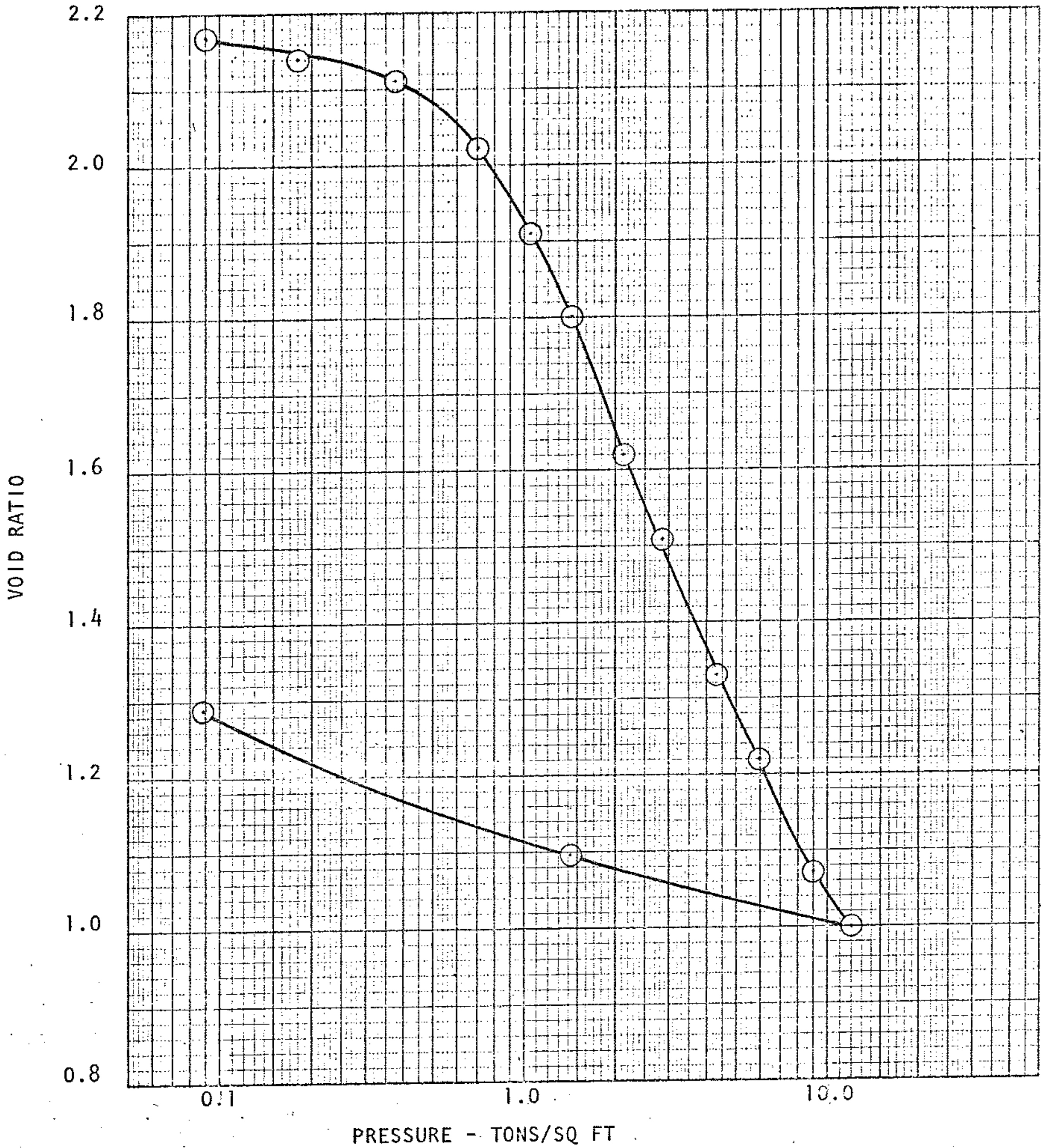
Liquid Limit - 280.0

Plastic Limit - 154.0

CONSOLIDATION TEST

e-log p CURVE

Fig. 6.1-2

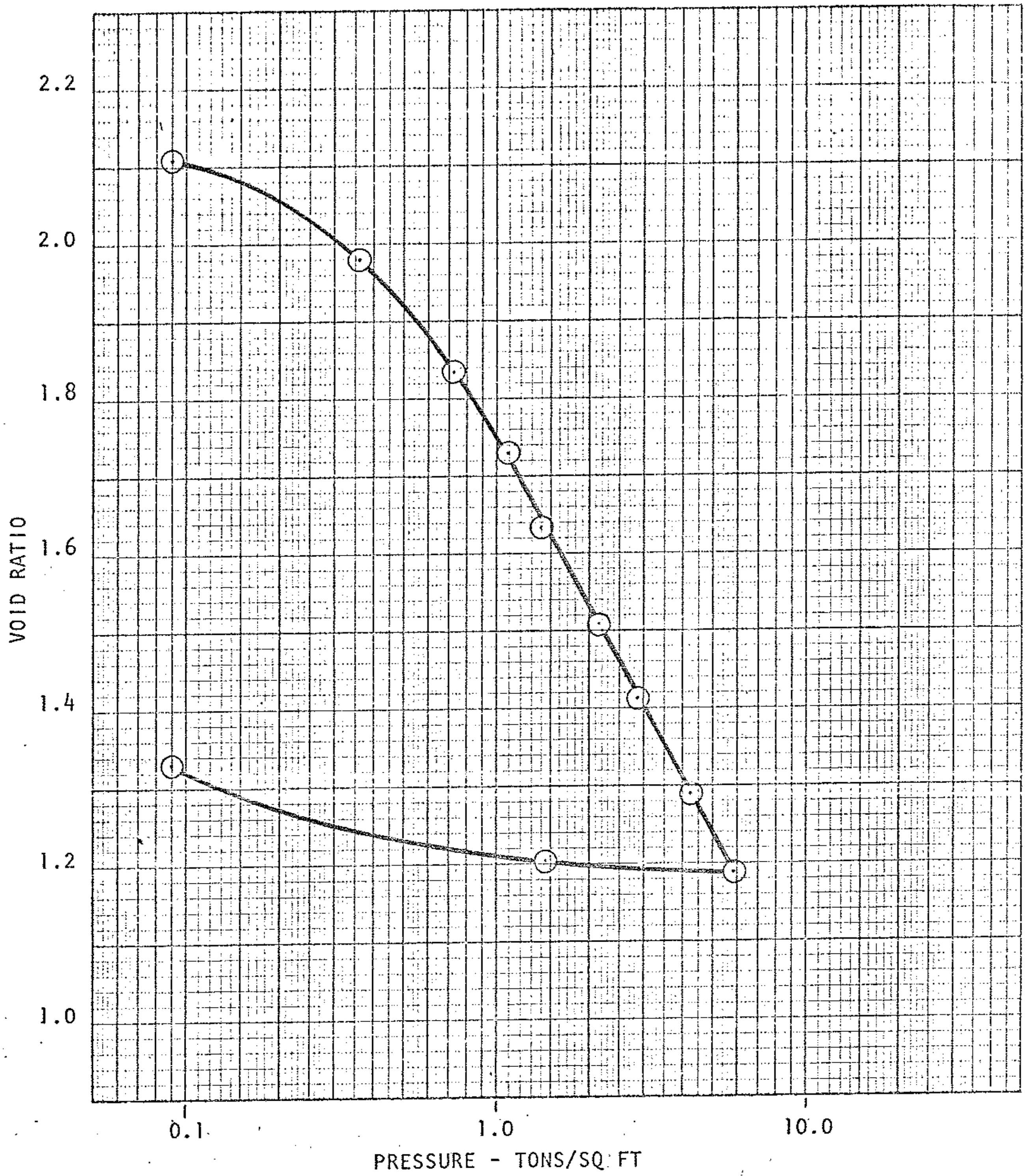


Drill Hole No. - 75-7
Sample Depth - 30.0'-32.0'
Description - Silt, organic, grey

