A COMPARISON OF ESTIMATED COSTS OF CONSTRUCTED WETLANDS AND OTHER ALTERNATIVES FOR IMPROVING LAGOON EFFLUENT QUALITY

prepared for

Environment Canada's Great Lakes 2000 Cleanup Fund

Ontario Ministry of the Environment

and

The Friends of Fort George

by

James W. Schmidt Associates Inc.

in collaboration with

E.E.C. Environmental Engineering Consultants

and

Stantec Consulting Ltd.
(formerly McNeely Engineering Consultants Ltd.)

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Although this document was subject to technical review, it does not necessarily reflect the views of the Cleanup Fund or Environment Canada or the Ontario Ministry of the Environment.

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EXECUTIVE SUMMARY

1.1 Background

In the Fall of 1991, The Friends of Fort George and the Regional Municipality of Niagara initiated a research project to evaluate a constructed wetland alternative (to a proposed physical chemical facility) to upgrade the effluent quality from the sewage lagoon treatment system serving the town of Niagara-on-the-Lake (NOTL), Ontario. It was recognized at the outset that a number of key issues would have to be resolved in order to treat the effluent effectively on a year round basis. These problems/challenges related to winter time freezing, cold temperatures, generally a lack of oxygen in the root-bed zone and effective nitrogen and phosphorus removal in a cold climate situation. These difficulties had been experienced in constructed wetland trials in Ontario at Listowel and Port Perry in the 1980s.

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Project results were presented in two interim reports and the final report. In addition, in 1996, as a consequence of the encouraging experimental results, it became desirable to determine the preliminary engineering feasibility of, and develop planning level cost estimates for a full scale constructed wetland facility capable of treating a flow of 5,710 m^3/day. This flow was specified in the Certificate of Authority for the NOTL STP issued in conjunction with the implementation of the physical chemical treatment facilities. This report was completed by James W. Schmidt Associates Inc. (JWSA) in February, 1997.

Subsequently, the MOE identified a requirement to develop and provide a more definitive planning level cost estimate for a full scale vertical subsurface flow wetland system and also, for a number of alternative design scenarios and technologies. These technologies were to meet the same effluent quality requirements as the NOTL STP. This work was undertaken by James W. Schmidt Associates in conjunction with E.E.C. Environmental Engineering Consultants (Sherwood [Woody] Reed) and McNeely Engineering Consultants Ltd (now Stantec Consulting Ltd.). McNeely Engineering was responsible for costing all the design scenarios, as well as developing a number of designs based on MOE guidelines, and the preparation of an appropriate report. They also subcontracted with Alfred College, University of Guelph, to develop design data for the flow distribution and underdrain piping networks associated with the vertical, subsurface flow wetlands. E.E.C. and JWSA were responsible for the preparation of reports, the development of conceptual designs for the wetlands and for reviewing the costing report prepared by McNeely Engineering Consultants.

Because of the requirement to complete the costing report and the SWAMP final report by March 31, 1997, it was not possible to provide McNeely Engineering with the final conclusions from the SWAMP project prior to March 31 and consequently, some of the preliminary conceptual designs (original designs) proposed for the vertical subsurface flow wetlands were superseded subsequently.

Two draft reports were prepared; one by McNeely Engineering Consultants and a second, summary report by JWSA in collaboration with E.E.C. and McNeely Engineering Consultants.

The purpose of this report is to combine the results of the two reports drafted in 1997 and add one additional full scale design scenario to the previously identified alternatives.

1.2 Alternative Design Scenarios

The following scenarios for the removal of BOD_5, TSS, NH_3-N and TP were identified by MOE. The first four scenarios were identified in 1996 and the fifth, in 1998.

1. Lagoon and vertical subsurface flow wetland based on the SWAMP project results
2. Lagoon and horizontal subsurface flow wetland
3. Lagoon and intermittent sand filter ("New Hamburg process")
4. Nitrifying activated sludge
5. Lagoon and free water surface wetland

For scenarios 1, 2 and 5, two alternative designs were to be developed where (1) TP is removed by the constructed wetland (CW), and (2) TP is removed by the addition of a chemical (alum) to the lagoon prior to the wetland.

1.3 Effluent Quality Requirements

The effluent quality requirements to be met are the same as for the NOTL STP and are as set out in Table 1.

Table 1. NOTL STP Effluent Requirements

Parameter[1]	Objective (mg/L)	Non-Compliance (mg/L)
BOD_5	15	25
TSS	15	25
TP	0.5	1
TNH_3-N (summer)[2] (winter)	5[3] 12[3]	10 20

1. Values are based on monthly averages of biweekly samples.
2. Summer is defined in the Certificate of Authority as May 01 to Oct 31.
3. In summer the average monthly loading cannot exceed 28.55 kg/d and in winter, 68.52 kg/d.

1.4 Methodology and Results

Based on the requirement to meet the effluent criteria, conceptual designs were developed, and capital, operation and maintenance, and "annualized" cost estimates were prepared. To compute the process and system costs, unit costs for various items, such as excavation, piping, etc., of the designs were estimated based on the experience of McNeely Engineering Consultants in eastern Ontario and costs developed for the preliminary feasibility study by JWSA. A significant component of the capital costs of the lagoons and wetlands, because of the substantial land area needed, relates to the requirement to provide a liner, and therefore, the final estimates were prepared for systems with and without a liner.

Annualized costs comprised two components. One was the estimated annual operation and maintenance (O & M) cost, which was assumed to be constant over a 20 year period, and the other was the capital cost debentured over 20 years at an interest rate of 10 %. On this basis a "true" comparison of the relative total system costs was made.

The capital and O & M costs for the various alternative systems are summarized in Table 2 and the annualized costs in Table 3.

Table 2. Alternative System Costs, Capital and O & M (Dollars)

System	Capital Cost (With Liner)	Capital Cost (Without Liner)	Annual O & M Costs
A/F Lagoon + VSF CW (TP removal in wetland)	8,597,120	6,907,520	179,500
A/F Lagoon + VSF CW (TP removal in lagoon)	4,983,073	3,663,073	163,500
A/F Lagoon + HSF CW (TP removal in wetland)	17,015,362	11,312,962	187,500
A/F Lagoon + HSF CW (TP removal in lagoon)	13,667,566	9,029,402	207,500
A/F Lagoon + FWS CW (TP removal in wetland)	10,862,182	5,318,182	187,500
A/F Lagoon + FWS CW (TP removal in lagoon)	4,699,599	2,841,039	167,500
Facultative Lagoon + ISF (TP removal in lagoon)	8,415,175	4,119,895	110,000
Activated Sludge Plant	(Not applicable)	12,921,000	799,400

Table 3. Summary of Treatment System Costs on an Annual Basis (Dollars)

System	Annualized Cost (With Liner)	Annualized Cost (Without Liner)
A/F Lagoon + VSF CW (TP removal in wetland)	1,185,365	987,680
A/F Lagoon + VSF CW (TP removal in lagoon)	746,520	592,080
A/F Lagoon + HSF CW (TP removal in wetland)	2,178,300	1,511,115
A/F Lagoon + HSF CW (TP removal in lagoon)	1,806,605	1,263,940
A/F Lagoon + FWS CW (TP removal in wetland)	1,458,375	809,725
A/F Lagoon + FWS CW (TP removal in lagoon)	717,355	499,900
Facultative Lagoon + ISF (TP removal in lagoon)	1,094,575	592,030
Activated Sludge Plant	(not applicable)	2,311,155

6.0 Summary and Conclusions

Based on the *NOTL effluent requirements*, for which alternative conceptual designs have been developed, and for which cost estimates have been prepared, the least cost system on an annualized basis is the combination of an aerated/facultative lagoon with TP removal, and a free water surface wetland, either with or without a liner.

The second lowest annualized cost system with a liner is the same lagoon system but with a vertical subsurface flow wetland. Without a liner, the second lowest annualized cost system is a facultative lagoon combined with an intermittent sand filter. However, (without a liner) the annualized costs of an A/F lagoon with TP removal, combined with a vertical subsurface wetland are essentially the same.

The rank order of the treatment systems, based on annualized costs, is presented in Table 4.

Table 4. Treatment System Ranking

System	Rank (With Liner)	Rank (Without Liner)
A/F Lagoon + VSF CW (TP removal in wetland)	4	5
A/F Lagoon + VSF CW (TP removal in lagoon)	**2**	**2**
A/F Lagoon + HSF CW (TP removal in wetland)	7	7
A/F Lagoon + HSF CW (TP removal in lagoon)	6	6
A/F Lagoon + FWS CW (TP removal in wetland)	5	4
A/F Lagoon + FWS CW (TP removal in lagoon)	**1**	**1**
Facultative Lagoon + ISF (TP removal in lagoon)	**3**	**2**
Activated Sludge Plant	N.A.	8

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1.0 INTRODUCTION

1.1 Background

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Subsequently, the MOE identified a requirement to develop and provide a more definitive planning level cost estimate for a full scale vertical subsurface flow wetland system and also, for a number of alternative design scenarios and technologies. These technologies were to meet the same effluent quality requirements as the NOTL STP. This work was undertaken by James W. Schmidt Associates (JWSA) in conjunction with E.E.C. Environmental Engineering Consultants (Sherwood [Woody] Reed) and McNeely Engineering Consultants Ltd. (now Stantec Consulting Ltd.). McNeely Engineering was responsible for costing all the design scenarios, as well as developing a number of designs based on MOE guidelines and the preparation of an appropriate report. They also subcontracted with Alfred College, University of Guelph, to develop design data for the flow distribution and underdrain piping networks associated with the vertical, subsurface flow wetlands. E.E.C. and JWSA were responsible for the preparation of reports, the development of conceptual designs for the wetlands and for reviewing the costing report prepared by McNeely Engineering Consultants.

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For scenarios 1, 2 and 5, two alternative designs were to be developed where (1) TP is removed by the constructed wetland (CW), and (2) TP is removed by the addition of a chemical (alum) to the lagoon prior to the wetland.

2.0 DESIGN REQUIREMENTS AND ASSUMPTIONS

2.1 Effluent Quality Requirements

The effluent quality requirements to be met are the same as for the NOTL STP and are as set out in Table 2.1.

Table 2.1. NOTL STP Effluent Requirements

Parameter[1]	Objective (mg/L)	Non-Compliance (mg/L)
BOD_5	15	25
TSS	15	25
TP	0.5	1
TNH_3-N (summer)[2] (winter)	5[3] 12[3]	10 20

1. Values are based on monthly averages of biweekly samples.
2. Summer is defined in the Certificate of Authority as May 01 to Oct 31.
3. In summer the average monthly loading cannot exceed 28.55 kg/d and in winter, 68.52 kg/d.

2.2 Design Boundary Conditions

Effluent requirements:

It is assumed that the design basis would be for systems to meet the effluent objective values and "never to exceed" the non-compliance values.

Flow:

Flow to the plant is to be the same as the Certificate of Authority for the NOTL STP, that is 5,710 m^3/day with a peak flow factor of 2. It is assumed that for scenarios 1, 2, 3 and 5 this will be accommodated in the design of the lagoons.

STP Influent Quality:

Data on the influent to the NOTL STP are provided in Table 2.2. These data are for the period 1991 through 1995 for the key parameters and are annual averages based on the monthly averages. The average at the bottom of the table is an average over this five year period and is based on the annual averages. From this table influent design concentrations were assumed. The monthly data are presented in Appendix A.

Table 2.2. NOTL STP Influent Quality

Year	Flow (ML/d)	BOD_5 (mg/L)	TSS (mg/L)	TKN-N (mg/L)	TP (mg/L)
1991	3.81	111	112	32.12	4.3
1992	4.04	102	102	29.85	4.1
1993	3.865	108	103	26.48	3.6
1994	4.015	124	121	23.43	4.42
1995	3.855	141	137	25.01	4.23
Average	**3.917**	**117**	**115**	**27.38**	**4.13**

From Table 2.2 it is noted that the influent concentration of BOD_5 appeared to be increasing in 1994 and 1995 and therefore, a conservative value of 150 mg/L was assumed as the influent for the various design scenarios. Similarly, a more conservative value of 5 mg/L was assumed for the influent TP concentration.

Lagoon Effluent Quality:
It was assumed that the effluent quality from the lagoon, in each scenario, would be similar to the effluent quality from the existing lagoon system at NOTL. These data are presented in Table 2.3.

Table 2.3. Lagoon Effluent Quality (Average Values, mg/L)

Year	BOD_5	TSS	TKN-N	NH_3-N	TP
1993	13.0	20.1	10.94	8.11	3.00
1994	14.8	45.0	11.58	7.83	2.71
1995[1]	21.2	40.0	9.80	6.64	2.54
Average	**16.3**	**35.1**	**10.77**	**7.53**	**2.75**

1. Data are from January 1 to July 12, 1995.

Soil and Site Conditions:
Although the existing lagoons at NOTL are constructed in blue clay and not lined, it is realized that at any particular site, a comprehensive soils investigation would have to

be undertaken before detailed engineering cost estimates could be completed. For lagoon systems the need for a liner is a significant issue because of the typically large land requirements. At this stage of cost estimation (i.e. hypothetical scenarios) it is assumed that there is essentially no information on the soils at the proposed site. Therefore, for purposes of these estimates a number of options were selected for costing, from essentially "no liner" to a quality synthetic liner for the lagoon system scenarios.

It was also assumed that the site is essentially level, that there is up to 30 cm of topsoil on the site and that it is covered in brush and a few small trees, necessitating "clearing and grubbing " (site preparation).

Processes:

For purposes of this report, information and cost estimates were prepared for the following processes:

(1) Aerated/facultative lagoon (continuous discharge)
(2) Facultative lagoon (four months of storage)
(3) Vertical subsurface flow constructed wetland (CW) (with TP removal capability)
(4) Vertical subsurface flow CW (TP removal in the lagoon)
(5) Horizontal subsurface flow CW (with TP removal capability)
(6) Horizontal subsurface flow CW (TP removal in the lagoon)
(7) Free water surface wetland (with TP removal capability)
(8) Free water surface wetland (TP removal in the lagoon)
(9) Intermittent sand filter (New Hamburg process)
(10) Nitrifying activated sludge process

Basic information on these processes is provided in the sections which follow.

2.3 Lagoon Facilities

A lagoon pre-treatment facility will be required for the constructed wetlands and the intermittent sand filter alternatives. As noted, the lagoon facilities are to provide an effluent quality similar to that of the existing NOTL treatment system. Based on the effluent quality data in Table 2.3 and data in the report, "Alternative Approaches for Upgrading Effluent Quality for Lagoon Based Systems,"(Anderson and XCG, 1992), the effluent quality to be achieved in the pre-treatment lagoons is as follows:

BOD_5	25 mg/L
TSS	40 mg/L (facultative lagoons can produce significantly greater TSS effluent in the summer due to an algae bloom. Estimated summer

	effluent in facultative lagoons can range between 30 and 60 mg/L)
TKN-N	12 mg/L
NH_3-N	8 mg/L
TP	4 mg/L (< 1 mg/L for the intermittent sand filter option)

Phosphorus (TP) removal will be achieved either in the constructed wetland or in the lagoon. For the intermittent sand filter alternative, TP will be removed in the lagoon.

2.4 Constructed Wetlands

Conceptual design information for the constructed wetlands was prepared by JWSA and E.E.C. Environmental Engineering Consultants (Appendix B). The following is a brief description of the concepts for the proposed wetland systems.

2.4.1 Vertical Subsurface Flow Wetland

Two conceptual designs were identified for the each of these alternatives (i.e., TP removal by the wetland and TP removal in the lagoon). The initial preliminary design for a wetland to remove TP consisted of a vertical flow system designed with a hydraulic loading rate (HLR) of 60 L/m^2.d and a total surface area of 95,167 m^2 (daily flow of 5,710 m^3/day). The system was comprised of three cells in series with dimensions of approximately 178 m x 178 m x 1.2 m deep. Because the cells would be in series, the first cell would be loaded at an average rate of 180 L/m^2.d (which then flows to the other cells). The proposed media in the cells was Queenston shale for the first two cells and Lockport Dolomite in the third cell. However, this design was modified during the course of the study to allow for a smaller footprint (thereby reducing the cost of the cells) and to improve its long term performance. The layout of the modified system consists of three groups of three cells in series (total of nine cells). The media are the same as the previous option. Due to the smaller footprint (63,444 m^2 as opposed to 95,167 m^2) the depth of the cells is greater (1.8 m) and consequently the HLR becomes 90 L/m^2.d because the flow rate (L/d) is constant. However, to ensure that there will be sufficient oxygen at lower depths, a perforated ventilation pipe system has also been introduced in the first row of cells. Figure 2.1 shows a typical system layout.

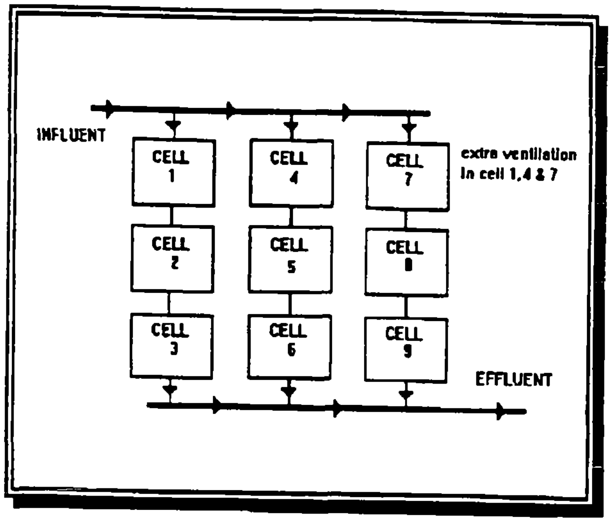

Figure 2.1. Layout of Final, Modified Design, Vertical Subsurface Flow Wetland (TP Removal in the Wetland)

The following two tables, 2.4 and 2.5, summarize the design parameters for the vertical subsurface flow wetlands.

Table 2.4. Summary of Vertical Subsurface Flow Wetland Parameters (TP Removal in the Wetland)

Parameter	Preliminary Design	Final, Modified Design
Flow (m^3/day)	5,710	5,710
HLR (L/m^2.d)	60	90
Cell Depth (m)	1.2	1.8
Total Surface Area (m^2)	95,167	63,444
Volume of Media (m^3)	114,200	114,200
Number of Cells	3	9
Area of Each Cell (m^2)	31,722	7,050
Cell Dimensions (m x m)	178 x 178	84 x 84

The area of the above wetlands is large due to the requirement for phosphorus removal. However, if phosphorus removal is achieved in the lagoons, the wetland area can be reduced by two thirds (Appendix B). This system will consist of three cells in parallel. The proposed media in the cells is Queenston shale and the HLR is 180 L/m^2.d. The perforated ventilation pipe system is not required as these cells are only 1.2 m deep. Figure 2.2 shows the system layout.

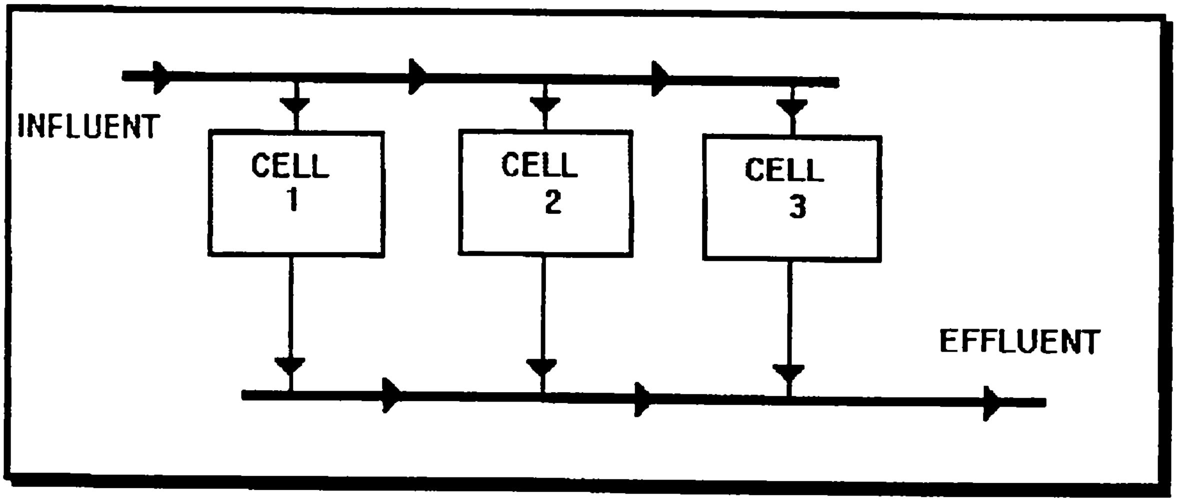

Figure 2.2. Layout of Vertical Subsurface Flow CW System (TP Removal in the Lagoon)

Table 2.5. Summary of Vertical Subsurface Flow CW Parameters (TP Removal in the Lagoon)

Parameter	Final Design
Flow (m^3/day)	5,710
HLR (L/m^2.d)	180
Cell Depth (m)	1.2
Total Surface Area (m^2)	31,722
Volume of Media (m^3)	38,066
Number of Cells	3
Area of Each Cell (m^2)	10,574
Cell Dimensions (m x m)	103 x 103

2.4.2 Horizontal Subsurface Flow Wetland

Two horizontal subsurface flow systems are proposed with the limiting parameter being either NH_3 removal or TP removal. The first system is designed to meet the TP requirement of 0.5 mg/L (as well as BOD_5, TSS, and NH_3-N requirements). To achieve this goal a total cell area of 434,931 m^2 (43.5 ha) is required. The second system is designed to meet the NH_3-N objective of 5 mg/L in summer and 12 mg/L in winter and requires a total cell area of 334,220 m^2 (33.4 ha). It is assumed that TP removal for the second system will be achieved in the pre-treatment lagoons through chemical addition. Cell depth is assumed at 0.6 m with an additional 0.6 m for freeboard.

2.4.3 Free Water Surface Wetland

Two free water surface wetlands are proposed. The limiting design parameters are again TP removal for one wetland and NH_3 removal for the other with TP removal in the pre-treatment lagoons. To achieve these goals an active cell area of 434,931 m^2 (43.5 ha) is required for the first wetland and 86,600 m^2 (8.66 ha) is required for the second. Cell water depth is assumed at 0.5 m with an additional 0.5 m for freeboard.

2.5 Intermittent Sand Filter

The conceptual design for the intermittent sand filter is based on previous designs from systems in New Hamburg and Schomberg, Ontario. The design assumes that the filters will operate over an eight month period and will be dormant during the winter. For this reason, a facultative lagoon with approximately 120 days of storage will be required for pre-treatment. It is assumed that TP removal will not be achieved in the sand filter but will be accomplished through chemical addition in the lagoons.

2.6 Nitrifying Activated Sludge Plant

The conceptual design for the nitrifying activated sludge plant is based on the extended aeration modification of the conventional activated sludge system, operated on a plug flow basis. The plant is assumed capable of treating the average daily flow of 5,710 m^3/day and a peak flow of approximately 14,275 m^3/day. The effluent requirements for the plant (which are based on the MOEE, "Manual of Environmental Policies and Guidelines" - December, 1994) is assumed to be 15 mg/L BOD_5, 15 mg/L TSS, 3 mg/L NH_3-N and 1.0 mg/L TP. The treatment process is described in further detail in Section 3.4.

3.0 CONCEPTUAL DESIGNS

3.1 Lagoon Facilities

The following section provides conceptual design information for the pre-treatment lagoons. These lagoon systems will be combined with either constructed wetlands or an intermittent sand filter to form various treatment design alternatives.

The preliminary lagoon alternatives presented are based on the MOEE manual, "Guidelines for the Design of Water Plants and Sewage Treatment Plants", July 1984.

3.1.1 Aerated/Facultative Lagoon (Continuous Discharge)

The aerated/facultative (A/F) lagoon is to provide pre-treatment prior to discharge into the constructed wetland facilities. Since the constructed wetlands are assumed to operate on a continuous basis, the A/F lagoon is designed to operate on a continuous basis to minimize the total volume required. As noted previously, the following effluent objectives for the lagoon need to be met:

- BOD_5 25 mg/L
- TSS 15 to 35 mg/L
- TKN-N 12 mg/L
- NH_3-N 8 mg/L
- TP 5 mg/L

To achieve the above objectives, the proposed A/F lagoon consists of a multiple cell system that includes a 5-day retention aerated cell and two facultative (quiescent) cells with a combined retention time of 25 days for a total system retention time of 30 days. Chemical TP removal will be required in the lagoon system for the design alternatives where the wetlands are not designed for TP removal. The A/F lagoon, therefore, consists of the following:

Design flow	5,710 m^3/day (average daily)
5 day Aeration cell	5,710 x 5 = 28,550 m^3
Operating depth	3.6 m
Area	7,930 m^2
25 day Quiescent cell	5,710 x 25 = 142,750 m^3
Assume 2 cells	71,375 m^3 each
Operating depth	1.8 m
Area	39,653 m^2

The quiescent portion of the lagoon system is divided into two cells of equal volume to allow for maintenance. The wetland system can operate for a short period of time under increased loading conditions to allow for cell maintenance (assumed once every 10 years). Further aeration in the quiescent portion of the lagoon (e.g., through windmill aerators) is not necessary due to the continuous flow operation and the aerobic nature of the wetland cells, particularly the vertical subsurface flow wetlands.

Figure C-1, Appendix C shows a conceptual layout for the proposed lagoon system.

3.1.2 Facultative Lagoon (120 days storage)

The facultative lagoon design is to provide retention of the flow in the winter prior to discharge to the intermittent sand filter (ISF) system. Since the ISF is assumed to be operational for only eight months of the year, the pre-treatment lagoon must have at least 4 months of storage capacity, identified here as 120 days. ISFs can accept effluent from facultative lagoons, therefore, the proposed design will be for a facultative lagoon. However, some mechanical aeration of the cells is assumed required in order to minimize H_2S concentrations during the winter months (This item can be removed from the cost estimates, if it is deemed that H_2S is not a concern). Chemical TP removal will be required in the lagoon system since existing ISFs have not been designed for TP removal. (In systems in Ontario, TP removal is achieved in the lagoon cells).

The proposed facultative lagoon design consists of two cells, each with 60 days of sewage flow retention. The following is a summary of the conceptual design:

Design flow	5,710 m^3/day (average daily)
Retention Time	120 days
Required Volume	685,200 m^3
Assumed 2 Cells	342,600 m^3 each
Operating Depth	1.8 m
Required Area	190,333 m^2 each cell

The cells will be drained on a continuous basis over an eight month period, and therefore, the drainage flow is the following:

Lagoon Volume	685,200	m^3
Drainage Period	245	days
Drainage Flow	2,797	m^3/day
Incoming Flow	5,710	m^3/day
Total Flow	8,507	m^3/day

Figure C-2, Appendix C shows a conceptual layout for the proposed lagoon design.

3.2 Constructed Wetlands

This section provides conceptual design information for the constructed wetland systems. It is assumed in the cost estimates that construction of the wetlands will coincide with the construction of the pre-treatment lagoons.

3.2.1 Vertical Subsurface Flow Wetland (preliminary, original design)

This design, in the course of the original work, was superceded with a "final" modified design. However, McNeely Engineering Consultants subcontractor, Alfred College, R&TT Section, University of Guelph did their principal calculations for both the influent subsurface piping network and the underdrain network on this design (Appendix D) with an addendum for the final modified design. As this preliminary wetland design was superceded it will not be discussed further.

3.2.2 Vertical Subsurface Flow Wetland (final, modified design)

Detailed design information for this wetland system is presented in Appendix B. This vertical flow system has a total active area of 63,444 m^2 and operates at a hydraulic loading rate of 90 $L/m^2.d$. The system consists of nine cells, in which the cells are arranged in three groups of three cells. Each group of three cells operates in parallel to the other two groups, and the cells within each group operate in series. (See Section 2.4.1, Figure 2.1). The approximate dimension of each cell is 84 m x 84 m x 1.8 m deep. The depth of the cells allows for 0.2 m of bottom gravel, 1.4 m of media and 0.2 m of distribution gravel at the top of the cells. Because the cells are in series within each group, the loading rate becomes 270 $L/m^2.d$ per cell (which is equivalent to an average loading rate of 90 $L/m^2.d$ over the entire area). The proposed media in each cell group consists of Queenston shale for the first two cells and Lockport Dolomite for the third cell. A series of air entrainment pipes, consisting of small diameter perforated pipes, has been introduced in the lower layer of the cell media to ensure an adequate supply of O_2 and improve wetland performance. It is proposed to separate the pipes in the same manner as the collection pipes.

The influent subsurface distribution and effluent collection piping networks were evaluated by the R&TT Section, Alfred College with the following results:

- The influent distribution system should consist of a pressurized network of pipes operating at a pressure of 50 Kpa (7.25 psi). To achieve a uniform distribution of the flow, the orifices in the pipe should be spaced at 2.5 m and the pipes themselves should be spaced at 2.5 m (which will form a 2.5 m grid for the distribution orifices). The orifices should be 3 mm in diameter to minimize clogging. The flow is to be pulsed at 4 hour intervals with each pulse lasting 111 minutes. This will provide a total of 666 minutes per day of flow over the wetland. The detailed computations for the orifice and pipe spacing are in

Appendix D.

- The collection system should consist of 200 mm diameter drainage pipes, separated by 12.5 m and underlain by a 0.15 m layer of gravel. The pipes should be laid on a slope of 0.2 % (Appendix D).

Using the information provided by Alfred College, a 50 mm diameter distribution network, consisting of watermain grade PVC pipe, is proposed for each cell. The network will have a 150 mm diameter perimeter pipe around each cell to equalize the pressure in the network. The 50 mm diameter pipe will be connected to the 150 mm diameter perimeter pipe from one end of the cell to the other and will be spaced by 2.5 m. As noted the pipes will have 3 mm diameter orifices drilled at every 2.5 m along the pipe length. These pipes will be arranged such that the orifices are staggered from side to side. The entire piping network will be placed below the top gravel layer which will be 0.2 m thick. Preliminary hydraulic calculations indicate a 2.2 m head loss in the network, assuming 0.17 L/s at each orifice. The required pumping head is therefore estimated at 2.2 m (network loss) + 2 m (station and fittings) + 3 m (static loss) + 5 m (residual pressure at 50 Kpa) for a total of 12.2 m (say 15 m).

The pumping station for the wetland is required to pump 150 L/s at approximately 15 m of head. This will be achieved by using a Flygt C3300 MT pump with a 639 impeller at 1170 RPM (60 HP).

Schematic details of a typical cell are shown in Figure C-3, Appendix C.

For the alternative in which TP removal is achieved in the lagoon as opposed to in the wetland, the system consists of three cells in parallel with a total areal of 31,722 m^2 (three cells of 103 m x 103 m). One pumping station is required to feed the system and the pulse dosing duration is estimated to be 300 minutes as opposed to 111 minutes. This will provide one half hour resting periods every 4 hours. Schematic details of a typical cell are shown in Figure C-4, Appendix C.

3.2.3 Horizontal Subsurface Flow Wetland

Two horizontal subsurface flow constructed wetland systems have been proposed and the design details are provided in Appendix B. The first system is designed to meet the TP requirement of 0.5 mg/L (as well as BOD_5, TSS, and NH_3 requirements) and requires a total cell area of 434,931 m^2 (43.5 ha). The second system is designed to meet the nitrogen objective of 12 mg/L NH_3-N in winter and requires a total cell area of 334,220 m^2 (33.4 ha). It is assumed that TP removal for the second system will be achieved in the pre-treatment lagoons through chemical addition. Cell depth for both systems is assumed to be 0.6 m with an additional 0.6 m for freeboard.

To achieve gravity flow throughout the wetland cells, it is preferable to divide each system into smaller cells operating in parallel. For the first option it would be impractical to construct one large 43.5 ha wetland. It is therefore assumed that six (6 smaller square cells would be constructed, each having a dimension of approximately 270 m X 270 m to reduce the hydraulic gradient of the system as well as ease construction. (The number of cells is arbitrary, however the greater the number of cells, the greater the cost of the earth works). The cells would operate in parallel, therefore requiring only one pumping station to supply the entire system.

