DRAFT

SEWAGE COLLECTION AND DISPOSAL STUDY

CONCEPTION BAY SOUTH AREA

CANADA
DEPARTMENT OF REGIONAL ECONOMIC EXPANSION

GOVERNMENT OF NEWFOUNDLAND AND LABRADOR
DEPARTMENT OF MUNICIPAL AFFAIRS & HOUSING

FENCO

Foundation of Canada Engineering Corporation Limited

DRAFT

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INTRODUCTION

1.1 ISSUE

In the area of Conception Bay South, from Topsail to Seal Cove, some 11,000 people live under conditions of relatively dense development. Water, which is drawn from local individual wells, does not meet bateriological standards in about 50 per cent of the cases. Sewage facilities include either outdoor privies or septic tank systems, of which about 90 per cent and 70 per cent, respectively, are estimated to be improperly constructed.

Conception Bay South Area is included in the proposed St. John's Regional Water System. Once this system is expanded to supply water, in adequate quantity, quality and pressure, to Conception Bay South area, the local sanitary conditions will create a serious health hazard.

Planning of a sewage collection and disposal system(s) is required to eliminate the unsatisfactory sanitary conditions, and potential health hazards, in the Conception Bay South area.

1.2 GENERAL DESCRIPTION OF STUDY AREA

The study area consists of the Town of Conception Bay South, Paradise, Foxtrap, Seal Cove, and an area around and to the west of Three Island Pond, which is controlled under the St. John's Metropolitan Area Board. The area covered by the study may be considered as a satellite region of St. John's, and is shown on Drawing No. 1.

The constituent districts of the study area are administered by different authorities, but they have been considered as one for the purposes of a regional sewage collection and disposal system(s).

The total area considered is about 14 miles long and on the average of about three miles wide, lying on the South-Eastern shore of Conception Bay and extending along the shoreline for about 12 miles from Indian Pond to Topsail Cove. The shoreline is for the most part gently sloping with some abruptly falling sections of land. The sea bed has a smooth outward slope, but near Kelligrews area, it contains large rocks in random grouping. The Conception Bay Highway passes through the area, and connects it to other townships to the west and to the east of St. John's. The Conception Bay Highway has narrow shoulders and is designed for two lanes of traffic. It is 15 miles in length within the Conception Bay South study area. Most of the local roads are still unpaved.

The study area consists of ribbon development along the Conception Bay Highway and its sideroads. The areas served by local side roads on the south east of Conception Bay Highway are not, in general, densely populated. All this development has proceeded more or less unchecked by planning controls prior to 1970. Even now, Foxtrap and other unincorporated areas do not seem to exercise effective planning controls.

The Canadian National Railway runs north-south through the study area. From the Trans Canada Highway over head crossing the railway runs east of the Conception Bay Highway. It crosses the highway near the Greeley Town Road, and then runs along the shoreline.

There are three islands offshore, Little Bell Island, Kellys Island and Bell Island, the latter of which is the only populated island.

Conception Bay is used for recreation and fishing. The axis of the bay is oriented almost north-south. The depth distribution in the bay is characterized by a deep trench situated to the west of Bell Island, the depth of which is in excess of 600 feet. To the south and east of Bell Island, the bay is shallower. Tidal observations show a modest tidal range in the order of 3 feet. The bottom of the bay at its shallower sections show signs of ice erosion.

1.3 REVIEW OF PREVIOUS REPORTS

The requirement for a sewage collection and disposal system(s) in the Conception Bay South area is covered in the St. John's Urban Region Study (S.J.U.R.S.).

Based on Public Health surveys (see Section 2.3 of this Report), the S.J.U.R.S. report postulates a need for a modern sanitary sewage collection and disposal system for the area. Land requirements for properly installed and maintained on-site services (septic tanks) and the fact that roads are already constructed and do not require to be improved much beyond their present standard, substantiate this postulation.

In the context of the S.J.U.R.S. report, there are eleven major drainage basins in the area. Consequently, the alternatives for a sewage scheme "boil down to whether there should be one integrated system or separate systems in each drainage basin, or something in between". The report, subsequently, evaluates a scheme of two central treatment plants, one at Chamberlains West and one at Kelligrews, versus a scheme of individual local treatment plants.

Preliminary cost estimates including pumping stations, forcemains, and treatment facilities, indicate that the least capital expenditure for preliminary treatment only, is for the scheme of individual local facilities. However, when complete treatment is considered, the more favourable outlay is for the scheme of two central treatment plants.

It should be noted here that the S.J.U.R.S. report does not spell out the provisions made for outfalls and sludge disposal, nor does it specify the process and type of treatment.

Separate sewage treatment plants are proposed for the communities of Seal-Cove and Paradise.

As a result of their study the St. John's Urban Region Plan recommends that "Local development plans should be prepared immediately incorporating the general recommendations made on local roads, particularly in the Conception Bay South Area. These would form an input complementary to this report for the development of an integrated waste water disposal system for the area."

The report warns its readers that all the projects should be scrutinised individually and in detail prior to commitment.

1.4 SCOPE OF THE WORK

As part of an overall programme to provide the area of Conception Bay South with sanitary services (Water supply, sewage collection and disposal, refuse collection and disposal), the Department of Regional Economic Expansion together with the Department of Municipal Affairs and Housing of the Province of Newfoundland and Labrador retained Foundation of Canada Engineering Corporation Limited (FENCO) to perform a comprehensive study on the collection and disposal of sewage in this area.

The scope of the study was detailed in FENCO's submission of Conception Bay South Area Sewage Disposal Study. The Terms of Reference indicate that the study would lead to a report covering conceptual design of sewage disposal for the study area, covering collection methods and systems, suitable treatment process(es) and plant(s), and disposal of the effluent into the bay.

The boundaries of the area to be covered by the study have been determined generally from the St. John's Urban Region Study. Other information taken from that study has included present and projected populations and land usage over the 20 year design period to 1995. The quantity and characteristics of the sewage have been determined from present conditions, and this data has been developed to cover projections for the future, with reference having been made to the parallel study of water supply for the Conception Bay South Area, as part of the St. John's Regional Water System. The study has covered different systems and methods of collection and treatment of the liquid waste products, with their eventual disposal into Conception Bay by means of outfalls. Information has been collected on existing marine life, local fishing areas and oceanographic data, and this information, together with calculations regarding the dispersion of effluent products, has been used in formulating the proposed treatment and outfall installations.

Preliminary estimates have been made of construction costs for the alternative disposal schemes. Using these costs a phased program of implementation has been developed together with cost flows, with due consideration being paid to the long term schedule contained in the St. John's Regional Water System Study Report.

LOCAL CONDITIONS

2.1 PHYSICAL FEATURES

The area of the study consists of a series of land spurs which are perpendicular to the coastline and produce a generally hilly and uneven land formation with little in the way of flat and level ground. Maximum elevations are in the order of 700 feet with many of the high spots being in the 300 to 500 foot range above sea level. Within the area, most of which is tree covered, there are many muskeg sections, shallow ponds and innumerable small streams.

The underlying bedrock is composed of late precambrian rock which grades upwards to cambrian rock. The rock formation of the Conception Bay area is mainly composed of slate and argillites with some deep water deposits produced by turbulent currents. ?

Glacial till and drift cover the rock, and deposits of gravel and accumulations of peat are widespread except on the high ridges. The soils are deficient in carbonates and have been leached to an acidic state, so that the groundwater is relatively soft and acidic.

Along the shoreline are gently sloping beaches formed of cobbles, gravel, pebbles and some sand, with a few outcrops of rock forming small cliffs.

Manuels River, together with the coastline irregularities around Long Pond, and the steep spur which terminates at Manuels combine to form somewhat of a natural division within the Conception Bay South Area.

The climatic conditions of the area are strongly affected by the cold Labrador current. The area is exposed to northerly winds, and these combined with the Labrador current bring ice-bergs into Conception Bay during early spring. Midwinter temperatures are normally around the mid 20's, and the coastline waters are sometimes covered by ice. Normal winter snow precipitation is about 140 inches, and although rain occurs in winter the snow cover sometimes persists from mid-December to mid-April.

Spring is usually late, coming towards the end of May or early June, with temperatures being held down in the mid-40^{o}F range by the cold onshore winds. The summers are short and humid, temperatures being from 60^{o} to 80^{o}F.

2.2 EXISTING SERVICES

Water supply in the area consists of individual wells, both of the dug and drilled type, and the water is generally acidic. There are proposals for a piped water supply system as part of a regional scheme, and these are covered in the report for the St. John's Region Water System study.

Where sanitation facilities do not include privies, the sanitary sewage is collected and treated anaerobically in individual septic tanks. The liquid phase of the sewage overflowing from these tanks is disposed of in drainage (soak-away) tiled fields, stone-filled trenches and pits.

The solid phase of the sewage is bio-degraded in the septic tanks (if and when maintained properly) and is occasionally hauled away by tank trucks. Prior to 1970, there was little in the way of formal planning or building approval for new installations, but since 1970 all new installations require approval by the town Engineer and the Health Department. An exception to this requirement is Foxtrap, where limited control is only exercised and consequently development is fairly sub-standard.

Another important factor related to present arrangements for sewage disposal within the study area is that there is no control of the cleaning and maintenance of the septic tanks and soak-away fields. The storm water drainage system in the area is by and large natural, except for some culverts that were built under roads. The area is served by one main paved road, the Conception Bay Highway which is a two lane road with narrow shoulders. Most of the local roads within the area are unpaved.

A major power distribution line runs more or less parallel to the coast in a South-East axis to Conception Bay Highway.

2.3 HEALTH HAZARDS

As mentioned previously, the study area developed (until 1970) without building controls. This kind of development coupled with uncontrolled installation of individual water supply and sewage disposal systems, resulted in a sanitary problem.

Many of the water supply wells are poorly located in respect to the sewage systems, natural drainage and high water tables, and in some cases they are in proximity to barns and refuse piles.

Disposal of sanitary sewage relies on solids separation and degradation in septic tanks, and effluent seepage in soak-away fields or pits . Poor maintenance resulted in solids discharges in the effluent thereby clogging the seepage medium. There are many places in which effluent discharges can be seen seeping and flowing in open cuts and shallow ditches. Some of the older houses discharge raw sewage into totally inadequate subdrains and drainage pits. The common earlier practice was to use a septic tank of 200 gallon capacity, from which a 20 to 30 feet long pipe or trench led into (usually) a 3 feet diameter by 3 feet depth stone-filled drainage pit.

A sanitary survey to determine the bacteriological quality of water in the study area was carried out by the Department of Health in early 1967 and again in late 1969. The findings are presented in Table 2.1.

TABLE 2.1

BACTERIOLOGICAL QUALITY OF WATER

ITEM	1967 SURVEY	1969 SURVEY
Number of samples collected	317 (100%)	492 (100%)
Samples of satisfactory quality	149 (47.0%)	159 (32.3%)
Sample of unsatisfactory quality	151 (47.6%	287 (58.3%)
Number of inconclusive samples	17 (5.4%)	46 (9.4%)

In addition, 105 samples were analysed for fecal coliform organisms. The results showed 20 samples (19%) to be positive.

A survey of the sewage disposal facilities (in 1967) disclosed that an estimated 90 per cent of the privies inspected were of unsatisfactory construction. Generally, unsanitary conditions existed around the privies with pits filled and odours and flies present. In a number of cases, privies were found to be located in high water tables and close to wells.

Of the septic tank systems inspected, an estimated 70% were improperly installed and half of these were not working properly.

Generally, the unsanitary conditions resulting from these systems range from direct or indirect drainage into roadside ditches, small streams, and wells. In some cases effluent discharges overflow onto other properties and roadways.

Drawing No. 2 shows areas with the more severe health hazards.

As of 1970 measures have been introduced for the Town of Conception Bay South, Paradise and the St. John's Metropolitan area such that new buildings are now regulated, and construction of houses is only permitted on lots of 15,000 square feet or over with minimum frontages of 75 feet, subject to satisfactory soil and water conditions These new regulations have to some extent been effective in preventing the situation from deteriorating, but of course they have not affected the pre - 1970 installations, nor the day to day upkeep and maintenance of the sewage and water supply systems. It should also be noted that within the study area there are communities, outside the Town of Conception Bay South, which are still not subject to desirable planning controls.

2.4 RECREATIONAL AREAS

Recreational activities within the study area are for the most part carried out along, or near to the shoreline. There are several small harbours at which pleasure boating is based.

The Royal Newfoundland Yacht Squadron at Long Pond is the most widely recognized centre for motor yachts, its membership including local residents and many from the St. John's area. Many small boats are owned privately and used for leisure time fishing purposes. Coves and wharves along the shoreline serve as mooring areas for these boats, and many people travel by boat into the area to fish, as it is known to be one of the best fishing grounds in Conception Bay. Tuna, cod, and sea trout fishing are all major attractions. Although no sailing clubs are located within the area, there is a club based nearby at Holyrood.