Initial Moisture Content - 82.6%
Liquid Limit - 120.9
Plastic Limit - 55.1

CONSOLIDATION TEST
e-log p CURVE

Fig. 6.1-3

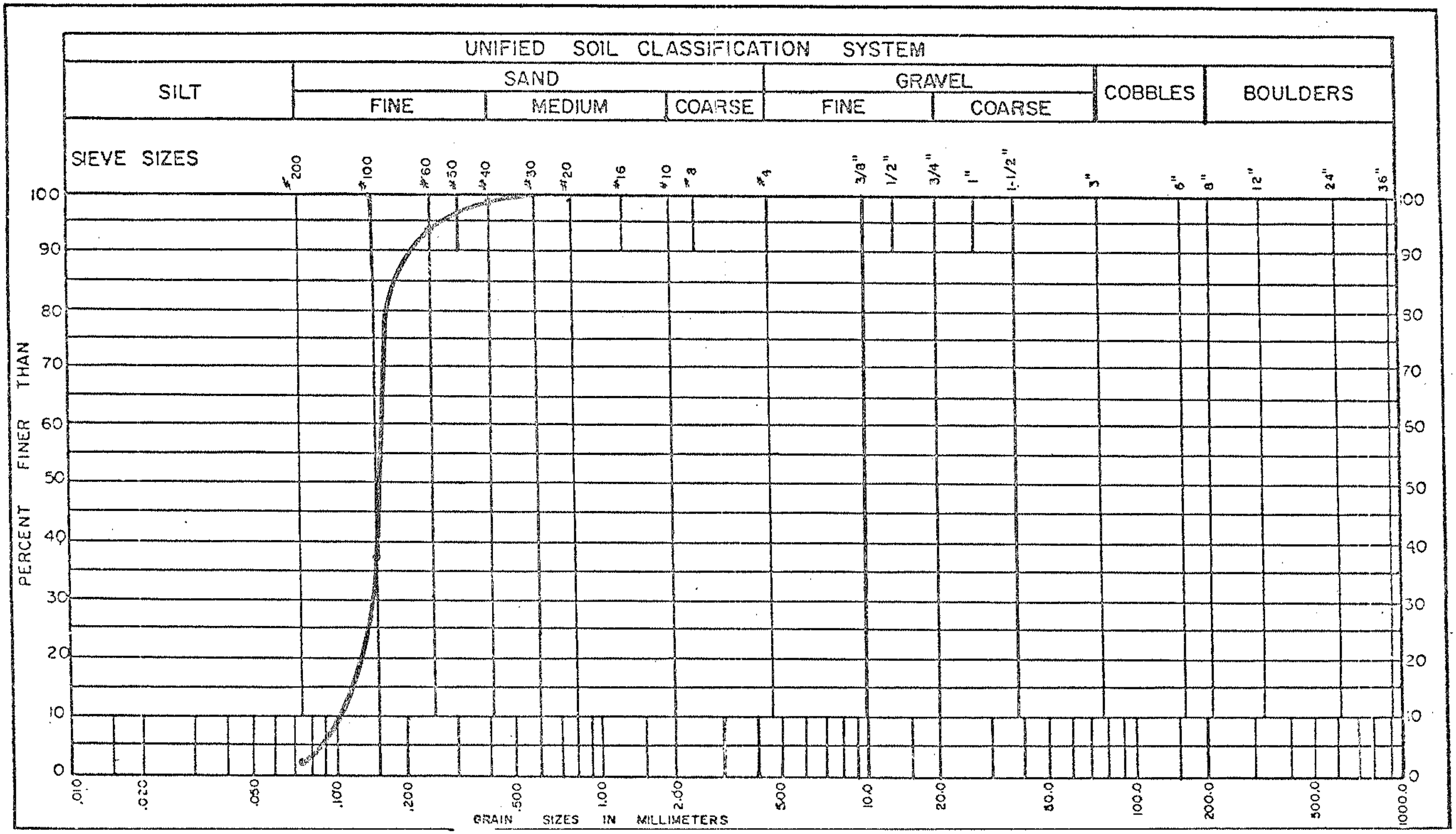


Drill Hole No. - 75-13
Sample Depth - 22.0'-24.0'
Description - Silt, organic, grey