It is assumed that the bottom of each cell will require a slope of 0.12% (assuming a hydraulic conductivity of 5000 m^3/m^2/day for the media). The end of each cell will therefore be 0.32 m lower than the front end. The inlet of each cell will consist of a perforated pipe buried into a coarse gravel trench approximately 1 m wide by 270 m long. The outlet of each cell will also consist of a perforated pipe buried in a coarse gravel trench. The dimensions of each cell, therefore, become 272 m long by 270 m wide.

The pumping station for the system is required to pump approximately 132 L/s at about 5 meters of head. This will be achieved by using a Flygt CP 3170 LT pump with a 607 impeller (25 HP).

A schematic layout of the system is shown in Figure C-5, Appendix C.

3.2.4. Free Water Surface Wetland

Two free water surface wetlands are proposed, as noted previously. The first system is designed to meet the TP requirement of 0.5 mg/L (as well as the requirements for BOD_5, TSS, and NH_3-N) and requires an active cell area of 434,931 m^2 (i.e., excludes area required for berms). The second system is designed to meet the effluent NH_3-N requirements (with TP removal in the pre-treatment lagoon) and requires an active cell area of 86,600 m^2 (8.66 ha). Cell water depth for both systems is assumed to be 0.5 m with an additional 0.5 m for freeboard.

To achieve gravity flow throughout the system and for practical maintenance purposes, it is preferable to divide the systems into smaller cells. For the first option 12 cells were assumed as four sets of parallel cells of three in series. A schematic is presented in Appendix C, Figure C-6. The size of each cell (bottom dimensions) then becomes 134 m wide by 270 m long. The cell sets operate in parallel and therefore, only one pumping station is required.

The bottom of each cell will slope 0.5 %. Flow through the wetland will be controlled by using a perforated manifold to distribute flow into the first row of cells and collect the effluent from the final row of cells. Weir boxes on about 40 m centres with adjustable weir plates will be located in the interior berms between cells.

For the second option, as noted, the active cell area requirement is 86,600 m^2 (8.66 ha). It is assumed that six cells will be appropriate, divided into two sets of parallel cells with three cells in series. Each cell will be 85 m wide by 170 m long (bottom dimensions). All other conditions will be the same as the previous option except that the weir boxes will be spaced at 20 m centres. A schematic is presented in Figure C-7, Appendix C.

As noted, the free water surface wetlands will be preceded by a lagoon. In the opinion of S. Reed (JWSA,1999), in southern Ontario (and presumably in other similar climatic regions in Canada) there would be no requirement for storage of the wastewater over the winter months. This is based upon the design objectives, particularly for ammonia, as free water surface wetlands are not very effective, summer or winter, for ammonia removal unless they are very large. In this case, given the influent quality and the design objectives, no ammonia removal in winter was required. Therefore, the chosen system consists of an aerated/facultative lagoon followed by a free water surface wetland. (It is recognized, that in Ontario in the 1980s, trials of free water surface wetlands were not considered successful, since the investigators had expectations of significant removals of ammonia and phosphorus in the winter months, which did not occur.)

In colder regions of Ontario (and Canada), if more stringent ammonia requirements were to prevail, then storage of the wastewater over the winter months might be a cost-effective alternative. This could be accomplished by using a facultative lagoon with, probably, at least four months of wastewater storage capacity.

3.3 Intermittent Sand Filter

The intermittent sand filter design is based on the New Hamburg process, which consists of applying intermittently over a number of slow sand filters on a rotational basis, effluent from either an aerated or a facultative lagoon. As the effluent seeps through the filter media, particulate matter, including algae cells, are strained from the wastewater and accumulate on the upper layers of the sand media forming a schmutzdeck. Lagoon effluent is pumped through a distribution system to the multi-media filters (sand and gravel). These gravity filters are flooded on an intermittent basis and underdrains are provided to collect the filtrate. Since no backwash mechanism is provided, a filter is operated until it has plugged with particulate matter, typically many weeks. It is subsequently taken off-line for resting, cleaning and maintenance, as appropriate.

Experience with the New Hamburg system had demonstrated that winter operation was not practical and that night time operation of the filters was preferable to avoid algae growth on the surface of the filters. Cleaning and maintenance of the filters typically included burning the vegetation and replacing the top layer of sand. Although commonly used in the United States, at the time of preparation of this report, only two systems were in operation in Ontario; in New Hamburg and in Schomberg.

Design Considerations:
The proposed design assumes that the filter operates over an eight month period and is dormant over the winter months. For this reason, the effluent from the facultative lagoon in Section 3.1.2 (with TP removal) is assumed as the influent to the sand filter. It is also assumed that the filter will have the same daily loading over the eight month operation period.

Design Flow:	5,710 m^3/day (avge daily nominal flow)
Discharge period	8 months
Lagoon storage required	685,200 m^3 (4 months)

Based on the report titled "Alternative Approaches for Upgrading Effluent Quality for Lagoon Based Systems", intermittent sand filters have been designed using nominal flows and loading rates ranging from 480 L/m^2.d to 560 L/m^2.d. As a conservative assumption, the proposed design is based on a loading rate of 480 L/m^2.d.

Size of filter to be based on	5,710 m^3/day (nominal flow)
Loading rate:	480 L/m^2.d. (nominal loading rate)
Filter Surface Area (5710/0.48)	11,900 m^2 (say 1.2 ha)

Use four cells of 3,000 m^2 each

(Note: Based on the lagoon drainage flow rate of 8,505 m^3/day, the actual loading rate becomes 715 L/m^2.d.)

Figure C-8, Appendix C shows a typical layout for the proposed intermittent sand filter.

3.4 Nitrifying Activated Sludge Plant (Extended Aeration)
The selected sewage treatment process is a modification of the conventional activated sludge system, which is operated on a plug flow basis (As per the MOEE guidelines for the design of sewage treatment works). The process is different somewhat from the conventional process in that no primary clarifiers are provided. The process is summarized in the following paragraphs.

Wastewater is pumped to the plant site via a forcemain which discharges into a screening channel (pumping to the plant is not included in this design). The main flow from this point throughout the plant and to the outlet is by gravity. The wastewater passes through a fine screen to grit removal facilities. Both screenings and grit are discharged to separate steel containers for disposal. The degritted wastewater then flows to an inlet distribution chamber where it is mixed with return activated sludge and distributed to aeration basins. Mixed liquor (wastewater plus activated sludge) is collected in an outlet collection channel to allow equal distribution to secondary clarifiers. A coagulant and flocculant aid may be added to the aeration basins just ahead of the outlet channel. The mixed liquor then flows to the secondary clarifiers. The effluent from the secondary clarifiers is disinfected with sodium hypochlorite and discharged to the outfall pipe through a Parshall flume chamber. The outfall pipe provides sufficient residence time to act as the disinfectant contact tank.

The plant is designed for dry weather flows of 5,710 m^3/day and is capable of accepting peak flows of 14,275 m^3/d. On the rare occasion that flows to the plant exceed this volume, a by-pass is provided that will allow excess flows to be by-passed from the degritter to the flume for disinfection.

Waste activated sludge is aerobically digested in a two-stage digester. It is assumed that digested sludge is applied to agricultural lands. During off-season periods for land application, the sludge is stored in a mixed, aerated, open top storage tank for later disposal.

The design also includes a control building to house a reception area, supervisor's office, control room, lunch room, laboratory, mechanical room, lockers, washroom, workshop and garage.

Figure C-9, Appendix C presents a process flow diagram for the treatment plant.

3.5 Areal Requirements

To obtain an appreciation for the relative size of the various processes, Table 3.1 was compiled which identifies the land requirements and provides to an extent an indication of what the relative magnitude of the capital costs will be, particularly for the lagoon based systems.

Table 3.1. Areal Requirements of the Various Processes*

Process	Area (ha)
Aerated/Facultative lagoon	15.1
Facultative lagoon	34.3
Vertical subsurface flow wetland (with TP removal capability)	6.4
Vertical subsurface flow wetland (TP removal in the lagoon)	3.2
Horizontal subsurface flow wetland (with TP removal capability)	43.5
Horizontal subsurface flow wetland (TP removal in the lagoon)	33.4
Free water surface wetland (TP removal capability)	43.5
Free water surface wetland (TP removal in lagoon)	8.7
Intermittent sand filter	2.5
Nitrifying activated sludge process	4

Note: * Exclusive of berms, roads, etc.

4.0 COST ESTIMATES

4.1 Aerated/Facultative Lagoon (Continuous Discharge)

4.1.1 Capital Costs

The cost of constructing the A/F lagoon, shown in Figure C-1, is estimated to be \$2,475,870, including land. Table 4.1 presents a detailed cost breakdown for this lagoon. The cost of items identified in Table 4.1 is based on actual construction costs for similar systems in eastern Ontario. The following is a brief description of some of the key components of the A/F lagoon.

- Site preparation includes clearing, grubbing and stripping of topsoil.

- Excavation volumes are estimated on the basis of balanced cut and fill on flat land. Berm heights allow for 0.6 meters of freeboard. Berm slopes are assumed to be 3:1.

- It is assumed that a liner is required. The cost of the liner is estimated on the following basis:

  Liners come in different materials ranging from natural clays to synthetic polyethylene geomembranes. Costs for the geomembranes range from \$5/m^2 to \$10/m^2. (The cost of polyethelene is often affected by fluctuations in petroleum costs).

  The liner however, may require the addition of an underdrain system to permit construction and eliminate excess hydrostatic uplift pressure conditions on the underside of the geomembranes (during maintenance when the lagoon is emptied). An underdrain system is estimated to cost \$5/m^2. The total costs of synthetic liners can range, therefore, from \$10 to \$15/m^2 if an underdrain system is required.

  If an adequate source of clay can be found within a reasonable distance from the site (within 20 km) a price of \$15/m^3 can be assumed for the clay liner. The cost is for placed material and includes additional excavation at the bottom of the lagoon to accept the clay liner. In our case a minimum thickness of 0.75 meters would be required, therefore the liner would cost approximately \$11.25/m^2 in place.

  For the purpose of this assignment, it is assumed that a suitable site can be found where an underdrain system would not be required. Therefore, the cost of placing a liner is estimated to be \$8/m^2 (average between clay and lower cost geomembrane).

- Perimeter fencing around the lagoon is assumed required (chain link fence).
- Site restoration includes placing of topsoil and seeding of disturbed areas (berms).
- Concrete distribution chamber includes valves and appurtenances for directing flow to quiescent cells.
- An alum injection system is only required where TP removal will be in the lagoon (and not by the wetland).
- Aerator system consists of an aeration building (complete with louvers and vents) to house the blower units, an air blower and submerged air diffusers.
- Magnetic flow meter is assumed for the flow metering system.
- Instrumentation and controls include chart recorder, auto dialer, emergency lighting, low voltage distribution and other electrical requirements.
- Effluent chamber includes a concrete structure, complete with valves, to control the lagoon effluent.
- Internal piping includes all pipes and appurtenances to direct the influent to the aerated cells, the quiescent cells and to the effluent chamber.
- Outlet sewer consists of a gravity system that is directed to the next phase of treatment.
- An electrical connection fee has been included. It is assumed that 3 phase power is not available at the side.
- Miscellaneous items include minor piping and valves, mechanical items, field office, trial operation and training, etc.
- Land costs are assumed to be $12,250 per ha ($5,000/acre) as per information provided in the James W. Schmidt Associates report (JWSA, 1997). For the purpose of this study land costs are assumed for poor clay lands in an undesirable location (option 1 in the JWSA. report). It is assumed that the preferred site would not be located in an area of high valued land. For the sake

of comparison, all options in this report will assume the same per ha land costs.

Table 4.1. Cost Estimate - Aerated/Facultative Lagoon

Items	Unit	Estimated Quantity	Unit Price (Dollars)	Amount (Dollars)
Mobilization/Demobilization	Lump Sum	1	50,000	50,000
Site Preparation	ha	18	5,000	90,000
Excavation/Berm Work	m^3	50,000	6	300,000
Liner	m^2	90,000	8	720,000
Fencing	m	1,500	45	67,500
Site Restoration	m^2	15,000	5	75,000
Influent Chamber	L.S.	1	15,000	15,000
Aeration Building	L.S.	1	25,000	25,000
Blower Unit	L.S.	1	75,000	75,000
Aerators	L.S.	1	30,000	30,000
Air Header Pipes	m	60	100	6,000
Flow Metering	L.S.	1	25,000	25,000
Instrumentation & Controls	L.S.	1	50,000	50,000
Effluent Chamber	L.S.	1	25,000	25,000
Internal Piping (300 mm dia.)	m	400	175	70,000
Outlet Sewer (500 mm dia.)	m	50	175	8,750
Electrical Connection	L.S.	1	25,000	25,000
Miscellaneous Items	L.S.	1	50,000	50,000
Contingency (10 %)				170,725
Design & Supervision (20 %)				375,595
Land	ha	18	12,350	222,300
Total				**2,475,870**

The estimated cost for the addition of an alum injection system to the aerated/facultative lagoon is presented in Table 4.2.

Table 4.2. Cost Estimate - A/F Lagoon and Alum Injection System

Items	Unit	Estimated Quantity	Unit Price (Dollars)	Amount (Dollars)
Alum Feed Building	L.S.	1	40,000	40,000
Chemical Injection Pumps	L.S.	1	30,000	30,000
Alum Storage	L.S.	1	50,000	50,000
Contingency (10%)				12,000
Design & Supervision (20%)				26,400
Sub-total				**158,400**
Total - Lagoon with alum				**2,634,270**

4.1.2 Operation and maintenance (O&M) costs:
The estimated annual operation and maintenance costs for the A/F lagoon are $105,000/yr. Electrical and chemical costs are based on existing data from the NOTL treatment system (JWSA, 1997). Excluding pumping and post lagoon treatment, the costs are comprised of the following:

• Energy: electricity for the blowers, instrumentation and controls.	$ 55,000/yr.
• Monitoring: Lab Costs	$ 10,000.
• Labour (28 person hours per week)	$ 30,000
• Equipment replacement:	$ 5,000
• Repairs	$ 5,000
Total	**$ 105,000/yr**

Note: Add $20,000 per year, to the O & M costs if an alum injection system is required.

4.2 Facultative Lagoon (4 months of storage)

4.2.1 Capital Costs

The cost of the facultative lagoon shown in Figure C-2 is estimated to be approximately $6,874,550 including land. Table 4.3 provides a detailed cost breakdown for the above system. The cost of items identified in Table 4.3 is based on actual construction costs for similar systems in eastern Ontario. The following is a brief description of some of the key components of the facultative lagoon which differ from those identified in the AF lagoon design.

- Alum injection is necessary since the intermittent sand filter is not designed to provide TP removal.

- Windmill aerators are used to minimize H_2S concentrations. It is assumed that approximately 1 aerator per ha of lagoon is required. If H_2S concentrations are not a concern, this item can be deleted.

4.2.2. Operation and maintenance costs

The estimated annual operation and maintenance costs for the facultative lagoon are $80,000/yr and are broken down as follows (costs do not include pumping and post lagoon treatment):

• Energy: electricity for instrumentation and controls	$ 5,000/yr.
• Monitoring: Lab costs	$10,000
• Chemical (alum)	$20,000
• Labour (28 person hours per week)	$30,000
• Equipment replacement:	$10,000
• Repairs	$ 5,000
Total	**$ 80,000**

Table 4.3. Cost Estimate - Facultative Lagoon

Items	Unit	Estimated Quantity	Unit Price (Dollars)	Amount (Dollars)
Mobilization/demobilization	Lump Sum	1	50,000	50,000
Site preparation	ha	40	5,000	200,000
Excavation/berm work	m^3	90,000	6	540,000
Liner	m^2	400,000	8	3,200,000
Fencing	m	2,500	45	112,500
Site restoration	m^2	31,000	5	155,000
Influent chamber	L.S.	1	20,000	20,000
Alum feed building	L.S.	1	40,000	40,000
Chemical injection pumps	L.S.	1	30,000	30,000
Alum storage	L.S.	1	50,000	50,000
Windmill aerators	Each	36	5,000	180,000
Flow metering	L.S.	1	25,000	25,000
Instrumentation & controls	L.S.	1	50,000	50,000
Effluent chamber	L.S.	1	25,000	25,000
Internal piping (300 mm dia.)	m	400	175	70,000
Outlet sewer (500 mm dia.)	m	50	225	11,250
Electrical connection	L.S.	1	25,000	25,000
Misc. Items	L.S.	1	50,000	50,000
Contingency (10 %)				483,375
Design & supervision (20 %)				1,063,425
Land	ha	40	12,350	494,000
Total				**6,874,550**

4.3 Vertical Subsurface Flow Wetland (Final, Modified Design)

4.3.1 Capital Costs

The estimated construction cost for the vertical flow wetland, shown in Figure C-3 (with TP removal capability) is $6,121,250, including land. Table 4.4 provides a detailed cost breakdown for the system. The cost of items identified in Table 4.4 are estimated using actual construction figures for similar items on other projects as well as information provided by JWSA (JWSA, 1997). The following is a brief description of some of the key components of the estimate for the final, modified design, vertical flow wetland.

- It is assumed that construction of the wetlands will coincide with the construction of the aerated lagoon, and therefore, there is no requirement for additional mobilization/demobilization costs.

- Site preparation includes clearing, grubbing and stripping of topsoil.

- Excavation volumes have been estimated on the basis of a balanced cut and fill on flat land and include the additional depth required to install the collection system at a 0.2% slope. Berm heights allow for 0.6 meters of freeboard and 1.8 meters of media. Berm slopes are assumed at 3:1 (outside) and 2:1 (inside).

- It is assumed that a liner is required. The cost of the liner is estimated on the same basis as the lagoons. It may be argued however, that natural attenuation would eliminate off site migration of pollutants entering the groundwater, thereby eliminating the need for a liner. As a conservative assumption, however, a liner is included in the costs.

- Media includes Queenston Shale for the first two cells and Lockport Dolomite for the last cell. Costs are based on average figures provided by JWSA.

- Site restoration includes placing of topsoil and seeding of disturbed areas (berms).

- Distribution pipes include 50 mm diameter watermain grade PVC pipe spaced at 2.5 meters and all appurtenances.

- Perimeter piping includes 150 mm diameter watermain grade PVC pipe around the perimeter of each cell. Costs include all pipes and appurtenances.

- Collection pipes include 200 mm diameter drainage pipes spaced at 12.5 meters and all appurtenances.

- Collection drain cleanouts are assumed at both extremities of pipes.
- Air entrainment pipes (for the alternative with TP removal by the wetland only) consist of small diameter tile drainage pipes (100 mm) and are spaced at 12.5 meters (same as collection pipes).
- Vegetation includes \$2,500/ha for purchase and \$1,000/ha for planting. Cost of vegetation is based on previous experience in eastern Ontario.
- Each pumping station includes two Flygt C3300 MT pumps with 638 impellers. Cost includes wet well, pumps, piping and valves, metal work (platforms), electrical and site work.
- Miscellaneous items include minor piping and valves, miscellaneous mechanical items, trial operation and training, etc.
- Land costs have been assumed at \$12,250 per ha (\$5,000/acre).

If it is assumed that TP removal will be achieved in the lagoon as opposed to the wetland, the wetland is modified to a three cell system (in parallel) with each cell having dimensions of approximately 103 m x 103 m. The aeration drains are removed and the hydraulic loading rate is increased to 180 L/m^2.d, which increases the pulse dosing to 300 minutes per pulse as opposed to 111 minutes. Table 4.5 provides a detailed cost breakdown for the system. Figure C-4, Appendix C shows a typical cell layout.

Table 4.4. Cost Estimate - Vertical Subsurface Flow Wetland (Final, Modified Design)

Items	Unit	Estimated Quantity	Unit Price (Dollars)	Amount (Dollars)
Mobilization/Demobilization	N.A.			0
Site Preparation	ha	7	5,000	35,000
Excavation/Berm Work	m^3	44,300	6	265,800
Cell Liner	m^2	70,000	8	560,000
Media (Queenston shale)	m^3	76,133	15.5	1,180,062
Media (Lockport dolomite)	m^3	38,067	18	685,206
Site Restoration	m^2	30,000	5	150,000
Distribution Piping (50 mm)	m	29,100	10	291,000
Distribution Piping (150 mm)	m	3,100	25	77,500
Distribution Network Gravel	m^3	13,500	18	243,000
Aeration Drains	m	5,000	5	25,000
Collection Drain Piping	m	6,600	15	99,000
Collection Drain Cleanout	m	126	75	9,450
Collection Drain Gravel	m	10,000	18	180,000
Outlet Sewer (500 mm dia)	m	50	175	8,750
Vegetation	ha	6.3	3,500	22,050
Miscellaneous Items	L.S.	1	50,000	50,000
Pumping Station (to wetlands)	Each	3	230,000	690,000
Contingency (10 %)				457,182
Design & Supervision (20 %)				1,005,800
Land	ha	7	12,350	86,450
Total				**6,121,250**

Table 4.5. Cost Estimate - Vertical Subsurface Flow Wetland (TP Removal in the Lagoon)

Items	**Unit**	**Estimated Quantity**	**Unit Price (Dollars)**	**Amount (Dollars)**
Mobilization/Demobilization	N.A.*			0
Site Preparation	ha	3.5	5,000	17,500
Excavation/Berm Work	m^3	13,000	6	78,000
Cell Liner	m^2	35,000	8	280,000
Media (Queenston shale)	m^3	39,000	15.5	604,500
Site Restoration	m^2	30,000	5	150,000
Distribution Piping (50 mm)	m	13,000	10	130,000
Distribution Piping (150 mm)	m	1,250	25	31,250
Distribution Network Gravel	m^3	2,300	18	41,400
Collection Drain Piping	m	2,700	15	40,500
Collection Drain Cleanout	m	42	75	3,150
Collection Drain Gravel	m	5,300	18	95,400
Outlet Sewer (500 mm dia)	m	50	175	8,750
Vegetation	ha	3.2	3,500	11,200
Miscellaneous Items	L.S.	1	25,000	25,000
Pumping Station (to wetlands)	Each	1	230,000	230,000
Contingency (10 %)				174,665
Design & Supervision (20 %)				384,263
Land	ha	3.5	12,350	43,225
Total				**2,348,803**

* N. A. = not applicable

4.3.2 Operation and Maintenance Costs

The estimated annual operation and maintenance costs for the final, modified design, vertical subsurface flow wetland (with TP removal) are $74,500/yr and are broken down as follows.

• Energy: electricity for the pumps and controls	$45,000/yr.
• Monitoring: Lab costs	$10,000
• Labour (7 person hours per week - pumps)	$ 7,500
• Equipment replacement and repairs	$ 5,000
• Landscaping (weekly maintenance):	$ 7,000
Total	**$74,500/yr.**

For the three cell system (TP removal in the lagoon), the operation and maintenance costs are:

• Energy: electricity for the pumps and controls	$15,000/yr.
• Monitoring: Lab costs	$10,000
• Labour (7 person hours per week - pumps)	$ 7,500
• Equipment replacement and repairs	$ 2,500
• Landscaping (weekly maintenance):	$ 3,500
Total	**$38,500/yr.**

4.5 Horizontal Subsurface Flow Wetland

4.5.1 Capital Costs

The estimated construction costs for the horizontal subsurface flow wetland, shown in Figure C-5, is $ 14,539,492, including land. Table 4.6 provides a detailed cost breakdown for the wetland system. The cost of items identified in Table 4.6 are estimated using actual construction costs for similar items on other projects as well as information provided by JWSA (JWSA, 1997). The following is a brief description of some of the key components of the horizontal subsurface flow wetland.

- Site preparation includes clearing, grubbing and stripping of topsoil

- Excavation volumes have been estimated on the basis of a balanced cut and fill on flat land and include the additional depth required to allow for a slope of
- 0.12 % to satisfy hydraulic gradient requirements. Berm heights allow for 0.6 m of freeboard. Berm slopes are assumed at 3:1 exterior and 2:1 interior.

- It is assumed that a liner is required. The cost of the liner is estimated on the same basis as the lagoons. It may be argued however, that natural attenuation would eliminate off site migration of pollutants entering the groundwater, thereby eliminating the need for a liner. As a conservative assumption, however, a liner is included in the costs.

- It is assumed that gravel is the media used in all cells.

- Site restoration includes placing of topsoil and seeding of disturbed areas (berms).

- The inlet pipe for each cell is assumed to be a 300 mm diameter perforated pipe, installed in the upper layer of a 1 m wide by 1 m deep, coarse gravel trench. The trench is tapered to join with the cell media at 0.6 m thickness. The cost includes gravel media for the trench.

- The outlet pipe for each cell is a 200 mm diameter perforated drain pipe installed in a 1 m wide by 1 m deep gravel trench. The cost includes gravel media for the trench.

- Inlet and outlet distribution chambers consist of typical "maintenance holes" (manholes) with valves and appurtenances to allow for distribution of flow or draining of each cell.

- Internal pressure piping is to connect the distribution chambers.

- Internal gravity piping is to collect all flow from the effluent chambers and direct it to the outlet.

- A 300 mm diameter outlet sewer is assumed at the end of the system.

- The pumping station includes two Flygt CP 3170 pumps with 607 impellers. The cost includes wet well, pumps, piping and valves, metal work (platforms), electrical and site work.

- Miscellaneous items include minor piping and valves, miscellaneous mechanical items, trial operation and training, etc.

- Land costs have been assumed to be $ 12,250 per ha ($ 5,000/acre).

Table 4.6. Cost Estimate - Horizontal Subsurface Flow Wetland

Items	Unit	Estimated Quantity	Unit Price (Dollars)	Amount (Dollars)
Mobilization/Demobilization	N.A.			0
Site Preparation	ha	44	5,000	220,000
Excavation/Berm Work	m^3	72,000	6	432,000
Cell Liner	m^2	450,000	8	3,600,000
Media (gravel)	m^3	278,000	18	5,004,000
Site Restoration	m^2	65,000	5	325,000
Inlet Pipe (incl. gravel trench)	m	1,620	35	56,700
Outlet Pipe (incl. gravel trench)	m	1,620	35	56,700
Distribution Chambers	Each	6	2,500	15,000
Outlet Chambers	Each	6	2,500	15,000
Internal Piping (pressure)	m	2,500	75	187,500
Internal Piping (gravity)	m	1,200	125	150,000
Outlet Sewer (300 mm dia)	m	50	175	8,750
Vegetation	ha	43.5	3,500	152,250
Miscellaneous Items	L.S.	1	50,000	50,000
Pumping Station	L.S.	1	330,200	330,200
Contingency (10 %)				1,060,310
Design & Supervision (20 %)				2,332,682
Land	ha	44	12,350	543,400
Total				**14,539,492**

4.5.2 Operation and Maintenance Costs

The estimated annual operation and maintenance costs for the horizontal flow wetland are \$ 82,500, broken down as follows:

• Energy: electricity for pumps and controls	\$ 5,000/yr
• Monitoring: Lab costs	\$ 10,000
• Labour (1 person hour per day - pumps)	\$ 7,500
• Equipment replacement and repairs	\$ 10,000
• Landscaping (weekly maintenance)	\$ 50,000
Total	**\$ 82,500/yr**

For a horizontal subsurface flow wetland to meet the nitrogen requirements, with TP removal in the lagoon, the total cell area becomes 334,220 m^2 and the cell dimensions become 236 m x 236 m (compared to 270 m x 270 m). The layout would be the same as for the previous case. The resulting total cost for this system is **\$ 11,033,296** with a liner and **\$ 7,345,532** without a liner. The annual operation and maintenance costs remain the same, **\$ 82,500/yr**.

4.6 Free Water Surface Wetland

4.6.1 Capital Costs

The estimated construction cost for the free water surface wetland shown in Figure C-6, Appendix C, with TP removal by the wetland, is $ 8,386,312, including land. Table 4.7 provides a detailed cost breakdown for the wetland system. The unit costs are derived on the same basis as the previous wetlands. Most of the components are the same or similar to the horizontal flow wetland with some differences noted as follows:

- Excavation volumes have been estimated on the basis of a balanced cut and fill on flat land and include additional depth required for a slope of 0.5 % and to allow for 150 mm of rooting media. Berm heights allow for the rooting media depth, a water depth of 0.5 m and 0.5 m of freeboard. The top width of the berms is 3 m with an exterior slope of 3:1 and interior slope of 2:1.

The estimated capital cost for a free water surface wetland with TP removal in the pre-treatment lagoon system is $ 2,065,329. Costing details are presented in Table 4.8

4.6.2 Operation and Maintenance Costs

The estimated annual operation and maintenance costs for the free water surface wetland with TP removal by the wetland are $ 82,500/yr, consisting of the following:

• Energy: electricity for pumps and controls	$ 5,000 /yr
• Monitoring: Lab costs	$ 10,000
• Labour (1 person hour per day - pumps)	$ 7,500
• Equipment replacement and repairs	$ 10,000
• Landscaping (weekly maintenance)	$ 50,000
Total	**$ 82,500 /yr**

The estimated annual operation and maintenance costs for the free water surface wetland with TP removal in the lagoon are $ 42,500/yr as follows:

• Energy: electricity for pumps and controls	$ 5,000 /yr
• Monitoring: Lab costs	$ 10,000
• Labour (1 person hour per day - pumps)	$ 7,500
• Equipment replacement and repairs	$ 10,000
• Landscaping (weekly maintenance)	$ 10,000
Total	**$ 42,500 /yr**

Table 4.7. Cost Estimate - Free Water Surface Wetland (TP Removal by the Wetland)

Items	Unit	Estimated Quantity	Unit Price	Amount
Mobilization/Demobilization	N.A.			0
Site Preparation	ha	48.4	5,000	242,000
Excavation/Berm Work	m^3	217,000	6	1,302,000
Cell Liner	m^2	435,000	8	3,480,000
Site Restoration	m^2	60,000	5	300,000
Site Piping	m	1,060	125	132,500
Chamber	Each	2	2,800	5,600
Weir Box	Each	8	1,000	8,000
Vegetation	ha	44	3,500	154,000
Miscellaneous Items	L.S.	1	50,000	50,000
Pumping Station	Each	1	230,000	230,000
Contingency (10 %)				590,410
Design & Supervision (20 %)				1,298,902
Land	ha	48.4	12,250	592,900
Total				**8,386,312**

Table 4.8. Cost Estimate - Free Water Surface Wetland (TP Removal in the Lagoon)

Items	Unit	Estimated Quantity	Unit Price	Amount
Mobilization/Demobilization	N.A.			0
Site Preparation	ha	10.5	5,000	52,500
Excavation/Berm Work	m^3	43,500	6	261,000
Cell Liner	m^2	86,000	8	688,000
Site Restoration	m^2	26,000	5	130,000
Site Piping	m	250	125	31,250
Chamber	Each	2	2,500	5,000
Weir Box	Each	4	1,000	4,000
Vegetation	ha	8.7	3,500	30,450
Miscellaneous Items	L.S.	1	50,000	35,000
Pumping Station	Each	1	230,000	230,000
Contingency (10 %)				146,720
Design & Supervision (20 %)				322,784
Land	ha	10.5	12,250	128,625
Total				**2,065,329**

4.7 Intermittent Sand Filter

4.7.1 Capital Costs

The cost of the intermittent sand filter in Figure C-8, Appendix C is estimated to be $ 1,540,625, including land. Table 4.9 provides a detailed cost breakdown for the system. The items identified in Table 4.9 are estimated using construction industry figures and information in the report, " Alternative approaches for Upgrading Effluent Quality for Lagoon Based Systems". The following is a brief description of some of the key components of the estimate for the intermittent sand filter.

- Site preparation includes clearing, grubbing and stripping of topsoil.

- Excavation volumes have been estimated on the basis of a balanced cut and fill on flat land. Berm heights allow for 0.3 m of freeboard. Berm slopes are 3:1 outside and 2:1 inside.