The water along the shore stays fairly cold even in summer, but in spite of this many people are attracted to Topsail beach, and one or two other beaches for picnicing but virtually no swimming due to cold water. There are two children's playgrounds in the area, and a third one is being constructed at Kelligrews. Facilities at the playgrounds include softball and soccer pitches, swings, a swimming pool and private areas. A swimming pool and stadium are planned for Killigrews. The coastal drive through the area is popular with residents and tourists alike, since it provides varied scenery and picturesque traditional fishing areas within easy reach of St. John's.

2.5 FISHING ACTIVITIES

The communities along the south-eastern shore of Conception Bay derive their origins primarily from fishing and agricultural activities. Although the present economy of the area is based on St. John's, the old ties with the sea still exist. While from a commercial point of view the fishing industry is tapering off and contributes little to the area, with only a small number of people relying for their livelihood on fishing, a much larger number of people do part-time fishing.

Cod, salmon and lobster form the main catches in the waters immediately off the shoreline of the study area. For these types of fish the area is one of the best fishing grounds in Conception Bay, and in addition to residents of the area who participate in this activity people from Port au Grave come to the area for cod and salmon. The main fishing grounds are shown on Drawing No. 2. In addition to the types already mentioned, relatively large amounts of capelin are taken in late Spring from the reaches of the Upper Gullies and Seal Cove area, and other fish caught in the area include plaice, mackerel and herring. The capelin are used both for food and for fertilizing agricultural areas.

In general, the part-time fishermen work either in the local communities or elsewhere in the St. John's region. They haul their nets mainly during the evenings and weekends, and keep some of their catches for their own consumption and sell the balance. Along with those who fish either full or part-time are many who fish purely for recreational purposes. As already mentioned, the Tuna fishing boats are based at Long Pond, and in addition to this activity a significant number of residents and tourists fish for cod and sea trout.

In summary, fishing is no longer a major economic activity in the direct sense for the area, but it institutes an important feature to the communities and as such contributes indirectly to the local economy.

Table 2.2 illustrates the fishing activity in the study area by presenting statistical data on quantity and variety of fish catches (based on records provided by the Fisheries Department).

TABLE 2.2

STATISTICAL DATA ON FISH CATCHES

VARIETY OF FISH	YEAR OF CATCH	AMOUNT OF CATCH in 1,000 lbs.
COD	1969	619
	1970	546
	1971	224
	1972	326
	1973	598
HERRING	1969	22
	1970	19
	1971	5
	1972	36
	1973	0
SALMON	1969	12
	1970	28
	1971	15
	1972	18
	1973	71
LOBSTER	1969	13
	1970	14
	1971	18
	1972	27
	1973	57
MACKEREL	1969	0
	1970	1
	1971	3
	1972	1
	1973	4

STATISTICAL DATA ON FISH CATCHES

TABLE 2.2 Cont'd

VARIETY OF FISH	YEAR OF CATCH	AMOUNT OF CATCH in 1,000 lbs.
CAPELIN	1969	0
	1970	1
	1971	0
	1972	0
	1973	18
SQUID	1971	1
HADDOCK	1970	0
PLAICE	1969	11
	1970	0
	1971	0
	1972	0
	1973	6
GREENLAND HALIBUT	1969	3

2.6 MARINE LIFE

Input contributions to this section was made by Dr. J. M. Green of M.U.N. (as put forward in our proposal). His report on this matter is contained in Appendix A. Data on marine life produced by Dr. Green is based on a survey he carried out in February 1974. Three sites were included in this survey, two near Chamberlains Pond and the third near Kelligrews. The purpose of the survey was to determine the species diversity and relative abundance of organisms at these sites as a partial basis for evaluating the environmental consequences of placing sewage outfalls at or near these locations, and to provide baseline biological data for these sites.

The three sites of the survey and records of species present, location, depth, abundance (percent cover or relative) are shown in Drawings No. 3 and 4.

The findings of the marine life survey are quite typical of Conception Bay in particular and much of the Newfoundland coast in general. The most notable are probably the algae Chorda filium, forming extensive beds with large amounts of associated Ectocarpus, Dictyosiphon and Ceramium, and fish Of the latter species such as capelin, winter flounder, cunner, rock eels and radiated shanny would be expected to be abundant at these sites at various times of the year. In addition, such species as the lobster would be more active in the summer, and so many more would be seen. In general, it can be assumed that more organisms would be evident in summer.

The main differences between the three stations seem substrate dependent. Station 3 with the most cover from large boulders has the greatest abundance and variety of organisms while the unstable rock and shifting sand of Station 2 has the least.

All stations, especially 1 and 2 show evidence of severe winter ice scouring to depths of at least 20 feet. Station 3 appeared to be somewhat more protected from ice damage as indicated by the relatively more developed Fucus and astophyllum beds.

DESIGN DATA

3.1 LAND USE

In terms of urban development, the Conception Bay South area can be considered to comprise of two parts, the Topsail to Seal Cove portion along Route 3 and the Local Improvement District of Paradise. The communities of Topsail, Chamberlains, Manuels, Long Pond, Foxtrap, Kelligrews, Gullies and Seal Cove are so homogeneously developed and geographically similar that it is difficult to identify the beginning and end of each community. These communities, with the exception of Foxtrap, recently combined to form the Town of Conception Bay South.

Although originally a series of small rural communities contributing to the agricultural needs of St. John's, the area now can be classified neither as purely rural nor as urban development, but comprises of a conglomerate of old and new houses, commercial and institutional establishments (including schools, churches, restaurants, small stores and gas stations), pasture land, vacant land, and summer cottages. The Town of Conception Bay South is presently attacking the problem of how to best create planned and orderly development in the area.

Most of the working inhabitants of the area travel daily to St. John's and other parts of the region (eg. Holyrood) and in common with other regional "suburban" residents, they do the bulk of their shopping in St. John's, except for day to day needs which they obtain locally.

In the local Improvement District of Paradise most of the development is in the form of residential ribbon development extending along Topsail Road and a road adjacent to it, and is considered by the Urban Regional Study to be a residential suburb of the capital city, St. John's.

The Regional Plan policy for the Conception Bay South Area required that the development of a full range of local services consistent with the anticipated size of the community should be encouraged. When municipal services are installed, the permitted uses of the land shall include a wide range of residential land use densities compatible with prevailing socio-economic capabilities and needs, sub-urban, local and highway commercial facilities; parks and community facilities, elementary and high school facilities and other uses consistent with a suburban area.

Once the maximum level of infilling in a specific part of the Conception Bay South area has been achieved, then the development of limited local additional areas may be permitted provided it is within the urban service area designated on the Regional Plan and provided such development is accompanied by the installation of appropriate urban services, such as sewers, water mains and roads.

The Provincial Planning Office is preparing a detailed plan for this area but it is not available as yet. Based on their advice, the extent and type of development used for the purposes of this study, and for this particular area, are those contained in the Urban Regional Study and Plan, as shown in Drawing No. 1.

The Regional Plan proposes that for the local centre of Paradise the pattern of future development should be as follows:

(a) Residential development to be established at low densities;

(b) Commercial development to be limited and local, but is to include both retail and highway commercial functions.

3.2 POPULATION

3.2.1 General

The study region, in common with most urbanized areas, is growing in population thus making the adequacy of any plan for sewage collection and disposal largely dependent upon the rate of population growth. The problem, accordingly, is to forecast population in the design year, namely twenty years hence (1995).

A basic source of population figures can be found in the

Statistics Canada census, carried out every five years. Projection of future population, from this basic source, can be done by several different methods. However, for this project it was decided that population figures and projections as assembled for the St. John's Urban Regional Plan Study (S.J.U.R.P.S.) will serve as the basis for design. The Municipal Services plan of this study recommends a population distribution and projection, for that study, for the years 1971 to 1991 after careful consideration of four criteria, namely:

(i) Existing and required road capacity
(ii) Infilling capacity
(iii) School capacity
(iv) The necessity to provide for the needs of the local population (that wishes to live there.)

3.2.2 Recommended Design Population

Table 3.1 summarizes our anticipated population for the design period identified by communities and reflecting five year periods of incremental rate of growth. An "s" shaped growth curve, known as the logistic curve, which describes a theory of P.F. Verhulst in mathematical terms, was assumed to be representative of conditions in the study area.

These future growth patterns were reviewed and evaluated by key local officials, responsible for planning the key communities, and their views on the anticipated future growth patterns have been correlated in the dispersement of the future population.

TABLE 3.1

Projections of Population Growth

	Community	U.R.S.* Estimate	1971 Census	Projections 1975	1980	1985	1990	1995
	Seal Cove	1,080	706	780	950	1,100	1,170	1,220
	Gullies	1,160	728	920	1,350	1,450	1,520	1,570
	Kelligrews	2,010	2,046	2,300	2,690	2,690	2,690	2,690
	Foxtrap	1,630	908	1,100	1,250	1,600	2,050	2,110
	Long Pond	2,050	1,758	1,900	2,200	2,500	2,850	3,320
	Manuels	980	1,200	1,220	1,240	1,260	1,270	1,280
	Chamberlains Topsail	3,950	3,078	4,100	4,500	4,800	5,000	5,220
	Paradise	1,250	524	650	900	1,200	1,350	1,430
**	Topsail Road							740
**	Metropolitan Area							1,420
	Total population within the boundary of the Study Area.							21,000

* Urban region study estimate

** Proportionate parts of the estimated area populations

3.3 PROJECTED WATER USAGE

3.3.1 General

Forecasts for water usage within the study area form the basis for developing design volumes for sewage flows. It is, therefore, considered important to include in this report relevent information taken from the St. John's Regional Water System Study. In that study, it has been assumed that water use in the Conception Bay South area would follow the patterns experienced in the Town of Mount Pearl; (both areas being of sub-urban development nature). Accordingly, present water use and trends in Mount Pearl have served as the baseline for projecting future water consumption in the study area.

3.3.2 Residential Customers

The largest single consumer of water is the residential class. In projecting this class's future requirements for water, the following principal factors that influence increase and decrease in water use have been considered:

(a) Factors influencing increase in water use

Economic conditions within a community have a significant effect on residential customer water use trends. Compared with other Metropolitan areas, the study region has not been known to have a favourable economic climate (see Chapter 2.2). Recent development trends, however, indicate that efforts are being made to accelerate the economic conditions of the region. Experience has shown that water use will follow the general trend in the economic situation of a region.

(b) Factors influencing decrease in water use

The most significant factor in influencing decrease in water use is user charges. [7]

Changes in residential living patterns such as a marked shift from single family dwellings to multi-family units, and changes in established area occupancy such as urban redevelopment, are other factors that may cause a decrease in water use.

After assessing the above cited factors, it has been recommended that for design purposes of a regional system an increase in residential water use during the design period be allowed. A compounded increase of about 2 percent per year has been recommended. Under this premise, the residential water use at the end of the design period will be 54 GPCD compared to the current 34 GPCD as presently experienced in the Town of Mount Pearl.

3.3.3 Industrial and Commercial Customers

Industrial and commercial customers in the Town of Mount Pearl have been using water equivalent to 16 GPCD. Since Mount Pearl has basically reached saturation in its development, it has been recommended that this value of 16 GPCD be used through the design period for all suburban development areas.

This same value of 16 GPCD will be applied to suburban areas using water for agricultural needs in lieu of industry.

3.3.4 Public Customers

It has been assumed that basically the ratio of public services to population as it exists presently will be maintained through the design period. Accordingly, the water use by this customer class will remain in the order of magnitude 5 GPCD for the suburban developments.

3.3.5 Waste

All water supply systems experience waste through underground leakage. The aggregate amount of this waste may be from 10 to 25 percent, or more, of the total output. A system experiencing leakage of about 10 percent of its output would be considered a well managed one.

Under the supposition that a regional managing organization will be equipped and staffed to adequately repair pipe leaks, and accounting for a longer system of pipe lines but with controlled pressures, it has been assumed that waste could be maintained at no more than 15 percent of the total output. To provide flexibility in design, it has been recommended that about 17 percent of the total water output be alloted to this 'use'.

3.3.6 Summary

A summary of basic water design parameters as analysed in the previous sections is presented in Table 3.2.

TABLE 3.2

RATE OF WATER USE - 1995

CUSTOMER CLASS	SUBURBAN DEVELOPMENT
1. Residential	54 GPCD
2. Industrial & Commercial	16 GPCD
3. Public	5 GPCD
4. Waste	15 GPCD
5. Total	90 GPCD

3.4 PROJECTED SEWAGE FLOWS

Forecasts for sewage design flows were derived from the projected rates of water use. As shown in Table 3.2, the total rate of water use (for the design year of 1995) has been projected to be 90 GPCD. Of this quantity, it has been assumed that only 75 GPCD will reach the customers. Assuming that water uses that do not reach the sewer (eg. car washing, lawn watering..) are negligible, this total amount of 75 GPCD can be considered as sewage flow. To this value infiltration has to be added.