Initial Moisture Content - 82.1%
Liquid Limit - 81.4%
Plastic Limit - 40.7%

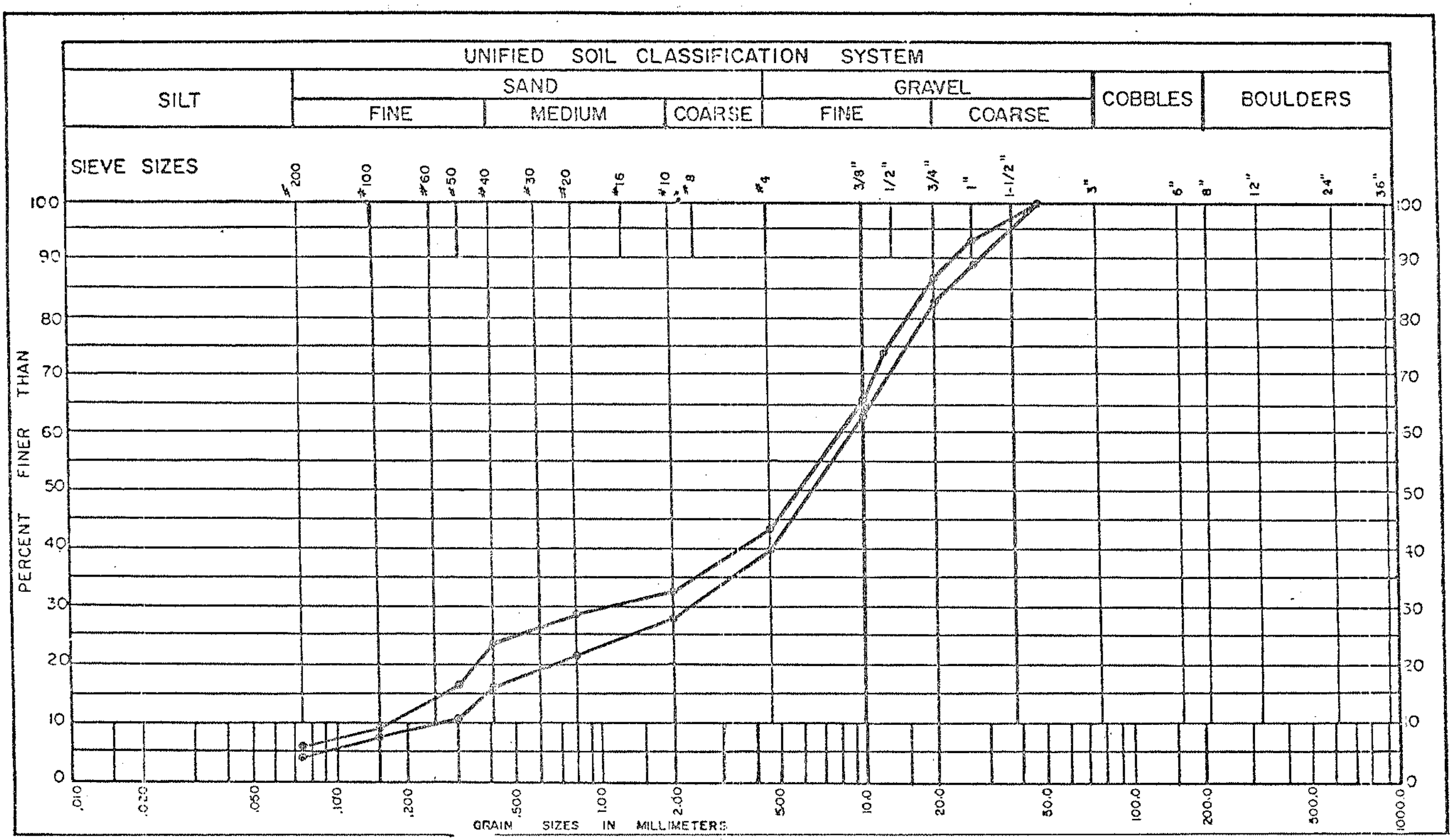
CONSOLIDATION TEST
e-log p CURVE

Fig. 6.1-4



FRASER RIVER DREDGED SAND
GRADATION CURVE

Fig. 6.1-5



SAND AND GRAVEL BORROW AREA
GRADATION CURVES

Fig. 6.1-6

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HOLE NO. 75-1
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 78700.86N,33031.21E
ELEVATION 9.6
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 14 - 15 Jan 1975

LEGEND
SPLIT SPOON
WASH SAMPLE
SHELBY TUBE
CORE SAMPLE

SHEAR STRENGTH
UNCONFINED COMPRESSION
LAB. VANE
PENETRATION RESISTANCE
STANDARD N - VALUE
ATTERBERG LIMITS
P.L. L.L.
MOISTURE CONTENT

DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF; 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
		9.6			
FILL, well graded sand and gravel	4.5	5.1			
SILT organic, numerous roots	6.0	3.6		1	20/24
PEAT, dark brown amorphous with frequent roots and pieces of decomposed wood			350%	2	24/24
	20.0	10.4	110%	3	24/24
SILT, organic, grey, soft with roots and pieces of decomposed wood, organic matter in thin layers				4	18/18
				5	24/24
				6	18/18
				7	19/24
				8	18/18
	49.0	39.4			
SILT, grey, trace clay and fine sand				9	18/18
	53.0	43.4			
SAND, grey, fine, trace silt, becoming coarser with depth				10	9/18
				11	6/18
	61.5	52.1			
End of hole					

Depth scale marks: 10, 20, 30, 40

CRIPPEN ENGINEERING LTD.
NORTH VANCOUVER B.C.