- It is assumed that a liner would be required due to the assumption that existing soils consist of poor clay lands (assumption used for land costing purposes). This assumption has also been used for the lagoons. It may be argued that natural attenuation would eliminate off site migration of pollutants entering the groundwater, thereby eliminating the need for a liner. However, as a conservative assumption, a liner is included in the costs.

- Site restoration includes placing of topsoil and seeding of disturbed areas (berms, etc.)

- Distribution piping includes pressure pipe with perforations spaced by 20 m to provide flood irrigation over the surface of the sand filters. Costs include appurtenances such as collars, thrust blocks, etc.

- Collection piping includes drainage pipes spaced at 10 m. The cost includes all appurtenances.

- Pumping station includes Flygt CS 170 pumps capable of providing 200L/s at 7 m of head (25 HP motor). Station includes 250 mm diameter outlet, valves and housing structure.

- Miscellaneous items include minor piping and valves, field office, trial operation and training, etc.

Table 4.9. Cost Estimate - Intermittent Sand Filter

Items	Unit	Estimated Quantity	Unit Price	Amount
Mobilization/Demobilization	N.A.			0
Site Preparation	ha	2.5	5,000	12,500
Excavation/Berm Work	m^3	8,800	6	52,800
Cell Liner	m^2	13,000	8	104,000
Filter Material (placed)	m^3	24,000	15	360,000
Site Restoration	m^2	35,000	5	175,000
Distribution Piping	m	500	100	50,000
Collection Piping	m	1,700	15	25,500
Outlet Sewer (300 mm dia)	m	50	175	8,750
Miscellaneous Items	L.S.	1	25,000	25,000
Pumping Station (to dose filters)	L.S.	1	330,200	330,200
Contingency (10 %)				114,375
Design & Supervision (20 %)				251,625
Land	ha	2.5	12,350	30,875
Total				**1,540,625**

4.7.2 Operation and Maintenance Costs

The estimated annual operation and maintenance costs for the intermittent sand filter are $ 30,000/yr and consist of the following:

• Labour: Estimated 4 person hours /day for 8 months	$ 20,000/yr
• Energy: electricity for pumps	$ 8,000
• Maintenance	$ 2,000
Total	**$ 30,000/yr**

4.8 Nitrifying Activated Sludge Plant

4.8.1 Capital Cost

The cost of the nitrifying activated sludge (extended aeration) plant in Figure C-7 is estimated to be \$ 12,921,000, including land. A detailed breakdown of the cost of this plant is not provided since the estimate is based on total construction costs of a similar sized plant in eastern Ontario. The cost was adjusted using the American Association of Cost Engineers (AACE) formula of:

$$\$ B / \$ A = (C_B/C_A)^n$$

where C_B = capacity of plant B (5,710 m3/day)
C_A = capacity of plant A (4,045 m3/day)
n = adjustment factor (0.6)
\$A = \$ 7,927,545

The cost, therefore, for a 5,710 m^3/day plant is \$ 9,759,880. Adding a contingency allowance of 10 % and 20 % fees for engineering and supervision, the total cost becomes \$ 12,871,162. The total estimated land area for the treatment plant is approximately 4 ha. Using a cost of \$ 12,350/ha the total capital cost for the plant then, becomes **\$ 12,920,562.**

4.8.2 Operation and Maintenance Costs

The estimated annual operation and maintenance costs for this plant are also based on the cost of operating and maintaining the same plant used in the capital cost estimate. Using the AACE formula identified above, the estimated annual operation and maintenance costs for the extended aeration plant are **\$ 799, 400** per year.

5.0 Estimated Costs of Design Alternatives

5.1 Process and Treatment System Costs - Capital and O & M

Design information and costs have been compiled on the following treatment processes and systems:

(1) Aerated/facultative lagoon (A/F lagoon)
(2) Facultative lagoon (4 months storage)
(3) Vertical subsurface flow (VSF) constructed wetland (CW) with TP removal capability
(4) Vertical subsurface flow CW with TP removal in the pre-treatment lagoon
(5) Horizontal subsurface flow (HSF) CW with TP removal capability
(6) Horizontal subsurface flow CW with TP removal in the pre-treatment lagoon
(7) Free water surface (FWS) CW with TP removal capability
(8) Free water surface CW with TP removal in the pre-treatment lagoon
(9) Intermittent sand filter (ISF) - ("New Hamburg process")
(10) Nitrifying activated sludge plant (extended aeration)

The capital costs, with and without a liner, and operating and maintenance costs associated with these processes are summarized in Table 5.1.

Based on the processes listed above, a number of complete alternative treatment systems have been identified. These are the following:

(1) A/F lagoon and VSF CW (TP removal in the wetland)
(2) A/F lagoon and VSF CW (TP removal in the lagoon)
(3) A/F lagoon and HSF CW (TP removal in the wetland)
(4) A/F lagoon and HSF CW (TP removal in the lagoon)
(5) A/F lagoon and FWS CW (TP removal in the wetland)
(6) A/F lagoon and FWS CW (TP removal in the lagoon)
(7) Facultative lagoon and intermittent sand filter
(8) Nitrifying activated sludge (extended aeration) plant

The capital costs, with and without a liner, and the operation and maintenance costs for these alternative treatment systems are presented in Table 5.2.

Table 5.1. Summary of Process Capital and O & M Costs (Dollars)

Process	Capital Cost (With Liner)	Capital Cost (Without Liner)	Annual Operation and Maintenance Costs
A/F Lagoon	2,475,870	1,525,470	105,000
A/F Lagoon with TP removal	2,634,270	1,683,870	125,000
Facultative Lagoon with TP Removal	6,874,550	2,716,550	80,000
VSF CW (TP removal in wetland)	6,121,250	5,382,050	74,500
VSF CW (TP removal in lagoon)	2,348,803	1,979,203	38,500
HSF CW (TP removal in wetland)	14,539,492	9,787,492	82,500
HSF CW (TP removal in lagoon)	11,033,296	7,345,532	82,500
FWS CW (TP removal in wetland)	8,386,312	3,792,712	82,500
FWS CW (TP removal in lagoon)	2,065,329	1,157,169	42,500
Intermittent Sand Filter (ISF)	1,540,625	1,403,345	30,000
Nitrifying Activated Sludge Plant	(Not applicable)	12,921,000	799,400

Table 5.2. Alternative System Costs, Capital and O & M (Dollars)

System	Capital Cost (With Liner)	Capital Cost (Without Liner)	Annual O & M Costs
A/F Lagoon + VSF CW (TP removal in wetland)	8,597,120	6,907,520	179,500
A/F Lagoon + VSF CW (TP removal in lagoon)	4,983,073	3,663,073	163,500
A/F Lagoon + HSF CW (TP removal in wetland)	17,015,362	11,312,962	187,500
A/F Lagoon + HSF CW (TP removal in lagoon)	13,667,566	9,029,402	207,500
A/F Lagoon + FWS CW (TP removal in wetland)	10,862,182	5,318,182	187,500
A/F Lagoon + FWS CW (TP removal in lagoon)	4,699,599	2,841,039	167,500
Fac. Lagoon + ISF (TP removal in lagoon)	8,415,175	4,119,895	110,000
Act. Sludge Plant	(Not applicable)	12,921,000	799,400

5.2 Treatment System Annualized Costs

To compare the cost of the alternatives presented in Table 5.2, a blended capital and operation and maintenance cost has been computed for each. These costs are presented in Table 5.3 and provide the total annual cost of each alternative. The capital costs are annualized by assuming a 20 year debenture at an annual interest rate of 10 %. The annual operation and maintenance costs are assumed constant over that period. The sum of these two costs is the total annualized cost for the system.

Table 5.3. Summary of Treatment System Costs on an Annual Basis (Dollars)

System	Annualized Cost (With Liner)	Annualized Cost (Without Liner)
A/F Lagoon + VSF CW (TP removal in wetland)	1,185,365	987,680
A/F Lagoon + VSF CW (TP removal in lagoon)	746,520	592,080
A/F Lagoon + HSF CW (TP removal in wetland)	2,178,300	1,511,115
A/F Lagoon + HSF CW (TP removal in lagoon)	1,806,605	1,263,940
A/F Lagoon + FWS CW (TP removal in wetland)	1,458,375	809,725
A/F Lagoon + FWS CW (TP removal in lagoon)	717,355	499,900
Facultative Lagoon + ISF (TP removal in lagoon)	1,094,575	592,030
Activated Sludge Plant	(not applicable)	2,311,155

5.3 Discussion of Cost Estimates

5.3.1 Capital Costs

Systems with a liner:

From Table 5.2, it can be seen that the least capital cost option is the treatment system consisting of an A/F lagoon with TP removal, combined with a FWS CW. For a similar lagoon system with a VSF CW the capital cost is approximately $ 285,000 more.

The capital cost of a facultative lagoon with TP removal combined with an ISF is significantly more than the above two systems but approximately $ 182,000 less than the cost of an A/F lagoon combined with a VSF CW (TP removal by the wetland).

The highest capital cost is for the combination of an A/F lagoon with a HSF CW with TP removal by the wetland.

Systems without a liner:

The least cost option is, again, an A/F lagoon with TP removal, combined with a FWS CW. In this case, the capital cost of a similar lagoon option with the VSF CW is significantly greater (about 30 %).

The most expensive option is the nitrifying activated sludge plant.

5.3.2 Operation and Maintenance Costs

The lowest costs are associated with the facultative lagoon (with TP removal) combined with the ISF. The wetland system combinations are approximately 150 % to 200 % of these costs. The greatest O & M cost is associated with the nitrifying activated sludge plant.

5.3.3 Annualized Costs

An examination of Table 5.3 reveals that, again, the least cost option for a system, with or without a liner, is the combination of an A/F lagoon with TP removal, and a FWS CW. With a liner, the combination of an A/F lagoon with TP removal, and a VSF CW is approximately $ 30,000 more per year but significantly less than the ISF system.

Without a liner, the annualized costs of an A/F lagoon with TP removal, combined with a VSF CW are essentially the same as the ISF system, but about $ 90,000 more than the comparable FWS system.

The annualized costs of the other systems are significantly greater, with the highest being the nitrifying activated sludge system.

In the opinion of S. Reed (JWSA, 1999b), if the effluent ammonia criteria on an annual basis had been set at 2-3 mg/L, then a much larger FWS CW would have been required and it is likely that the VSF CW would have been the optimum choice. This is based on the fact that FWS wetlands are not very effective, summer or winter, for ammonia removal, unless they are very large.

6.0 Summary and Conclusions

Based on the ***NOTL effluent requirements*** (Table 2.1) for which alternative conceptual designs have been developed, and for which cost estimates have been prepared, the least cost system on an annualized basis is the combination of an aerated/facultative lagoon, with TP removal, and a free water surface wetland, either with or without a liner.

With a liner, the second lowest annualized cost system is the same lagoon system but with a vertical subsurface flow wetland. Without a liner, the second lowest annualized cost system is a facultative lagoon combined with an intermittent sand filter. However, the annualized costs of an A/F lagoon with TP removal, combined with a vertical subsurface flow wetland are essentially the same.

The rank order of the treatment systems based on annualized costs is presented in Table 6.1.

Table 6.1. Treatment System Ranking by Annualized Costs

System	Rank (With Liner)	Rank (Without Liner)
A/F Lagoon + VSF CW (TP removal in wetland)	4	5
A/F Lagoon + VSF CW (TP removal in lagoon)	2	2
A/F Lagoon + HSF CW (TP removal in wetland)	7	7
A/F Lagoon + HSF CW (TP removal in lagoon)	6	6
A/F Lagoon + FWS CW (TP removal in wetland)	5	4
A/F Lagoon + FWS CW (TP removal in lagoon)	1	1
Facultative Lagoon + ISF (TP removal in lagoon)	3	2
Activated Sludge Plant	N.A.	8

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APPENDIX A

NOTL STP INFLUENT DATA

NOTL STP Influent Data

Month	Flow ML/d	BOD (mg/L)	TSS (mg/L)	TKN - N (mg/L)	TP (mg/L)
1995					
Jan	6.067	56	89	14.49	2.15
Feb	3.894	102	115	17.57	3.02
Mar	4.435	101	107	13.63	2.76
Apr	3.373	125	244	24.00	4.69
May	3.317	154	163	29.60	4.89
Jun	3.072	167	129	32.80	5.00
Jul	3.21	181	140	36.50	5.80
Aug	3.294	183	136	34.80	6.00
Sep	3.027	172	140	29.20	5.30
Oct	3.884	181	155	25.90	4.30
Nov	4.783	120	101	20.27	3.10
Dec	3.902	146	129	21.37	3.80
Ann. Avg.	**3.855**	**141**	**137**	**25.01**	**4.23**
1994					
Jan	2.734	181	113	30.30	6.50
Feb	3.665	122	126	19.70	3.60
Mar	5.166	86	65	15.00	2.40
Apr	6.106	90	89	13.20	2.20
May	3.012	85	93	21.27	3.20
Jun	2.900	67	62	22.80	3.60
Jul	3.621	136	109	27.50	5.50
Aug	3.554	131	109	33.90	5.00
Sep	3.684	141	95	21.75	4.90
Oct	3.878	273	376	36.00	10.00
Nov	4.443	105	117	23.42	3.10
Dec	5.422	66	100	16.30	3.00
Ann. Avg.	**4.015**	**124**	**121**	**23.43**	**4.42**

Month	Flow ML/d	BOD (mg/L)	TSS (mg/L)	TKN - N (mg/L)	TP (mg/L)
1993					
Jan	6.41	112	80	19.45	2.6
Feb	3.31	90	84	23.08	3.8
Mar	5.73	57	52	17.08	2.4
Apr	5.99	62	61	18.07	2.3
May	3.65	99	132	28.30	3.0
Jun	3.96	124	157		3.6
Jul	3.25	118	111	40.40	4.3
Aug	2.87	184	136	41.10	4.1
Sep	2.82	163	166	34.60	3.8
Oct	2.80	125	104	35.70	5.3
Nov	2.86	87	69	22.10	4.0
Dec	2.73	78	78	11.40	4.5
Ann. Avg.	**3.865**	**108**	**103**	**26.48**	**3.6**
1992					
Jan	2.81	100	122	35.73	4.0
Feb	2.98	78	76	26.40	3.9
Mar	3.08	120	121	31.65	4.4
Apr	5.67	73	79	22.38	4.6
May	4.28	77	80	29.23	4.2
Jun	3.15	145	152	34.58	4.8
Jul	4.75	78	81	41.46	4.9
Aug	4.40	131	123	35.63	4.5
Sep	3.31	126	115	34.86	5.0
Oct	3.46	131	103	31.23	4.3
Nov	5.17	71	80	16.15	2.4
Dec	5.43	92	95	18.95	2.6
Ann. Avg.	**4.04**	**102**	**102**	**29.85**	**4.1**

Month	Flow ML/d	BOD (mg/L)	TSS (mg/L)	TKN - N (mg/L)	TP (mg/L)
1991					
Jan	5.15	58	58	41.70	3.2
Feb	3.81	56	107		2.8
Mar	6.74	48	77	25.60	2.6
Apr	6.52	54	90	14.40	3.0
May	3.89	82	104	21.90	3.5
Jun	3.03	108	142	35.00	4.9
Jul	2.96	137	155	34.20	5.6
Aug	2.85	140	159	34.60	5.6
Sep	2.90	193	133	37.50	5.3
Oct	2.57	238	131	43.35	6.3
Nov	2.35	101	82	33.75	4.8
Dec	2.91	119	108	31.30	4.3
Ann. Avg.	**3.81**	**111**	**112**	**32.12**	**4.3**

APPENDIX B

CONSTRUCTED WETLANDS - CONCEPTUAL DESIGNS

CONCEPTUAL DESIGNS

1.0 Conceptual Design - Vertical Flow Subsurface Wetland

The conceptual design for this vertical flow subsurface constructed wetland is based primarily on the results of Experiment 2 of the SWAMP project (SWAMP 2). SWAMP 2 consisted of three cells in series, each 5 m x 5 m x 1.2 m deep, buried in the ground. The lagoon effluent was pumped into the first cell through a perforated pipe 10.2 cm in diameter with the bottom of the pipe located at the 30 cm depth. Fluid collected from the bottom of the first cell was pumped to the top of the second cell and similarly for the third cell. The first two cells were filled with Queenston shale and the third with Lockport Dolomite. All cells were planted with cattails. Further details are provided in the SWAMP Interim Report (Lemon et al., 1995) and the feasibility study report (JWSA, 1997).

In the preliminary scale-up feasibility study (JWSA, 1997), it was concluded that the only way to scale-up the experimental wetlands would be on the basis of geometry, media and hydraulic loading rate. The limiting parameter for scale-up was identified as TP in order to meet the NOTL STP requirements for all parameters. On this basis conceptual design scenario no. 1 was generated as set out in Table 1.

Table 1. Conceptual Design Scenario No. 1

Design Scenario	No. 1 (BOD_5, TSS, NH_3-N & TP)
Hydraulic loading rate	60 L/m^2/day
Flow	5,710 m^3/day
Total surface area	95,167 m^2 or 9.5 ha.
Surface area - each cell (3 cells)	31,722 m^2
Cell dimensions	178 m x 178 m
Cell depth	1.2 m
Volume of media required per cell	38,066 m^3
Assumed hydraulic residence time	6 - 7 days[1]

1. Tracer studies have indicated an actual residence time closer to 2 days

The conclusions with respect to TP removal were as follows:

(1) The SWAMP 2 system has been demonstrated to effectively remove TP from secondary lagoon effluent at a hydraulic loading rate of 60 L/m^2/day and meet the NOTL STP effluent requirement.

(2) It may be possible to effectively treat secondary lagoon effluent to remove TP and meet the

NOTL STP effluent requirement at a hydraulic loading rate of 120 L/m^2/day, *but* more definitive data are required.

(3) It may also be possible to treat "primary" lagoon effluent (likely from aerated lagoon no. 2) to remove TP, at a hydraulic loading rate of 60 L/m^2/day, but again more definitive data are required.

(4) There is still some uncertainty with respect to the long term sustainability of TP removal because of the uncertainty associated with the TP adsorption capacity of the media at this time and the incomplete understanding of the phosphorus removal mechanisms for this type of wetland.

(5) Further, it was concluded that at a hydraulic loading rate of 180 L/m^2/day, only one cell would be sufficient to reduce BOD, TSS and NH_3-N to concentrations that would meet the effluent requirements for these parameters for the NOTL STP.

Subsequent to this preliminary feasibility study additional information became available concerning conditions within the cells that would impact on their capability to remove contaminants at greater depth. These data related to redox potential measurements and oxygen movement. It had been noted that while there was generally a significant concentration of dissolved oxygen (> 2 mg/L) in the effluents from the three cells but there were no acceptable measurements within the cells themselves. However, measurements of the redox potential were examined and these suggested that the redox potential remained high (+ 200 mv) at depth, thus suggesting that the cells were well oxygenated and, therefore, could safely be made deeper. It was noted that the "Phytofilt" vertical flow system in Europe were constructed to a depth of 1.8 m and it was concluded that this should be feasible for a full scale wetland, although it was considered desirable, based upon European experience, to add a supplementary ventilation/aeration system at depth.

It was also believed that at full-scale, three large cells in series would not be appropriate for maintenance reasons. A better configuration would be three sets of three smaller cells in series based on the SWAMP 2 experience. It was also felt that it would be desirable to feed the cells in any order. Since pumping is required between the cells, when in series, for flow distribution purposes, changing the order in which the cells were fed was not expected to be too difficult. The original (preliminary) and final conceptual designs are set out in Table 2.

As indicated above, the second case considered that TP will not be removed in the wetland but will be removed in the lagoon with the addition of alum. The wetland will have to remove only BOD, TSS and NH_3-N. It should be recognized, however, that the wetland will remove some phosphorus, probably 30 to 50 % of the applied load. The alum dosage, therefore could likely be less as the lagoon effluent would likely only have to be reduced to 1 mg/L TP. The original design premise and the final conceptual design are set out in Table 3.

In this second case, it was felt that because the hydraulic loading rate was so high, 180 L/m^2/day, that the depth of the bed should not be increased. Therefore, no increase in depth beyond 1.2 m is proposed because of the already high hydraulic loading rate. (If the depth was to be increased, and the influent flow rate kept constant, the surface area would decrease and the hydraulic load increase correspondingly).

In addition, for maintenance reasons, more than one cell was considered essential.

Table 2. Case 1. Removal of BOD, TSS, NH_3-N and TP

Parameter	Original Premise	Final Conceptual Design
Flow (m^3/day)	5,710	5,710
HLR ($L/m^2/d$)	60	90
Cell Depth (m)	1.2	1.8
Total Surface Area (m^2)	95,167	63,444
Volume of media (m^3)	114,200	114,200
Number of Cells	3	9 (3 sets of 3 in series)
Area - each cell (m^2)	31,722	7,050
Cell Dimensions (m x m)	178 x 178	84 x 84

Table 3. Case 2. Removal of BOD, TSS, and NH_3-N (TP removal in the lagoon)

Parameter	Original Premise	Final Conceptual Design
Flow (m^3/day)	5,710	5,710
HLR ($L/m^2/d$)	180	180
Cell Depth (m)	1.2	1.2
Total Surface Area (m^2)	31,722	31,722
Volume of media (m^3)	38,066	38,066
Number of Cells	1	3
Area - each cell (m^2)	31,722	10,574
Cell Dimensions (m x m)	178 x 178	103 x 103

The cell media would be Queenston shale. The feeding arrangement should be flexible to permit shutting down of one cell so that maintenance work etc. can be done. Because these cells are only 1.2 m deep there is no need for an additional ventilation/aeration system.

2.0 Conceptual Design - Horizontal Flow Subsurface Wetland

The design is based on the approach by Reed et al, 1996 for this type of wetland and the equations used are from their book "Natural Systems for Waste Management and Treatment, Second Edition".

The influent quality to the wetland is assumed to have the following characteristics, based in part on the effluent characteristics of the NOTL lagoon system:

BOD = 25 mg/L
TSS = 40 mg/L and possibly as high as 90 mg/L during the algal season
TKN = 12 mg/L
NH_3-N = 8 mg/L
TP = 4 mg/L

Assume the critical water temperature is 4 ° C.

Case 1. Size wetland for effluent BOD = 5 mg/L

Assume the depth of gravel in the wetland = 0.6 m, porosity = 0.38

Design equation:

Area = $Q[\ln C_o - \ln C_e]/[K_T(y)(n)]$ [Eqn 6.36, p224]

where

A = surface area of wetland, m^2
C_o = influent BOD, mg/L
C_e = effluent BOD, mg/L
K_T = temperature-dependent, first order rate constant, 1/d
y = average water depth in the system, m
n = porosity of system, decimal fraction
Q = average flow in the system, m^3/d

At 4 °C $K_T = 1.104(1.06)^{-16} = 0.4346$

Area = $(5710)[\ln(25/5)]/[0.4346(0.6)(0.38)]$

= 92,744 m^2 = 9.3 ha

HRT = 3.7 days (Eqn HRT = $[\ln(Ce/Co)]/-K_T$)

Volume = 55,643 m^3

Materials: 55,643 m^3 of 2 cm gravel, 56,000 plants (approx) at 1 m spacing

Effluent ammonia: (Note that this is calculated based on lagoon effluent TKN)

Aerated lagoon as influent

Eqn: $Ce = Co[\, e^{-K.HRT}]$

$K = 0.1367(1.15)^{T-10}$

Co = TKN value

$C_e = 12(e^{-0.0392(3.7)})$

$= 10.4$ mg/L

Facultative lagoon as influent

$C_e = 12\ (e^{-0.0392(3.7)})$

$= 10.4$ mg/L

Effluent Phosphorus:

HLR = 5710/92,744 = 0.062 m/d = 6.2 cm/d

$C_e = 4(e^{-(2.73/6.2)})$

$= 2.6$ mg/L

Expected Wetland Effluent Quality :

BOD = 5 mg/L
TSS < 10 mg/L
TP = 2.6 mg/L
NO_3 = 0 mg/L (Approx. As the wetland is essentially anoxic, nitrification would not be expected to occur)
TKN same as NH_3
NH_3 -N = 10.4 mg/L from aerated lagoon effluent
= 10.4 mg/L from facultative lagoon effluent

The area calculated above is again the bottom area of the wetland and does not include berms. Berms should have about 0.6 m freeboard, 2:1 slope inside and 3:1 slope outside.

This wetland configuration does satisfy the nitrogen objective of 12 mg/L NH_3-N in winter but not the phosphorus objective of 0.5 mg/L TP.

Check to see if the summer objective of 5 mg/L NH_3-N will be met.

Assume an average water temperature of 15 °C

Recalculate a wetland size to produce an effluent ammonia of 5 mg/L.

For both the aerated lagoon effluent and facultative lagoon effluent as influent to the wetland use the same equation as for BOD but C_e = effluent NH_3 -N = 5 and C_o = influent TKN = 12 mg/L.

However, $K_T = K_{NH}(1.048)^{T-20}$

$$K_{NH} = 0.01854 + 0.3922(rz)^{2.6077}$$

where rz = % of SSF wetland bed depth occupied by the root zone = 0.5

$$\text{Area} = 5710[\ln(12/5)]/[0.0656(0.6)(0.38)]$$

$$= 334{,}220 \text{ m}^2 = 33.4 \text{ ha.}$$

This would be the determining area for NH_3-N removal.

It is not known whether the above would satisfy the requirements for TP removal. A conservative assumption for the NOTL STP influent TP is 5 mg/L. If we assume that at least 1 mg/L will be removed in the lagoons (without chemical added) then the influent TP to the wetland will be TP = 4 mg/L = C_o.

Design Eqn:

$$A = (b)(Q)[\ln(C_o/C_e)]/ K_p$$

where

A = wetland area, m^2
b = conversion coefficient = 100 cm/m
Q = average flow, m^3/day
C_e= effluent TP = "objective" value = 0.5 mg/L
K_p= rate constant = 2.73 cm/day

$$A = (100)(5710)[\ln(4/0.5)]/ 2.73$$

$$= 434{,}931 \text{ m}^2$$

The above results for a horizontal subsurface flow wetland are summarized in Table 4.

Table 4. Subsurface Horizontal Flow Wetland Sizing Area, m^2 (ha)

Parameter to be Removed	Influent from Aerated Lagoon	Influent from Facultative Lagoon
BOD	92,744 (9.3)	92,744 (9.2)
NH_3	334,220 (33.4)	334,220 (33.4)
TP	434,931 (43.5)	434,931 (43.5)

Based on Table 4, Table 5 below then sets out the design areas for a subsurface horizontal flow wetland for the two required cases:

Case 1 - the removal of BOD, TSS, NH_3 and TP, and
Case 2 - the removal of BOD, TSS and NH_3 (TP is to be removed in the lagoon).

Table 5. Subsurface Horizontal Flow Wetland Design Scenarios, Areal Requirements

Design Scenario	Influent from Aerated Lagoon	Influent from Facultative Lagoon
Case 1	434,931 m^2 (43.5 ha)	434,931 m^2 (43.5 ha)
Case 2	334,220 m^2 (33.4 ha)	334,220 m^2 (33.4 ha)

3.0 Conceptual Design - Free Water Surface Wetland

The design is based on the approach by Reed et al., 1996 for this type of wetland and the equations used are from their book, "Natural Systems for Waste Management and Treatment, Second Edition".

The influent quality to the wetland is assumed to have the same characteristics as for the horizontal flow wetland. In addition the following assumptions have been made:

• winter water temperature	5 °C
• summer water temperature	20 °C
• wetland water depth	0.5 m
• porosity	0.75 (porosity is the void space in the wetland, not occupied by plants, litter and detritus, and available for flow)

The basic design model used is:

$$A = (Q)(\ln[C_o/C_e])/ (K)(d)(n) \qquad \text{(Equation 1)}$$

Where

A = treatment area of the wetland (bottom area), m^2
Q = average flow through the wetland, m^3/d
= 5,710 m^3/d
C_o = wetland influent concentration, mg/L
C_e = wetland effluent concentration, mg/L
K = first order, plug flow rate constant, d-1
(temperature dependent for BOD, NH_3-N)
d = depth of water in wetland, m
= 0.5 m assumed for this case
n = porosity of the wetland, % as a decimal
= 0.75 assumed for this case

The temperature dependence for BOD and NH3-N is expressed by:

$$K_{20} = K_{20}\,(\theta)^{(T-20)}$$

Where:

KT = rate constant at temperature T, °C
K_{20} = rate constant at 20 °C
θ = temperature factor

For BOD $K_{20} = 0.678\ d^{-1}$, $\theta = 1.06$
NH_3-N $K_{20} = 0.2187\ d^{-1}$, $\theta = 1.048$

BOD calculations

Determine wetland area required to produce an effluent BOD = 5 mg/L. Using equation 1 with the appropriate temperature dependent rate constants the required areas are:

For winter performance Area = 86,597 m^2 or 8.66 ha
For summer performance Area = 36,146 m^2 or 3.61 ha

NH_3-N calculations

Determine wetland area required to produce an effluent quality of 12 mg/L in winter and 5 mg/L in summer.

It is prudent, because of the nitrogen transformations which can occur in a wetland, to assume that all of the entering TKN can appear as NH_3-N. The assumed C_o for these calculations is, therefore, 12 mg/L.

For winter performance	Area = 0 m^2 (since 12 mg/L is effluent limit)
For summer performance	Area = 60,954 m^2 or 6.1 ha

TSS Calculations

The required area for TSS removal can be estimated with:

$$A = Q/\left[(C_e/C_o - 0.1139)/0.213\right]$$

The removal of TSS is not temperature sensitive. Winter influent TSS = 40 mg/L assumed and summer influent TSS = 90 mg/L (because of algae blooms in the lagoons).

For winter performance	Area = 4,658 m^2 or 0.47 ha
For summer performance	Area = 23,049 m^2 or 2.3 ha

TP Calculations

The area required for TP removal can be estimated with:

$$A = [(Q)(\ln (C_o/C_e))]/0.0273$$

The model calculates an area based on the annual average effluent TP concentration. The actual effluent concentration may exceed the value on occasion. In this case Ce = 0.5 mg/L and Co = 4 mg/L.

Required Area = 434,931 m^2 or 43.4 ha

Design Scenarios

From the preceding information, two design scenarios can be developed based upon whether or not TP is removed in the wetland or in the lagoon. Note that for the wetland sizing in the first case the controlling parameter is TP , but for the second case it is BOD removal in the winter. Therefore, the area requirements are as follows:

Design Scenario 1, TP removal by the wetland	Area = 434,931 m2 or 43.4 ha
Design Scenario 2, TP removal in the lagoon	Area = 86,597 m2 or 8.66 ha

From the above, the following two designs, set out in Table 6, were developed.

Table 6. Free Water Surface Wetland Design Scenarios

Item	**Scenario 1 (TP removal in the lagoon)**	**Scenario 2 (TP removal by the wetland)**
Area, m^2 (ha)	86,597 (8.66)	434,931 (43.5)
No. of cells	6	12
Cell area, m^2	14,450	36,180
Cell width, m	85	134
Cell length, m	170	270

A free water surface wetland with a treatment area of 8.66 ha and a design flow of 5,710 m^3/d would be expected to produce an effluent with the characteristics noted in Table 7.

Table 7. Free Water Surface Wetland Performance Expectations

Parameter	**Winter Effluent (mg/L)**	**Summer Effluent (mg/L)**
BOD_5	5	5
TSS	5	12[1]
NH_3-N	6.5	3.5
TP	0.3[2]	0.3[2]

Note: 1. Assuming up to 90 mg/L input from the lagoon
2. An annual average value, based on 0.5 mg/L TP from the lagoon

References

JWSA, 1997. *Niagara-on-the-Lake Constructed Wetlands Scale-up Feasibility Study*. A report prepared for The Friends of Fort George SWAMP Project by James W. Schmidt Associates Inc., February.