Sanitary sewers must be designed to carry unavoidable amounts of groundwater infiltration or seepage in addition to the sewage flows. Groundwater gains entrance to sewers through faulty pipe joints, defective Wye branches (for house connections), cracks or openings in manholes. It is obvious that infiltration rates should be kept at minimum since they affect the cost of essentially all components of sewage collection and disposal facilities. High workmanship standards should be adhered to and compression-type joints should have preference.

For design purposes, it is recommended that an infiltration rate of 10,000 GPD per mile of pipe be allowed. The equivalent per capita flow of this infiltration allowance will be 20 GPD, bringing the total sewage design flow to 95 GPCD.

The daily sewage flow for each community in the study area is presented in Table 3.3.

TABLE 3.3

DAILY SEWAGE FLOWS

COMMUNITY	SEWAGE FLOW, IGPD
Seal Cove	115,900
Gullies	149,150
Kelligrews	255,550
Foxtrap	200,450
Long trap	315,400
Manuels	121,600
Chamberlains) Topsail)	495,900
Paradise	135,850
Topsail Road	70,300
Metropolitan Board Area	134,900
TOTAL DAILY SEWAGE FLOW -IG	1,995,000

3.5 SEWAGE CHARACTERISTICS

3.5.1 General

Sewage characteristics that are of significant importance to aspects of treatment and disposal include BOD_5, suspended solids, nutrients, and coliform organisms.

(a) BOD_5 - is that fraction of the organic material in sewage used by aerobic organisms as a source of food under controlled conditions of incubation at $20^{\circ}C$ for a period of 5 days. It is measured in equivalent units of oxygen uptake by the aerobic organisms. The term BOD_5 is universally accepted to express the organic strength of sewage, even though in raw sewage it represents only carbonaceous organic material, not to its ultimate value.

(b) Suspended Solids - are the nonfiltrable residues in sewage. The fraction of these residues that is volatile denotes organic matter.

(c) Nutrients - phosphorus and nitrogen are considered as nutrients. Of the two the former is more critical as it contributes to the over-fertilization of surface waters. Whereas this situation could enhance the yield of fish, it may, on the other hand, accelerate the growth of such marine organisms as algae which tend to reduce and deplete the dissolved oxygen in the water.

(d) Coliform Organisms - are indicative of pathogens that may spread or transmit infection. Common modes of transmission (other than through drinking water) are through shellfish harvested from polluted water and bathing in water with high coliform count.

3.5.2 Methodology

The best method to establish the characteristics of sewage is by undertaking a sampling and analysis programme in which truly representative samples, that reflect the wide fluctuations in the quantity and quality of the sewage discharges, are collected and analysed. In the absence of a sewage collection system, as is the case in this project, common practice is to correlate sewage characteristic data from other similar communities.

On this project, FENCO's original intent was to assess and extrapolate the analytical data from the St. John's Sewage Disposal Study (Dree Project 3.2). It has been assumed that such detailed information as Dry Weather Flow, Peak and Minimum flow rate Factors, infiltration rates, would be available for different areas of the City, together with key sewage characteristics as outlined above. This type of information would have been analysed by us in conjunction with comparable data developed on the water study (Dree Project 3.1), and basic sewage characteristic criteria would have been extrapolated for Conception Bay South Area. The information we had received on this matter did not enable us to pursue our original intent. The following methodology has, therefore, been adopted:

(a) Analyse the sewage characteristic data recommended for St. John's (Dree Project 3.2);

(b) Analyse the sewage characteristic data available for Mount Pearl;

(c) Analyse sewage characteristic data available for other communities and endeavour to develop a probability curve for extrapolation purposes;

(d) Compare the analysis for items (a) through (c) above with universally accepted medium strength sewage characteristics.

3.5.3 St. John's Sewage Characteristic Data

Sewage characteristic data recommended for St. John's (Dree Project 3.2) are as follows:

- BOD_5 concentration 126 ppm
- Suspended solids concentration 163 ppm

It should be noted that comparison of concentrations is grossly inadequate since they are a product of sewage characteristics and water use (Dry Weather Flow). The presentation of sewage characteristics should be by the equivalent load of contaminant contributed by each person per day. Using our water use forecasts (Dree Project 3.1), the ratio of sewage flow to water use and infiltration rates as presented previously in Section 3.4 of this report, the above concentrations would be equivalent to the following order of magnitude of loads:

- BOD_5, 0.17 lbs. per capita per day
- Suspended solids, 0.22 lbs. per capita per day.

3.5.4 Mount Pearl Sewage Characteristic Data

Our analyses here have been based on the sampling programme carried out by Canadian-British Consultants Limited in January, 1972. Their findings show an average BOD_5 concentration of 65 ppm. This low level of BOD_5 could be attributed to the following factors:

(a) Very high infiltration which diluted the sewage strength. The flow measured during the sampling period averaged about 1.1 MGD, roughly twice the amount of water actually used by the consumers.

(b) The nature of the town, being essentially a residential suburb of the City of St. John's.

(c) The sampling technique which extended from 8:30 A.M. to 5:30 P.M. only, and which involved the collection of one sample every three (3) hours.

Using the BOD_5 concentration and flow figures as given above, the equivalent load would be:

- BOD_5, 0.10 lbs. per capita per day.

3.5.5. Other Communities

As can be deduced from the previous sections, the correlation and extrapolation of sewage characteristics from other communities is not a simple chore. Complicating the issue is the fact that sewage characteristics are measured in concentrations, and the interpretation to equivalent loads per capita is often missing due to lack of data such as connected population.

Sewage characteristic data that we did collect from other communities show a relatively high number of occurrences far from the expected mean value. Consequently, any probability distribution fitting these occurences would have a very large standard deviation. Furthermore, the occurrence of very small relative frequencies near the expected mean and larger relative frequencies further away appears to be anomalous. It could be that the sample data is too small to be conclusive.

3.5.6 Discussion and Recommendations

Universally accepted loads for medium strength sewage are as follows:

- BOD_5, 0.17 lbs. per capita per day.
- Suspended solids, 0.20 lbs. per capita per day.

The figures adopted for St. John's (Dree Project 3.2) seem to be within these limits. However, the survey of Mount Pearl sewage indicates a weaker wastewater.

It is our understanding that for design purposes a BOD_5 concentration of between 80 and 100 ppm has been recommended for Mount Pearl. This concentration would be equivalent to a load of between 0.125 and 0.155 lbs. per capita per day.

Data from other communities (and our own experience) indicate that in about 2/3rds of the cases the BOD_5 load is less than 0.17 lbs. per day. Regarding suspended solids, in about 50 per cent of the cases the load is less than 0.20 lbs. per capita per day.

Based on the above information and experience, and allowing for a conservative approach, it is recommended that the sewage characteristics as outlined in Table 3.4 be adopted for this project.

TABLE 3.4

PROJECTED SEWAGE CHARACTERISTICS

Constituent	Load lbs./c/day	Concentration ppm
BOD_5	0.16	170
Suspended Solids	0.18	190
Phosphorus*	-	6-8

* The annual quantity of phosphorus resulting from human excretion reportedly ranges from 0.5 to 2.3 lbs. per capita. Assuming the contribution to be the mean of these values, and allowing a similar value of contribution from other sources (such as limited synthetic detergents), the concentration of elemental phosphorus in the sewage would be in the range shown in the table.

SEWAGE COLLECTION SYSTEMS

4.1 METHODOLOGY

As previously described, the study area is about 14 miles long, its topography is undulating, its road systems are largely unplanned, the building developments are scattered in an unplanned pattern, streams and ponds intrude into the area. All of these factors make the planning of a sewerage system a complicated task. We have, therefore, taken the following approach in planning the sewerage system:

(a) Divided the study area into sewerage areas that more or less follow natural drainage patterns;

(b) Studied alternative methods for sewage collection;

(c) Applied the findings from item (b) above to the planning of a collection system for each sewerage area, viewing each such system as part of a regional scheme.

It is our considered opinion that the above methodology is the best rational approach to the design of sewage collection system(s) for an area such as Conception Bay South.

4.2 SEWERAGE AREAS

The study area can be viewed as comprised of two major sewerage basins, one north of Manuels River and the other south of the river. The former basin is less undulated whereas the latter is very undulated. Consequently, the basin north of Manuels River has been divided into 5 sewerage areas and the basin south of the river has been divided into 12 sewerage areas with pertinent design parameters for each area.

A brief description of the sewerage areas in a northerly direction from Seal Cove to Paradise is presented below.

4.2.1 Southern Basin

- Sewerage Area 1: Includes the area immediately to the north of Indian Pond. Its collection point is north of the community of Indian Pond.
- Sewerage Area 2: Includes the area between the community of Indian Pond and the estuary of Seal Cove Pond. Its collection point is at the estuary of Seal Cove Pond.
- Sewerage Area 3: Extends from Seal Cove to Lance Cove. Its main collection point is at the estuary of the Lower Gullies River.
- Sewerage Area 4: Extends from Lance Cove through Upper Gullies. Its main collection point is by the shoreline of Upper Gullies at Breakwater Road.
- Sewerage Area 5: Extends from Upper Gullies to Lower Gullies. Its collection point is at the estuary of the Lower Gullies River.
- Sewerage Area 6: Extends from the Lower Gullies to Kelligrews. Its major collection point is at the estuary of the Kelligrews River.

- Sewerage Areas 7, 8: Include the community of Kelligrews. The collection point of these areas is the low grounds at the shoreline where the pollution control plant (P.C.P.) for the southern basin is proposed. Area No. 7 discharges to the P.C.P. from the south, delivering the flows from areas 1 through 7. Area No. 8 discharges to the P.C.P. from the north, delivering the flows from areas 8 through 12.
- Sewerage Area 9: Includes parts of the communities of Kelligrews and Foxtrap. Its collection point is at the shoreline boundary of Foxtrap.
- Sewerage Area 10: Includes the community of Foxtrap. Its major collection point is at the estuary of Steadywater Brook.
- Sewerage Area 11: Extends from Foxtrap to Long Pond. Its major collection point is at Long Pond (South).
- Sewerage Area 12: Extends from Long Pond to Manuels. Its major collection point is at Long Pond (North).

4.2.2 Northern Basin

- Sewerage Area 13: Includes the community of Manuels East on both banks of the Manuels River. Its collection point is at the Manuels River west of Conception Bay Highway.
- Sewerage Area 14: Extends from Manuels to Chamberlains. It's collection point is just north of Chamberlains Pond where pollution control plant (P.C.P.) No. 2 is proposed.

- Sewerage Area 15: Includes parts of the communities of Chamberlains and Topsail. Its collection point is at the shoreline of Chamberlains.
- Sewerage Area 16: Extends from Topsail to Paradise, and includes the area around Topsail Pond and Three Island Pond which is administered by the St. John's Metropolitan Area Board. Its collection point is at the shoreline of Topsail.
- Sewerage Area 17: Includes most of Paradise. Its collection point is at Adams Pond.

It should be noted that the area east of Paradise Road (within the administrative boundary of Paradise) has not been included in any sewerage area because its natural drainage pattern is eastward. This area could be considered as part of an eastern region sewerage system.

4.3 ALTERNATIVE COLLECTION METHODS

Three alternative collection methods have been considered for this project, as follows:

- Conventional gravity - pressure sewers;
- Low pressure sewers;
- Vacuum sewers.

4.3.1 Conventional Gravity-Pressure Sewers

The gravity sewer is the most common and economical system for sewage collection. Under steady and uniform flow conditions, and provided the sewer has a free water surface (i.e. partially full sewer), the slope of the sewer will determine the flow velocity and discharge (along with the pipe roughness coefficient and hydraulic radius).

For a gradually varied flow, that is a steady flow whose depth varies gradually along the sewer line, the slope developed by the free water surface relative to the slope of the sewer will control the flow conditions. It follows then that the slope of sewers is a most important parameter in gravity systems. Therefore, an undulating or flat terrain could interfere with the economical design of gravity sewers. The latter form of a terrain would necessitate the use of lift stations to prevent excessively deep cuts, whereas the former would require pump stations and forcemains to overcome local ridges. Both of these conditions are common in the Conception Bay South area.

4.3.2. Low Pressure Sewers

This system was developed as a competitive alternative to gravity sewers and for servicing low-lying marginal lands.

Where the installation of gravity sewers becomes prohibitively costly due to deep trenches, excessive excavation in rock, high groundwater conditions; or where low-lying land falling away from the collector sewer requires the installation of duplicate sewers, one gravity in the direction of grade slope and one pressure back onto the collector sewer, the low pressure sewer system can be found to be an economic alternative.