HOLE NO. 75-2
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 78129.07N, 30231.09E
ELEVATION 12.2
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 13 Jan. 1975

LEGEND
- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH
- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE
- STANDARD N - VALUE

ATTERBERG LIMITS
P.L. — MOISTURE CONTENT — L.L.

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 / 400 / 600 / 800 PSF; 20 / 40 / 60 / 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			12.2			
	FILL, silty and clayey sand with gravel, some gypsum from nearby gyproc plant				1	6/18
	SILT, grey, firm, trace organic matter	8.0	4.2			
	SILT, organic, grey, clayey with layers of dark brown fibrous peat and pieces of decomposed wood	10.0	1.7		2	22/24
	SAND, silty, fine, grey, soil with layers of silt, trace organic matter	16.0	-3.8		3	18/18
	SAND, fine, grey	21.0	-8.8		4	18/24
					5	18/18
					6	18/18
	End of Hole	31.5	-19.3			

CRIPPEN ENGINEERING LTD.

NORTH VANCOUVER B.C.

HOLE NO. 75-3

SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes

LOCATION OF HOLE 76208.78N,27503.25E

ELEVATION 11.5

CONTRACTOR Keller Soiltest Drilling

TYPE OF DRILL Rotary

DATE OF DRILLING 10 Jan 1975

LEGEND

- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH

- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE

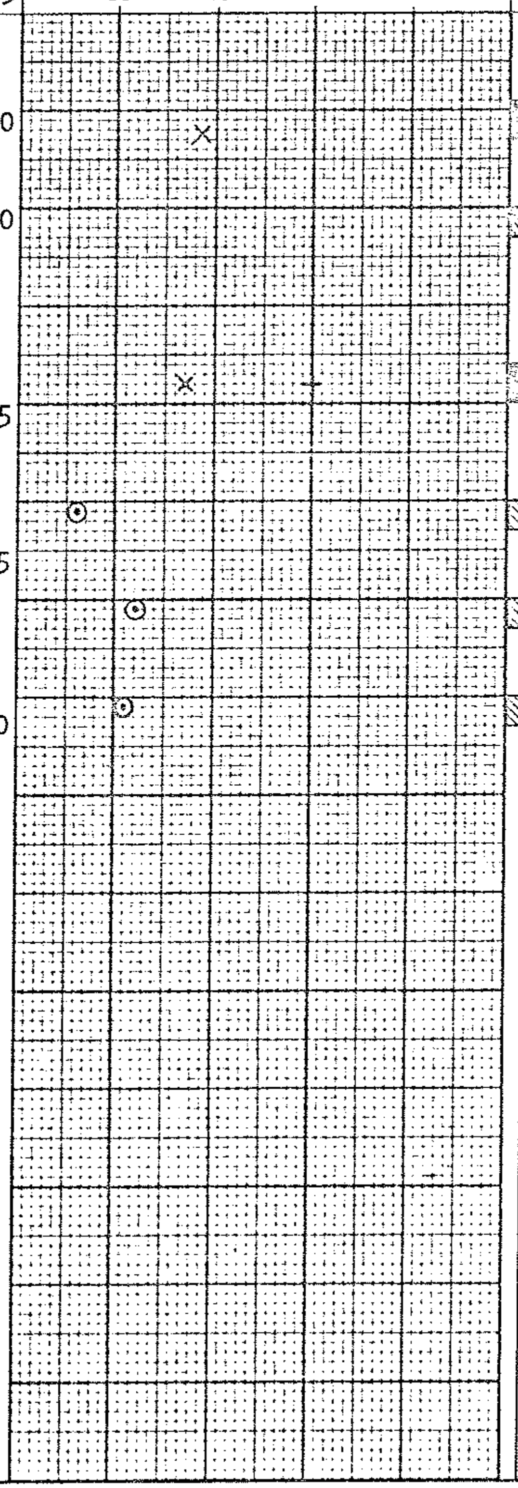
- STANDARD N - VALUE

ATTERBERG LIMITS

P.L. — L.L.

MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF / 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			11.5			
	FILL, topsoil, gravel and oily sand	5.5	6.0		1	24/24
	SILT, clayey trace fine sand, weathered	10.5	1.0		2	14/18
	SAND, fine, silty, trace organic matter, silt layers to 15-ft depth	21.0	-9.5		3	18/24
	SAND, grey, fine to medium with roots and decomposed wood pieces	28.0	-16.5		4	12/18
	SAND, brown to grey, fine, trace silt	36.5	-25.0		5	13/18
					6	13/18
	End of Hole					



CRIPPEN ENGINEERING LTD.

NORTH VANCOUVER B.C.

HOLE NO. 75-4

SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes

LOCATION OF HOLE 74288.25N, 25942.07E

ELEVATION 9.9

CONTRACTOR Keller Soiltest Drilling

TYPE OF DRILL Rotary

DATE OF DRILLING 2 - 3 Jan 1975

LEGEND

- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH

- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE

- STANDARD N - VALUE

ATTERBERG LIMITS

P.L. — MOISTURE CONTENT — L.L.