Lemon, E.R., G. Bis, L. Rozema and I. Smith, 1995. The Friends of Fort George *Sewage Waste Amendment Marsh Process Project (S.W.A.M.P.), Interim Report*, Niagara-on-the-Lake, Ontario, September.

Reed, S.C., R.W. Crites and E.J. Middlebrooks, 1995. *Natural Systems for Waste Management and Treatment, Second Edition*, McGraw-Hill Inc., New York.

APPENDIX C
CONCEPTUAL DESIGN SKETCHES

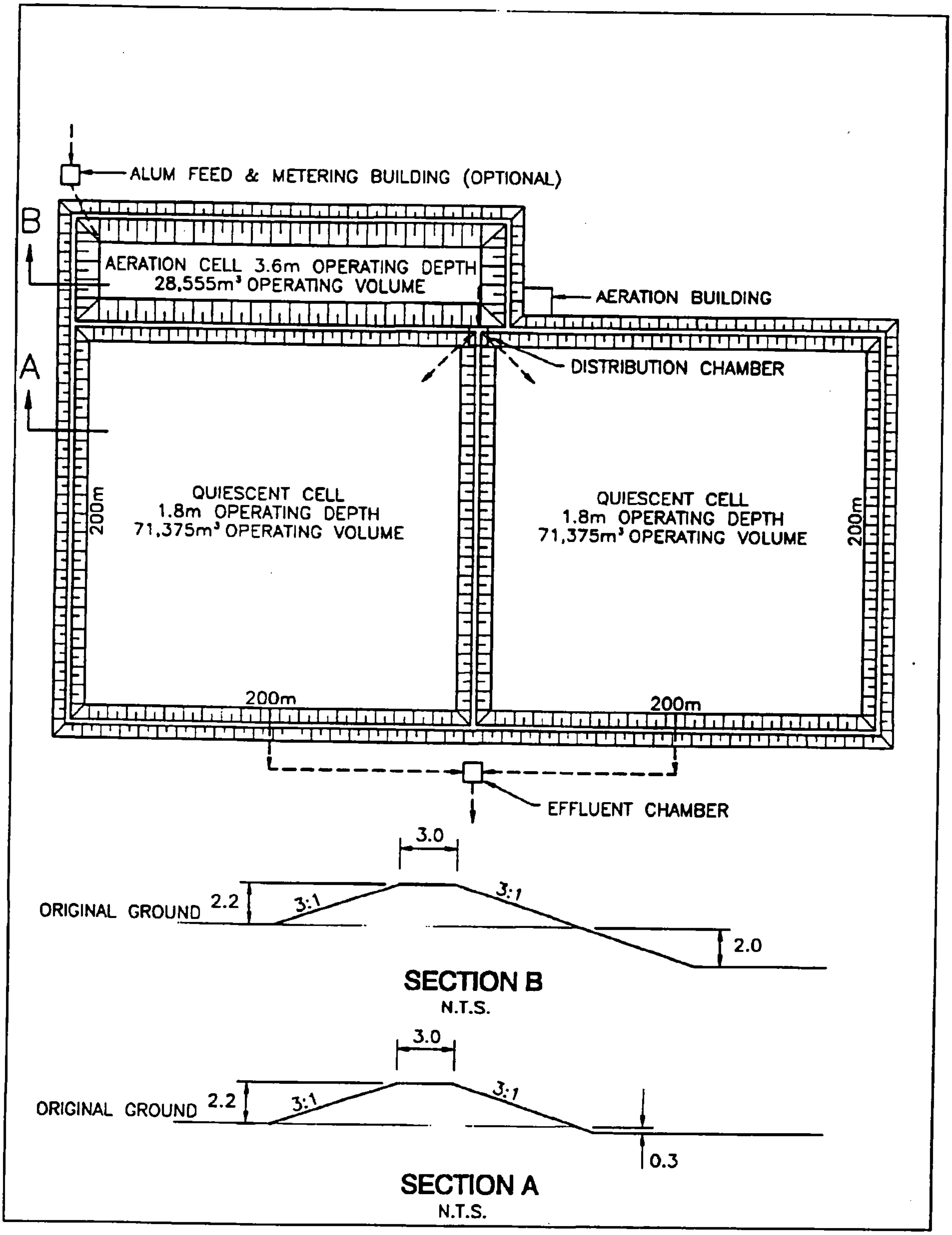

Figure C-1. Aerated Facultative Lagoon

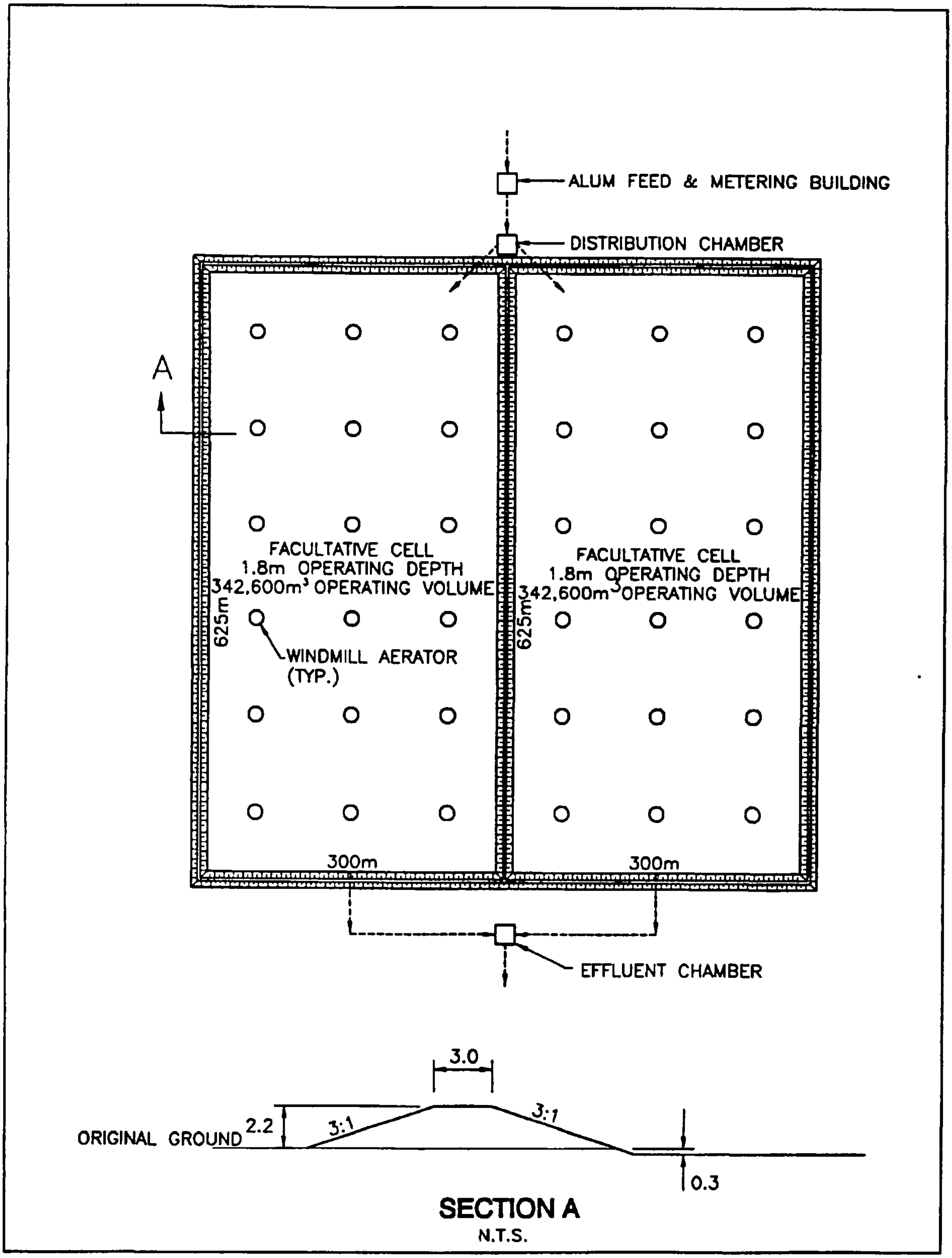

Figure C-2. Facultative Lagoon

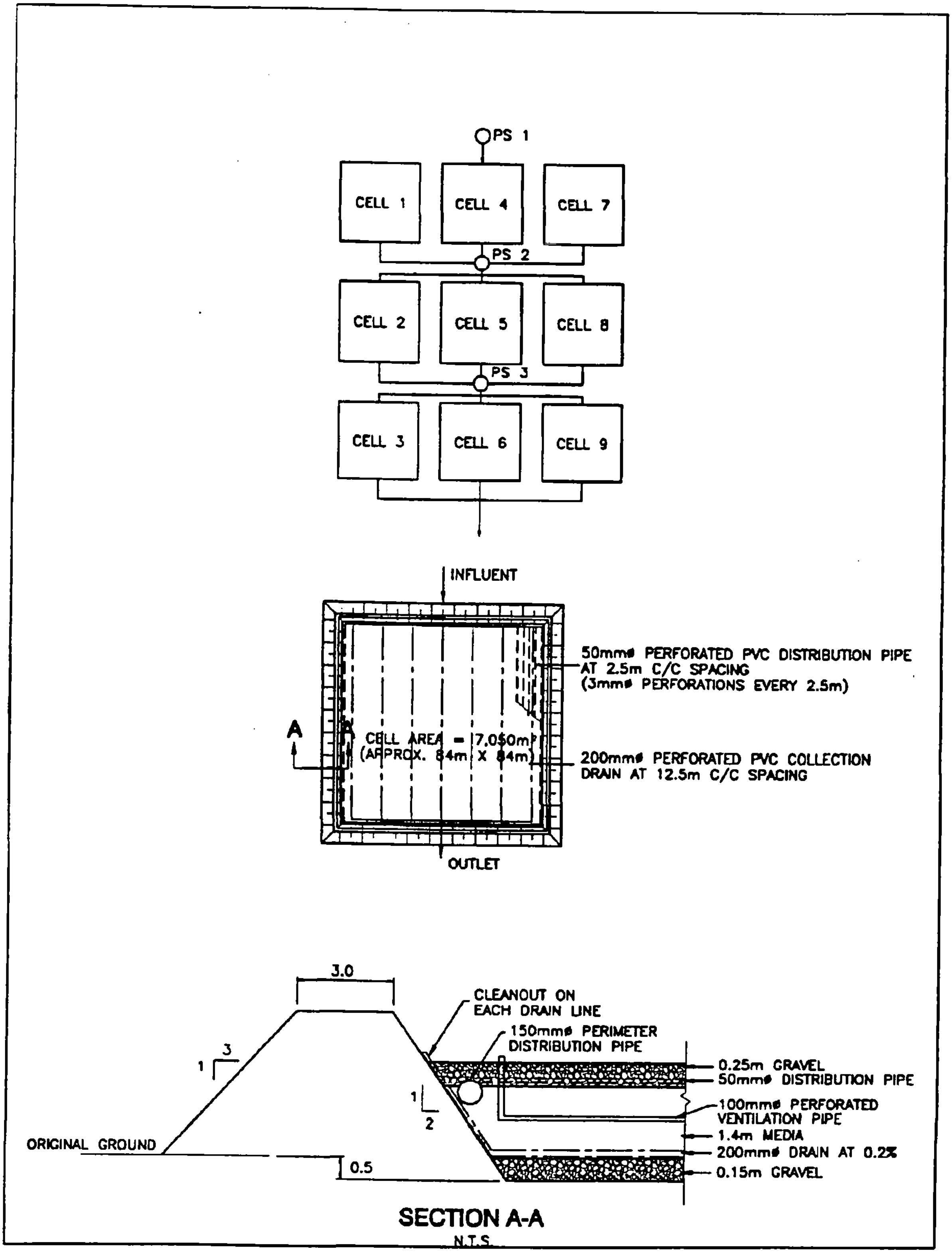

Figure C-3. Vertical Subsurface Flow Constructed Wetland (Final, Modified Design)

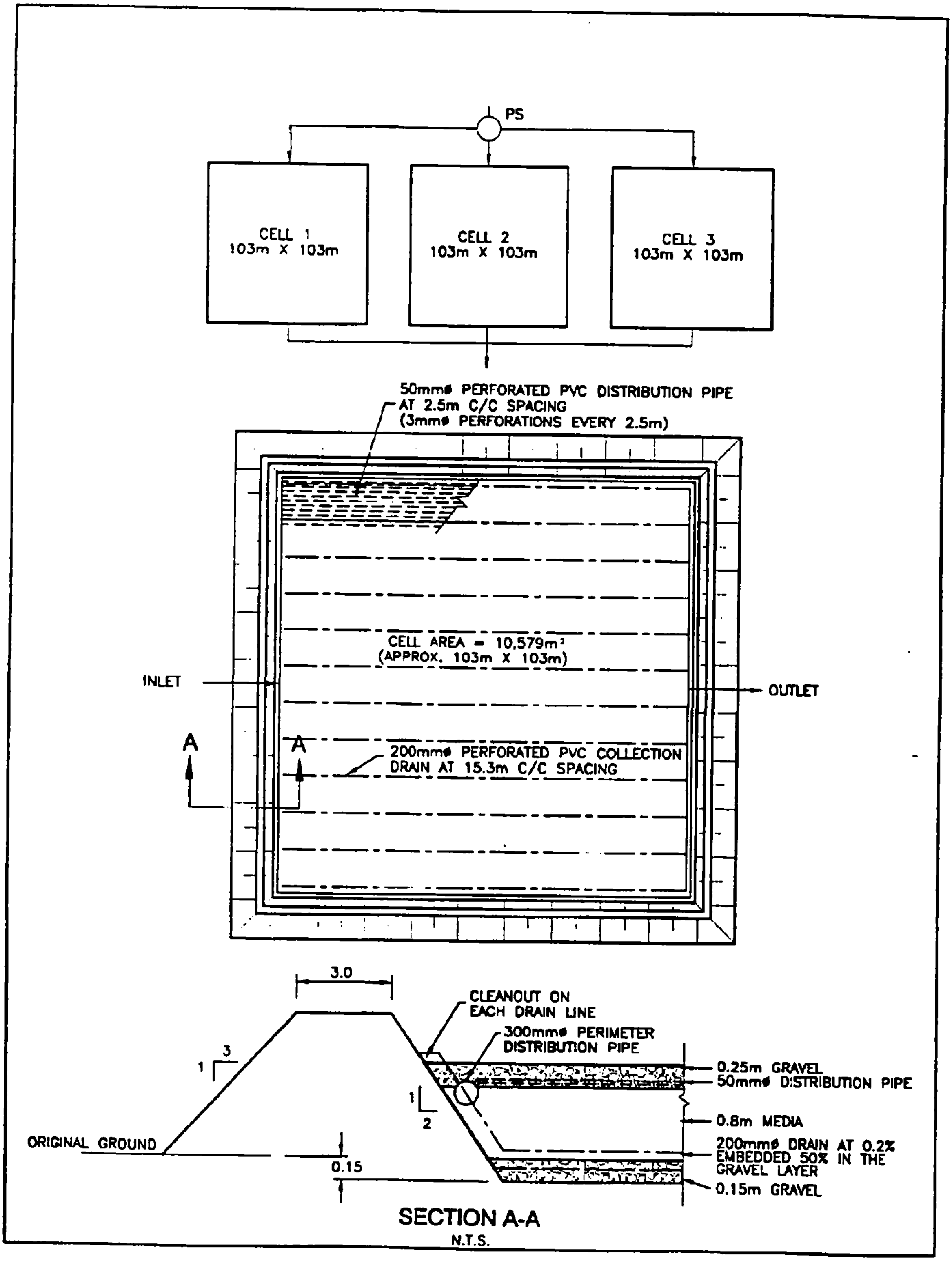

Figure C-4. Vertical Subsurface Flow Constructed Wetland (TP Removal in the Lagoon)

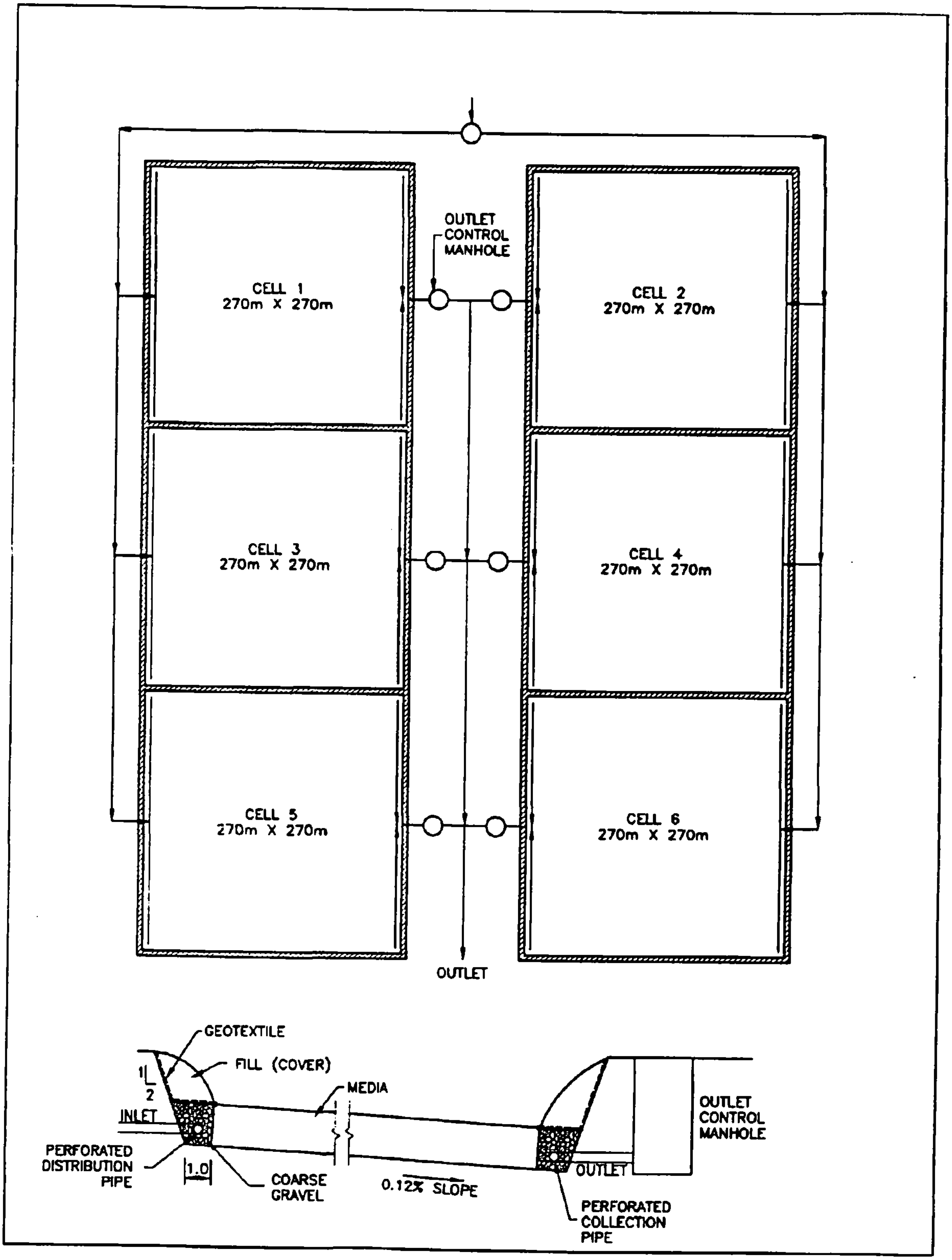

Figure C-5. Horizontal Subsurface Flow Constructed Wetland

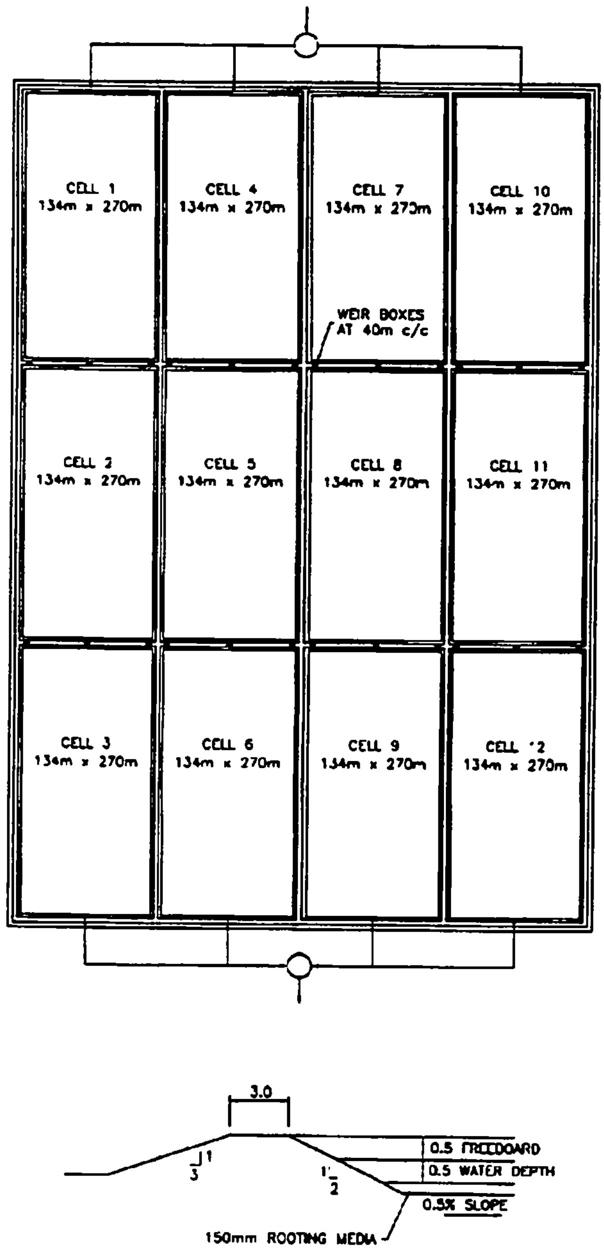

Figure C-6. Free Water Surface Wetland
(TP Removal by the Wetland)

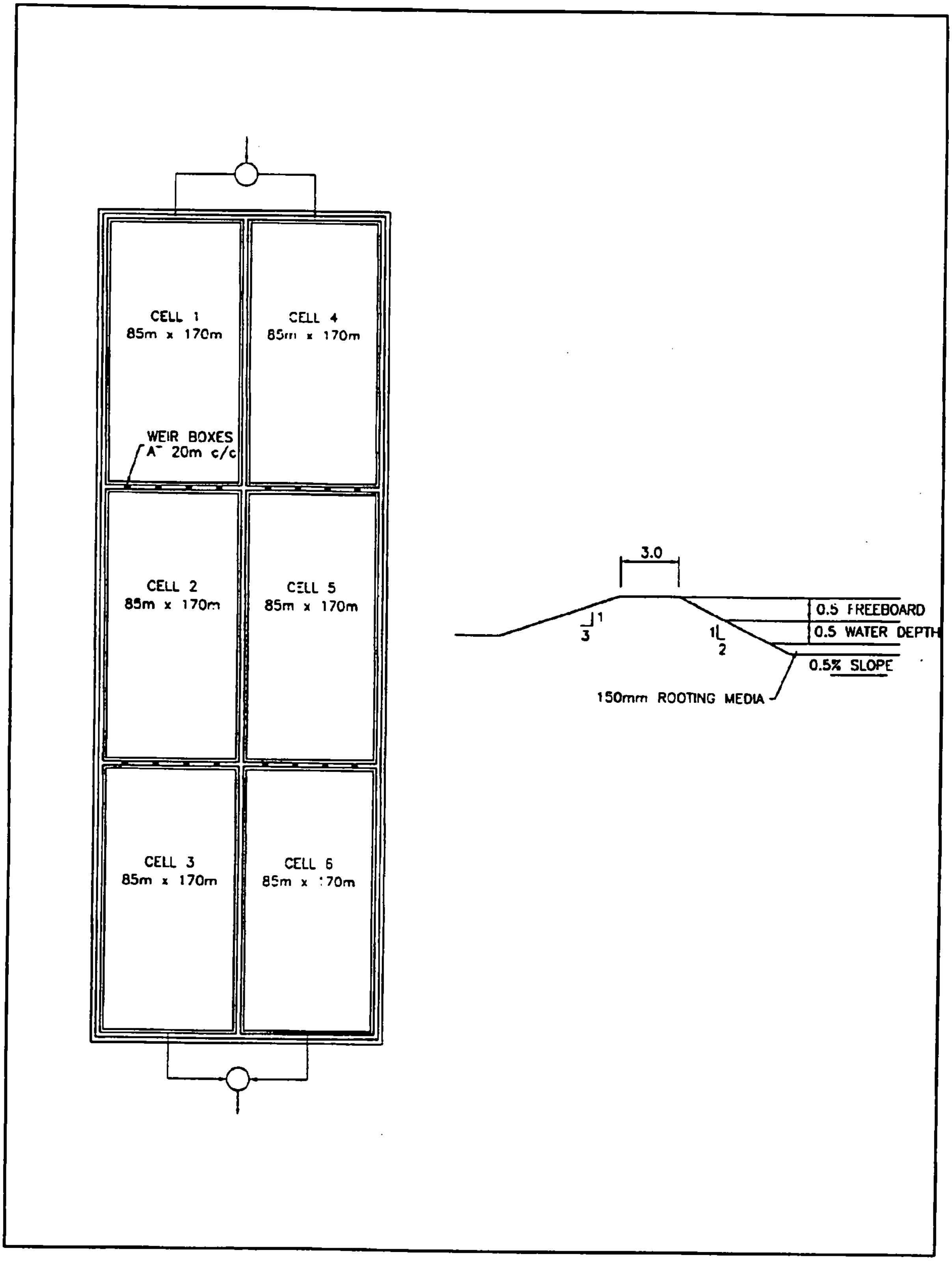

Figure C-7. Free Water Surface Wetland (TP Removal in the Lagoon)

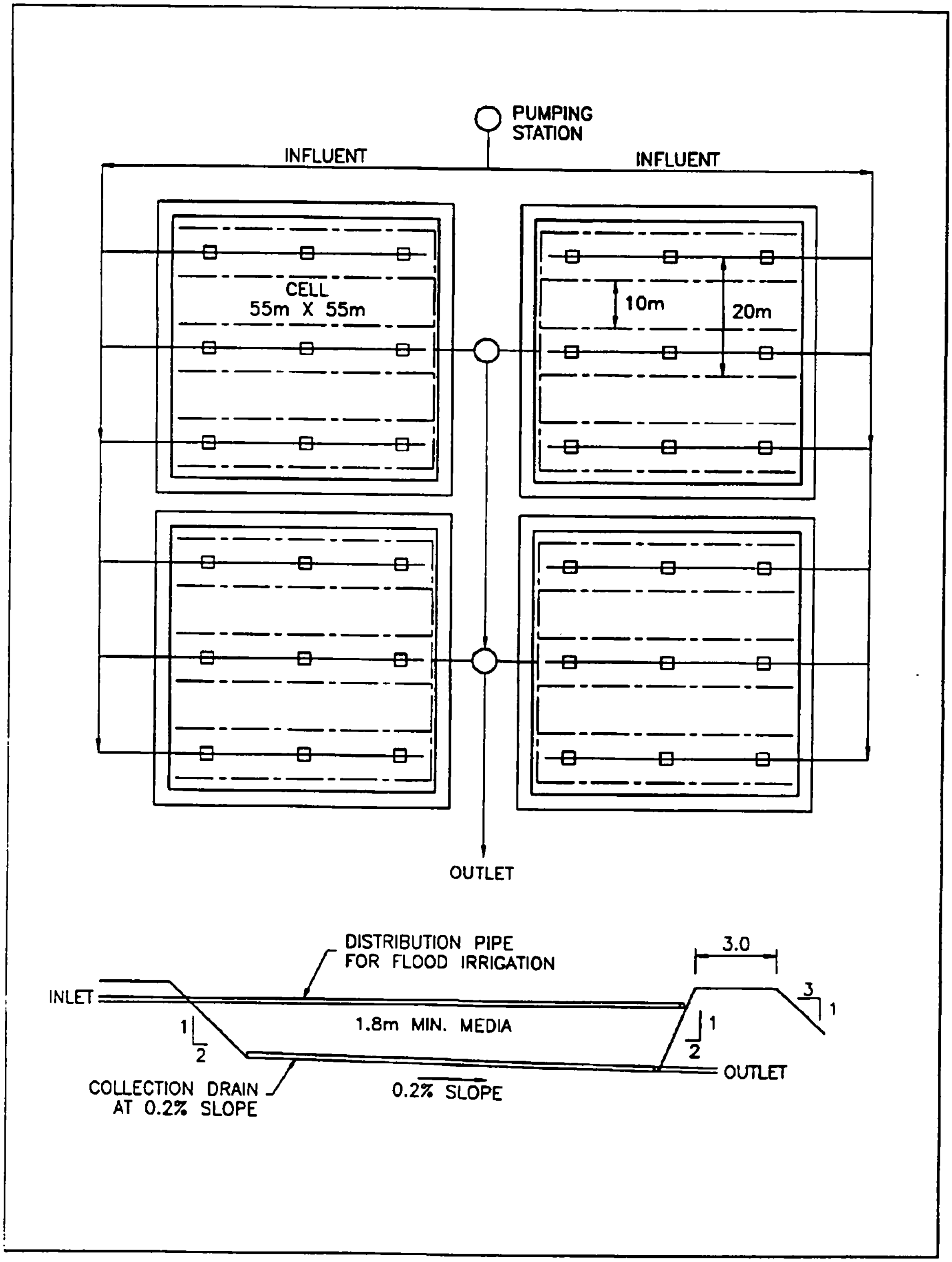

Figure C-8. Intermittent Sand Filter

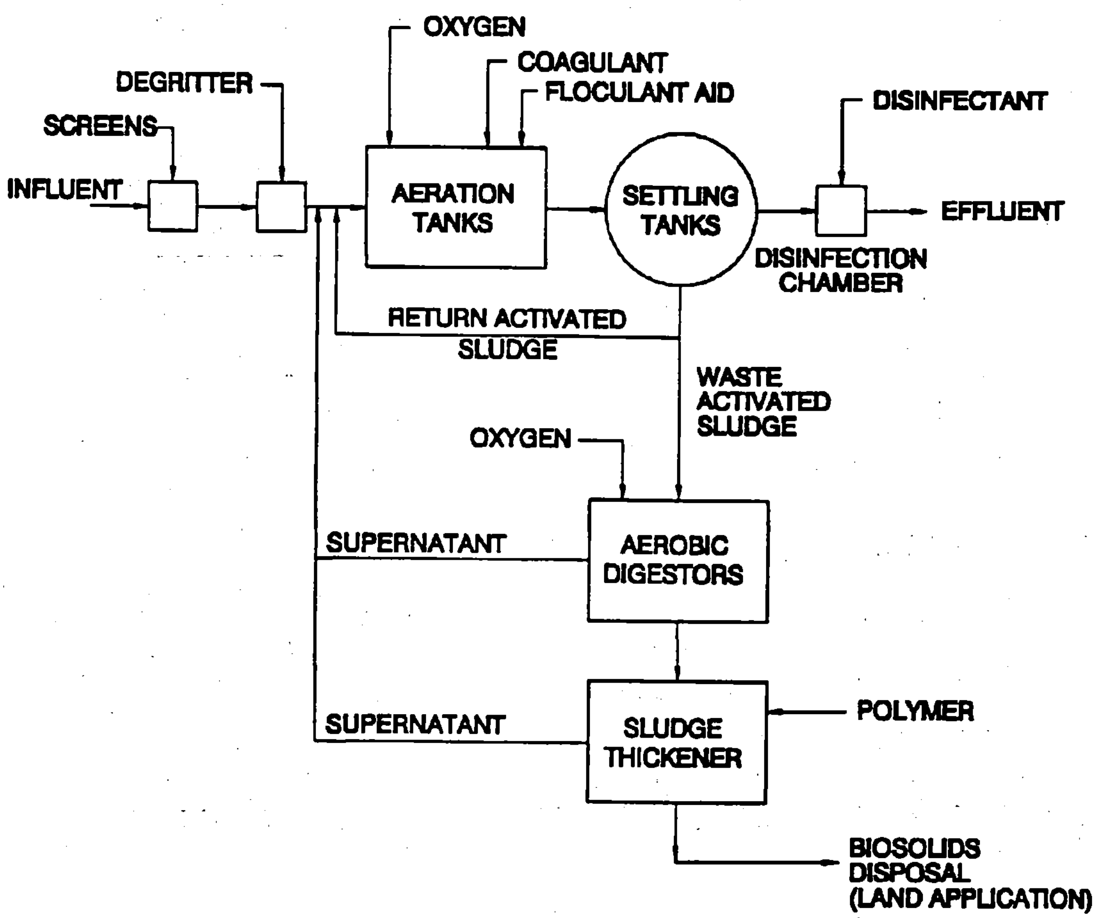

Figure C-9. Nitrifying Activated Sludge Plant (Extended Aeration) (Process Flow Diagram)

APPENDIX D
DRAINAGE AND FEED PIPE EVALUATION REPORT

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Scaling-up of a subsurface vertical flow wetland:

drainage and feeding pipes evaluation

prepared by

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April 1997

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Scaling-up of a subsurface vertical flow wetland: drainage and feeding pipes evaluation

1. INTRODUCTION

In the fall of 1991, the Friends of Fort George and the Regional Municipality of Niagara initiated a research project to evaluate a constructed wetland alternative to treat the effluent from the sewage lagoon treatment system serving Niagara-on-the-Lake. Among the different types of wetlands, a pulsed, vertical flow, subsurface wetland was chosen for testing. As a consequence of the encouraging results, the possibility to scale-up that process is being assessed. This document reviews the drainage layout and the associated costs.