The "low pressure sewer system" includes a holding tank, 120 gallons capacity, and a specially designed grinder pump of the semi-positive displacement type. The pump is self-priming and is equipped with an anti-siphon valve (to automatically correct problems from any vagrant vacuum pockets in the main), and a built-in check valve. An automatic reset thermal overload protector is built into the pump motor. The entire electrical system is built to meet the most stringent electrical codes. All sewage handling operations (as well as odours) are sealed inside the holding tank. The unit is installed inside and adjacent to a home. Its power consumption and cost is comparable to a home appliance.

Sewage collected in the holding tank is ground ~~grinded~~ by the pump while being pumped into the main. This grinding operation ensures that equipment and pipelines are not clogged. A typical house connection is 1¼ inch. The collecting main is usually 2-3 inches in diameter, depending on the number of houses connected to the system. The semi-positive displacement characteristic of the pump enables parallel operation of many units simultaneously into the collecting main. Tank volume and pump rate selection match domestic sewage flow volumes and patterns so as to eliminate overflows. However, emergency measures for periods of power failure or equipment outage should be provided. These measures could include by-passing provisions from the holding tank to an abandoned septic tank, soil absorption system, and the like.

Appurtenances on the collecting main are similar to those commonly used on forcemains.

The undulating nature of the area, which creates small pockets of low-lying land, warrants considering the "low pressure sewer system" as an alternative for such situations.

4.3.3 Vacuum Sewers

This system was originally developed for the vacuum transport of sewage from vacuum toilets. The latter is a specially designed fixture utilizing only 10% of the flush water used by the conventional toilet. Consequently, house connections are 1½ inches in diameter and the collecting main is 2 to 3 inches in diameter.

The system transports sewage under surges rather than continuous flow. The collection system is held under a constant vacuum of one half an atmosphere. "Plugs" of sewage are admitted into the collection system in a pre-determined ratio of atmospheric air to waste. The higher atmospheric pressure behind these "plugs" is rapidly propelled along the pipe with vacuum replacing gravity as the motive power. At the end of the collecting main is the vacuum collecting station which consists of vacuum pumps, a collection tank, and a vacuum reserve tank. Because the pressure difference between absolute vacuum and atmospheric pressure is only some 14.7 lbs. per square inch, the head to overcome friction and vertical lift in a vacuum sewer system is limited. To overcome this limitation, a series of vacuum collecting stations should be considered. The sewage from the vacuum collecting station can be discharged to a treatment plant, or into a conventional collection system.

To avoid the need for two sewers, one for the vacuum toilet and the other for all other fixtures in the home, the flow from the latter can be connected to a larger diameter vacuum sewer through a special interface valve.

Theoretically it is possible to use the vacuum sewer system as a transport means of sewage from conventional toilets and other home fixtures. However, experience with such applications is quite limited. Since transport is based on "plug" flow, a buffer tank would be required at the home to provide the dose of atmospheric air waste into the system. In addition, at the present stage of vacuum technology, the vacuum collecting stations are only capable of handling 30 gallons per minute.

In summary, without vacuum toilets, the capacity and advantages of the vacuum sewer system are severely curtailed. The system could be considered for low-lying lands as an alternative to the "low pressure sewer system" described above. However, difficulties may be encountered if only the vacuum sewer system were to be supplied, exclusive of the complete package that includes the vacuum toilets.

4.4 SEWERAGE SUB-SYSTEMS

The concepts presented in the previous sections were used to develop a sewage collection system for Conception Bay South area. This system, comprising sewerage sub-systems compatible with the sewerage areas, is shown on a general orientation plan (Drawing No. 6), and four detailed plans (Drawings No. 6-1 through 6-4). A description of the sewerage sub-systems is included herein.

4.4.1 Southern Basin

- Sewerage Sub-System 1:

  Gravity sewers will collect the sewage onto a low collecting point at the shoreline, north of the community of Indian Pond. A pumping station proposed at this point will deliver the sewage via a forcemain to be located along the shoreline to Sewerage Area 2.

- Sewerage Sub-System 2:

  Gravity sewers will collect the sewage onto a low collecting point at the estuary of Seal Cove Pond. A pumping station proposed at this point will deliver the sewage via an easterly forcemain to Sewerage Area 3.

- Sewerage Sub-System 3:

  The undulating effects of the terrain are pronounced in this area. A gravity sewer along Conception Bay Highway will collect the sewage from houses along the highway and east of it. A proposed pumping station north of the Seal Cove Power House will lift the sewage across a local ridge and thence it will gravitate along the highway to the major collecting point at Lance Cove.

  The sewage from the area south of Seal Cove Pond, and the sewage pumped from Areas 1 and 2, will gravitate to a low collecting point at the inlet to Seal Cove Road. A pumping station proposed at this point will pump the sewage towards the major collecting point at Lance Cove.

A pumping station proposed at Lance Cove will lift the entire sewage flow from Areas 1, 2 and 3 to the north bank of Lance Cove. From this point, a "shoreline" gravity trunk sewer will run to Kelligrews Point.

It should be noted here that the hydraulic features of the above trunk sewer designated P.C.P.1 Southern Trunk Sewer will be determined on a computer model. (See Section 7). Specifically, topographic conditions will be accounted for to establish the economic depth of a gravity sewer vis-a-vis a lift station and shallower gravity sewer.

- <u>Sewerage Sub-System 4</u>:

  Sewage collected by gravity sewers will be discharged into the P.C.P. 1 Southern Trunk Sewer.

- <u>Sewerage Sub-System 5</u>:

  Sewage collected by gravity sewers will be discharged into the P.C.P. 1 Southern Trunk Sewer. A pump station at the estuary of the Lower Gullies River will discharge the entire sewage flow (Areas 1 through 5) across the river to the continuation of the southern trunk sewer in Area 6.

- <u>Sewerage Sub-System 6</u>:

  Sewage collected by gravity sewers will be discharged into the P.C.P. 1 Southern Trunk Sewer. At Kelligrews Point a central pump station will pump the sewage via a forcemain along the shoreline to Area 7.

- Sewerage Sub-System 7:

Sewage collected by gravity sewers will be discharged into the lower run of P.C.P. 1 Southern Trunk Sewer, which will outlet into Pollution Control Plant No. 1 proposed to be located at Kelligrews shoreline (See Section 6.1).

- Sewerage Sub-System 8:

Sewage collected by gravity sewers will discharge into a gravity trunk sewer designated P.C.P. 1 Northern Trunk Sewer. The lower reach of this trunk sewer will run along the shoreline from the proposed site of Pollution Control Plant No. 1 to the administrative boundary of Foxtrap.

Sewerage sub-systems 8 through 12 will be served by this trunk sewer.

- Sewerage Sub-System 9:

Gravity sewers in this area will discharge into P.C.P. 1 Northern Trunk Sewer which will be routed in a southerly direction between Conception Bay Highway and the shoreline. At Foxtrap boundary line the trunk sewer will change direction and run west to a low collecting point at the shoreline. A pumping station will discharge the sewage from this point, via a forcemain along the shoreline, to Area 8.

A "low pressure sewer system" has been considered for pockets of low-lying land west of the trunk sewer, along Foxtrap River and the shoreline.

- <u>Sewerage Sub-System 10:</u>

  Gravity sewers in this area will discharge into P.C.P. 1 Northern Trunk Sewer which will be routed along Conception Bay Highway. This trunk sewer will outlet to a pump station located at a low point of the Highway near Steadywater Brook bridge. A forcemain along the Highway will discharge the sewage to Area 9.

  The sewers in the area south of Long Pond will gravitate to a local pump station which will lift the sewage to the trunk sewer along the Highway.

  A "low pressure sewer system" has been considered for pockets of low-lying land around Steadywater Pond.

- <u>Sewerage Sub-System 11:</u>

  P.C.P. 1 Northern Trunk Sewer will be routed along Conception Bay Highway in a southerly direction as far as Conway Brook bridge. The trunk sewer will then run westward to a low collecting point on the southern bank of Long Pond. A pump station at this location will lift the sewage for discharge to Area 10.

  Sewers in the area east of the Highway will gravitate to the trunk sewer. Sewers in the Long Pond peninsula west of the Highway will gravitate to a local pump station. A forcemain from this pump station will discharge the sewage to the trunk sewer.

- Sewerage Sub-System 12:

  P.C.P. 1 Northern Trunk Sewer will be routed along Conception Bay Highway in a southerly direction as far as Long Pond tip. A pump station at this location will discharge the sewage, via a forcemain along the Highway, to Area 11.

  Sewers in the area east of the Highway will gravitate to the trunk sewer. Sewers west of the Highway, in the area between Manuels River and Long Pond, will gravitate to a local pump station. A forcemain from this pump station will discharge the sewage to the trunk sewer.

  A "low pressure sewer system" has been considered for pockets of low-lying land around Long Pond.

4.4.2 Northern Basin

- Sewerage Sub-System 13:

  This sub-system will service the immediate area on both sides of Manuels River. A pump station at Manuels River, west of Conception Bay Highway, will discharge the sewage to Area 14.

- Sewerage Sub-System 14:

  Sewers in the area east of Conception Bay Highway will gravitate to a collector sewer routed along the Highway in a northerly direction.

  Sewers in the area west of the Highway will gravitate to a collector sewer routed along Manuels River and the shoreline. Both collector sewers will merge north of Chamberlains Pond and outlet into proposed Pollution Control Plant No. 2.

- <u>Sewerage Sub-System 15</u>:

Gravity sewers will outlet into a "shoreline" gravity trunk sewer designated P.C.P. 2 Northern Trunk Sewer.

The hydraulic features of this trunk sewer will be determined on a computer model, the purpose of which will be similar to that described previously for the trunk sewers in the Southern Basin.

- <u>Sewerage Sub-System 16</u>:

P.C.P. 2 Northern Trunk sewer will be routed along the shoreline as far as Topsail Pond. It will then be routed along Conception Bay Highway as far as Paradise Road.

Sewers in this area will gravitate to the above trunk sewer, except for the low-lying land around Topsail Pond for which a "low pressure sewer system" has been considered.

- <u>Sewerage Sub-System 17</u>:

This sub-system will serve the larger part of Paradise. The collector sewer for this area will be routed in a north-westerly direction, away from Conception Bay Highway. North of Adams Pond the collector sewer will change direction and run southward, west of Adams Pond, to connect with the trunk sewer in Area 16.

4.5 SEWER DESIGN CRITERIA

The design period has been taken as 20 years. The following criteria were used in the preparation of this report and are recommended for adoption in the detailed design of sanitary sewers.

1. Total Flow

   Allowing for 1995 population figures and an average volume of waste water of 95 I.G.P.C.D. the total flow for the whole study area is 2 M.I.G.P.D.

2. Peak Factors

   The peak factors used for the design of pipes would be as derived from Babbit's formula. The factors for various populations are given below:

Population	Peak Factor
1,000 to 3,000	4.0
3,500	3.89
4,000	3.78
5,000	3.62
6,000	3.49
7,000	3.38
8,000	3.29
9,000	3.22
10,000	3.15
15,000	2.90
20,000	2.74

3. Pipe Design

   For hydraulic computations of gravity sewers, Manning's formula would be used. A roughness factor n, of 0.013 would be used for concrete and asbestos cement pipes. For pressure sewers, Hazen

Williams equation would be used with a coefficient C of 130 for Polyethyl Vinyl Chloride pipe PEVC. Where local conditions warrant use of cast iron or ductile iron pipes, a coefficient C of 110 should be considered.

Minimum and maximum velocities of flow would be 2.5 ft/sec and 12 ft/sec., respectively for gravity sewers, and 3.5 ft/sec and 14 ft/sec., respectively for forcemains.

The minimum diameter for gravity sewers would be 8 inches, and for forcemains 4 inches with the exception of low pressure collection systems. Minimum slopes for the various sizes of gravity sewer pipes would be as follows:

Pipe Diameter, Inches	Minimum slope, %
8	0.40
10	0.28
12	0.22
14	0.17
15	0.15
16	0.14
18	0.12
21	0.10
24	0.08

For slopes of more than 15%, concrete anchor blocks would be used to hold the pipes and joints in position. Anchorages would also be provided in pressure mains wherever required.

4. Pipe Bedding

Bedding material surrounding pipes would be either A or B gravel, depending on loading conditions.

5. Manholes

Manholes would generally be of the circular type, precast or cast in-situ, with a minimum internal diameter of 48 inches.

The maximum spacing from centre to centre of adjacent manholes for the various pipe sizes would be:

Pipe Diameter, Inches	Spacing, Ft.
8, 10	300
12,14,15, and 16	400
18 and above	500

6. Building Connections

Y- and T- fittings would be provided on the collector pipes for individual connections. Sizes of connection fittings would generally be:

Minimum connection size	-	6 inches dia.
Double connection	-	8 inches dia.
Commercial	-	8 inches dia.
Institutional	-	8 inches dia.