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF; 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			9.9			
	FILL, sand and gravel, hogfuel, cinders and wood planks	9.0	0.9		1	24/24
	SAND, fine, silty, grey with silt layers, organic matter in thin layers, pieces of decomposed wood	27.5	-17.6		2	3/18
					3	4/18
	SILT, organic, grey, soft becoming firm with depth, organic matter in thin layers, only trace organic matter below 40-ft depth	46.0	-36.1		4	18/24
					5	24/24
					6	24/24
					7	18/24
	SAND, fine, silty, grey, trace organic matter to 52-ft depth, sand becomes cleaner and coarser with depth	61.5	-51.6		8	18/18
					9	18/18
					10	18/18
	End of Hole					

Depth scale: 20, 30, 40, 50

CRIPPEN ENGINEERING LTD.
NORTH VANCOUVER B.C.

HOLE NO. 75-5
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 73620.86N,25393.21E
ELEVATION 11.9
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 3 - 6 Jan 1975

LEGEND
- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH
- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE
- STANDARD N - VALUE

ATTERBERG LIMITS
P.L. — L.L.
MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF; 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			11.9			
	DYKE FILL, silt, clayey, brown, weathered with wood particles (may be hogfuel base for dyke fill)				1	20/24
	SILT, grey, soft with silty fine sand layers, organic matter in thin layers	9.5	2.4		2	24/24
					3	18/18
		20			4	24/24
	SILT, organic, soft, grey with layers of peat and decomposed wood, trace fine sand below 40-ft depth, organic content decreasing with depth	22	-10.1		5	18/18
		30			6	24/24
					7	18/18
		40			8	24/24
					9	18/18
	SAND, fine, grey with silt layers, sand becomes cleaner and coarser with depth	50.0	-38.1		10	21/24
					11	18/18
					12	16/24
	End of Hole	62.0	-50.1			

CRIPPEN ENGINEERING LTD.

NORTH VANCOUVER B.C.

HOLE NO. 75-6

SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes

LOCATION OF HOLE 72983.96N
24876.78E

ELEVATION 11.1

CONTRACTOR Keller Soiltest Drilling

TYPE OF DRILL Rotary

DATE OF DRILLING 7-8 Jan. 1975

LEGEND

- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH

- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE

- STANDARD N - VALUE

ATTERBERG LIMITS

P.L. — L.L.

MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF / 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			11.1			
	FILL, sand and gravel, a thin layer of hogfuel below fill	3.0	8.1			
	SILT, clayey, brown, trace organic matter, trace fine sand	9.0	2.1		1	22/24
	SILT, grey, with layers of silty fine sand, trace organic matter	17.5	-6.4			0/18
					2	22/24
	SILT, organic, clayey, grey with pieces of decomposed wood and peat layers	20, 30, 40, 48.0	-36.9		3	25/30
					4	18/18
					5	24/24
					6	18/18
					7	24/24
					8	18/18
	SAND, fine, silty, trace organic matter; a layer of organic silt from 55' to 55.5'	56.5	-45.4		9	12/24
					10	18/18
	End of Hole					

CRIPPEN ENGINEERING LTD.

NORTH VANCOUVER B.C.

HOLE NO. 75-7

SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes

LOCATION OF HOLE 72340.60N
24463.40E

ELEVATION 11.9

CONTRACTOR Keller Soiltest Drilling

TYPE OF DRILL Rotary

DATE OF DRILLING 8-9 Jan. 1975

LEGEND

- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH

- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE

- STANDARD N - VALUE

ATTERBERG LIMITS

P.L. — L.L.
MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF / 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			11.9			
	DYKE FILL, Organic silt and peat with layer of hogfuel at bottom of fill				1	24/24
		8.0	3.9			
	PEAT, brown, fibrous with layers of organic grey silt, pieces of decomposed wood			117%	2	8/24
				243%	3	24/24
		17.0	-5.1			
	SILT, organic, clayey, grey, soft, becoming firm with depth, occasional layers of peat and pieces of decomposed wood				4	18/24
					5	18/18
				L.L. = 121	6	24/24
					7	18/18
					8	24/24
					9	18/18
		49.0	-37.1			
	SAND, fine, silty, grey with layers of silt, sand becomes cleaner with depth				10	12/18
		61.5	-49.6		11	12/18
	End of Hole					

CRIPPEN ENGINEERING LTD.

NORTH VANCOUVER B.C.

HOLE NO. 75-10

SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes

LOCATION OF HOLE 78397.90N
31179.61E

ELEVATION 11.0

CONTRACTOR Keller Soiltest Drilling

TYPE OF DRILL Rotary

DATE OF DRILLING 13-14 Jan. 1975

LEGEND

- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH

- ⊕ UNCONFINED COMPRESSION
- + LAB. VANE

PENETRATION RESISTANCE

- ⊙ STANDARD N - VALUE

ATTERBERG LIMITS

P.L. ——×—— L.L.
MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 / 400 / 600 / 800 PSF; 20 / 40 / 60 / 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			11.0			
	FILL, sand and gravel surface course of road, hog-fuel	7.0	4.0		1	24/24
	CLAY, silty, weathered some organic matter	11.0	0.0	198%	2	24/24
	PEAT, fibrous, brown	14.0	-3.0			
	SILT, organic, clayey some roots	15.5	-4.5		3	24/24
	SAND, fine, silty, grey becoming clean below 20' depth, occasional roots to 17' depth	20			4	12/18
					5	15/18
		30			6	18/18
		36.5	-25.5		7	15/18
	End of Hole					

CRIPPEN ENGINEERING LTD.
NORTH VANCOUVER B.C.

HOLE NO. 75-11
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 75304.05N,26908.41E
ELEVATION 12.9
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 9 Jan. 75

LEGEND
- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH
- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE
- STANDARD N-VALUE

ATTERBERG LIMITS
P.L. — L.L.
MOISTURE CONTENT

DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF; 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
		12.9			
FILL, sand and gravel, gravel layer at 11' depth. hogfuel from about 12' to about 14' depth	10			1	3/18
				2	0/18
SILT, organic, grey, only trace organic matter in thin layers below 20' depth	15.0	-2.1		3	8/24
				4	12/18
SAND, fine, grey, trace silt	21.5	-8.6		5	12/18
	30			6	12/18
SAND, fine, grey, trace gravel to approx. 3/4" size	35.0	-22.1		7	12/18
				8	9/18
End of Hole	41.5	-28.6			

CRIPPEN ENGINEERING LTD.
NORTH VANCOUVER B.C.