2. DESCRIPTION OF TASKS:

As part of the evaluation of the scale-up costs of a vegetated submerged bed wetland that would be constructed in Niagara-on-the-Lake, our mandate was:

A) To review background materials (i.e. conceptual design information, previous reports, etc.)

B) To optimize the sub-drain spacing and sizing in order to reduce costs

C) To propose an appropriate spacing of the influent pipes

3. DESIGN BOUNDARY CONDITIONS

Wetland Layout

The wetland is composed of three cells in series (fig. 1). Each cell covers an area of 178 by 178 metres. Cattails are planted in the media (fig. 2), which is Queenstone Shale for the first two cells and Lockport Dolomite for the last. A layer of gravel (15 - 20 cm thick) is located about 10 - 15 cm under the top of the cell and a feeding pipe runs through it at a depth of 30 cm. The water that flows from the feeding pipe percolates through a layer of approximately 70 - 80 cm of media and is drained by 15-20cm of gravel in which drains are located. A pump collects that water and transfers it to the top of the next cell.

Design Flow

The flow that will run through the wetland is approximately 5710 m^3/day, with a peak flow of approximately 11000 m^3/day. The lagoon that precedes the wetland should accommodate this variation. As the total surface area of the wetland is 95 167 m^2, the average hydraulic loading rate is 60L/m^2/day. However, as the cells are in series, the design hydraulic loading rate for each cell will be 180L/m^2/day.

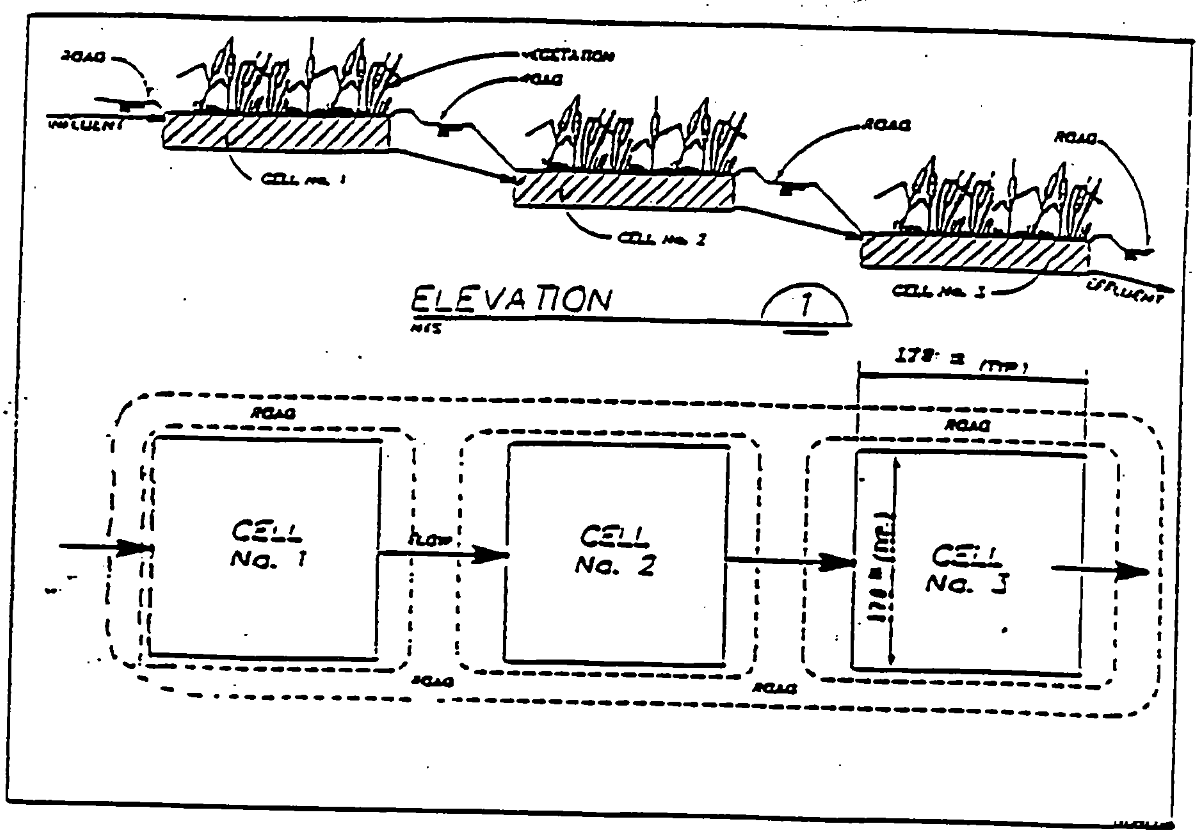

Figure 1: Plan View of the Wetland

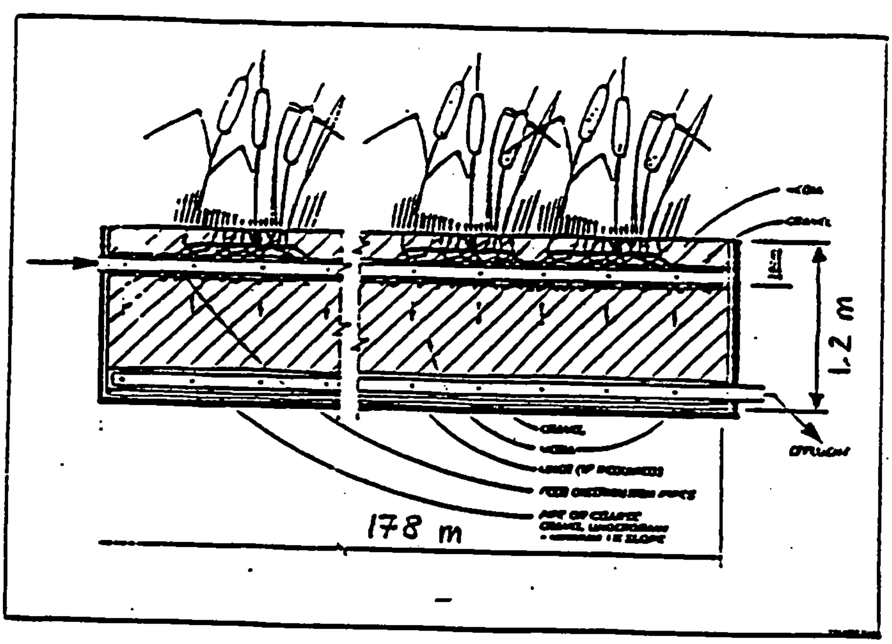

Figure 2: Cell Schematic

In order to promote nitrification-denitrification, both aerobic and anaerobic conditions must exist in the media. For this reason, and also to let the system rest regularly, the flow is pulsed. The drainage design will also be affected by the need to have aerobic and anaerobic regions in the media. If water level rises to the feeding pipes (approx. 85cm above drains), the conditions in the media will be mainly anaerobic. The water level should remain approximately 40 cm above the drains. However, this implies reducing the space between the drains and thus increasing costs.

Media Characteristics

Two media types are used for the subsurface vertical flow wetland: Queenstone Shale for the first two cells and Lockport Dolomite for the last.

A sieve analysis was undertaken by the Laboratories of Walker Industries (Lemon, 1996). The Queenstone Shale is heterogeneous and is composed mainly of particles whose size is between 0.6 and 2.36 mm. This range of particles corresponds to coarse sands and fine gravels. The Lockport Dolomite is dominated by sands and fine gravels; however, the distribution is much more regular. A textural analysis demonstrated that this media is primarily sand (93.9%), containing little silt (5.3%) and only 0.8% clay. The Queenstone Shale texture is primarily silt (78%), with only 2.8% sand and 19.2% clay.

These characteristics influence the hydraulic conductivity of each media. According to the reports submitted (Rozema, 1997; Lemon, 1996; Anonymous, 1996), the hydraulic conductivity of Queenstone Shale ranges from 4.18 to 20.07 cm/hr. As Lockport Dolomite has less coarse particles, it has a lower conductivity (2.0-8.4 cm/hr). Thus, the hydraulic conductivity of the two media is in the range of 10^{-5} m/s, compared to a conductivity of 10^{-2}-10^{3} m/s for clean gravel and 10^{-8}-10^{-4} m/s for fine sands and silts.

4. DRAINAGE DESIGN

4.1 Assumptions

Different assumptions had to be made in order to compute the spacing and diameter of the drains. These include: type of flow, the presence of a gravel layer, the height of water above the drains and the slope of the drains and collectors.

Type of flow

A permanent flow regime was assumed for the calculation of the drainage spacing. The drains were spaced such that the water table should remain at a constant level above the drainage pipes.

Gravel layer or coating

As the documents did not specifically mention if a complete layer of gravel will be provided at the depth of the drains or if gravel will only be placed locally around the drains, two scenarios were studied. Scenario 1 assumes that the drains are directly on the impermeable layer and that they are just surrounded by a coat of gravel. Scenario 2 assumes that there is 15-20 cm complete layer of gravel at the level of the drains. The gavel layer is supposed to dramatically reduce the number of drains that will be needed.

The hydraulic conductivity of the gravel was estimated to be approximately 10^{-3} m/s (Musy and Soutter, 1991). This conductivity is lower than that usually used for gravel (10^{-2}-10^{3} m/s). To reduce the costs, no geotextile will be installed over the gravel layer. It was assumed that over time, part of the media and other particles would clog the gravel. Thus, the chosen conductivity is similar to that of a mix of gravel with fine sands and silts.

Height of water table above drains

The pilot scale experiments were conducted with water at a depth of 30 and 80cm below the ground surface (Rozema, 1997). This corresponds to a water table set at approximately 85 and 40 cm above the centre of the drains. Maintaining the water table 85cm above the drains is less expensive as the spacing between the drains can be higher, but keeping a water table 40 cm above the drains may be necessary for treatment purposes.

Slope of drains and collectors

In order to reduce the costs of excavation, media and pumping, the slope of the drains should be minimal (0.1%). However, with a smaller slope, the diameter must be greater. For that reason, slopes between 0.1-0.3% were considered for the drains. Slopes of 0.05 to 4% (CPVQ, 1989) can be considered for collectors (as low as reasonably possible).

4.2 Calculations

Drainage Spacing

As mentioned above, two scenarios were studied. For the first, the drains are directly above the impermeable layer. According to the equations developed by Guyon (1969), the spacing for a permanent flow can be estimated by:

$$E = 2h\sqrt{\frac{K}{q_c}}$$

E = Spacing (m)
h = Water height above drains plan (m)
K = Hydraulic conductivity of media (m/s)
q_c = Characteristic flow ($m^3/m^2/s$)

To avoid confusion, it is to be mentioned that q_c is actually the load of water that is applied per m^2, and is not the unitary flow (m^3/m length of drain/s). The equation is an applied case ($K_i = 0$) of the global equation for a drain that is between two layers with two different hydraulic conductivities (K_s for the above layer and K_i for the layer that is under the drains). The global equation will be used for the second scenario, where there is a layer of gravel under the drains:

$$E = \sqrt{\frac{8\ K_i\ h\ d}{q_c} + \frac{4\ K_s\ h^2}{q_c}}$$

where d = distance drain-impermeable layer when smaller than 50 cm

Diameter of drains and collectors

The diameter of the drains and collectors will be estimated using the Manning-Strickler equation:

$$Q = \frac{1}{n}\ S\ R^{\frac{2}{3}}\ J^{\frac{1}{2}}$$

Q = Flow (m^3/s)
S = Wet surface of the drain (m^2)
R = Hydraulic ray (m)
J = Slope (m/m)
n = Manning-Strickler coefficient

The hydraulic radius for a circular pipe is equivalent to the wet area divided by the wet perimeter. The term 1/n can be replaced by the Strickler coefficient K. Values of n vary from 0.0148 to 0.0176 for PVC drainage pipes (CPVQ, 1989). A final value of 0.015 was chosen for n (K=67).

4.3 Results

Scenario 1: Drains on impermeable layer

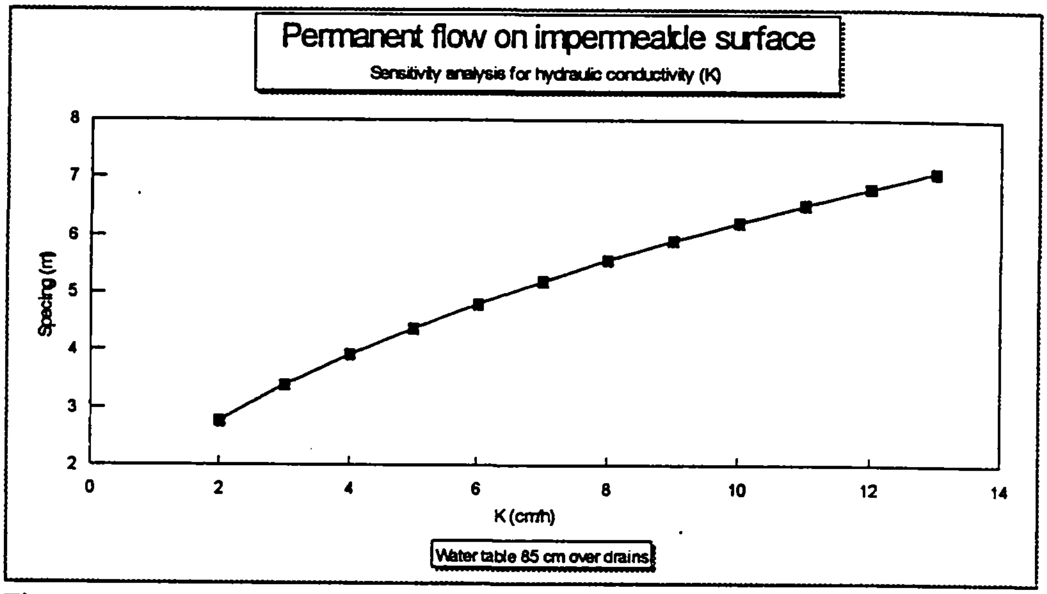

Figure 3. Spacing required to maintain water table 85 cm above drain (no continuous gravel layer provided)

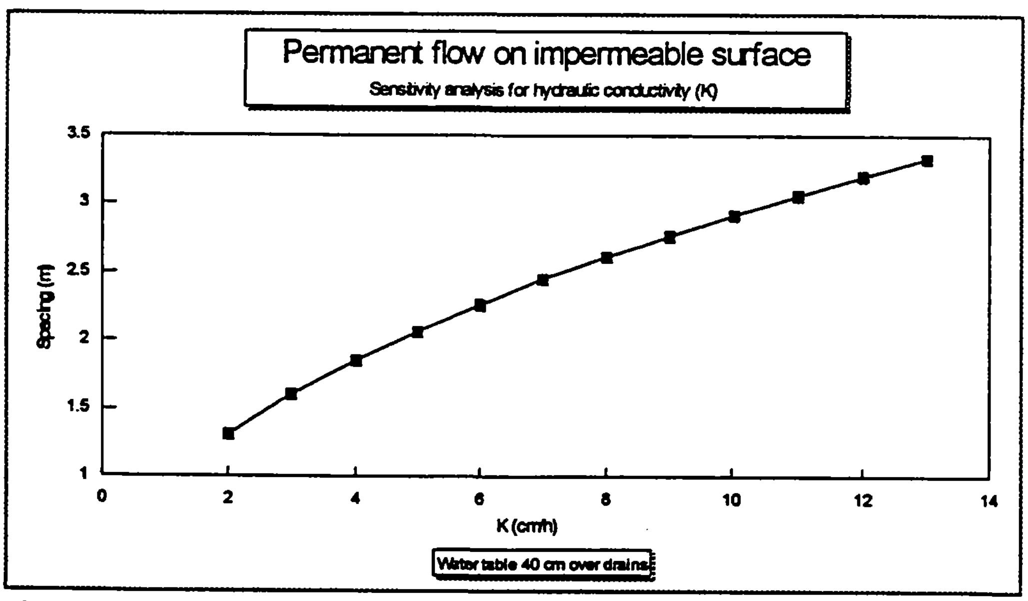

Figure 4. Spacing required to maintain water table 40 cm above drains (no continuous gravel layer provided)

A sensitivity analysis on the effect of a variation in the hydraulic conductivity of the media was conducted. For a variation between 2 and 13 cm/h, the spacing varies between 2.8 and 7.1 metres, if the water table is 0.85 metres above the drains (fig. 3). In the case where the water table is just 0.4 metres above the drains, the spacing varies between 1.3-3.3 metres (fig. 4).

Table 1 shows the spacing, diameter, length of drains in a cell and total cost for the wetland for an hydraulic conductivity equivalent to 5 cm/h, which represents an average for the conductivities encountered. The pipe cost is based on 15$/m. The spacing is quite small, which is the reason why the inclusion of a layer of gravel below the drains could save a substantial amount of money.

Table 1: Design values for K_{media} = 5 cm/h and a slope of 0.2%

Water table (cm)	Spacing (m)	Diameter (cm)	Length per cell (m)	Total cost ($)
40	2.1	10	15306	688783
85	4.4	10	7118	320312

Scenario 2: Drains on a layer of gravel

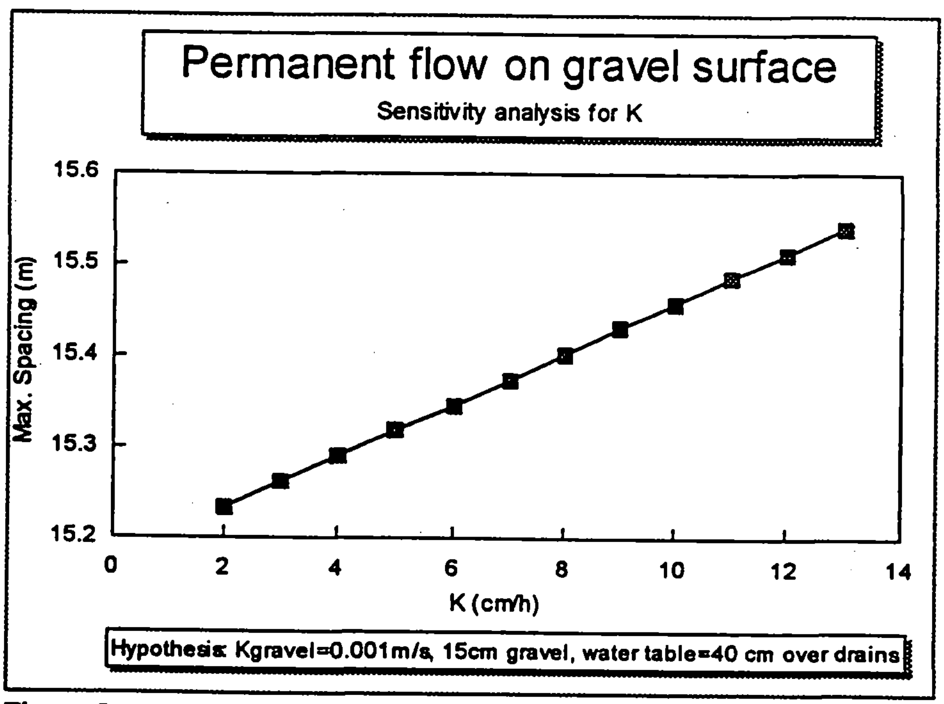

Figure 5. Drainage spacing required to maintain water table 40 cm above pipes if gravel layer is provided.

The inclusion of 15 cm of gravel makes the spacing virtually independent of the hydraulic conductivity of the media (fig. 5). The variation of its conductivity from 2-13 cm/h results in a change of spacing of only 30 cm. Figures 6 and 7 indicate the spacing and diameter of the pipes in function of the hydraulic conductivity of the gravel. Its variation results in a large modification in the spacing.

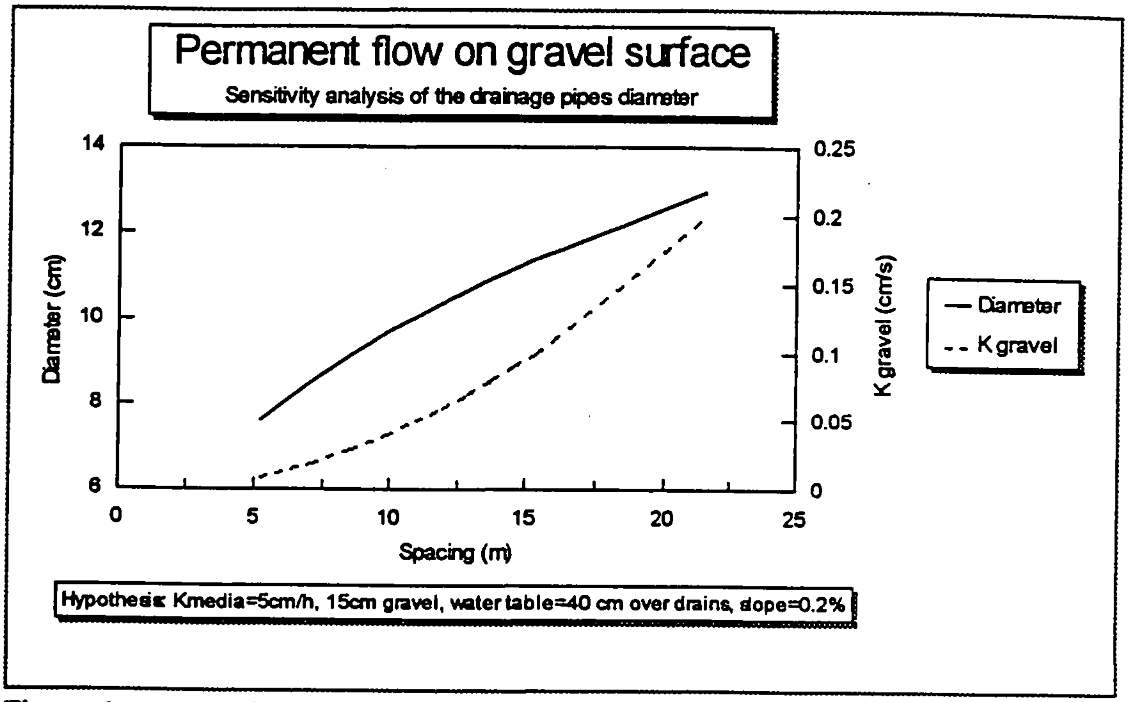

Figure 6. Drainage spacing and diameter required to maintain water table 40 cm above drainage pipes if layer of gravel is provided.

As stated previously, an hydraulic conductivity of 10^{-3} m/s is a good estimate for gravel which is partially clogged with sand, silt and other particles. For the considered conductivity, table 2 indicates what spacing, diameter, length of pipes and costs are to be expected. The costs are based on a price for the drains of 15\$/m and 18\$/m^3 for the gravel. For a water table 40 cm above the drains, as desired, the cost is half of what it would be in a design without a gravel layer. However, some additional gravel might be necessary in order to surround the part of the pipes that is in the media layer.

Table 2: Design values for K_{media} = 5 cm/h, K_{gravel} = 10^{-3} m/s and a slope of 0.2%

Water table (cm)	Spacing (m)	Diameter (cm)	Length per cell (m)	Total cost (\$)
40	15.3	15	1952	344488
85	22.6	15	1244	312600

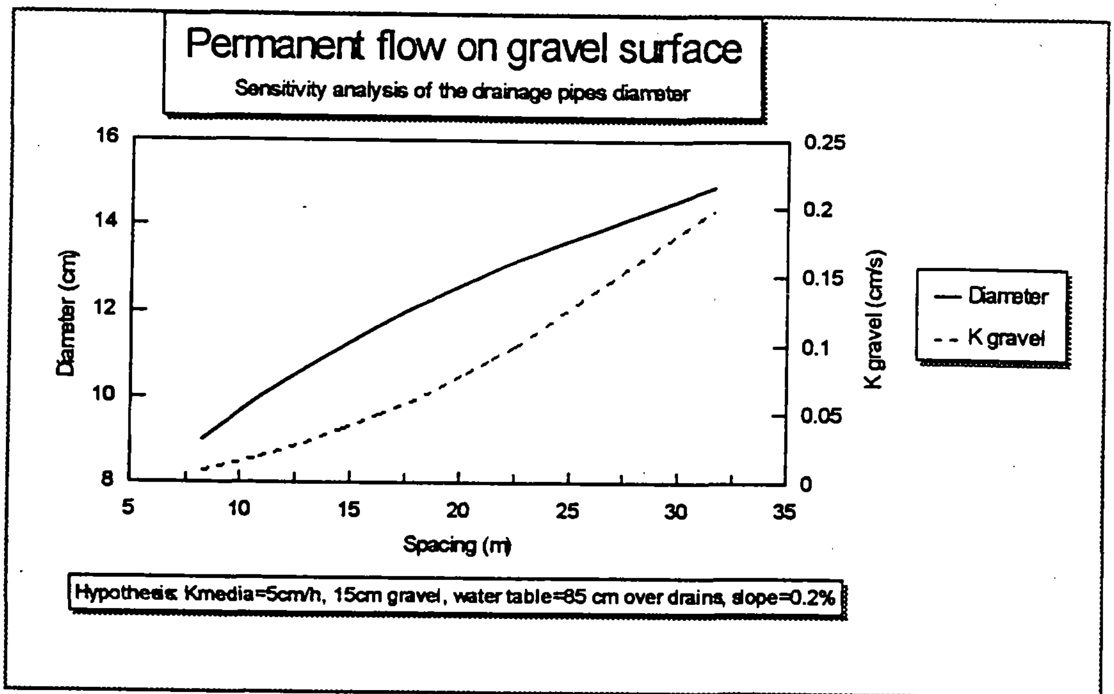

Figure 7. Drainage spacing and diameter required if water table is to be maintained 85 cm above drainage pipes and a gravel layer is provided.

Collector Size:

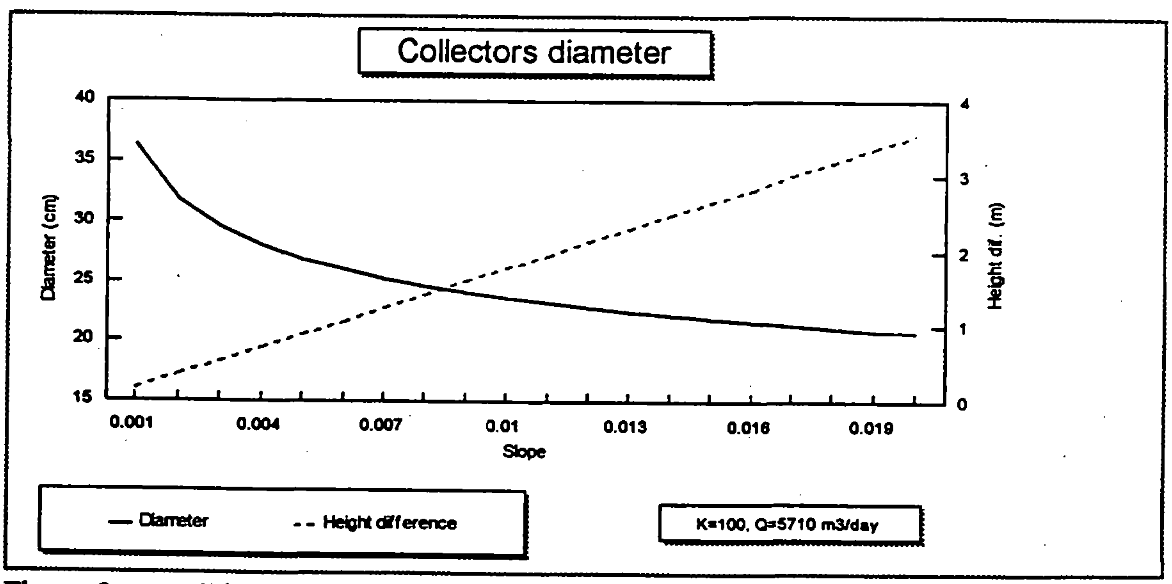

Figure 8. Diameter of PVC collector main required versus slope provided.

The size of the collectors was calculated using the Manning-Strickler equations. According to the literature (Bourrier, 1991), the coefficient for that size of pipes (PVC) is approximately n=0.01 (K=100). Depending on the height difference between the beginning and the end of the collector in the cell, the diameter will vary between 30-40 cm (fig. 8). Among the different possibilities, a slope of 0.35% results in a diameter of 30 cm and a height difference of 62 cm. A slope of 1% results in a diameter of 25cm and a height difference of 1.78 metres. The cost of having a bigger head for the pumps should be compared to the advantage of having a smaller collector.

4.4 Summary

If the goal is to set the water table at a depth of approximately 80 cm from the cell surface (40cm above the drains), then the use of a gravel layer is clearly advantageous. As the drains are located at the interface of the media and gravel layers, the latter does not need to be thicker than 15cm. The proposed design should thus be the following:

Spacing = 15.3 metres
Diameter of drainage pipes = 15 cm
Gravel Layer under drains = 15 cm
Slope = 0.2%

5. FEEDING PIPES

As part of our mandate, we also had to indicate the distance between the holes on each pipe, the distance between each pipe and the size of the holes. In the following paragraphs, a list of assumptions will be presented, the calculations will be explained and the results will be presented.

5.1 Assumptions

Flow

The influent surface loading is 180L/m^2/day for each cell. The flow passing through each hole is a function of the spacing between the pipes and the holes. During the pilot scale study, the pulses were extremely short (one pulse of about one minute every four hours), which is difficult to scale-up (high flow through the gravel layer, over-dimensioned pipes that function only a short time). For that reason, it was decided to increase the duration of the pulses (between 10 minutes to two hours) in the calculations. If the pulses last longer, the drainage system will also work better.

Orifice Size

The system of feeding pipes is similar to that used for agricultural irrigation. The following calculations are based on irrigation principles. For drip irrigation, the diameter of the holes is usually between 0.5 and 1.5 mm (Mermoud et al., 1989). To avoid clogging problems, it is suggested that the diameter should be greater than 1 mm. The diameters of the holes was set at 3 mm to lower the risk of clogging.