7. Flushing

Where sewers would require periodic flushing due to special ground conditions which might dictate flatter than normal grades, flushing tanks or flushing manholes would be provided to operate on time clocks, or manually as required.

8. Depth of Bury

In general all the gravity sewers would have a minimum of 6 feet of cover and pressure lines a minimum of 5 feet of cover. However, the possibility of using insulating materials and less ground cover is presently being investigated with a view to establishing technical feasibility, and comparative costs.

9. Distance from Water Mains

Sewer pipe location would be arranged to give a minimum of 10 ft. horizontal clearance to parallel water mains. Since trench concept of water supply over sewer pipe may be utilised with caution and only if the sewer line is deep enough and subsoil water level is not high. Gravity sewers and pressure sewers could be combined in one trench wherever feasible.

10. Alignment of Sewer Inverts in Manholes

In manholes where there is a change in incoming and outgoing pipe sizes, adjustment of inverts would be such that the 0.8 depth point of both sewers will be at the same level.

11. Construction Materials

The following construction materials are recommended for use:

Gravity Sewer Pipes	-	Asbestos cement and concrete pipes
Pipe Joints	-	Push-on type with neoprene gaskets

Forcemains	-	Polyethylene-Vinyl-Chloride pipes (PEVC)
Manholes	-	Precast or in-situ concrete
Manhole frames and covers	-	Cast iron and steel with weight to be compatible with local load conditions.
Manhole steps	-	Galvanized steel or Aluminum (Safety type)
Cement	-	Ordinary Portland for general use, Sulphate resisting for pipes manholes, and underground structures in acidic environment.

SEWAGE DISPOSAL CONSIDERATIONS

5.1 FEDERAL AND PROVINCIAL REGULATIONS

The available standards Federal and Provincial regulations are reviewed and recommendations of the treatment and effluent disposal are made on the basis of guidelines laid down by the authorities.

5.1.1 Public Health Engineering Division, Department of National Health and Welfare

The general objectives described are that the waste discharged in receiving waters shall not create health hazards nor produce adverse conditions through abnormal temperature colour, turbidity or other changes. It further adds that wherever needed, biological nutrient substances in waste waters such as phosphates and nitrates shall be reduced to lowest concentration compatible with existing technology. Toxic wastes harmful to aquatic life (or other potable and recreational uses) shall not be discharged.

The required effluent standards after various treatment processes are described as follows:

(a) Primary Treatment

Primary treatment shall be considered and equate only if the water receiving effluents will meet the following objectives:

Dissolved Oxygen	- 5 mg/l or 60% of saturation whichever is lesser.
Biochemical Oxygen Demand	- 4 mg/l maximum.
Total Coliforms	- not to exceed a median of 500 coliform bacteria per 100 ml.
pH	- 6.7 to 8.5
Odour	- Threshold odour number = 4 maximum
Phenols	- 2 μ mg/l average 5 μ mg/l maximum

(b) Effluent Standards from Secondary Treatment Plant

Biochemical Oxygen Demand	- 15 mg/l maximum
Suspended Solids	- 15 mg/l maximum
Total Coliforms	- Not to exceed a median of 1,000 Coliforms 1000 ml
Chlorine Residual	- 0.75 mg/l minimum
pH	- 6 to 9

Odour	- Threshold odour number = 8 maximum
Phenols	- 20 μ mg/l maximum
Oils	- 15 mg/l maximum

5.1. Following are the guidelines suggested by the District Manager ENVIRONMENTAL PROTECTION SERVICE of St. John's. The concentrations of various parameters not to be exceeded.

1.	BOD	=	20 mg/l
2.	S.S.	=	25 mg/l
3.	Fecal Coliforms	=	400/100 ml
4.	Chlorine Residual after 15 minutes contact time		
	maximum concentration of chlorine	=	1.5 mg/l
	minimum concentration of chlorine	=	0.75 mg/l
5.	pH range	=	6 - 9
6.	Phenol	=	20 μ mg/l
7.	Oil and Grease	=	15 mg/l
8.	Temperature	-	not to increase embient water temperature more than 3^{o}c at the perimeter of mixing zone

9. Dilution with fresh water is not permitted to achieve compliance with the guidelines.

5.1.3 CLEAN AIR, WATER AND SOIL AUTHORITY POLICY STATEMENT

(1) Each sewage system would be evaluated separately.

(2) Primary and secondary treatment would be required prior to discharge into (a) fresh water, (b) comparatively enclosed bodies of sea water, i.e. if sea water movement is minimal, the relative volume of sewage discharged to the body of sea water must be evaluated stringently.

(3) Raw sewage may be approved for discharge to the ocean if the following conditions are met:

(i) Rapid dissipation due to currents and wave action may be expected.

(ii) No beneficial use of sea water would likely be affected, e.g. fish processing, shell-fish beds, swimming or recreational use.

5.1.4 Objective and Recommended Standards

The guidelines described by various agencies and authorities are all the same in principle.

The objective is to find out and recommend the method of a combination treatment process and disposal system which is of acceptable standard and economical. The specific standards for various types of treatments have been recommended by various authorities as detailed earlier; but for disposal, there are only guidelines which are under constant progressive rationalization. The dilution of secondary treated effluent in the receiving body of water used to be about 6/1. It has now been rationalized. The condition and quality of the receiving body of water is fully taken into account before the degree of treatment and subsequent dilution is allowed by the controlling authority.

In this study, the receiving water body is the ocean (Conception Bay) and the existing pollution at the point of outfalls is insignificant. The Bay waters are not used for swimming or for such purposes where this water is used for human consumption or domestic use, directly or indirectly. There is no Fishing Plant in the area. The recreational activity of fishing, yachting and sailing are such activities which definitely restrict indiscriminate disposal of wastes into the Bay. Consequently, the floating unsightly matters have to be removed before effluent is discharged. Similarly, odours have to be eliminated, therefore, BOD and S.S. standards should be maintained.

The other important aspect is its effect on fauna and flora of Conception Bay at the points of disposal which have been studied by Dr. J.M. Green. His report states that the marine and plant life of the disposal areas would not be affected adversely by the domestic sewage disposal, if secondary treatment is done (see Appendix B). At the same time Dr. G. T. Csanady recommends a long outfall (about 1300 feet of outfall pipe with about 75 feet of diffuser pipe) (See Appendix C).

Winter temperature difference of sea water and sewage effluent would be taken care of by diffusion of effluent. The pH, oil, and phenols would not present any problem, it being a domestic sewage.

For disinfection of waste water, chlorination is recommended. But in the case where the dilution of 200/1 with a deep and long outfall is accepted as per recommendations of Dr. G. T. Csanady, no treatment would be required, and no chlorination would be necessary.

In our opinion, subject to further economical evaluation, the mixing and dilution at the treatment site with sea water could be a potential alternative, instead of discharging the waste water at distant points into the ocean for mixing and dilution.

5.2 OUTFALL DIFFUSION MODEL

5.2.1 Basic Theoretical Concepts

Input contribution to this section was made by Dr. G. T. Csanady (as put forward in our proposal). His report on this matter is contained in Appendix B.

Essentially all outfall diffusion models are based on the fundamental premise that the initial dilution, So at the top of a rising plume formed by a horizontal jet is a function of the following four variables:

y_o = depth from surface to centre of discharge jet;

D = diameter of jet at point of discharge;

Q = jet discharge (of sewage effluent);

$g^1 = \frac{\Delta\rho}{\rho} g$ = apparent acceleration due to gravity, where

$\frac{\Delta\rho}{\rho}$ = the relative density difference, commonly about 0.026.

These four variables may be arranged into two dimensionless groups, namely:

$$\frac{y_o}{D}$$

$$F = \frac{Q}{\frac{\pi}{4} D^2 \sqrt{g^1 D}} = \frac{Y}{\sqrt{g^1 D}} = \text{Froude Number}$$

Therefore, the initial dilution So of a jet is a function of the depth of the outfall diffuser, the diameter of the diffuser ports, and Froude number:

$$S_o = f\left(\frac{Y_o}{D}, F\right)$$

The initial mixing (and dilution) of sewage jets from a diffuser with sea water produces a field of diluted sewage which drifts away with the ocean current. Accordingly, there is a second phenomenon of further dilution of the sewage field (after the initial jet dilution had occurred) by the natural turbulence in an ocean current. This diffusion type dilution is dependent on a diffusion coefficient, and is inversely proportioned to the ocean currents and the length of the diffuser. The factual meaning of this function (of diffusion) is that on shore ocean currents of high velocity not only drift the sewage field faster onto the shore, but also hinder further dilution by diffusion. As will be seen later, the effect of dilution by diffusion (in cases of on shore currents) has a secondary limited value, and by and large prime dilution should be based on initial jet dilution.

In the application of the above fundamental dilution function different techniques (models) have been developed.

5.2.2 Methodology

The methodology in applying an outfall diffusion model considers certain importants facets. Some of these facets are outlined in this section.

In order to provide effective dilution in situations of on shore and longshore ocean currents, a Wye diffuser has been considered.

A diffuser with as many ports as feasible operating as a line source is the most effective. In order to consider and analyse the diffuser as a line source, that is a summation of a collection of point sources, two conditions have to be met, namely,

a) that the outflow is shared equally by all ports of the diffuser (each of which is a point source), so that the discharge through any single port is Q/n, and

b) that the sewage fields emanating from the ports are non-overlapping.

The crucial requirement of equal outflows at all ports can only be achieved after a thorough computer analysis of the hydraulics of the outfall (a detailed design task). Suffice to mention here is the fact that an improperly designed manifold can degenerate, in the most extreme case, to a point source, thereby losing the extra dispersion introduced by employing a diffuser as a line source.

A design criterion used for a line source is that the ratio R (as defined below) should lie in the interval -

$$0.5 \leq R \leq 0.67$$

wherein values are small enough to obtain strong jet flow, but not so small as to increase the total head unduly.

The ratio R is defined by -

$$R = \frac{\text{sum of areas of all ports}}{\text{area of pipe}}$$

To prevent neighbouring sewage fields from interferring with each other, thereby increasing the concentration, it has been considered that adjacent ports will be at least two half-widths of a sewage field apart. Consequently, if for a given flow a constant number of ports is considered, the diffuser length must be increased as the length of the outfall (and its depth) are increased.

The implementation of the above facets ensures that the designed outfall will perform satisfactorily, as depicted by the model. An additional facet of significant importance is the ratio of the initial dilution.

A point made by Dr. Csanady relates to the fact that the model can produce any dilution ratio. However, to get a homogeneous mixture of the lighter density sewage and heavier density sea water, a minimum limit of dilution ratio should be considered. Below this empirical limit the two media would not mix properly, and a "floating lense" of sewage will be drifted on the water surface. Accordingly, we have considered the following values as the minimum dilution limits:

- Preliminary treated effluent 200:1
- Primary treated effluent 100:1
- Secondary treated effluent 20:1

5.2.3. Model Analysis

a) The model has been analysed for an average design flow of 1.0 IMGD and a maximum design flow of 3.0 IMGD. The minimum initial flow has been assumed to be o.3 IMGD.

To meet the requirement of the ratio $0.5 \leq R \leq 0.67$, the following has been considered:

- port size, 2½ inch diameter;
- diffuser diameter 10 inch diameter;
- number of ports in each Wye branch is 10.

This gives a value of $R = 0.625$.

The resulting range of Froude number is -

$$1.05 \leq F \leq 19.5$$

b) As mentioned previously in this section, the initial jet dilution is a function of Froude number, and the relationship of water depth to diffuser port size. Solving this function for the critical Froude number and different depths of water gives initial jet dilution values, S_O as shown on Figure 1. Note that S_O is related on the graph to outfall length in lieu of depth which is a more practical presentation.

c) To provide two half-widths of head of sewage plume between adjacent ports, diffuser lengths as related to outfall length are shown in Figure 2.

d) The effect of diffusion on dilution was determined from Fickian law, using Richardson's law to express the diffusion coefficient. Because of the many variables in the diffusion function, the curves shown on Figure 3 represent only one specific situation of a current velocity of 1 fps and a diffuser length perpendicular to onshore currents. Nonetheless, the curves are very meaningful as they express the following:

- For onshore currents the dilution factor for diffusion is negligible. For instance, the dilution factor for a 1500 foot outfall at a point 1000 feet from its source line (i.e. 500 feet from the shoreline), is about 1.3. This same outfall provides an initial jet dilution of 100 (Figure 1). The total dilution at the above point is therefore, 130. It can readily be seen that the prime dilution results from initial jet effects. For current velocities higher than 1 fps, as experienced by wind currents, the above factor of 1.3 will be still smaller,making the jet dilution effect even more pronounced.