HOLE NO. 75-12
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 74589.91N
26079.51E
ELEVATION 10.5
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 7 Jan. 1975

LEGEND
- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH
- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE
- STANDARD N - VALUE

ATTERBERG LIMITS
P.L. L.L.
MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF; 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			10.5			
	SILT, organic, grey, with silty fine sand layers, a thin layer of gravel at 10' depth	10			1	3/24
					2	24/24
		16.0	-5.5		3	18/18
	SAND, fine to medium, grey, occasional gravel	20			4	24/24
					5	6/18
		30			6	24/24
					7	8/18
					8	12/18
	End of Hole	41.5	-31.0			

CRIPPEN ENGINEERING LTD.

NORTH VANCOUVER B.C.

HOLE NO. 75-13
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 71968.93N
24197.81E
ELEVATION 7.7
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 15-16 Jan. 1975

LEGEND
- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH
- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE
- STANDARD N - VALUE

ATTERBERG LIMITS
P.L. — MOISTURE CONTENT — L.L.

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET
			7.7
	FILL, containing peat and organic silt with gravel surfacing	8.0	-0.3
	PEAT, dark brown, fibrous, with roots and pieces of decomposed wood	15.0	-7.3
	SILT, organic, with peat layers to about 25' depth, occasional pieces of decomposed wood, organic content decreasing with depth	48.5	-40.8
	SAND, fine, grey, silty near top with silt layers, becoming clean and coarser below 60' depth	71.5	-63.8
	End of Hole		

TEST RESULTS: 200 / 400 / 600 / 800 PSF; 20 / 40 / 60 / 80 BLOWS/FT. OR %

260%

SAMPLE NO.	RECOVERY INCHES
1	24/24
2	24/24
3	18/18
4	24/24
5	18/18
6	24/24
7	18/18
8	24/24
9	18/18
10	9/18
11	12/18
12	12/18
13/	12/18
14	16/18

CRIPPEN ENGINEERING LTD.
NORTH VANCOUVER B.C.

HOLE NO. 75-14
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 71431.88N
24864.73E
ELEVATION 4.5
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 17 Jan. 1975

LEGEND
- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH
- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE
- STANDARD N - VALUE

ATTERBERG LIMITS
P.L. — L.L.
MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF; 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			4.5			
	TOPSOIL, peat, organic material with roots	3.0	1.5			
	PEAT, fibrous, dark brown with pieces of decomposed wood				1	12/24
		11.0	-6.5	968% → X 308% → X	2	24/24
	SILT, organic, grey, soft becoming firm below 35' depth. Isolated 1" thick layers of grey, stiff clayey silt at 32' and 36'-3" depth; occasional fine sand partings				3	18/18
		20			4	24/24
					5	18/18
		30			6	24/24
					7	14/18
		40			8	24/24
		43.0	-38.5			
	SAND, fine, grey, trace silt, trace organic matter to 52' depth, sand becomes medium with trace gravel below 53' depth				9	12/18
					10	18/18
		56.5	-52.0		11	18/18
	End of Hole					

CRIPPEN ENGINEERING LTD.

NORTH VANCOUVER B.C.

HOLE NO. 75-15

SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes

LOCATION OF HOLE 70438.85N 26090.41E

ELEVATION 5.3

CONTRACTOR Keller Soiltest Drilling

TYPE OF DRILL Rotary

DATE OF DRILLING 21 Jan. 75

LEGEND
- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH
- ⊕ UNCONFINED COMPRESSION
- + LAB. VANE

PENETRATION RESISTANCE
- ⊙ STANDARD N - VALUE

ATTERBERG LIMITS
P.L. ——×—— L.L.
MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF; 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			5.3			
	FILL, sand and gravel	1.5	3.8			
	PEAT, dark brown, fibrous with roots			692% → ×	1	24/24
		10			2	6/18
		14.0	-8.7			
	SILT, organic, grey, soft with layers of peat, some roots			112% → ×	3	24/24
		20			4	18/18
				+ —— ×	5	24/24
		30.0	-24.7		6	18/18
	Only trace organic matter, trace fine sand in thin partings			+ ×	7	12/24
		40.0	-34.7		8	18/18
	Firm with high organic content					
		46.0	-40.7		9	0/24
	SAND, fine, trace silt	50		⊙	10	15/18
				⊙	11	12/18
		56.5	-51.2			
	End of Hole					

CRIPPEN ENGINEERING LTD.

NORTH VANCOUVER B.C.

HOLE NO. 75-17

SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes

LOCATION OF HOLE 68009.80N, 23698.47E

ELEVATION 15.7

CONTRACTOR Keller Soiltest Drilling

TYPE OF DRILL Rotary

DATE OF DRILLING 22 January 1975

LEGEND

- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH

- ⊕ UNCONFINED COMPRESSION
- + LAB. VANE

PENETRATION RESISTANCE

- ⊙ STANDARD N - VALUE

ATTERBERG LIMITS

P.L. — × — L.L.

MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 / 400 / 600 / 800 PSF; 20 / 40 / 60 / 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			15.7			
	FILL- sand and gravel with logs, hogfuel	9.5	6.2			
	PEAT, dark brown, fibrous with roots and pieces of decomposed wood.			820%	1	24/24
					2	3/18
		21.0	-5.3	590%	3	24/24
	SILT, organic, with pieces of decomposed wood and roots.				4	18/18
		30		102%	5	24/24
					6	18/18
		40			7	24/24
					8	18/18
	layer of peat between 50.5' and 51.0'	50			9	18/18
		56.0	-40.3		10	18/18
	SAND, fine, silty, grey				11	
	End of Hole	61.5	-45.8			

CRIPPEN ENGINEERING LTD.