Pressure in pipes

The outlet pressure in a drip irrigation system is between 100 and 200 kPa (14.5-29 psi). However, in irrigation pipes (a metallic or plastic pipe with holes every 0.5-1m and with a high loading rate) the pressure is 50-200 kPa (7.25-29 psi). To slow the velocity of the water entering the gravel bed, the pressure was set at 50 kPa (7.25 psi). The relation between the pressure and the flow through a hole is the following:

$$Q = CS\sqrt{2gH}$$

Q = flow through the hole (m3/s)
C = coefficient = 0.6
g = gravitational constant=9.81 m/s/s
H = pressure in water height (m)
S = surface area of the hole (m^2)

Spacing of feeding pipes and distance between holes

The spacing of pipes and the distance between holes are functions of the flow through the hole and of the type of media. These two factors affect the size of the wetting pattern. For drip irrigation, empirical values exist for flows of up to 8 litres per hour. A 100% efficiency can be obtained in a middle textured soil with a spacing between the holes of 1.3m and a spacing of the pipes of 1.5m (Mermoud et al., 1989). For a fine textured soil, these values are 1.7m and 2m. However, in this study, the flows are higher and these empirical values cannot be used. A simple model has been selected to calculate the wetting pattern (Jensen, 1983). The model approximates the wetted soil volume as an hemisphere:

$$r = 3\frac{qt}{2\,\pi\,.\,(\theta_f - \theta_i)^{\frac{1}{3}}}$$

r = radius of wetting pattern (m)
q = flow (m^3/h)
t = time of application (h)
θ_f = final moisture (-)
θ_i = initial moisture (-)

5.2 Results

Wetting pattern

As the flow rates are high, the approximation of the wetting pattern by a sphere demonstrates that the wetted area is always greater than required (see appendices 4A and 4B). This suggests that whatever the spacing the whole ground would be covered. However, since the model used overestimates the size of the wetting pattern near the surface, it was decided to set the spacing between the holes and the pipes at 2.5 metres (which is comparable to the empirical values used for smaller flows).

Flow, pressure, orifice diameter and duration of application

All these factors are interdependent. As mentioned above, proposed design values are the following: a pressure of 50 kPa (7.25 psi), a diameter of holes of 3mm and a pulse long enough to reduce the flow to an acceptable level. It was estimated that the pulse should last 74 minutes (cycle of four hours), resulting in a flow through the holes of 2.54 litres per minute (see appendix 5).

5.4 Summary

According to the calculations, the design for the feeding pipes is the following:

Spacing of holes = 2.5 m
Spacing of pipes = 2.5 m
Diameter of hole = 3mm
Pressure at hole = 50 kPa (7.25 psi)
Pulse duration = 74 minutes every 4 hours
Flow through hole during pulse = 2.54 l/min

Such a design should reduce the risks of clogging, diminish the costs of operation and allow the scale-up of the whole process. The scaling-up of the feeding pipes will be a major challenge, as such conditions are seldom encountered.

6. CONCLUSION

The optimization of the spacing between the drains can cut the cost by half of what is stated in the reports submitted (James W. Schmidt Associates Inc., 1997). The use of a gravel layer allows to increase the spacing dramatically. No geotextile will have to be installed over the gravel layer, as the assumed hydraulic conductivity for that material is quite low. It should to be mentioned the spacing was set making the assumption that the flow is permanent. That condition is more restrictive than for a "drying flow" regime, as the drains have to cope with a continuing incoming flow. However, the more regular the flow, the better the drainage system will work. This is also true for the feeding pipes, as a very short pulse results in a system that is over-designed and inefficiently operates for short intervals. For this reason, it is proposed to increase the pulse duration. This will also improve the flow around the feeding pipes.

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APPENDICES

APPENDIX 1

Drainage Calculations - no gravel layer

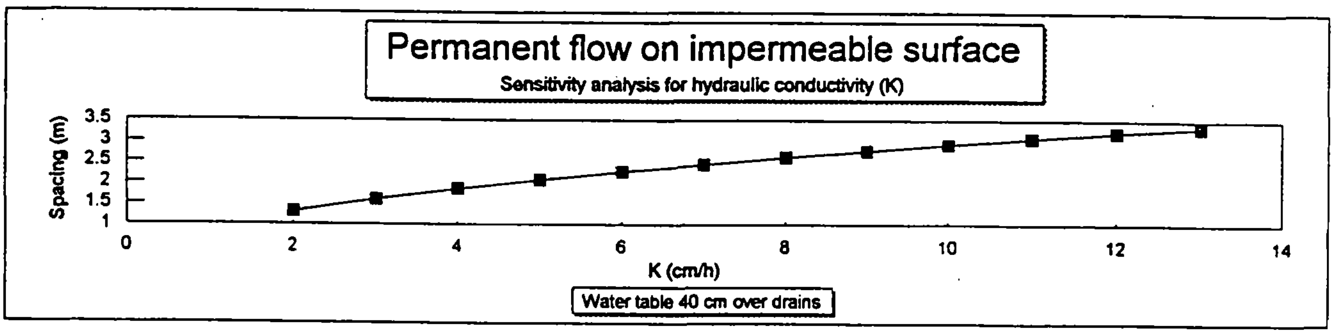
Permanent flow - drains placed on impermeable surface - h=40 cm

Entered values:

h (m)=	*0.4*	*Half-cell width =*	*89*	*Cell length (m)=*	*178*
K(cm/h)=	*5*	*K Strickler =*	*67*	n Manning =	0.015
qc(l/m2/d)=	*180*	*Pipes cost per m=*	*15*	*slope (m/m)=*	*0.002*

Hydraulic conductivity (K) variation:

K	K	qc	Spacing E	Q	D	nb drains	Length drains per cell	Cost/cell	For 3 cells
cm/hr	m/s	m3/m2/s	m	m3/s	cm	-	m	$	$
2	5.6E-06	2.1E-06	1.31	2.42E-04	4.5	137	24207	363105	1089316
3	8.3E-06	2.1E-06	1.60	2.97E-04	4.9	112	19757	296352	889056
4	1.1E-05	2.1E-06	1.85	3.43E-04	5.1	97	17087	256302	768906
5	**1.4E-05**	**2.1E-06**	**2.07**	**3.83E-04**	**5.4**	87	15306	**229594**	688783
6	1.7E-05	2.1E-06	2.26	4.20E-04	5.6	79	13883	208249	624746
7	1.9E-05	2.1E-06	2.44	4.53E-04	5.7	73	12816	192234	576701
8	2.2E-05	2.1E-06	2.61	4.84E-04	5.9	69	12102	181526	544577
9	2.5E-05	2.1E-06	2.77	5.14E-04	6.0	65	11390	170848	512544
10	2.8E-05	2.1E-06	2.92	5.42E-04	6.1	61	10680	160197	480591
11	3.1E-05	2.1E-06	3.06	5.68E-04	6.2	59	10321	154819	464456
12	3.3E-05	2.1E-06	3.20	5.93E-04	6.3	56	9789	146832	440496
13	3.6E-05	2.1E-06	3.33	6.18E-04	6.4	54	9432	141482	424446



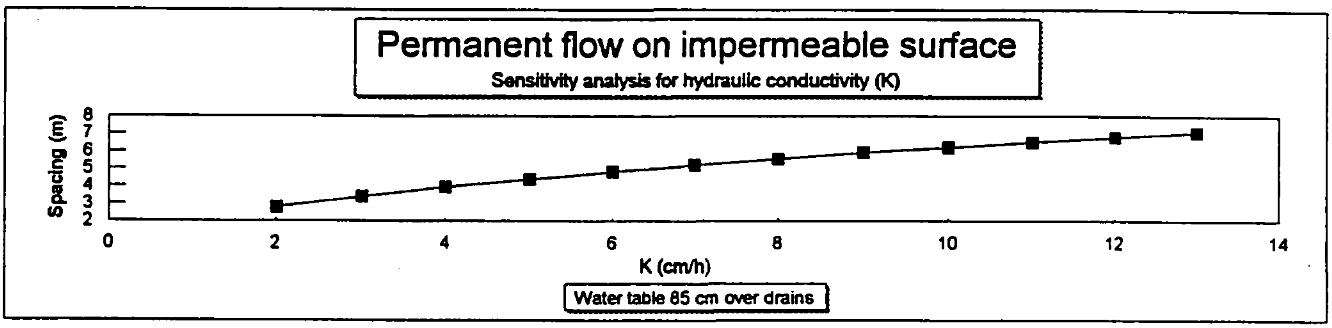
Permanent flow - drains placed on impermeable surface - h=85 cm

Entered values:

h (m)=	*0.85*	*Half-cell width =*	*89*	*Cell length (m)=*	*178*
K(cm/h)=	*5*	*K Strickler =*	*67*	n Manning =	0.015
qc(l/m2/d)=	*180*	*Pipes cost per m=*	*15*	*slope (m/m)=*	*0.002*

Hydraulic conductivity (K) variation:

K	K	qc	Spacing E	Q	D	nb drains	Length drains per cell	Cost/cell	For 3 cells
cm/hr	m/s	m3/m2/s	m	m3/s	cm	-	m	$	$
2	5.6E-06	2.1E-06	2.78	5.15E-04	6.0	65	11390	170843	512530
3	8.3E-06	2.1E-06	3.40	6.30E-04	6.5	53	9254	138807	416421
4	1.1E-05	2.1E-06	3.93	7.28E-04	6.8	46	8007	120111	360333
5	**1.4E-05**	**2.1E-06**	**4.39**	**8.14E-04**	**7.1**	**41**	**7118**	**106771**	**320312**
6	1.7E-05	2.1E-06	4.81	8.92E-04	7.4	38	6581	98719	296158
7	1.9E-05	2.1E-06	5.19	9.63E-04	7.6	35	6048	90723	272170
8	2.2E-05	2.1E-06	5.55	1.03E-03	7.8	33	5691	85362	256085
9	2.5E-05	2.1E-06	5.89	1.09E-03	8.0	31	5335	80032	240095
10	2.8E-05	2.1E-06	6.21	1.15E-03	8.1	29	4982	74730	224189
11	3.1E-05	2.1E-06	6.51	1.21E-03	8.3	28	4802	72026	216077
12	3.3E-05	2.1E-06	6.80	1.26E-03	8.4	27	4622	69336	208008
13	3.6E-05	2.1E-06	7.08	1.31E-03	8.5	26	4444	66660	199979



APPENDIX 2

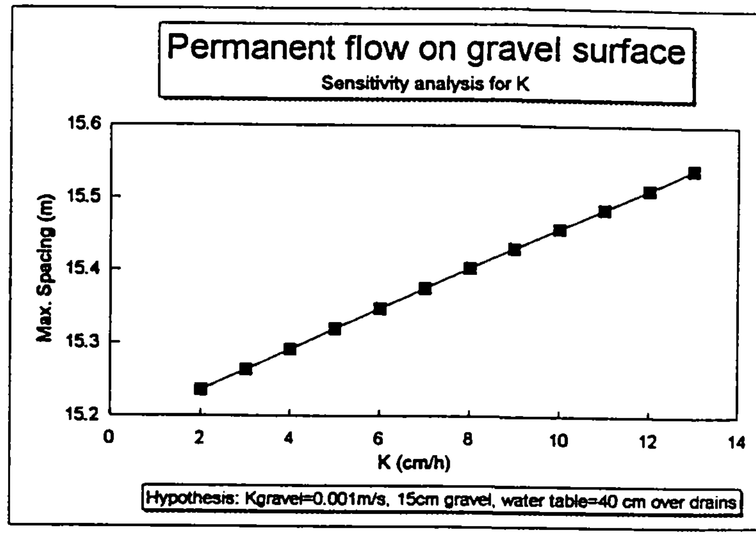
Drainage Calculations - with gravel layer

Permanent flow on gravel surface: variation of K media

Entered values:	*h(m)=*	*0.4*	*Kgravel (m/s)=*	*0.001*
	qc(l/m2/d)=	*180*	*Gravel depth d (m)=*	*0.15*

Variation of K media:

K cm/hr	K m/s	qc m3/m2/s	E m
2	5.6E-06	2.1E-06	15.24
3	8.3E-06	2.1E-06	15.26
4	0.000011	2.1E-06	15.29
5	0.000014	2.1E-06	15.32
6	0.000017	2.1E-06	15.35
7	0.000019	2.1E-06	15.37
8	0.000022	2.1E-06	15.40
9	0.000025	2.1E-06	15.43
10	0.000028	2.1E-06	15.46
11	0.000031	2.1E-06	15.49
12	0.000033	2.1E-06	15.51
13	0.000036	2.1E-06	15.54



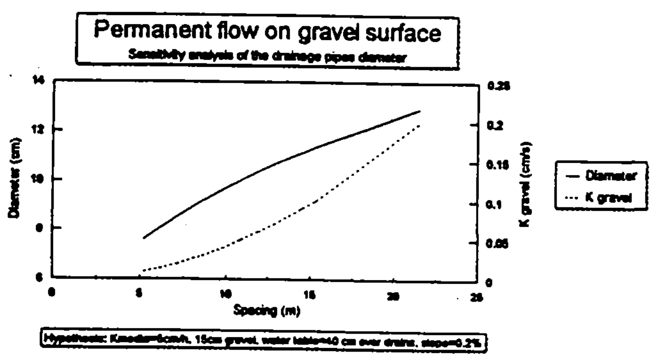
Permanent flow - drains placed on a gravel layer - h=40cm:

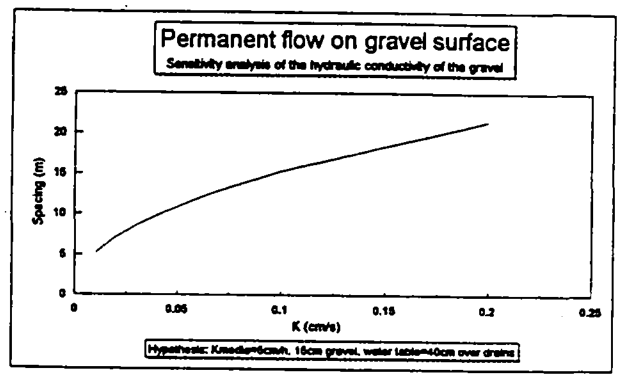
Entered values:

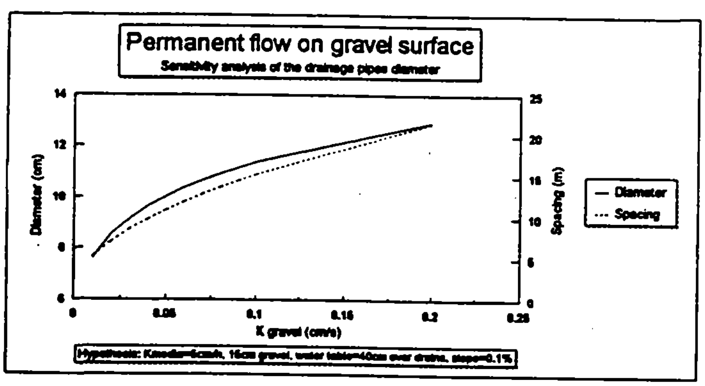
h(m)=	*0.4*	*K Strickler=*	*67*	*Half-cell width (m)=*	*89*
qc (l/m2/d) =	*180*	*n Manning=*	0.015	*Length of cell (m)=*	*178*
Kmedia (cm/h)=	*5*	*Slope (m/m)=*	*0.002*	*Gravel cost per m3=*	*18*
Gravel depth d (m)=	*0.15*	*Pipes cost per m=*	*15*		

Sensitivity analysis for the hydraulic conductivity of the gravel:

Kgravel cm/s	Kgravel m/s	qc m3/m2/s	Spacing m	Flow m3/s	Diameter cm	nb drains -	L drains m	Cost $	Gravel cost $	Total/cell $	For 3 cells $
0.01	1.0E-04	2.1E-06	5.23	9.69E-04	7.6	35	6047	90707	85547	176253	528760
0.02	2.0E-04	2.1E-06	7.10	1.32E-03	8.5	26	4444	66653	85547	152200	456599
0.03	3.0E-04	2.1E-06	8.57	1.59E-03	9.2	21	3558	53372	85547	138918	416755
0.04	4.0E-04	2.1E-06	9.82	1.82E-03	9.6	19	3195	47931	85547	133478	400435
0.05	5.0E-04	2.1E-06	10.93	2.03E-03	10.0	17	2840	42603	85547	128150	384449
0.06	6.0E-04	2.1E-06	11.94	2.21E-03	10.4	15	2491	37364	85547	122911	368733
0.07	7.0E-04	2.1E-06	12.87	2.39E-03	10.7	14	2312	34678	85547	120225	360675
0.08	8.0E-04	2.1E-06	13.73	2.55E-03	10.9	13	2135	32032	85547	117579	352737
0.09	9.0E-04	2.1E-06	14.55	2.70E-03	11.2	13	2125	31873	85547	117420	352260
0.1	1.0E-03	2.1E-06	15.32	2.84E-03	11.4	12	1962	29283	85547	114829	344488
0.2	2.0E-03	2.1E-06	21.57	4.00E-03	12.9	9	1408	21119	85547	106665	319996





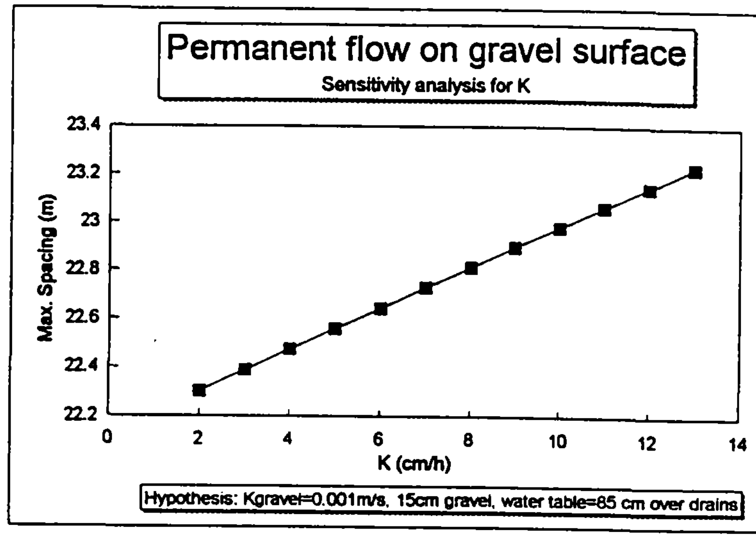


Permanent flow on gravel surface: variation of K media

Entered values:	*h(m)=*	*0.85*	*Kgravel (m/s)=*	*0.001*
	qc(l/m2/d)=	*180*	*Gravel depth d (m)=*	*0.15*

Variation of K media:

K cm/hr	K m/s	qc m3/m2/s	E m
2	5.6E-06	2.1E-06	22.30
3	8.3E-06	2.1E-06	22.39
4	0.000011	2.1E-06	22.47
5	0.000014	2.1E-06	22.56
6	0.000017	2.1E-06	22.64
7	0.000019	2.1E-06	22.73
8	0.000022	2.1E-06	22.81
9	0.000025	2.1E-06	22.90
10	0.000028	2.1E-06	22.98
11	0.000031	2.1E-06	23.06
12	0.000033	2.1E-06	23.15
13	0.000036	2.1E-06	23.23



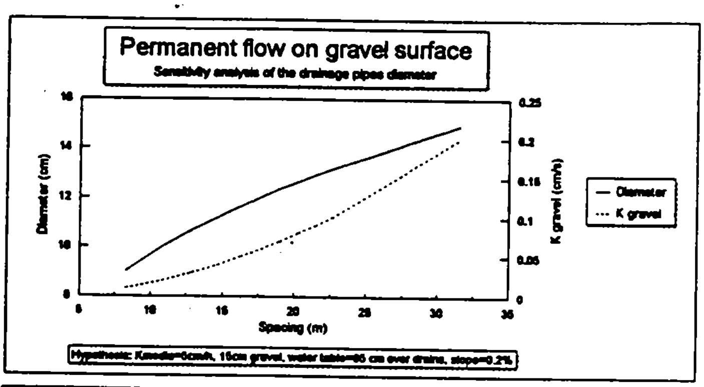
Permanent flow - drains placed on a gravel layer - h=65cm:

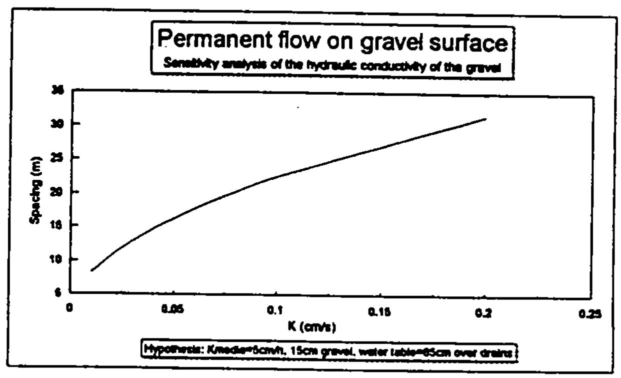
Entered values:

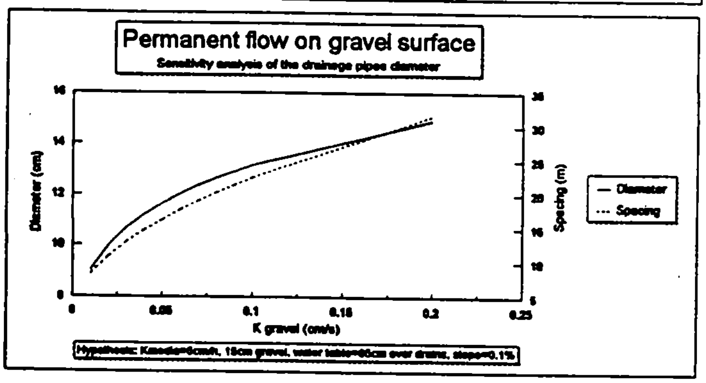
h(m)=	*0.65*	*K Strickler=*	*67*	*Half-cell width (m)=*	*89*
qc (l/m2/d) =	*180*	n Manning=	0.015	*Length of cell (m)=*	*178*
Kmedia (cm/h)=	*5*	*Slope (m/m)=*	*0.002*	*Gravel cost per m3=*	*18*
Gravel depth d (m)=	*0.15*	*Pipes cost per m=*	*15*		

Sensitivity analysis for the hydraulic conductivity of the gravel:

Kgravel cm/s	Kgravel m/s	qc m3/m2/s	Spacing m	Flow m3/s	Diameter cm	nb drains -	L drains m	Cost $	Gravel cost $	Total/cell $	For 3 cells $
0.01	1.0E-04	2.1E-06	8.26	1.53E-03	9.0	22	3734	56014	85547	141561	424683
0.02	2.0E-04	2.1E-06	10.83	2.01E-03	10.0	17	2842	42630	85547	128176	384529
0.03	3.0E-04	2.1E-06	12.69	2.39E-03	10.7	14	2312	34673	85547	120220	360660
0.04	4.0E-04	2.1E-06	14.67	2.72E-03	11.2	13	2123	31850	85547	117397	352190
0.05	5.0E-04	2.1E-06	16.25	3.01E-03	11.6	11	1779	26689	85547	112236	336707
0.06	6.0E-04	2.1E-06	17.69	3.28E-03	12.0	11	1763	26451	85547	111998	335993
0.07	7.0E-04	2.1E-06	19.03	3.53E-03	12.3	10	1590	23846	85547	109393	328179
0.08	8.0E-04	2.1E-06	20.27	3.76E-03	12.6	9	1420	21293	85547	106840	320520
0.09	9.0E-04	2.1E-06	21.45	3.98E-03	12.9	9	1409	21135	85547	106682	320045
0.1	1.0E-03	2.1E-06	22.56	4.18E-03	13.2	8	1244	18653	85547	104200	312599
0.2	2.0E-03	2.1E-06	31.60	5.86E-03	14.9	6	878	13176	85547	98723	296169







APPENDIX 3

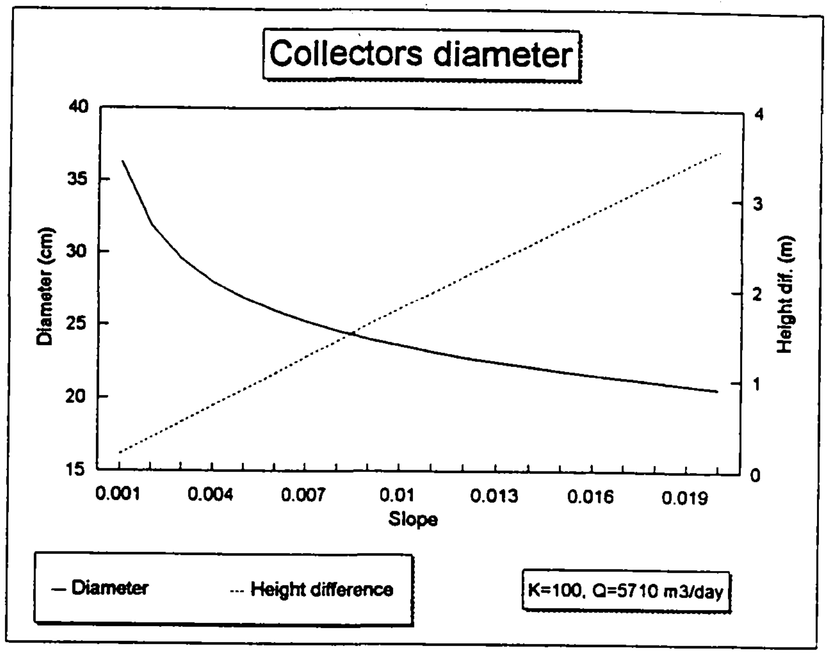
Calculation of collector main diameter

Size of the collector mains:

Entered values:	*Total Flow (m3/d)*	*5710*		
	Flow in each series(m3/d)=	*5710*	Flow(m3/s)=	0.066088
	K Strickler =	*100*	Manning n=	0.010
	Main length =	*178*		

Variation of the slope:

Slope (m/m)	Diameter cm	Height difference m
0.001	36.30	0.18
0.002	31.88	0.36
0.003	29.54	0.53
0.004	27.99	0.71
0.005	26.84	0.89
0.006	25.94	1.07
0.007	25.20	1.25
0.008	24.58	1.42
0.009	24.04	1.60
0.01	23.57	1.78
0.011	23.15	1.96
0.012	22.78	2.14
0.013	22.44	2.31
0.014	22.13	2.49
0.015	21.85	2.67
0.016	21.58	2.85
0.017	21.34	3.03
0.018	21.11	3.20
0.019	20.90	3.38
0.02	20.70	3.56



APPENDIX 4

Irrigation (feed) Calculations

Wetting pattern: Queenstone Shale

Q(l/m2/pulse) =	*30*	The incoming flow area is the area that provides such a flow
Initial moisture (-) =	*0.19*	
Final moisture (-) =	*0.42*	The wet area is the area that is wet by such a flow

Variation of the area that provides the flow and effect on the wetting pattern:

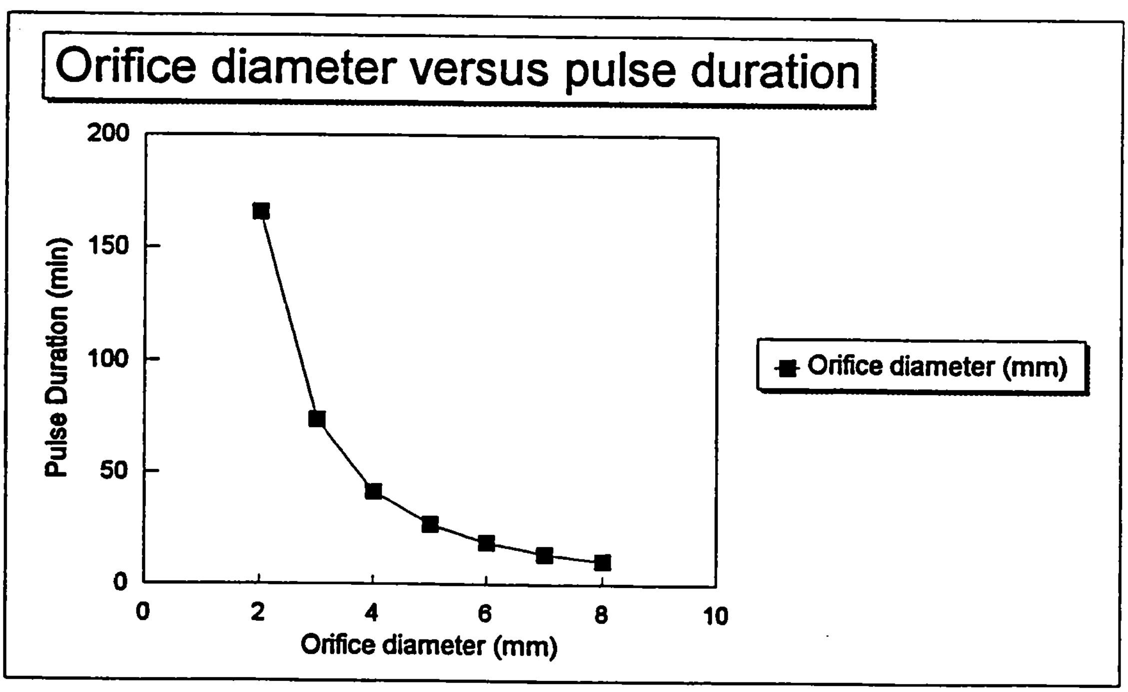
Area to be covered (A) m2	q m3/dose	r m	Wetted area (B) m2	B/A %
1.0	0.30	0.80	1.24	124
1.1	0.33	0.88	1.50	137
1.2	0.36	0.96	1.79	149
1.3	0.39	1.03	2.10	161
1.4	0.42	1.11	2.43	174
1.5	0.45	1.19	2.79	186
1.6	0.48	1.27	3.18	199
1.7	0.51	1.35	3.59	211
1.8	0.54	1.43	4.02	224
1.9	0.57	1.51	4.48	236
2.0	0.60	1.59	4.97	248
2.1	0.63	1.67	5.48	261
2.2	0.66	1.75	6.01	273
2.3	0.69	1.83	6.57	286
2.4	0.72	1.91	7.15	298
2.5	0.75	1.99	7.76	311
2.6	0.78	2.07	8.40	323
2.7	0.81	2.15	9.05	335
2.8	0.84	2.23	9.74	348
2.9	0.87	2.31	10.45	360
3.0	0.90	2.39	11.18	373
3.1	0.93	2.47	11.94	385
3.2	0.96	2.55	12.72	397
3.3	0.99	2.63	13.53	410
3.4	1.02	2.71	14.36	422
3.5	1.05	2.79	15.22	435
3.6	1.08	2.87	16.10	447
3.7	1.11	2.95	17.00	460
3.8	1.14	3.02	17.94	472
3.9	1.17	3.10	18.89	484
4.0	1.20	3.18	19.87	497
4.1	1.23	3.26	20.88	509
4.2	1.26	3.34	21.91	522
4.3	1.29	3.42	22.97	534
4.4	1.32	3.50	24.05	546
4.5	1.35	3.58	25.15	559
4.6	1.38	3.66	26.28	571
4.7	1.41	3.74	27.44	584
4.8	1.44	3.82	28.62	596
4.9	1.47	3.90	29.82	609
5.0	1.50	3.98	31.05	621
5.1	1.53	4.06	32.31	633
5.2	1.56	4.14	33.58	646
5.3	1.59	4.22	34.89	658
5.4	1.62	4.30	36.22	671
5.5	1.65	4.38	37.57	683
5.6	1.68	4.46	38.95	696
5.7	1.71	4.54	40.35	708
5.8	1.74	4.62	41.78	720
5.9	1.77	4.70	43.24	733
6.0	1.80	4.78	44.71	745
6.1	1.83	4.86	46.22	758
6.2	1.86	4.94	47.74	770
6.3	1.89	5.02	49.30	782
6.4	1.92	5.09	50.87	795
6.5	1.95	5.17	52.48	807
6.6	1.98	5.25	54.10	820
6.7	2.01	5.33	55.76	832
6.8	2.04	5.41	57.43	845
6.9	2.07	5.49	59.13	857
7.0	2.10	5.57	60.86	869
7.1	2.13	5.65	62.61	882
7.2	2.16	5.73	64.39	894
7.3	2.19	5.81	66.19	907
7.4	2.22	5.89	68.01	919
7.5	2.25	5.97	69.86	932
7.6	2.28	6.05	71.74	944
7.7	2.31	6.13	73.64	956
7.8	2.34	6.21	75.57	969
7.9	2.37	6.29	77.52	981

Feeding pipes: spacing, hole diameter, pressure, pulse duration

Holes spacing (m)=	*2.5*	Feeding area (m2)=	6.25
Pipes spacing (m)=	*2.5*		
Pulse (l/m2/dose)=	*30*	Pulse per hole (l/dose)=	187.5
Pressure (bar)=	*0.5*	Pressure (psi)=	7.25
		Pressure (kPa)=	50
		Pressure water (m)=	5.10
Coefficient C=	*0.6*		

Variation of pulse duration: **Q=CS(2gH)^0.5**

Orifice diameter (mm) mm	S m2	Q m3/s	Pulse Duration (min) hours
8	0.00005	0.000302	10.36
7	0.000038	0.000231	13.53
6	0.000028	0.00017	18.42
5	0.00002	0.000118	26.52
4	0.000013	0.000075	41.44
3	**7.1E-06**	**0.000042**	73.67
2	3.1E-06	0.000019	165.76



Recommended pipe sizes and spacing for a nine-cell subsurface wetland

Addendum to:

Scaling-up of a subsurface vertical flow wetland: drainage and feeding pipes evaluation

- Alternate Design

1. INTRODUCTION

The following calculations are presented as an addendum to the report *Scaling-up of a subsurface vertical flow wetland: drainage and feeding pipes evaluation* (Fankhauser & Weil, 1997). They reflect modifications considered for the design of the proposed subsurface wetland for Niagara-on-the-Lake.