- Using the same example for longshore currents, it can be seen (Figure 3) that the diffusion effect is appreciable. For instance, at a distance of 5000 feet from the diffuser the dilution factor is 3.5, bringing the total dilution (jet plus diffusion) to 350. This figure would be somewhat higher if we use the diffuser length perpendicular to longshore currents. However, again, higher current velocities will tend to reduce the dilution factor due to diffusion.

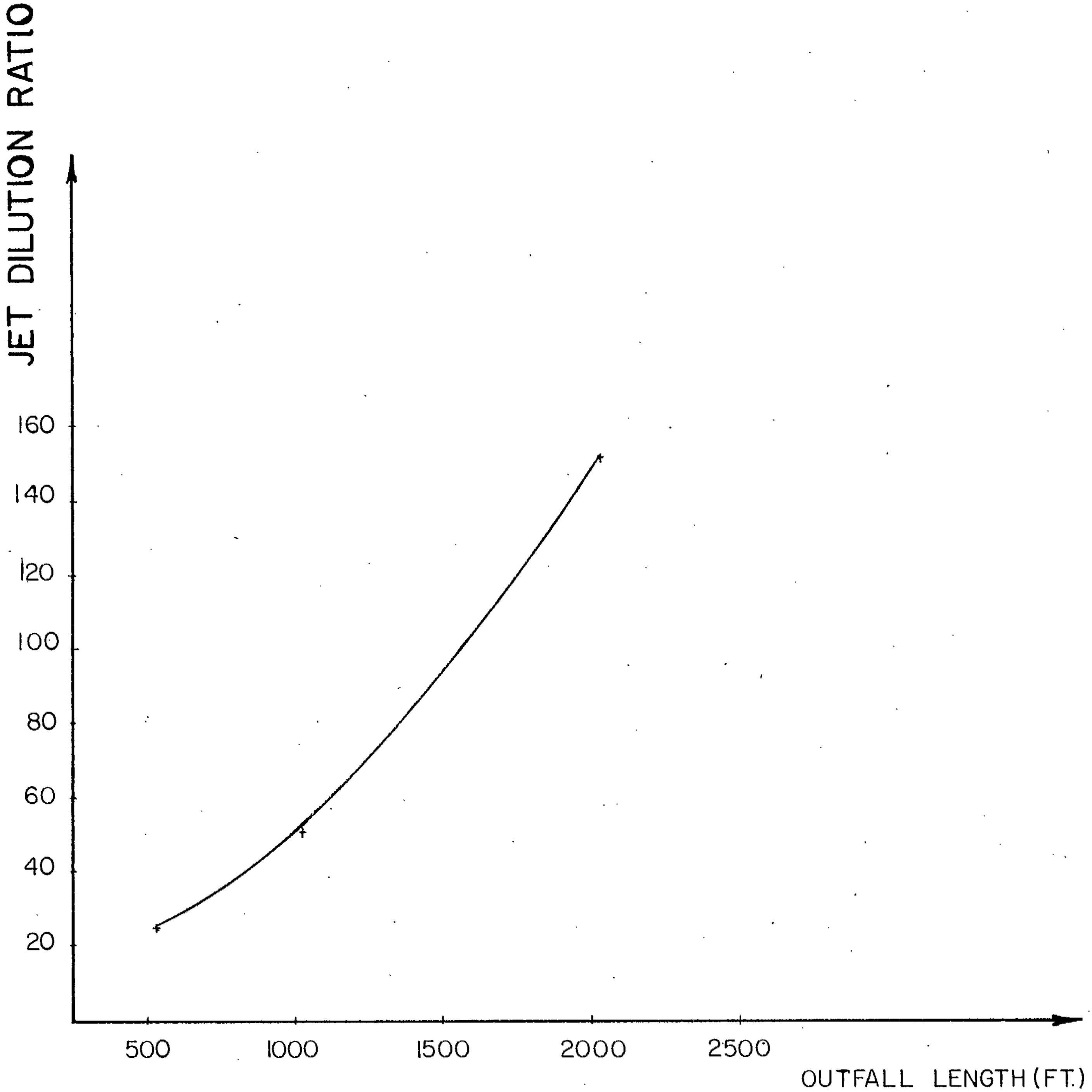

FIG. 1 INITIAL JET DILUTION

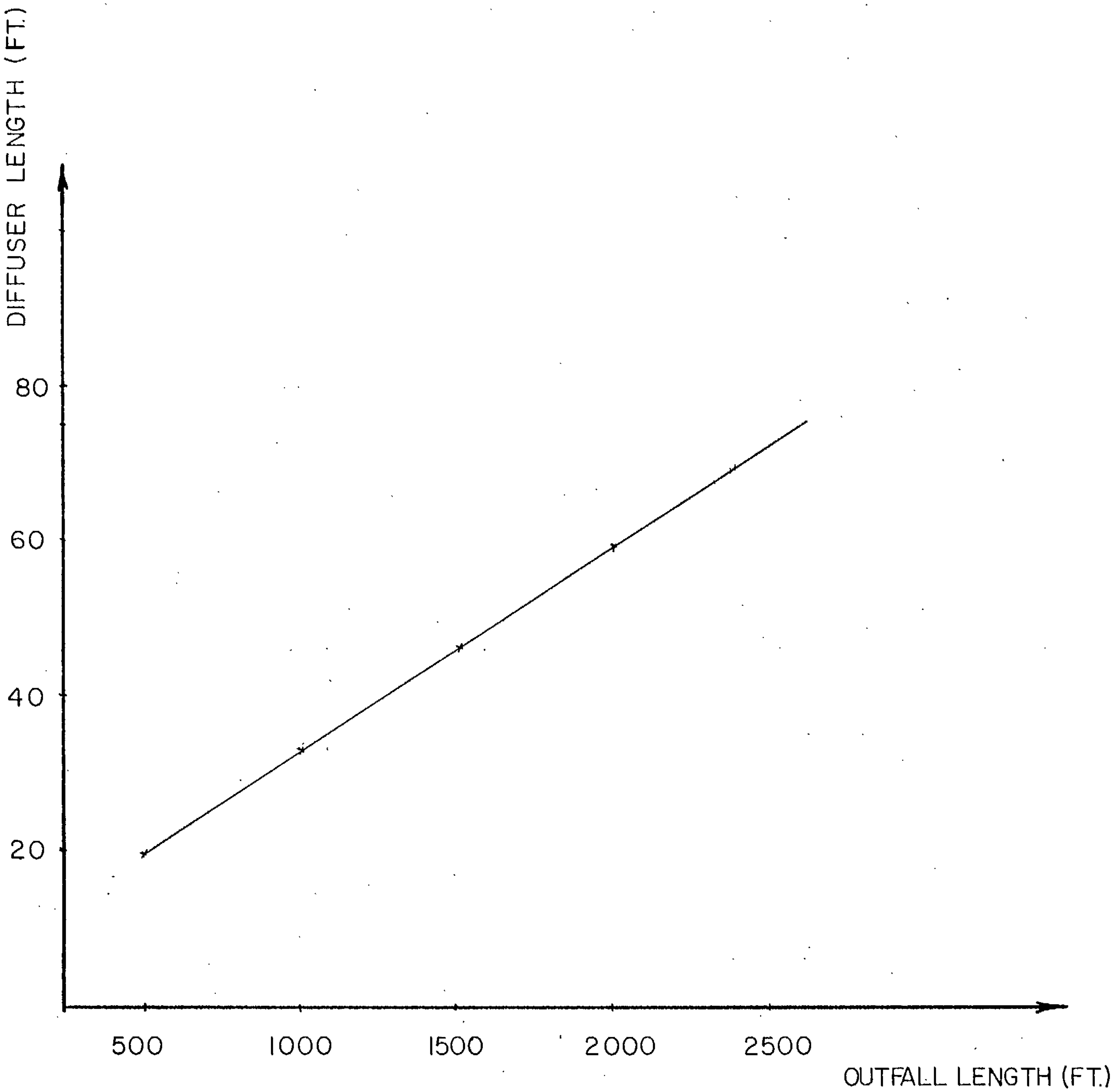

FIG. 2 DIFFUSER LENGTH

NOTE LENGTH SHOWN ON GRAPH IS FOR ONE BRANCH OF THE WYE DIFFUSER

FENCO

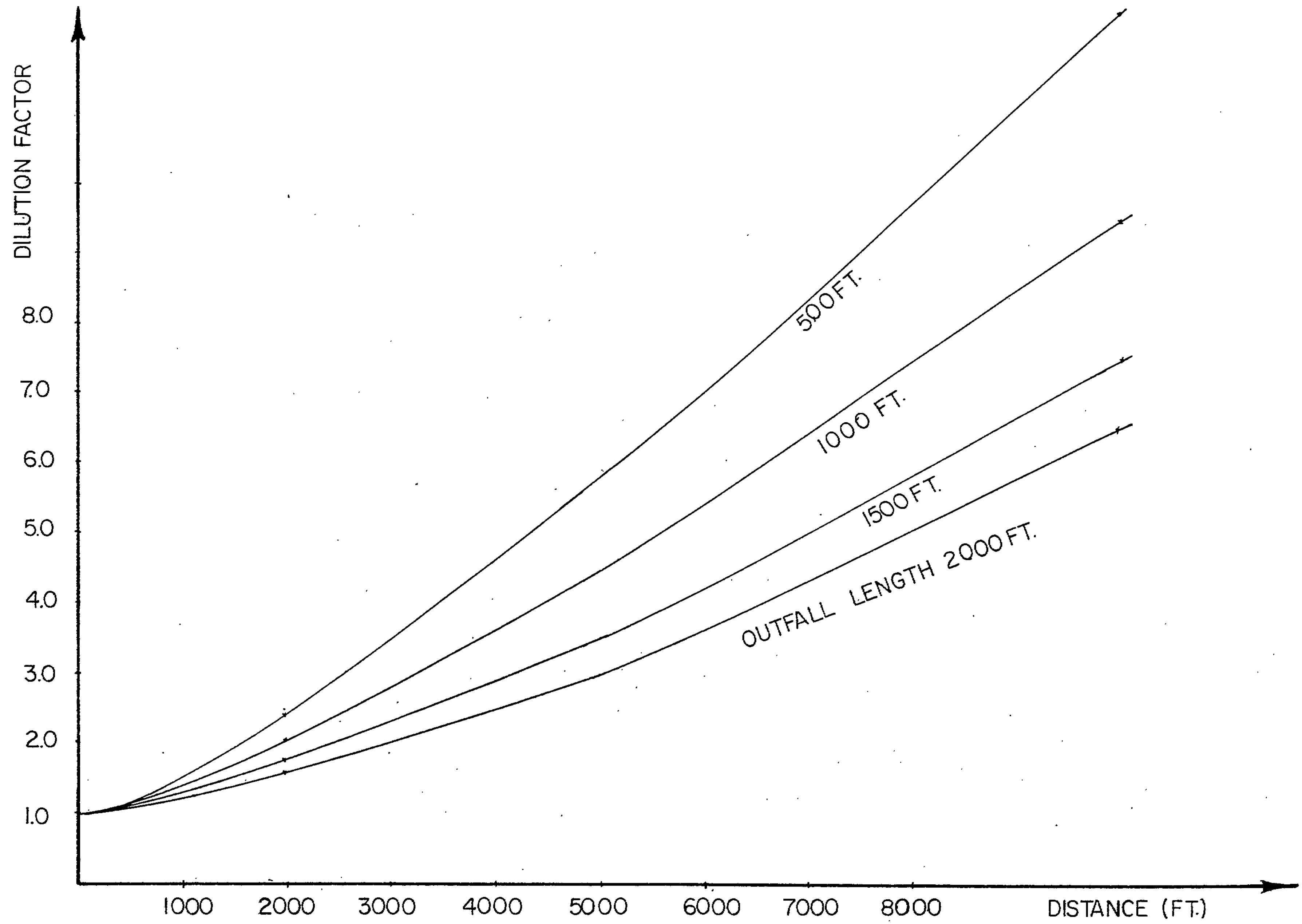

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FIG. 3 EFFECT OF DIFFUSION FOR DIFFERENT OUTFALL LENGTH

5.3 ASSESSMENT OF ECOLOGICAL IMPACT

Input contribution to this section was made by Dr. J. M. Green of M. U. N. His report on this matter is contained in Appendix C.

Dr. Green's assessment of the anticipated ecological impact considers the type of effluent discharges previously described, and their possible effects on marine biota.

5.3.1 Marine Life

The observations of the benthic marine life in the general areas of Chamberlain's Pond and Kelligrews Point, the two possible outfall locations, have been previously described.