NORTH VANCOUVER B.C.

HOLE NO. 75-18

SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes

LOCATION OF HOLE 68615.19N, 22883.54E

ELEVATION 11.3

CONTRACTOR Keller Soiltest Drilling

TYPE OF DRILL Rotary

DATE OF DRILLING 23 January 1975

LEGEND

- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH

- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE

- STANDARD N - VALUE

ATTERBERG LIMITS

P.L. — L.L.

MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 / 400 / 600 / 800 PSF; 20 / 40 / 60 / 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			11.3			
	FILL, fine sand, grey, dredged from river	7.0	4.3		1	6/18
	FILL - Hogfuel	9.0	2.3			
	SILT, organic, with thin peat layers, layer of peat 9'-9.5', pieces of wood.				2	24/24
					3	18/18
		19.0	-7.7			
	SAND, fine, silty, with silt layers, trace organic material to 25' depth. Silt content decreasing with depth.				4	14/18
					5	18/18
					6	18/18
		31.5	-20.2			
	End of Hole					

CRIPPEN ENGINEERING LTD.
NORTH VANCOUVER B.C.

HOLE NO. 75-19
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 68855.42N, 22493.23E
ELEVATION 12.2
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 23 January 1975

LEGEND
- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH
- ⊕ UNCONFINED COMPRESSION
- + LAB. VANE

PENETRATION RESISTANCE
- ⊙ STANDARD N - VALUE

ATTERBERG LIMITS
P.L. L.L.
MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF; 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			12.2			
	FILL, fine sand, grey, dredged from river, trace organic matter	9.5	2.7		1	12/18
	SILT, organic, stiff, grey with roots and pieces of wood, 6" peat layer at top.	21.0	-8.8		2	20/24
					3	18/18
					4	20/24
	SAND, fine, silty, grey, trace organic matter to 27' depth, sand becomes clean below 27'.	31.5	-19.3		5	15/18
					6	12/18
	End of Hole					

CRIPPEN ENGINEERING LTD.

NORTH VANCOUVER B.C.

HOLE NO. 75-20
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 74403.82N, 26134.90E
ELEVATION 9.5
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 24 January 1975

LEGEND
- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH
- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE
- STANDARD N - VALUE

ATTERBERG LIMITS
P.L. L.L.
MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF; 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			9.5			
	FILL- sand and gravel surfacing	2.0	7.5			
	SILT, clayey, brown, weathered occasional pieces of wood, (probably fill)	6.0	3.5		1	24/24
					2	9/18
	SAND, fine, silty, grey, trace organic matter, pieces of black decomposed wood at 25' depth.	10			3	12/18
					4	12/18
		20			5	14/18
		28.0	-18.5		6	18/18
	SILT, organic, with occasional pieces of decomposed wood and peat layers.	30			7	24/24
		40			8	18/18
		44.0	-34.5		9	20/24
	SAND, fine, silty with silt layers, trace organie matter to 52' depth. Sand becomes clean and coarser below 60' depth.				10	18/18
		50			11	12/18
		60			12	12/18
					13	12/18
	End of Hole	66.5	-57.0			

CRIPPEN ENGINEERING LTD.

NORTH VANCOUVER B.C.

HOLE NO. 75-21
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 74629.76N, 26250.06E
ELEVATION 15.6
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 25 January 1975

LEGEND
SPLIT SPOON
WASH SAMPLE
SHELBY TUBE
CORE SAMPLE

SHEAR STRENGTH
UNCONFINED COMPRESSION
LAB. VANE
PENETRATION RESISTANCE
STANDARD N - VALUE
ATTERBERG LIMITS
P.L. — L.L.
MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 / 400 / 600 / 800 PSF; 20 / 40 / 60 / 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			15.6			
	DYKE FILL - clayey silt, sand gravel mixture, well graded, dense	4.5	11.1		1 2	20/24 21/24
	DYKE FILL - sand, fine, brown, loose	8.0	7.6		3 4	24/24 24/24
	SILT, clayey, weathered, trace fine sand	13.0	2.6		5 6	24/24 24/24
	SAND, fine, silty with silt layers, trace organic matter. Concentration of organic layers and occasional pieces of wood between 20' and 25'. Sand becomes clean below 25' depth.	20 30 40.0	-24.4		7 8 9 10 11 12 13 14	24/24 24/24 24/24 24/24 24/24 24/24 12/24 18/24
	End of Hole					

CRIPPEN ENGINEERING LTD.
NORTH VANCOUVER B.C.

HOLE NO. 75-22
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 74942.27N, 26510.81E
ELEVATION 13.8
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 25 January 1975

LEGEND
- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH
- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE
- STANDARD N - VALUE

ATTERBERG LIMITS
P.L. — L.L.
MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF / 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			13.8			
	DYKE FILL 0-2' clayey silt, sand and gravel dense 2'-3' sand, loose 3'-8' sand and gravel, silty, clayey	8.0	11.8 10.8 5.8		1 2 3	18/24 12/24
	SILT, clayey, weathered, brown trace fine sand.	11.0	2.8		4	24/24
	SILT & silty fine sand, trace organic matter.	14.0	-0.2		5	
	End of hole					

CRIPPEN ENGINEERING LTD.

NORTH VANCOUVER B.C.