2. DESIGN CRITERIA

Wetland Layout

The proposed wetland design is composed of nine cells, three series of three cells (as shown in the sketch titled "Case 1. Plan View" included in Appendix 1). Each cell covers an area of 84 m by 84 m. The total depth of each cell is 1.8 m. The flow is split evenly among the three series.

Design Flow

The flow is approximately 5710 m^3/day, with a peak flow of approximately 11000 m^3 /day. The lagoon that precedes the wetland should accommodate this variation. As the total surface area of the wetland is 63 974 m^2, the average hydraulic loading rate is 90 L/m^2/day. However, as the cells are in three series, the design hydraulic loading rate for each cell is 270 L/m^2/day.

Design Water Level in the Wetland

Three water levels are considered: 40 cm, 70 cm and 85 cm above the pipes. Lower water levels require closer pipe spacing.

Media Characteristics

The same media is to be used as suggested in the previous design.

3. DRAINAGE DESIGN

The same assumptions and design calculations were followed as in the previous report. Calculations were made under two scenarios: 1) drains overlying impermeable layer and 2) drains placed in a layer of crushed stone or gravel.

Scenario 1: Drains overlying impermeable layer (no continuous layer of crushed stone)

Table 1 shows the spacing, diameter, length of drains in a cell and total cost for the wetland for an hydraulic conductivity equivalent to 5 cm/h, which represents an average for the conductivities encountered. The pipe cost is based on $15/m.

Table 1: Design values for K_{media} = 5 cm/h and a slope of 0.2% (no continuous layer of crushed stone)

Water table (cm)	Spacing (m)	Pipe Diameter (mm)	Length per cell (m)	Total cost ($)
40	1.69	100	4116	$555 616
70	2.95	100	2350	$317 305
85	3.58	100	1930	$260 548

The minimum pipe diameter was chosen to be 100 mm (commonly used in agricultural drainage). All of the calculated pipe diameters were below this minimum.

Calculations as well as a sensitivity analysis on the effect of a variation in the hydraulic conductivity of the media is presented in Appendix 2.

Scenario 2: Drains on a layer of gravel

Table 2: Design values for K_{media} = 5 cm/h, K_{gravel} = 10^{-3} m/s and a slope of 0.2%

Water table (cm)	Spacing (m)	Minimum Diameter (cm)	Length per cell (m)	Total cost ($)
40	12.5	15	500	$239 021
70	16.7	15	404	$226 007
85	18.4	15	328	$215 728

Calculations and sensitivity analysis of the variation of the hydraulic conductivity of the media is presented in Appendix 3. The inclusion of 15 cm of gravel makes the spacing virtually independent of the hydraulic conductivity of the media.

Collector Size:

Calculations are presented in Appendix 4.

The size of the collectors was calculated using the Manning-Strickler equations. According to the literature (Bourrier, 1991), the coefficient for that size of pipes (PVC) is approximately n=0.01 (K=100). Depending on the height difference between the beginning and the end of the collector in the cell, the calculated pipe diameter varied between 15-25 cm. Using a slope of 0.3%, the required diameter of the collector main was calculated as 20 cm (a least 25 cm is recommended) and the height difference as 25 cm. A slope of 1% resulted in a calculated diameter of 16 cm (at least 20 cm is recommended) and a height difference of 84 cm. The cost of having a bigger head for the pumps should be compared to the advantage of a smaller collector.

Summary

If the goal is to set the water table at a depth of approximately 40 cm above the drains, then the use of a gravel layer is clearly advantageous. As the drains are located at the interface of the media and gravel layers, the latter does not need to be thicker than 15 cm. The proposed design should thus be the following:

Spacing = 12.5 metres
Diameter of drainage pipes = 15 cm
Gravel Layer under drains = 15 cm
Slope of drainage pipes = 0.2%
Diameter of collector main = 25 cm (minimum)
Slope of collector main = 0.3%

4. FEEDING PIPES

The same assumptions and design calculations were made as in the previous report. Calculations are presented in Appendix 5.

Flow

The influent surface loading is 270L/m^2/day for each cell. It is suggested to dose the first cells every 4 hours (6 times daily) in pulses of 45 L/m^2.

Orifice Size

The diameter of the holes was set at 3 mm to lower the risk of clogging.

Pressure in pipes

The pressure was set at approximately 50 kPa (7.25 psi).

Spacing of feeding pipes and distance between holes

The spacing between the holes was set at 2.5 m. The separation of the pipes was also set at 2.5 metres. Calculations showed that a pulse of 45 L/m^2 aplied to such a network would ensure that the entire area would be covered.

Duration of application

It was estimated that the pulse should last 1 hour 51 minutes (cycle of four hours), which results in a flow through the holes of 2.6 litres per minute (see Appendix 4).

SUMMARY

According to the calculations, the design for the feeding pipes is the following:

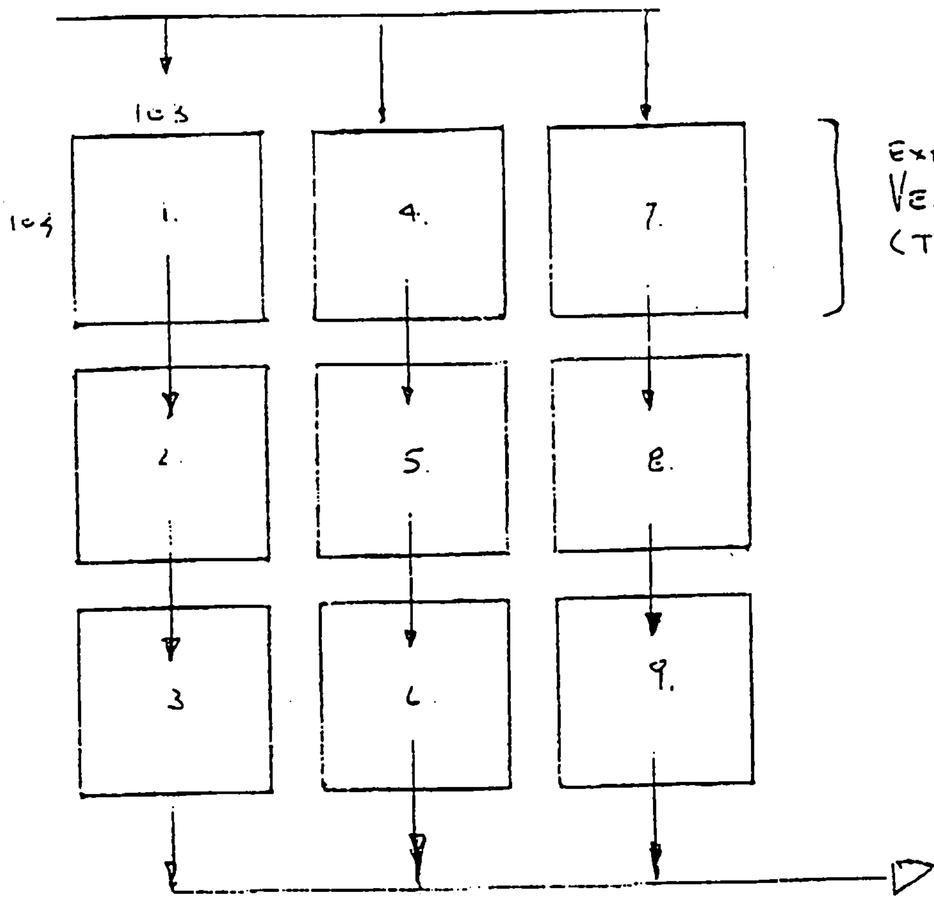
Spacing of holes = 2.5 m
Spacing of pipes = 2.5 m
Diameter of hole = 3mm
Pressure at hole = 50 kPa (7.25 psi)
Pulse duration = 1 hour 51 minutes every 4 hours
Flow through hole during pulse = 2.6 L/min

APPENDICES

APPENDIX 1

Sketch of proposed design

CASE 1. PLAN VIEW



EXTRA VENTILATION (THIS SET ONLY.)

PUMPING STILL REQUIRED BETWEEN CELLS

SHOULD HAVE THE ABILITY TO REVERSE FEEDING — IF FEASIBLE IF NOT - FORGET AT THIS STAGE - A FINAL DESIGN DETAIL AND GETS VERY COMPLICATED.

2 April 1997

APPENDIX 2

Drainage Calculations - no gravel layer

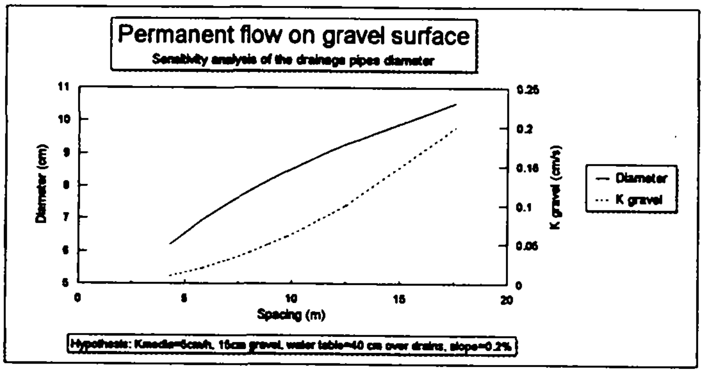
Permanent flow - drains placed on a gravel layer - h=40cm:

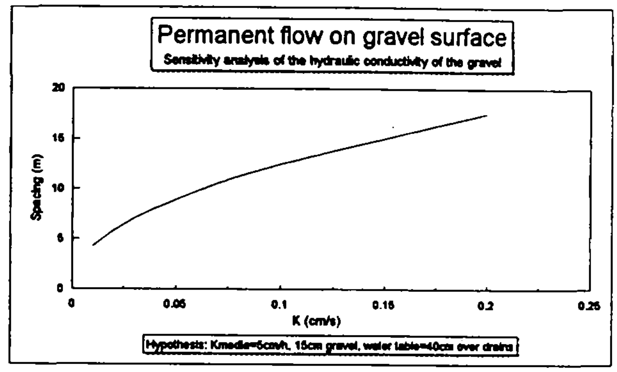
Entered values:

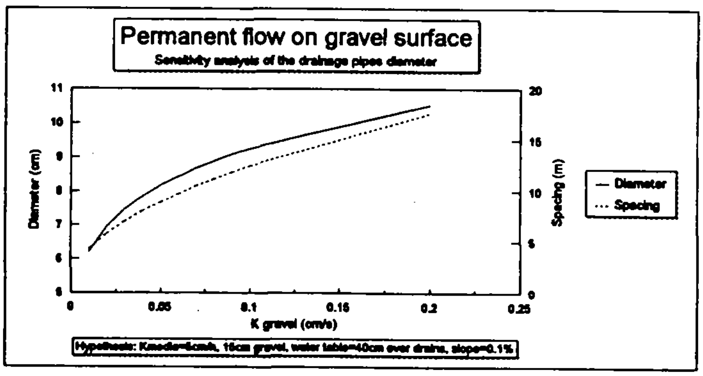
h(m)=	*0.4*	*K Strickler=*	*67*	*Half-cell width (m)=*	*42*
qc (l/m2/d) =	*270*	*n Manning=*	*0.015*	*Length of cell (m)=*	*84*
Kmedia (cm/h)=	*5*	*Slope (m/m)=*	*0.002*	*Gravel cost per m3=*	*18*
Gravel depth d (m)=	*0.15*	*Pipes cost per m=*	*15*		

Sensitivity analysis for the hydraulic conductivity of the gravel:

Kgravel cm/s	Kgravel m/s	qc m3/m2/s	Spacing m	Flow m3/s	Diameter cm	nb drains -	L drains m	Cost $	Gravel cost $	Total/cell $	For 9 cells $
0.01	1.0E-04	3.1E-06	4.27	5.60E-04	6.2	20	1595	23920	19051	42971	386741
0.02	2.0E-04	3.1E-06	5.79	7.60E-04	6.9	15	1173	17596	19051	36648	329829
0.03	3.0E-04	3.1E-06	6.99	9.18E-04	7.5	13	1001	15016	19051	34067	306605
0.04	4.0E-04	3.1E-06	8.02	1.05E-03	7.8	11	836	12537	19051	31588	284294
0.05	5.0E-04	3.1E-06	8.92	1.17E-03	8.2	10	751	11261	19051	30313	272813
0.06	6.0E-04	3.1E-06	9.75	1.28E-03	8.4	9	668	10024	19051	29075	261678
0.07	7.0E-04	3.1E-06	10.51	1.38E-03	8.7	8	588	8819	19051	27871	250835
0.08	8.0E-04	3.1E-06	11.21	1.47E-03	8.9	8	582	8734	19051	27786	250071
0.09	9.0E-04	3.1E-06	11.88	1.56E-03	9.1	8	577	8655	19051	27706	249353
0.1	1.0E-03	3.1E-06	12.61	1.64E-03	9.3	7	500	7507	19051	26558	239021
0.2	2.0E-03	3.1E-06	17.61	2.31E-03	10.5	5	332	4979	19051	24031	216275







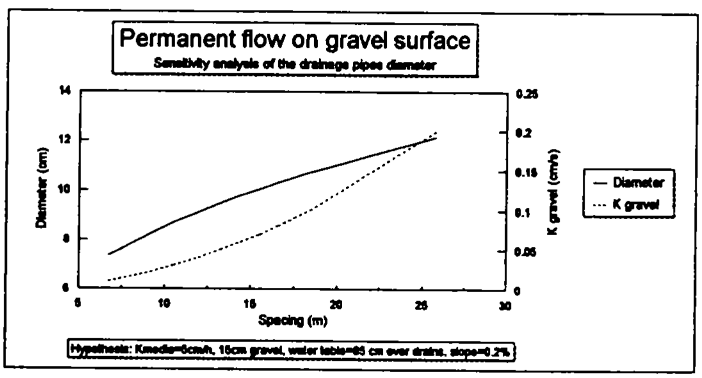
2B. Permanent flow - drains placed on a gravel layer - h=65cm:

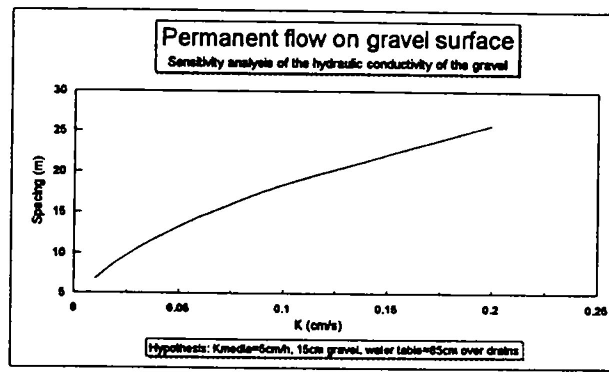
Entered values:

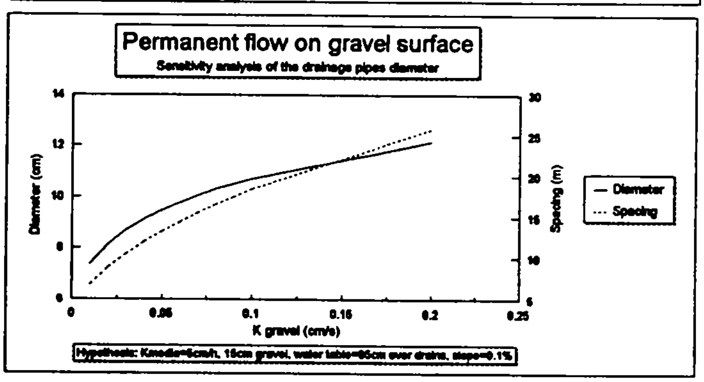
h(m)=	*0.85*	*K Strickler=*	*67*	*Half-cell width (m)=*	*42*
qc (l/m2/d) =	*270*	n Manning=	0.015	*Length of cell (m)=*	*84*
Kmedia (cm/h)=	*5*	*Slope (m/m)=*	*0.002*	*Gravel cost per m3=*	*18*
Gravel depth d (m)=	*0.15*	*Pipes cost per m=*	*15*		

Sensitivity analysis for the hydraulic conductivity of the gravel:

Kgravel cm/s	Kgravel m/s	qc m3/m2/s	Spacing m	Flow m3/s	Diameter cm	nb drains -	L drains m	Cost $	Gravel cost $	Total/cell $	For 9 cells $
0.01	1.0E-04	3.1E-06	6.74	8.85E-04	7.3	13	1004	15065	19051	34116	307045
0.02	2.0E-04	3.1E-06	8.84	1.16E-03	8.1	10	752	11274	19051	30325	272928
0.03	3.0E-04	3.1E-06	10.52	1.38E-03	8.7	8	588	8817	19051	27868	250814
0.04	4.0E-04	3.1E-06	11.98	1.57E-03	9.1	8	576	8643	19051	27694	249248
0.05	5.0E-04	3.1E-06	13.27	1.74E-03	9.5	7	495	7427	19051	26478	238302
0.06	6.0E-04	3.1E-06	14.45	1.90E-03	9.8	6	417	6260	19051	25311	227800
0.07	7.0E-04	3.1E-06	15.53	2.04E-03	10.0	6	411	6162	19051	25213	226918
0.08	8.0E-04	3.1E-06	16.55	2.17E-03	10.3	6	405	6070	19051	25122	226094
0.09	9.0E-04	3.1E-06	17.51	2.30E-03	10.5	5	332	4987	19051	24038	216341
0.1	1.0E-03	3.1E-06	18.42	2.42E-03	10.7	5	328	4919	19051	23970	215728
0.2	2.0E-03	3.1E-06	25.80	3.39E-03	12.2	4	233	3492	19051	22543	202889







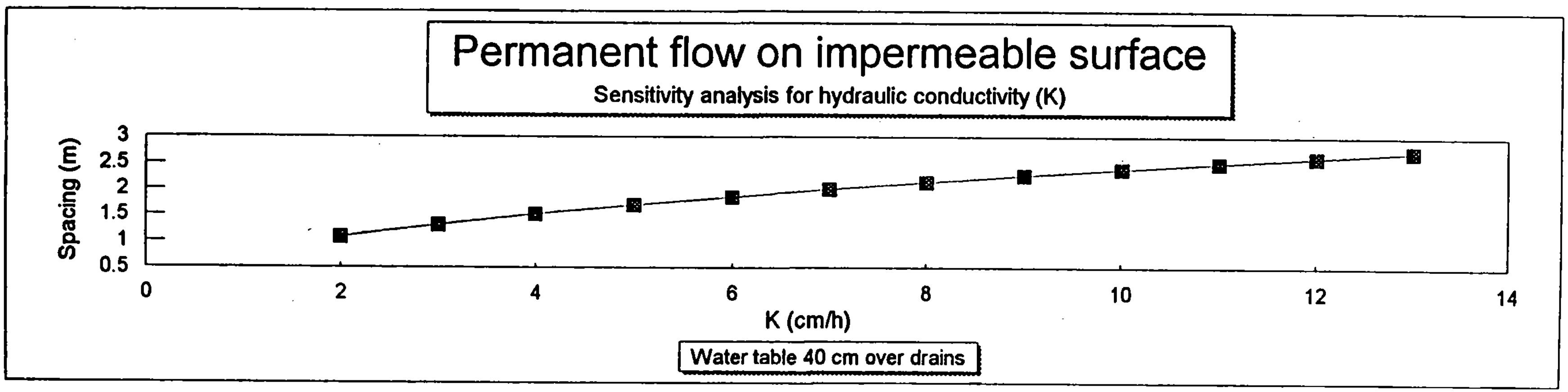
Permanent flow - drains placed on impermeable surface - h=40 cm

Entered values:

h (m)=	*0.4*	*Half-cell width =*	*42*	*Cell length (m)=*	*84*
K(cm/h)=	*5*	*K Strickler =*	*67*	n Manning	0.015
qc(l/m2/d)=	*270*	*Pipes cost per m=*	*15*	*slope (m/m)=*	*0.002*

Hydraulic conductivity (K) variation:

K	K	qc	Spacing E	Q	D	nb drains	Length drains per cell	Cost/cell	For 9 cells
cm/hr	m/s	m3/m2/s	m	m3/s	cm	-	m	$	$
2	5.6E-06	3.1E-06	1.07	1.40E-04	3.7	79	6552	98276	884484
3	8.3E-06	3.1E-06	1.31	1.71E-04	4.0	65	5375	80626	725636
4	1.1E-05	3.1E-06	1.51	1.98E-04	4.2	56	4620	69293	623636
5	**1.4E-05**	**3.1E-06**	**1.69**	**2.21E-04**	**4.4**	50	4116	**61735**	**555616**
6	1.7E-05	3.1E-06	1.85	2.42E-04	4.5	46	3779	56685	510167
7	1.9E-05	3.1E-06	2.00	2.62E-04	4.7	43	3526	52893	476036
8	2.2E-05	3.1E-06	2.13	2.80E-04	4.8	40	3275	49120	442080
9	2.5E-05	3.1E-06	2.26	2.97E-04	4.9	38	3106	46590	419312
10	2.8E-05	3.1E-06	2.39	3.13E-04	5.0	36	2938	44072	396648
11	3.1E-05	3.1E-06	2.50	3.28E-04	5.1	34	2771	41564	374078
12	3.3E-05	3.1E-06	2.61	3.43E-04	5.2	33	2686	40287	362580
13	3.6E-05	3.1E-06	2.72	3.57E-04	5.2	31	2520	37795	340159



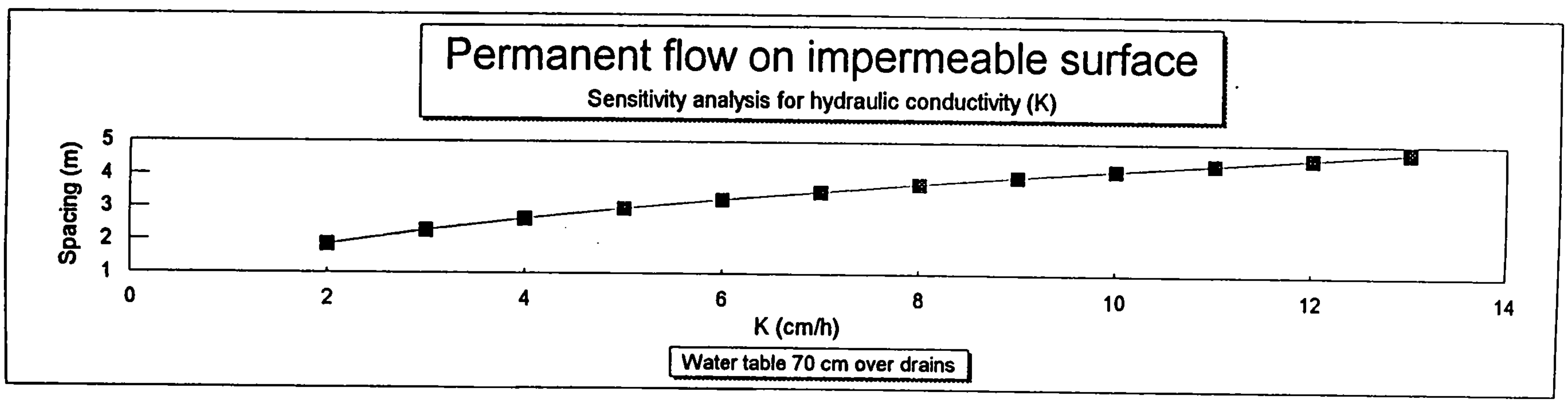
Permanent flow - drains placed on impermeable surface - h=70 cm

Entered values:

h (m)=	*0.7*	*Half-cell width =*	*42*	*Cell length (m)=*	*84*
K(cm/h)=	*5*	*K Strickler =*	*67*	n Manning	0.015
qc(l/m2/d)=	*270*	*Pipes cost per m=*	*15*	*slope (m/m)=*	*0.002*

Hydraulic conductivity (K) variation:

K cm/hr	K m/s	qc m3/m2/s	Spacing E m	Q m3/s	D cm	nb drains -	Length drains per cell m	Cost/cell $	For 9 cells $
2	5.6E-06	3.1E-06	1.87	2.45E-04	4.5	46	3778	56672	510048
3	8.3E-06	3.1E-06	2.29	3.00E-04	4.9	37	3023	45351	408160
4	1.1E-05	3.1E-06	2.64	3.46E-04	5.2	32	2604	39053	351476
5	**1.4E-05**	**3.1E-06**	**2.95**	**3.87E-04**	**5.4**	29	2350	**35256**	**317305**
6	1.7E-05	3.1E-06	3.23	4.24E-04	5.6	26	2100	31499	283492
7	1.9E-05	3.1E-06	3.49	4.58E-04	5.7	25	2013	30190	271714
8	2.2E-05	3.1E-06	3.73	4.90E-04	5.9	23	1846	27692	249228
9	2.5E-05	3.1E-06	3.96	5.20E-04	6.0	22	1761	26413	237719
10	2.8E-05	3.1E-06	4.17	5.48E-04	6.1	21	1676	25145	226307
11	3.1E-05	3.1E-06	4.38	5.75E-04	6.3	20	1592	23887	214980
12	3.3E-05	3.1E-06	4.57	6.00E-04	6.4	19	1509	22637	203732
13	3.6E-05	3.1E-06	4.76	6.25E-04	6.4	18	1426	21395	192555



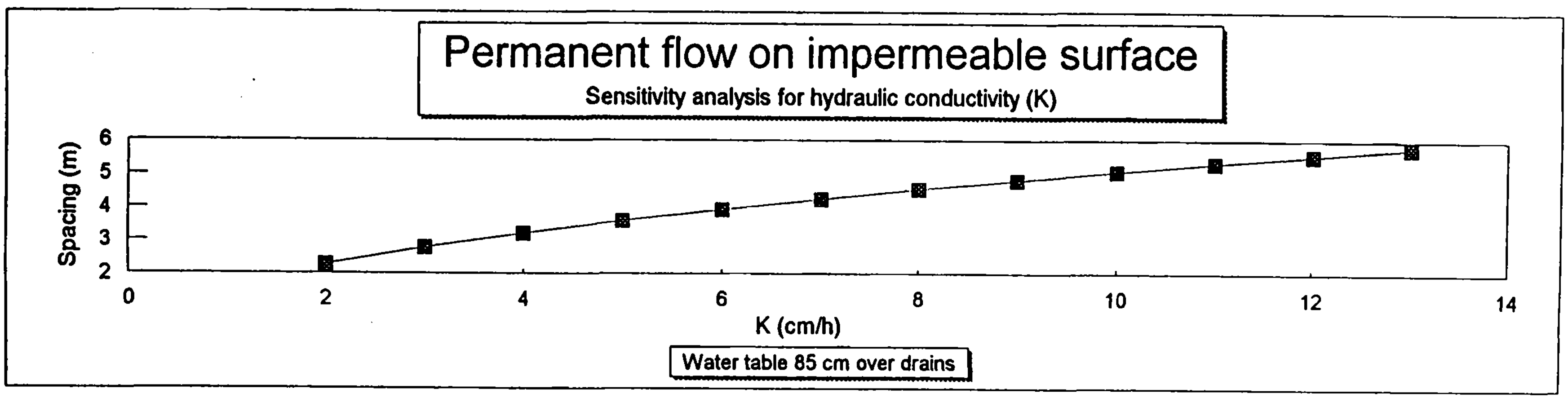
Permanent flow - drains placed on impermeable surface - h=85 cm

Entered values:

h (m)=	*0.85*	*Half-cell width =*	*42*	*Cell length (m)=*	*84*
K(cm/h)=	*5*	*K Strickler =*	*67*	n Manning	0.015
qc(l/m2/d)=	*270*	*Pipes cost per m=*	*15*	*slope (m/m)=*	*0.002*

Hydraulic conductivity (K) variation:

K	K	qc	Spacing E	Q	D	nb drains	Length drains per cell	Cost/cell	For 9 cells
cm/hr	m/s	m3/m2/s	m	m3/s	cm	-	m	$	$
2	5.6E-06	3.1E-06	2.27	2.98E-04	4.9	38	3106	46588	419292
3	8.3E-06	3.1E-06	2.78	3.64E-04	5.3	31	2518	37769	339922
4	1.1E-05	3.1E-06	3.21	4.21E-04	5.6	27	2181	32722	294496
5	**1.4E-05**	**3.1E-06**	**3.58**	**4.70E-04**	**5.8**	24	1930	**28950**	**260548**
6	1.7E-05	3.1E-06	3.93	5.15E-04	6.0	22	1762	26424	237820
7	1.9E-05	3.1E-06	4.24	5.57E-04	6.2	20	1595	23928	215351
8	2.2E-05	3.1E-06	4.53	5.95E-04	6.3	19	1510	22648	203832
9	2.5E-05	3.1E-06	4.81	6.31E-04	6.5	18	1425	21382	192436
10	2.8E-05	3.1E-06	5.07	6.65E-04	6.6	17	1342	20128	181148
11	3.1E-05	3.1E-06	5.32	6.98E-04	6.7	16	1259	18884	169958
12	3.3E-05	3.1E-06	5.55	7.29E-04	6.8	16	1255	18827	169447
13	3.6E-05	3.1E-06	5.78	7.58E-04	6.9	15	1173	17600	158398