The field survey conducted in February 1974, provided limited information on seasonal changes in the marine life of the areas, but the year round underwater observations since 1971 of the marine life at Broad Cove and Conception Bay (St. Phillips) by Dr. J. M. Green have provided seasonal data, particularly on the fishes. The survey sponsored by FENCO and conducted in February 1974, indicated that neither site has commerically valuable invertibrates with the exception of lobsters which are found at the Kelligrews site. This situation does not change with the seasons. It is to be noted, however, that the whole of the Conception Bay sea bed area in general slopes gradually and provides spawning grounds for several species of fish. Capelin, winter flounder and lump fish spawn at about eleven yards from the shore line. These fish lay eggs at the bottom of the sea and they remain there until their eggs are hatched.

The outfall's design should, therefore, provide effluent discharges that would not effect these eggs, nor should the diffusion be allowed to uplift the eggs. Since it is the egg and larvae life forms of fish which are most easily effected by pollution, the design of the outfalls should take into consideration their protection. It is, however, to be expected that the fish themselves would not lay eggs at places which are agitated with diffused discharges.

Other fish, like cunner, avoid areas of environmental stress. Only a sudden pollution in winter may harm such forms of fish life because they hide themselves in winter under rocks and become inactive; but due to regular continuous flows of sewage effluent, all the year round, the fish would keep hideouts elsewhere, away from the outfall diffuser. This would slightly change the local environment of a very small area without adverse effect on overall marine life.

5.3.2 Physical Environmental Conditions

Local physical conditions, such as gradually sloping sea bed and relatively shallow inshore water, the lack of strong tidal currents, onshore winds, inshore ice cover in winter, may increase the likelihood that effluent discharged inshore could adversely effect benthric marine life.

5.3.3 Ecological Recommendations

Local physical conditions, the nature of the marine communities, the desirability of maintaining water quality for mariculture, have led Dr. Green to the following recommendations:

- that primary treated effluent be discharged in deep water (in the order of magnitude of 60 feet).

- that only secondary treated effluent be considered for discharge in inshore water.

- that for any quality of effluent discharges a diffused outfall be used.

Dr. Green has also recommended that inshore discharge of the effluent from the secondary treatment plant should go through the diffuser system to prevent a nutrient rich lens of low salenity water from forming on the sea surface. Chlorination of effluent is not expected to have an adverse effect on marine life.

Dr. J. M. Green has concluded:-

"As far as locating treatment facilities and outfalls at Kelligrews Point and/or near Chamberlin's Pond, the field survey indicated that this would not disrupt or permanently change unique or especially productive or deverse marine communities."

TREATMENT AND DISPOSAL WORKS

6.1 TREATMENT SITES

Two treatment plant sites (P.C.P.) have been recommended as shown on the plan numbers 2 and 7. The Pollution Control Plant, Site I, shall serve the sewerage areas south of Manuel River up to and including Seal Cove. The other Pollution Control Plant, Site II, shall serve sewerage areas north of Manuel River up to and including Paradise.

The location of the site is tentative subject to further investigation and acquisition. It has been earlier pointed out in the description of sewerage areas that the natural barriers and topography suits the selection of at least two treatment sites.

The bed levels or say the depths of the ocean and outfall studies have suggested long lengths of untreated outfalls, therefore, it does not seem to be economical to have several untreated outfalls or several partially treated outfalls.

The economic justification would further be studied on computors.

Both the treatment plant or untreated outfall sites have been selected with the view to achieve the most suitable location for gravity flows and minimum pumping. A centralised situation to minimise the pumping is obviously most suitable. The pumping capacities and heads are also considered, for example, smaller pumpage for longer lengths of pressure mains against shorter lengths of pressure mains for larger pumping capacities. Therefore, for both the Pollution Control Plants the centralised locations selected are in keeping with the above guidelines. The trunk line alignments and outfall into the ocean were other important considerations.

The choice was further limited due to apparent soil conditions, surface elevations, availability of open land, proximity of housing and recreational sites, and also fauna and flora of the ocean, as well as the tides. In selection, some flexibility in P.C.P. site locations have been kept in consideration so that the location could be modified or relocated within the same area which would not alter the earlier described criterias or estimates.

A detail survey and investigation should be conducted of the two sites before starting the final detail design of the Pollution Control Plant.

Site for PCP I

It is located in Kelligrews area near the shore on the west of the Canadian National Railway. There is an unpaved approach road off the Conception Bay highway to this site. The road ends at the railway line and the treatment site is on the north side of this road near brush. The electric cables are close. The area gently slopes in between contours 25 to 15. There are boulders scattered around but brush area seems to be suitable after clearance. Though the bed of the ocean has gradual slopes but an alignment for outfall projected from this position is steeper in comparison to others and hence shorter.

Site for PCP II

It is situated north of Chamberlain Pond and playground, on the seashore. A small unpaved road off the Conception Bay highway approaches the site. The electricity is available close by. The site is uneven having contours between 36 to 10. The site can be levelled to suit the required flow system of different units of treatment. The soil seems to be medium (not hard) quality as normally available in the area. The seabed is gradual in slope but steeper than others to qualify for a shorter outfall.

6.2 DEGREE OF TREATMENT

In general, four degrees of treated effluent have been considered for marine disposal in Conception Bay. These effluent discharges are:-

- Preliminary treated effluent
- Primary treated effluent
- Secondary treated effluent without sludge
- Secondary treated effluent with sludge

6.2.1 Preliminary Treated Effluent

The degree of treatment for this effluent will include screening to remove floating material, grit removal, and shredding of solids. This preliminary treatment is to be considered the minimum requirement for diffused outfall disposal.

In order to ascertain proper dispersion of this type of effluent in Conception Bay water, a jet-dilution of 200:1 should be considered (see Section 5.2). This is a high dilution ratio, requiring a long outfall (in excess of 2,500 feet). Evidence of iceburg scouring effects on the bottom of the Bay will necessitate protection (cover) to the outfall, making its price costly.

Another important factor to consider with preliminary treated effluent is the coliform organisms. Without chlorination and count on natural die off dilution by itself would not reduce the coliform count to acceptable standard levels. Also, depending on the location of the outfalls and the proximity to fish catching areas, fisheries authorities may refuse to count on natural die off for coliform kill would be costly because of the affinity of the organic matter in the sewage (more of which has been removed or stabilized) to chlorine.

6.2.2 Primary Treated Effluent

This degree of treatment involves the removal of the settleable solids in the sewage and the BODs associated with it. The treatment is performed in a clarifier and usually includes preliminary treatment facilities. Up to 65 per cent of the suspended solids and 30 per cent of the BODs can be removed by primary treatment.

Primary treated effluent is commonly disposed of in a marine environment with a jet dilution of 100:1. Good practice would be to chlorinate this effluent prior to its discharge. A factor of concern is the primary sludge removed in the clarifier. This sludge is not stabilized and if not treated or disposed of properly would result in abnozious odours. A regional solids waste disposal scheme that could handle this sludge would make primary treatment attractive.

6.2.3 Secondary Treated Effluent (without sludge)

Scondary treatment is the biological removal of colloridal and soluble organic matter from the sewage. Depending on the process applied, different fractions of the organic material removed are converted into end products of water and CO_2 and into biological flocs. The latter is separated from the liquid phase in a clarifier (usually referred to as secondary or final clarifier). Most of the secondary treatment processes can remove 90 per cent or over of the suspended solids and BODs of the sewage.

Secondary treatment usually includes also preliminary treatment facilities. Primary clarifiers are optional with some processes and mandatory with others.

This alternative considers the discharge of a chlorinated effluent in a diffused outfall that will produce a jet-dilution of 20:1. All the sludge removed will be handled on land, either locally at the treatment plant, or at a regional solids waste disposal centre.

6.2.4 Secondary Treated Effluent (with sludge)

This alternative is the same as the previous one, except that sludge will undergo aerobic treatment (digestion) at the local treatment plant and be lended with the secondary effluent for marine disposal. The combined chlorinated flow will be regarded as primary effluent requiring an initial jet-dilution of 100:1.

6.3 ALTERNATIVE TREATMENT PROCESSES

6.3.1 General

Processes involved in the treatment of the liquid phase of sewage, as referred to in the previous section, include physical separation of solids and biological removal of colloidal and soluble organic matter.

Physical separation of sewage solids (be it settleable raw solids or biological flocs) follows a flocculent pattern of settling whereby both surace loading an retention time control the process. Acceptable design values for these parameters are given in Section 6.5.

The biological treatment of sewage is a more complex phenomenon, and a number of different processes can be used to implement this treatment. The processes considered for this project will be discussed in this section.

6.3.2. Basic Process Concepts

In the biological treatment of sewage, bacteria utilize (and thereby stabilize) the organic matter as a means of survival and perpetuation of the species. In all processes the same general metabolic scheme follows. Through a series of coupled oxidation - reduction reactions (in the form of dehydrogenation and hydrogenation), a portion of the energy produced from the oxidation of the organic matter is utilized in maintaining the system and a portion for synthesis (production of new bacterial cells). The higher the synthesis rate is, the more solids will the system produce. These solids (bacterial cells or flocs) require additional treatment. Ideally, synthesis should approach zero.

In applying the above metobolic scheme to a treatment process, the engineer utilizes three general phases of bacterial growth.

At a high ratio of organic matter to bacteria (i.e. much food in the system), the rate of metabolism and growth is also high. As new cells are being produced, the ratio is reduced and the rate of growth declines. When organic matter (food) had been essentially depleted, endogenous respiration follows. In this phase there is no production of new cells, but on the contrary, in order to maintain the system cells are being used as the source for energy.

In the engineering of a system that will perform the above two basic methods exist:

a) Bacteria are grown in the sewage in an aeration tank. They overflow with the effluent and have to be separated (in a clarifier) and returned to the aeration tank to continue the treatment process as described above. The system is known as "activated sludge". The rate of bacteria (sludge) return, relative to the size of the aeration tank and the organic matter in the sewage, will determine the type of the activated sludge process. Four types of this process have been considered for this project (Section 6.3.3.)

b) Bacteria are grown on a foreign media and the sewage is coming only in contact with the bacteria. Consequently, the solids in the system are low, they do require separation in a clarifier but there is no need to return them for process control. Systems utilizing this concept are the "trickling filter" and "biological rotating disc". The latter has been considered for this project.

6.3.3. Alternative Treatment Processes

The alternative treatment processes considered for this project are:

- Extended aeration
- Oxidation ditch
- Contact stabilization
- Combined tank high-rate
- Bio-surf

a) Extended Aeration

This process utilizes the concept of "synthesis approaching zero" to facilitate operation. Consequently, the process is extended well into the endogenous respiration phase where cells are used as the source for energy and the rate of metabolism is low.

Treatment units usually employed in this process are:

- Screens
- Grit chamber
- Aeration tank
- Final clarifier
- Chlorination

Some advantages of the process are:

- Operation is relatively simple
- Provides a buffer for shock loads.

Some disadvantages of the process are:

- Because of the extended aeration period biological solids do not have good settling properties.
- The system is relatively expensive.

b) Oxidation Ditch

The oxidation ditch is similar to the extended aeration process, the difference being that an extended aeration plant usually has a completely mixed aeration tank whereas the oxidation ditch is a plug flow system. Another difference is in the mode of air application. In an extended aeration plant diffused air or mechanical aeration is provided. In the oxidation ditch rotors provide air.

c) Contact Stabilization

Adsorption of organic matter by the bacteria is a precursor stage to oxidation. This process utilizes this phenomenon by providing a relatively short "contact" time of the total flow with the bacteria. After solids separation, the returned bacteria are kept for a longer period of time in a stabilization tank where they proceed with oxidation of the adsorbed organic matter. The bacteria are then added to the sewage in the contact tank.

The process rate is not as low as with extended aeration, consequently there is excess sludge build-up for wast. Treatment units usually include:

- Screens
- Grit chamber
- Aeration tank divided into two compartments, "contact" and "stabilization"
- Final clarifier
- Chlorination
- Aerobic digester.

d) Combined-Tank High-Rate

In this process the capibility of bacteria for high rate metabolism is utilized. Consequently, the ratio of organic matter to bacteria is the higher of the activated sludge processes, resulting in longer production of excess cells (solids) for waste. Treatment units include:

- Screens
- Grit chamber
- Aeration tank combined with a settling tank
- Chlorination
- Aerobic digester

Some of the advantages of this process are:

- Because of the high reaction rate the retention time in the aeration tank is relatively short. In case of process upset, this feature enables restoration to normal operations in a relatively short time (in terms of days).

Some of the disadvantages of this process are:

- Excessive quantities of waste sludge to handle.
- Skilled operation.
- Limited buffer capacity for shock loads. This is somewhat offset by the above advantage.

e) Bio-Surf

This system requires the development of a bacterial culture on the surace of a foreign media. It employs a large diameter corrugated plastic media which is mounted on a horizontal shaft placed in a contoured-bottom tank. While rotating slowly, the bacterial culture on the media comes in contact with the sewage that flows through the tank. Shearing forces keep the bacterial culture in equilibrium. Only excess biomass sloughs off from the media into the effluent. This is eventually removed in the final clarifier.