HOLE NO. 75-23

SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster, Dykes

LOCATION OF HOLE 68,255.94N
23,390.28E

ELEVATION 14.3

CONTRACTOR Keller Soiltest Drilling

TYPE OF DRILL Rotary

DATE OF DRILLING 6 March 1975

LEGEND

- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH

- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE

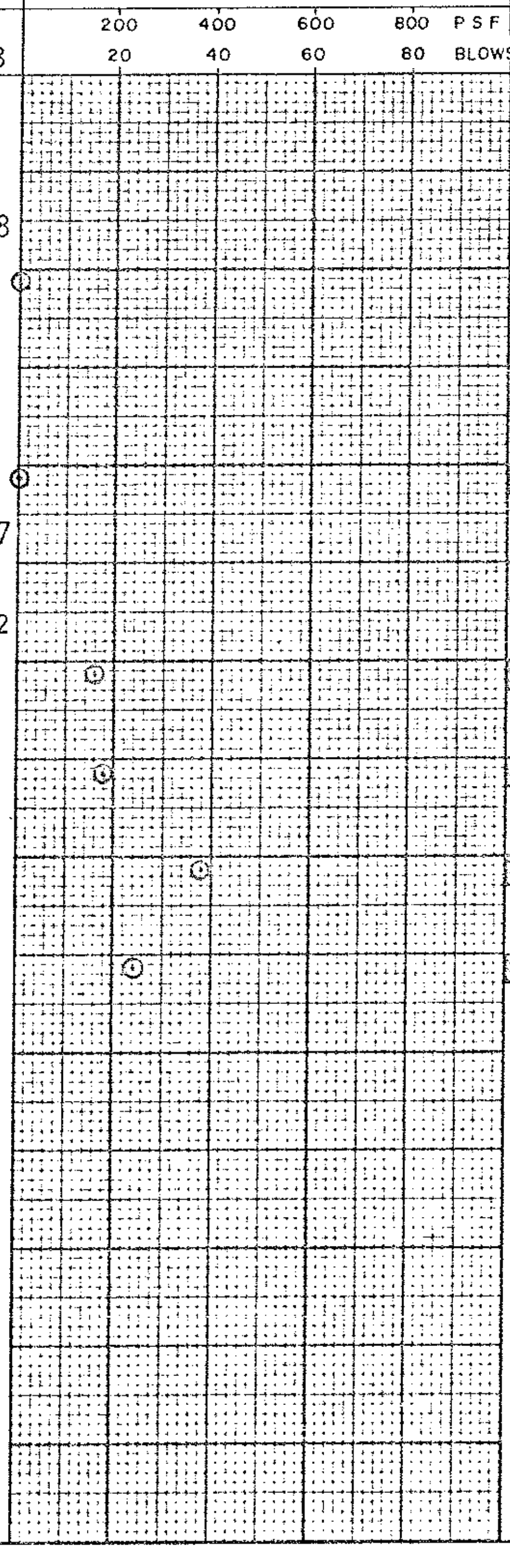
- STANDARD N - VALUE

ATTERBERG LIMITS

P.L. L.L.

MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF; 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			14.3			
	FILL, sand and gravel, predominantly hogfuel below 2.5'	8.5	5.8			
	SILT, organic, clayey, grey stiff, plastic, high organic content, weeds and roots below 15' depth.				1	12/18
					2	24/24
					3	3/18
		23.0	- 8.7			
	SILT, clayey, grey, firm trace organic matter				4	24/24
		28.5	-14.2			
	SAND, fine, grey, clean				5	9/18
					6	7/18
					7	7/18
		46.5	-32.2		8	10/18
	End of hole					



CRIPPEN ENGINEERING LTD.
NORTH VANCOUVER B.C.

HOLE NO. 75-24
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 69519.88N
21,844.73E
ELEVATION 12.3
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 7 March 1975

LEGEND
- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH
- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE
- STANDARD N - VALUE

ATTERBERG LIMITS
P.L. L.L.
MOISTURE CONTENT

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET	TEST RESULTS (200 400 600 800 PSF; 20 40 60 80 BLOWS/FT. OR %)	SAMPLE NO.	RECOVERY INCHES
			12.3			
	FILL, sand, trace gravel	4.0	8.3			
	SILT, organic, grey, firm, slightly plastic	10			1	15/24
					2	7/18
		14.0	1.7			
	SAND, fine, silty, grey, trace organic matter. Highly organic layer at 25.5' - 26.0' depth.				3	24/24
		20			4	9/18
					5	16/18
		30			6	8/18
					7	6/18
		41.5	-29.2		8	5/18
	End of Hole					

CRIPPEN ENGINEERING LTD.
NORTH VANCOUVER B.C.

HOLE NO. 75-25
SHEET 1 OF 1

LOG OF DRILL HOLE

PROJECT South Westminster Dykes
LOCATION OF HOLE 72,008.54N
23,752.97E
ELEVATION 10.4
CONTRACTOR Keller Soiltest Drilling
TYPE OF DRILL Rotary
DATE OF DRILLING 10 March 1975

LEGEND
- SPLIT SPOON
- WASH SAMPLE
- SHELBY TUBE
- CORE SAMPLE

SHEAR STRENGTH
- UNCONFINED COMPRESSION
- LAB. VANE

PENETRATION RESISTANCE
- STANDARD N-VALUE

ATTERBERG LIMITS
P.L. — L.L.
MOISTURE CONTENT

TEST RESULTS: 200 / 400 / 600 / 800 PSF; 20 / 40 / 60 / 80 BLOWS/FT. OR %

SYMBOL	DESCRIPTION	DEPTH FEET	ELEV. FEET
			10.4
	FILL, silty sand, trace gravel, hogfuel and pieces of decomposed wood.	8.5	1.9
	SILT, organic, grey, soft	18.0	-7.6
	SILT, sandy, trace organic matter in thin layers.	27.5	-17.1
	SAND, fine, silty, grey	34.0	-23.6
	SILT, organic, clayey, firm to stiff, moderately plastic	47.5	-37.1
	Sand, fine, trace silt	51.5	-41.1
	End of Hole		

SAMPLE NO.	RECOVERY INCHES
1	10/18
2	12/24
3	2/18
4	9/18
5	18/18
6	14/18
	0/18
7	
8	13/18
9	6/18