APPENDIX 3

Drainage Calculations - with gravel layer

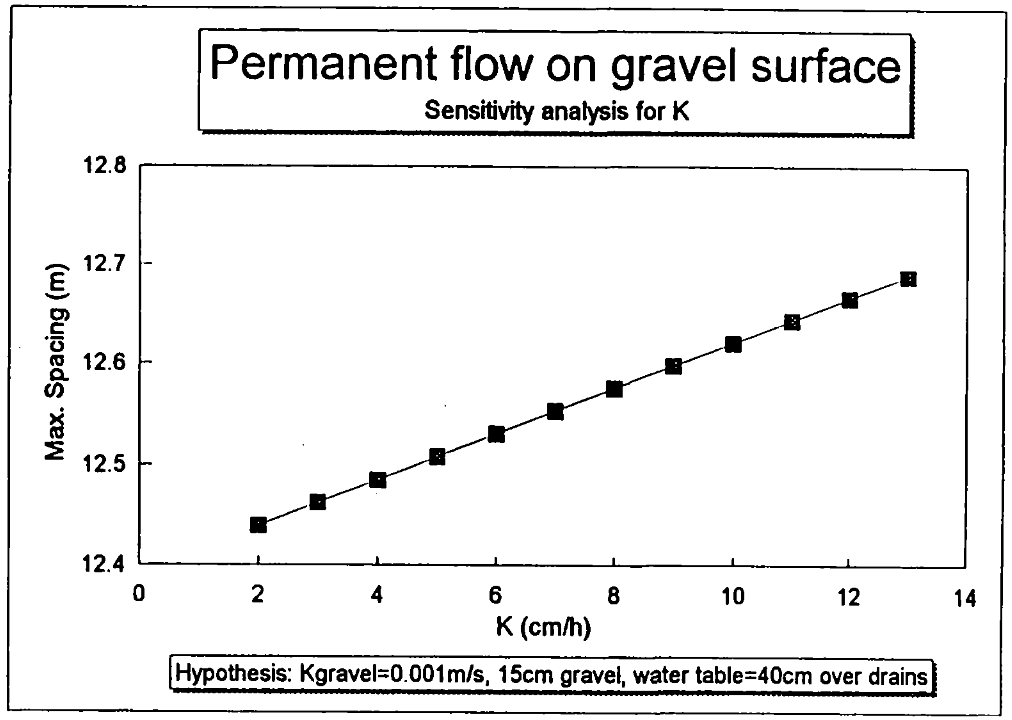
Permanent flow on gravel surface; variation of K media

Entered values:

h(m)=	*0.4*	*Kgravel (m/s)=*	*0.001*
qc(l/m2/d)=	*270*	*Gravel depth d (m)=*	*0.15*

Variation of K media:

K cm/hr	K m/s	qc m3/m2/s	E m
2	5.6E-06	3.1E-06	12.44
3	8.3E-06	3.1E-06	12.46
4	0.000011	3.1E-06	12.49
5	0.000014	3.1E-06	12.51
6	0.000017	3.1E-06	12.53
7	0.000019	3.1E-06	12.55
8	0.000022	3.1E-06	12.58
9	0.000025	3.1E-06	12.60
10	0.000028	3.1E-06	12.62
11	0.000031	3.1E-06	12.64
12	0.000033	3.1E-06	12.67
13	0.000036	3.1E-06	12.69



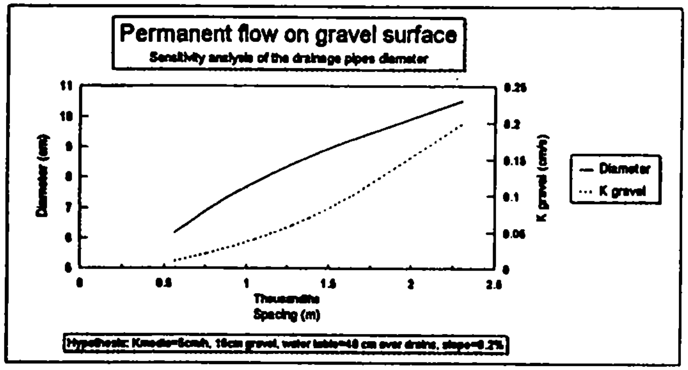
Permanent flow - drains placed on a gravel layer - h=40cm:

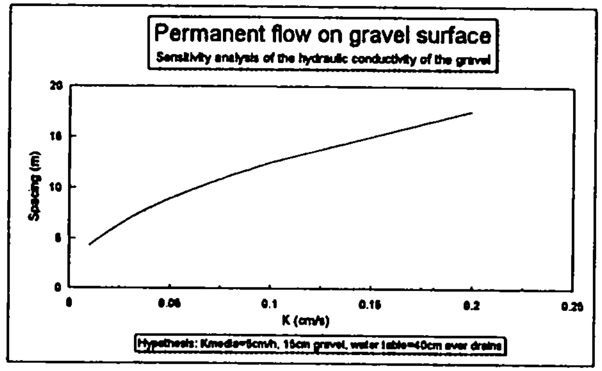
Entered values:

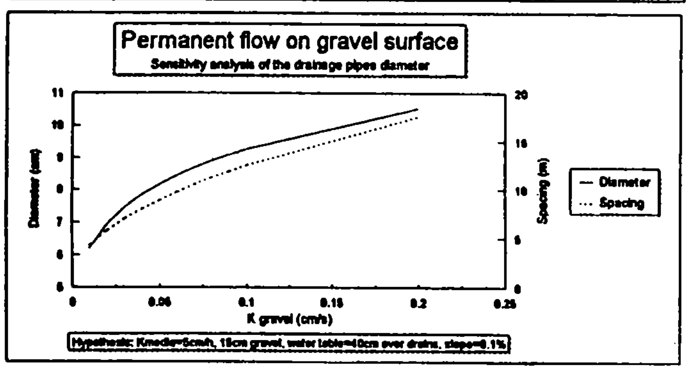
h(m)=	*0.4*	*K Strickler=*	*67*	*Half-cell width (m)=*	*42*
qc (l/m2/d) =	*270*	n Manning=	0.015	*Length of cell (m)=*	*84*
Kmedia (cm/h)=	*5*	*Slope (m/m)=*	*0.002*	*Gravel cost per m3=*	*18*
Gravel depth d (m)=	*0.15*	*Pipes cost per m=*	*15*		

Sensitivity analysis for the hydraulic conductivity of the gravel:

Kgravel cm/s	Kgravel m/s	qc m3/m2/s	Spacing m	Flow m3/s	Diameter cm	nb drains -	L drains m	Cost $	Gravel cost $	Total/cell $	For 9 cells $
0.01	1.0E-04	3.1E-06	4.27	5.60E-04	6.2	20	1595	23920	19051	42971	386741
0.02	2.0E-04	3.1E-06	5.79	7.60E-04	6.9	15	1173	17596	19051	36648	329829
0.03	3.0E-04	3.1E-06	6.99	9.18E-04	7.5	13	1001	15016	19051	34067	306605
0.04	4.0E-04	3.1E-06	8.02	1.05E-03	7.8	11	836	12537	19051	31588	284294
0.05	5.0E-04	3.1E-06	8.92	1.17E-03	8.2	10	751	11261	19051	30313	272813
0.06	6.0E-04	3.1E-06	9.75	1.28E-03	8.4	9	668	10024	19051	29075	261678
0.07	7.0E-04	3.1E-06	10.51	1.38E-03	8.7	8	588	8819	19051	27871	250835
0.08	8.0E-04	3.1E-06	11.21	1.47E-03	8.9	8	582	8734	19051	27786	250071
0.09	9.0E-04	3.1E-06	11.88	1.56E-03	9.1	8	577	8655	19051	27706	249353
0.1	1.0E-03	3.1E-06	12.61	1.64E-03	9.3	7	500	7507	19051	26558	239021
0.2	2.0E-03	3.1E-06	17.61	2.31E-03	10.5	5	332	4979	19051	24031	216275





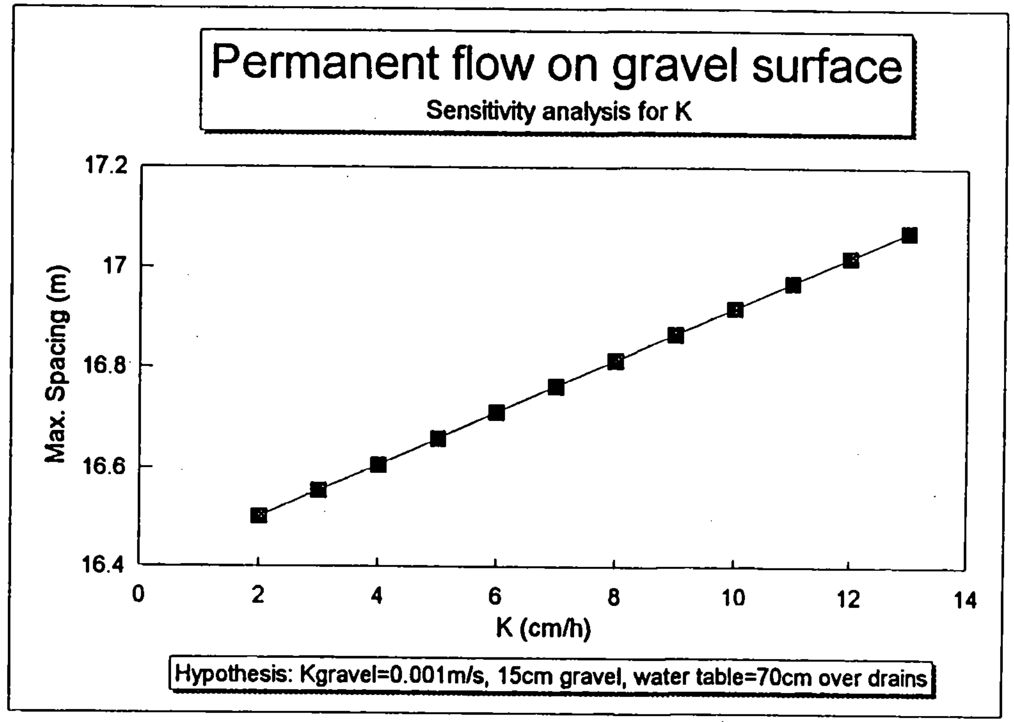


Permanent flow on gravel surface; variation of K media

Entered values: *h(m)=* 0.7 — *Kgravel (m/s)=* 0.001
qc(l/m2/d)= 270 — *Gravel depth d (m)=* 0.15

Variation of K media:

K cm/hr	K m/s	qc m3/m2/s	E m
2	5.6E-06	3.1E-06	16.50
3	8.3E-06	3.1E-06	16.55
4	0.000011	3.1E-06	16.61
5	0.000014	3.1E-06	16.66
6	0.000017	3.1E-06	16.71
7	0.000019	3.1E-06	16.76
8	0.000022	3.1E-06	16.81
9	0.000025	3.1E-06	16.87
10	0.000028	3.1E-06	16.92
11	0.000031	3.1E-06	16.97
12	0.000033	3.1E-06	17.02
13	0.000036	3.1E-06	17.07



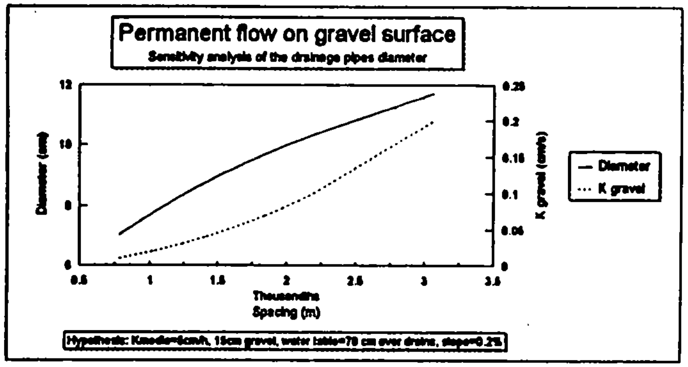
Permanent flow - drains placed on a gravel layer - h=70cm:

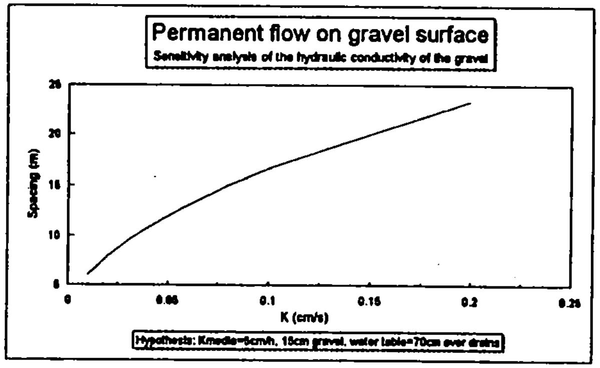
Entered values:

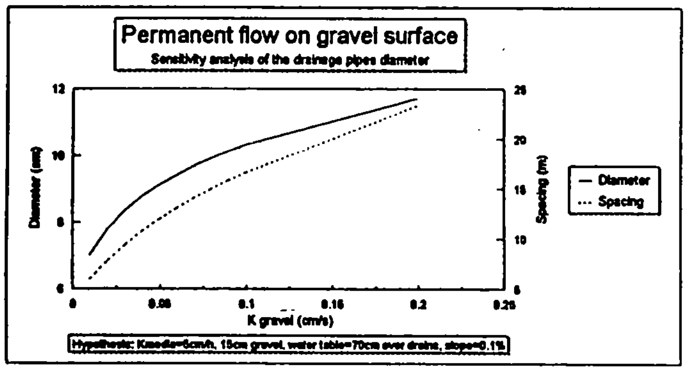
h(m)=	*0.7*	*K Strickler=*	*67*	*Half-cell width (m)=*	*42*
qc (l/m2/d) =	*270*	*n Manning=*	*0.015*	*Length of cell (m)=*	*84*
Kmedia (cm/h)=	*5*	*Slope (m/m)=*	*0.002*	*Gravel cost per m3=*	*18*
Gravel depth d (m)=	*0.15*	*Pipes cost per m=*	*15*		

Sensitivity analysis for the hydraulic conductivity of the gravel:

Kgravel cm/s	Kgravel m/s	qc m3/m2/s	Spacing m	Flow m3/s	Diameter cm	nb drains -	L drains m	Cost $	Gravel cost $	Total/cell $	For 9 cells $
0.01	1.0E-04	3.1E-06	5.97	7.83E-04	7.0	15	1171	17558	19051	36609	329480
0.02	2.0E-04	3.1E-06	7.90	1.04E-03	7.8	11	837	12556	19051	31607	284464
0.03	3.0E-04	3.1E-06	9.45	1.24E-03	8.3	9	671	10064	19051	29115	262036
0.04	4.0E-04	3.1E-06	10.78	1.42E-03	8.8	8	566	8786	19051	27837	250537
0.05	5.0E-04	3.1E-06	11.96	1.57E-03	9.1	8	576	8644	19051	27696	249261
0.06	6.0E-04	3.1E-06	13.04	1.71E-03	9.4	7	497	7451	19051	26502	238520
0.07	7.0E-04	3.1E-06	14.03	1.84E-03	9.7	6	420	6297	19051	25348	228136
0.08	8.0E-04	3.1E-06	14.96	1.96E-03	9.9	6	414	6214	19051	25265	227385
0.09	9.0E-04	3.1E-06	15.83	2.08E-03	10.1	6	409	6135	19051	25186	226677
0.1	1.0E-03	3.1E-06	16.66	2.19E-03	10.3	6	404	6061	19051	25112	226007
0.2	2.0E-03	3.1E-06	23.37	3.07E-03	11.7	4	243	3638	19051	22689	204199







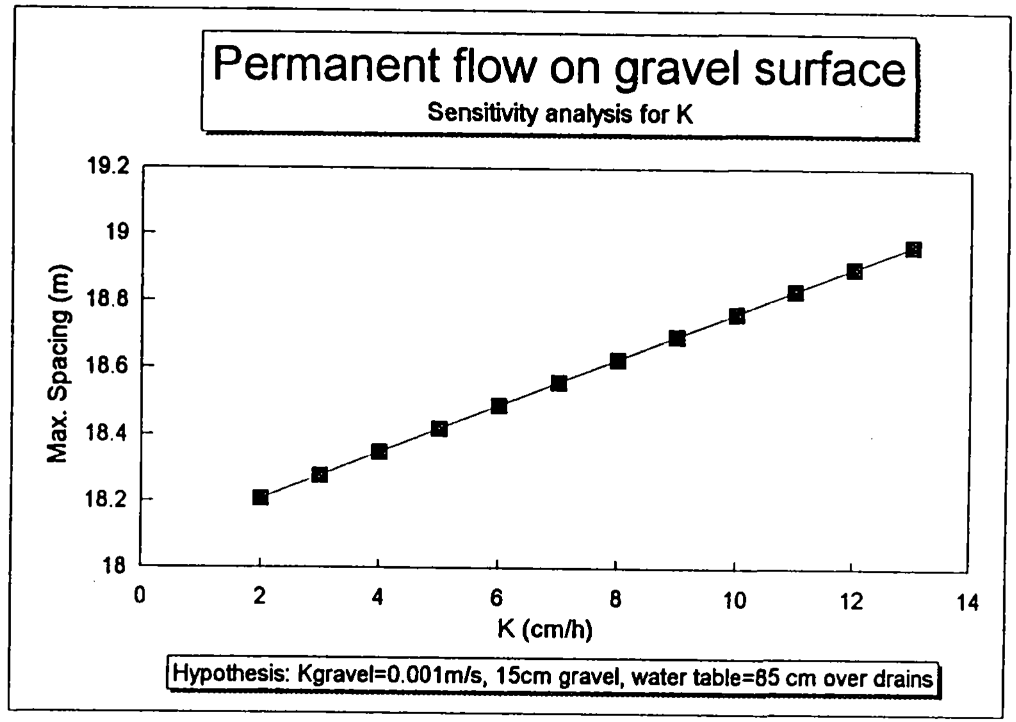
Permanent flow on gravel surface: variation of K media

Entered values:

h(m)=	*0.85*	*Kgravel (m/s)=*	*0.001*
qc(l/m2/d)=	*270*	*Gravel depth d (m)=*	*0.15*

Variation of K media:

K cm/hr	K m/s	qc m3/m2/s	E m
2	5.6E-06	3.1E-06	18.21
3	8.3E-06	3.1E-06	18.28
4	0.000011	3.1E-06	18.35
5	0.000014	3.1E-06	18.42
6	0.000017	3.1E-06	18.49
7	0.000019	3.1E-06	18.56
8	0.000022	3.1E-06	18.63
9	0.000025	3.1E-06	18.70
10	0.000028	3.1E-06	18.76
11	0.000031	3.1E-06	18.83
12	0.000033	3.1E-06	18.90
13	0.000036	3.1E-06	18.97



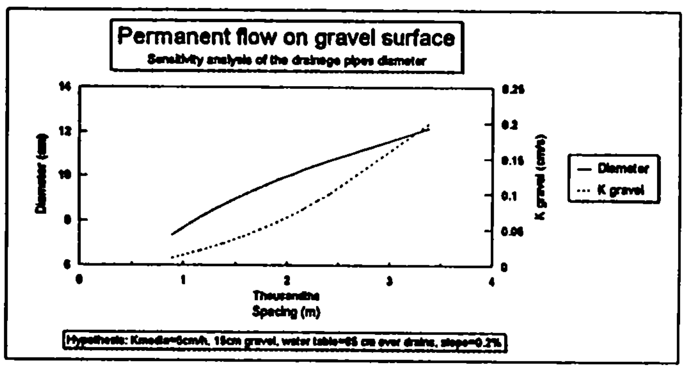
Permanent flow - drains placed on a gravel layer - h=85cm:

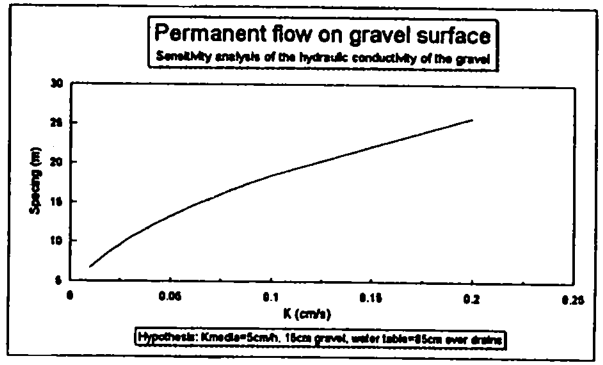
Entered values:

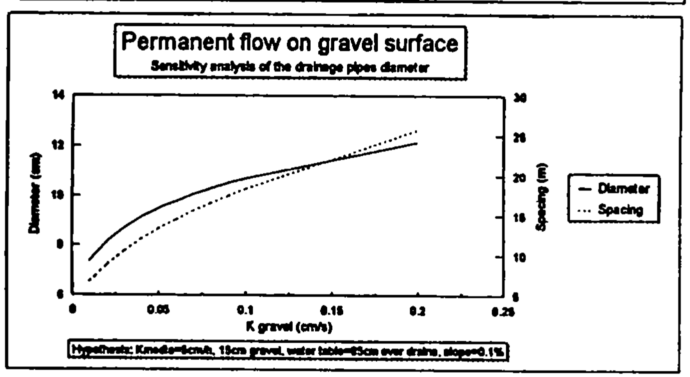
h(m)=	0.85	K Strickler=	67	Half-cell width (m)=	42
qc (l/m2/d) =	270	n Manning=	0.015	Length of cell (m)=	84
Kmedia (cm/h)=	5	Slope (m/m)=	0.002	Gravel cost per m3=	18
Gravel depth d (m)=	0.15	Pipes cost per m=	15		

Sensitivity analysis for the hydraulic conductivity of the gravel:

Kgravel cm/s	Kgravel m/s	qc m3/m2/s	Spacing m	Flow m3/s	Diameter cm	nb drains -	L drains m	Cost $	Gravel cost $	Total/cell $	For 9 cells $
0.01	1.0E-04	3.1E-06	6.74	6.85E-04	7.3	13	1004	15065	19051	34116	307045
0.02	2.0E-04	3.1E-06	8.84	1.16E-03	8.1	10	752	11274	19051	30325	272928
0.03	3.0E-04	3.1E-06	10.52	1.38E-03	8.7	8	588	8817	19051	27868	250814
0.04	4.0E-04	3.1E-06	11.98	1.57E-03	9.1	8	576	8643	19051	27694	249248
0.05	5.0E-04	3.1E-06	13.27	1.74E-03	9.5	7	495	7427	19051	26478	238302
0.06	6.0E-04	3.1E-06	14.45	1.90E-03	9.8	6	417	6260	19051	25311	227800
0.07	7.0E-04	3.1E-06	15.53	2.04E-03	10.0	6	411	6162	19051	25213	226918
0.08	8.0E-04	3.1E-06	16.55	2.17E-03	10.3	6	405	6070	19051	25122	226094
0.09	9.0E-04	3.1E-06	17.51	2.30E-03	10.5	5	332	4987	19051	24038	216341
0.1	1.0E-03	3.1E-06	18.42	2.42E-03	10.7	5	328	4918	19051	23970	215728
0.2	2.0E-03	3.1E-06	25.80	3.39E-03	12.2	4	233	3492	19051	22543	202889







APPENDIX 4

Calculation of collector main diameter

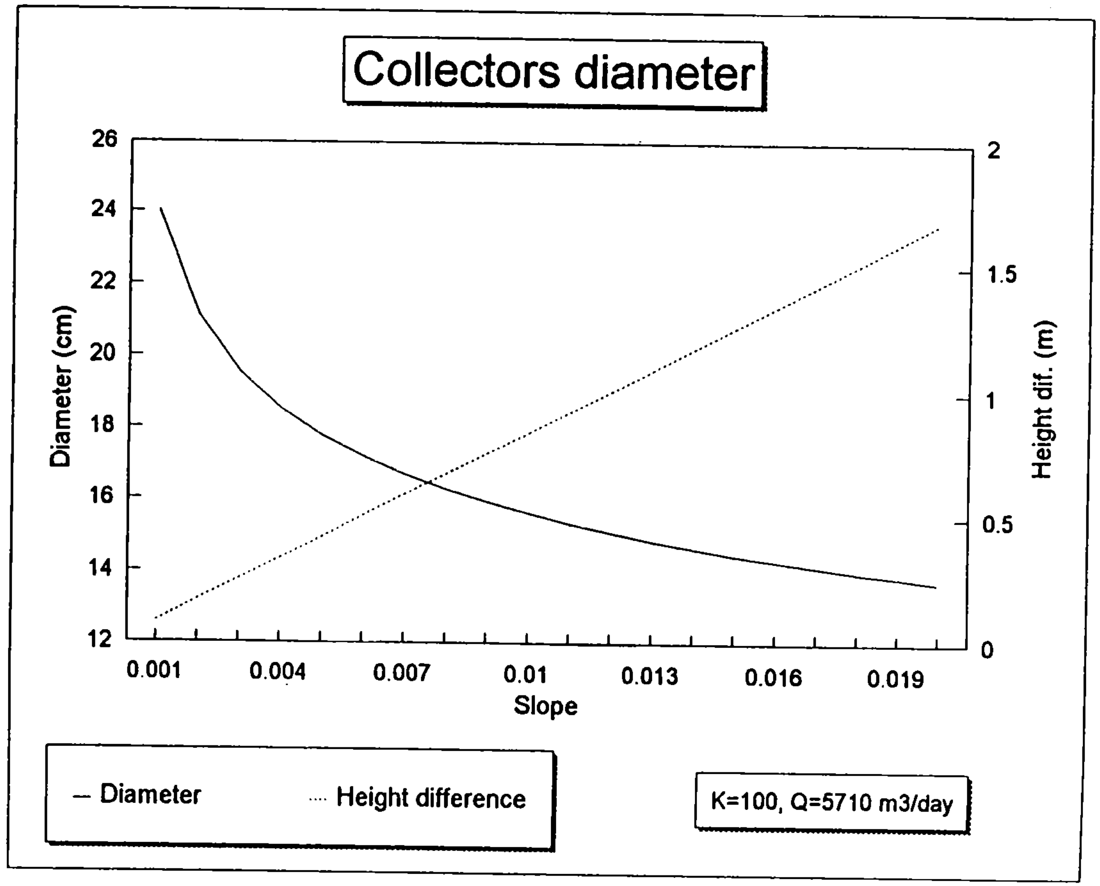
Size of the collector mains:

(based on flow split evenly among three collector mains)

Entered values:	*Total Flow (m3/d)*	*5710*		
	Flow in each series(m3/d)=	*1903.3333*	Flow(m3/s)=	0.022029
	K Strickler =	*100*	Manning n=	0.010
	Main length =	*84*		

Variation of the slope:

Slope (m/m)	Diameter cm	Height difference m
0.001	24.04	0.08
0.002	21.11	0.17
0.003	19.57	0.25
0.004	18.54	0.34
0.005	17.78	0.42
0.006	17.18	0.50
0.007	16.69	0.59
0.008	16.28	0.67
0.009	15.92	0.76
0.01	15.61	0.84
0.011	15.34	0.92
0.012	15.09	1.01
0.013	14.86	1.09
0.014	14.66	1.18
0.015	14.47	1.26
0.016	14.30	1.34
0.017	14.13	1.43
0.018	13.98	1.51
0.019	13.84	1.60
0.02	13.71	1.68



APPENDIX 5

Irrigation (feed) Calculations

Wetting pattern: Queenstone Shale

Q(l/m2/pulse) =	*45*	The incoming flow area is the area that provides such a flow
Initial moisture (-) =	*0.19*	
Final moisture (-) =	*0.42*	The wet area is the area that is wet by such a flow

Variation of the area that provides the flow and effect on the wetting pattern:

Area to be covered (A)	q	r	Wetted area (B)	B/A
m2	m3/dose	m	m2	%
1.0	0.45	1.19	2.79	279
1.1	0.50	1.31	3.38	307
1.2	0.54	1.43	4.02	335
1.3	0.59	1.55	4.72	363
1.4	0.63	1.67	5.48	391
1.5	0.68	1.79	6.29	419
1.6	0.72	1.91	7.15	447
1.7	0.77	2.03	8.08	475
1.8	0.81	2.15	9.05	503
1.9	0.86	2.27	10.09	531
2.0	0.90	2.39	11.18	559
2.1	0.95	2.51	12.32	587
2.2	0.99	2.63	13.53	615
2.3	1.04	2.75	14.78	643
2.4	1.08	2.87	16.10	671
2.5	1.13	2.99	17.47	699
2.6	1.17	3.10	18.89	727
2.7	1.22	3.22	20.37	755
2.8	1.26	3.34	21.91	782
2.9	1.31	3.46	23.50	810
3.0	1.35	3.58	25.15	838
3.1	1.40	3.70	26.86	866
3.2	1.44	3.82	28.62	894
3.3	1.49	3.94	30.43	922
3.4	1.53	4.06	32.31	950
3.5	1.58	4.18	34.23	978
3.6	1.62	4.30	36.22	1006
3.7	1.67	4.42	38.26	1034
3.8	1.71	4.54	40.35	1062
3.9	1.76	4.66	42.51	1090
4.0	1.80	4.78	44.71	1118
4.1	1.85	4.90	46.98	1146
4.2	1.89	5.02	49.30	1174
4.3	1.94	5.13	51.67	1202
4.4	1.98	5.25	54.10	1230
4.5	2.03	5.37	56.59	1258
4.6	2.07	5.49	59.13	1286
4.7	2.12	5.61	61.73	1313
4.8	2.16	5.73	64.39	1341
4.9	2.21	5.85	67.10	1369
5.0	2.25	5.97	69.86	1397
5.1	2.30	6.09	72.69	1425
5.2	2.34	6.21	75.57	1453
5.3	2.39	6.33	78.50	1481
5.4	2.43	6.45	81.49	1509
5.5	2.48	6.57	84.54	1537
5.6	2.52	6.69	87.64	1565
5.7	2.57	6.81	90.80	1593
5.8	2.61	6.93	94.01	1621
5.9	2.66	7.05	97.28	1649
6.0	2.70	7.16	100.61	1677
6.1	2.75	7.28	103.99	1705
6.2	2.79	7.40	107.42	1733
6.3	2.84	7.52	110.92	1761
6.4	2.88	7.64	114.47	1789
6.5	2.93	7.76	118.07	1816
6.6	2.97	7.88	121.73	1844
6.7	3.02	8.00	125.45	1872
6.8	3.06	8.12	129.22	1900
6.9	3.11	8.24	133.05	1928
7.0	3.15	8.36	136.94	1956
7.1	3.20	8.48	140.88	1984
7.2	3.24	8.60	144.87	2012
7.3	3.29	8.72	148.92	2040
7.4	3.33	8.84	153.03	2068
7.5	3.38	8.96	157.20	2096
7.6	3.42	9.07	161.42	2124
7.7	3.47	9.19	165.69	2152
7.8	3.51	9.31	170.02	2180
7.9	3.56	9.43	174.41	2208

Feeding pipes: spacing, hole diameter, pressure, pulse duration

Holes spacing (m)=	*2.5*	Feeding area (m2)=	6.25
Pipes spacing (m)=	*2.5*		
Pulse (l/m2/dose)=	*45*	Pulse per hole (l/dose)=	281.25
Pressure (bar)=	*0.5*	Pressure (psi)=	7.25
		Pressure (kPa)=	50
		Pressure water (m)=	5.10
Coefficient C=	*0.6*		

Variation of pulse duration:

Q=CS(2gH)^0.5

Pulse duration hours	Q l/min	Q l/h	S m2	Hole diameter mm
0.1	46.88	2812.50	0.0001302	12.87
0.2	23.44	1406.25	0.0000651	9.10
0.3	15.63	937.50	0.0000434	7.43
0.4	11.72	703.13	0.0000325	6.44
0.5	9.38	562.50	0.000026	5.76
0.6	7.81	468.75	0.0000217	5.26
0.7	6.70	401.79	0.0000186	4.87
0.8	5.86	351.56	0.0000163	4.55
0.9	5.21	312.50	0.0000145	4.29
1	4.69	281.25	0.000013	4.07
1.1	4.26	255.68	0.0000118	3.88
1.2	3.91	234.38	0.0000108	3.72
1.3	3.61	216.35	0.00001	3.57
1.4	3.35	200.89	9.30E-06	3.44
1.5	3.13	187.50	8.68E-06	3.32
1.6	2.93	175.78	8.14E-06	3.22
1.7	2.76	165.44	7.66E-06	3.12
1.8	**2.60**	**156.25**	**7.23E-06**	**3.03**
1.9	2.47	148.03	6.85E-06	2.95
2	2.34	140.63	6.51E-06	2.88

A COMPARISON OF ESTIMATED COSTS OF CONSTRUCTED WETLANDS AND OTHER ALTERNATIVES FOR IMPROVING LAGOON EFFLUENT QUALITY

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Cover Photo: Vertical flow, experimental constructed wetlands (SWAMP project) bordering the secondary treatment lagoon at the Niagara-on-the-Lake, Ontario sewage treatment plant. Photo courtesy of the Friends of Fort George.