Some of the advantages of the process are:

- One of the more stable and flexible processes.
- Small quantities of waste sludge to handle.
- Waste sludge is readily compactable.
- Lower maintenance and lower power consumption.
- Economical design of the final clarifier.

Some of the disadvantages of the process are:

- Necessity for a primary clarifier.

6.4 ALTERNATIVE DISPOSAL WORKS

Outfall disposal works compatible with the degree of treatment have been considered as follows:

Degree of Treatment	Length of Outfall - Ft.	Length of Diffuser - Ft. *
- Preliminary Effluent	2,500	75
- Primary Effluent	1,500	47
- Secondary Effluent (without digested sludge)	500	20
- Secondary Effluent (with digested sludge)	1,500	47

* Wye shaped. Length shown is for each branch of the Wye.

6.5 TREATMENT PLANT DESIGN CRITERIA

A number of authorities and agencies have laid down certain standards on the basis of experiences. The criteria to design treatment plant have been compiled for comparison and use. All the standards are similar in principle but no parameters have been laid, by any agencies for outfall design which have already been covered in section 6.4 earlier.

6.5.1 Public Health Engineering Division, Department of National Health and Welfare, Ottawa

Following design parameters have been recommended by the above said agency.

(1) Sedimentation

Primary Sedimentation -

i)	Detention time	=	1.0 - 3.0 Hrs.
ii)	Surface Loading Rate	=	500-1250 IGPD/ft^2
iii)	Weir Loading	=	4000-12,250 IGPD/ft. of Weir
iv)	Depth of Tank	=	6 - 15 ft.
v)	Overflow Velocity	=	2 - 12 ft./Hr.

Final Sedimentation

i)	Detention time	=	2.5 Hrs.
ii)	Surface Loading Rate	=	540 - 1000 IGPD/ft^2
iii)	Weir Loading	=	12,250 IGPD/ft. of Weir

(2) <u>Aeration</u>

Extended Aeration -

i)	Aeration time	=	24 Hrs.
ii)	Volume of tank	=	15 lb. BOD/day/1000 ft^3 of aeration tank capacity
iii)	Air supply	=	500 to 3500 ft^3/lb BOD (Applied)
iv)	Final settling tank		
	Detention Time	=	4 Hrs.
	Surface Loading Rate	=	290 to 580 IGPD/ft^2

(3) <u>Trickling Filters</u>

a) Low rate (less than 4 mgd/acre)

i)	BOD	=	200-600 lb.BOD/day/ acreft or 5-25 lb. BOD/day/1000 ft^3 of filter media
ii)	Depth	=	4 - 10 ft.

b) High rate 10-30 mgad.

i)	BOD load	=	2000 - 5000 lb. BOD/ day/acreft or 25-300 lb. BOD/day/ 1000 ft^3 of filter media
ii)	Depth	=	4 - 10 ft.

(4) <u>Contact Stabilization</u>

i)	Tank Volume	=	70 lb. BOD/Day/1000 ft^3 of Aeration capacity
ii)	Air Supply	=	750 ft^3/lb. BOD removed

(5) Combined Tank

i)	Tank Volume	=	125 - 180 lb. BOD/day/ 1000 ft^3 of Aeration tank capacity
ii)	Air Supply	=	500 - 700 ft^3/lb. BOD removed

(6) Imhoff Tank

i)	Detention Time	=	1.5 - 3.0 Hrs.
ii)	Overflow Rate	=	Less than 500 IG/Day/ Ft^2
iii)	Digestion Chamber	=	2.5 - 4 ft^3/CAP

(7) Chlorination

i)	Contact Time	=	15 to 30 minutes based on maximum daily flow
ii)	Chlorine Residual after 15 minutes	=	0.75 to 1.0

mg/l minimum contact time (in contact chamber)

6.1.2 The 10 States Standards (U.S.A.) Guidelines for Sewage Works

Design standards have been laid to produce acceptable effluent by guiding the methods of treatment by the 10 States of the United States.

(1) Settling Tanks

- The minimum length = 10 ft.
  Liquid depth for mechanically cleaned settling tanks shall be as shallow as possible but not less than 7 ft.
- Final Clarifier
  Depth not less than 8 ft.
- Weirs
  Overflow = Less than 10,000 US GPD/foot for plants 1.0 US MGPD or less
- Surface settling rate
  Primary Clarifier: (if not followed by secondary treatment)
  - Less than 600 US GPD/ft^2
  (if followed by secondary treatment)
  - Less than 1,000 US GPD/ft^2 based on design flow.

(2) Final Settling Tank for the following process

Types of Process	Detention Time Hrs.	Surface Settling Rate USGal.Day/ft^2
High Rate	2.5	700
Contact Stabilization	3.0	600
Extended Aeration	3.6	300

(3) Aeration tank capacities and permissible loadings

Process	Detention Time Hrs.	Loading lb. BOD Day/1000/ft^3	MLSS/lb BOD_5 Loading
Contact Stabilization	* 3.0 to 2.0 (In Contact Zone)	30 to 50	2/1
Extended Aeration	2.4	12.5	As low as 10/1 to As high as 20/1

* Contact Zone - 30% to 35% of total aeration capacity. Reaeration zone comprises the balance of the aeration capacity.

(4) Chlorination

Contact Time = 15 minutes at peak hourly flow. Doze depending on requirement to reduce bateria to acceptable limit.

6.1.3 Standards for Water and Sewer Works for Environment Division Dept. Provincial Affairs & Environment, Newfoundland

It states that - "The quality of effluent shall meet the standards and/or objectives for effluent and stream, established by the Authority".

The guidelines for design parameters are enumerated as follows:

(1) Primary Settling Tank

Detention Time	=	2 Hrs.
Surface Settling Rate	=	500 to 750 IGPD/ft^2

(2) Secondary Treatment

It states that - "No modified activated sludge process shall be approved for BOD reduction of more than 75% or for a solids/BOD ratio of more than 1/1. Although a higher solids to BOD ratio might theoretically result in BOD reduction of more than 75%, it is believed that a high sludge volume index and resulting sludge bulking, likely to occur at solids/BOD ratios between 1/1 and 2/1, might make the process inoperable".

(3) Lagoons - Aerated

Organic Loading	=	100 lbs. BOD/acre/day or more provided it can be justified.
Hydraulic Loading	=	It is recommended that the aerated lagoon to be considered should be designed on the basis of 10 to 20 days retention time.

(4) Trickling Filters (Low-Rate and High-Rate)

Low-Rate Trickling Filters

Hydraulic Loading	2 to 4 million gallons per acre per day

Organic Loading	10 to 20 lb. $BOD_5/1000/ft^3$ of media

<u>High-Rate Trickling Filters</u>

Hydraulic Loading	10 to 40 million gallons per acre per day
Organic Loading	Up to 90 lb. $BOD_5/1000\ ft^3$ of media

SYSTEMS ANALYSIS

7.1 METHODOLOGY

This section describes the computer analysis of the most economic sewerage system for Conception Bay South area, though the procedures developed are of a general nature and could be applied to many similar situations.

The real problem is a complex one. In order to solve it, a stepwise approach is used, in which additional variables entering into the design of the ultimate sewer scheme are included one by one, in computer programmes of increasing complexity. In order to do this, a certain number of problems are described, for each of which it is intended to develop a computer method of solution. In each case, a cost for the overall scheme will be found according to a procedure touched on subsequently.

Problem A

The first problem is one for which a readily-identifiable optimal solution exists, from first principles. Computer analysis is therfore not required to solve it. However the computer subroutines developed for solving it on a machine are important building bricks for subsequent developments:

Given a horizontal plain, across which a straight gravity sewer is to run from points A to Z (say), with intermediate inflows Q(B), Q(C) ..... at points B,C, with a pumping station at point Z only, find the minimum cost layout of the sewer.

Clearly the sewer must be designed to run from point A downwards at the minimum scour velocity, the sewer being designed to run say at 80% of full depth at this velocity.

Problem B

Same problem, but with two sets of flows, Q' (minimum) and Q" (maximum).

The sewer runs at the minimum scouring velocity at least for the Q' condition, and 80% depth at most for the Q" condition.

Problem C

Same problem as in B (a single gravity sewer with terminal pump station), but in non-horizontal terrain.

(i) If the sewer profile as established under B nowhere intersects (or more precisely, is at less than 6 ft. from the ground surface), the solution is as in B.

(ii) If however the B type profile intersects the ground surface at one or more points along the profile, it must be lowered at these points so that the surface is no longer intersected (or more precisely the requisite minimum cover is achieved). In order to achieve this object, the slope of the sewer before reaching the first critical point will be increased to the maximum compatible with allowable flow velocities and extended back either

(a) to meet the previous profile,
(b) if the previous, low-grade profile is not encountered prior to the preceding Q intake point being encountered, a drop will be inserted at this point.

Similar modifications to the sewer profile will be accomplished subsequently if points of intersection with the natural surface still remain.

Problem D

In earlier stages of the computer study, a procedure for design of a single gravity sewer has been outlined, under a fairly comprehensive set of conditions. In the present phase, the addition of one or more intermediate pumping stations is envisaged.

Incorporation of intermediate pumping states is conducted under the following set of rules:

(i) A pump will be placed only at a point of sewer inflow, a 'Q' point.

(ii) It will deliver flow to a force main which will be extended no further than the next Q point, e.g. from point B to C, C to D or even from A, the initial inflow point and origin of the sewer, to B.

The selection process adopted will be as follows:

(a) Insertion of one intermediate pump station at a Q point selected automatically by the computer as being the most favourable site for such an installation.

The selection criterion may include the following factors, singly or in combination:

(i) Greatest rise in level from one Q point to the next,

(ii) Greatest cumulative overburden measured above minimum grade sewer between one Q point and the next.

A negative factor, which may be considered in the selection criterion would be

(i) Re-emergence of minimum slope sewer above ground surface between delivery point and the (third) Q point following.

If the penultimate Q point Y say, is selected for that imtermediate pump station, no pumping station will be required at the last point Z. However, for the cost analysis, each intermediate station will be assumed to have a cost levied against it, apart from pumping, to allow for remoteness from the ultimate treatment plant, access, etc. According to this philosophy, location of a pump house at this penultimate station Y will therefore not necessarily prove more economical than the construction of a pump station Z, as it would appear from mere consideration of pumping and excavation costs.

(b) Repetition of the previous analysis for other possible locations ranked 2nd, 3rd etc. according to the desirability criterion.

(c) Study of two intermediate pumping stations, one at the Q point with the highest desirability criterion, and the other at the point next on the list.

(d) According to the complexity of the sewer line, i.e. for a line incorporating a large number of Q points and possible sites for intermediate pumping plants, other combinations may be considered.

Problem E

In the foregoing, the basic problem that has been elaborated on is the selection of a profile of a trunk sewer along a specified layout, with or without intermediate pumping stations. Additions to these programmes, such as the allowance in the excavation cost formulae for the presence of hard rock rather than overburden, may be made, but are hardly worthy of separate comment: They do not constitute a real added complication changing the nature of the solution process. This remark also applies to possible competing layouts in plan for the sewer, with different collection facilities, etc. Each can be optimized as above, and a comparison made between alternative layouts on a basis of cost.

In order to make the computer analysis respond fully to the problems faced at South Conception Bay, in Problem E, cost comparisons should be established for separate sewer units, constituted by a trunk main terminating at its own treatment plant.

Local topography will dictate the site of the treatment plants. Each individual main will be treated as above. The task for the computer is to be able to use the input data that would be prepared for the complete single trunk main problem, with the minimum of modification, and even, in an ultimate phase, to establish the cost of local treatment of each and every inflow Q and local reject of the treated effluent. In essence, therefore, in this stage, sections of trunk main will be successively deleted, and the cost of establishing separate treatment plants at the end of each truncated portion of main added to the cost of the remainder of the sewerage system. In the assessment of treatment plant costs, the cost of building the requisite effluent outfall will be added, as a last refinement.

APPENDIX A

BENTITIC MARINE BIOTA

IN

CONCEPTION BAY SOUTH

A REPORT

BY

DR. J.M. GREEN

MEMORIAL UNIVERSITY OF NEWFOUNDLAND

FENCO

INTRODUCTION

The purpose of this report is to provide a preliminary assessment of the ecological impact of discharging sewage of varying degrees of treatment into Conception Bay. It is assumed that the wastes referred to here are domestic sewage as defined in "Environmental Management Regulations 1972" issued by the Province of Newfoundland. It should also be noted that the report considers only the effects of effluents on the marine biota and does not consider aesthetic and human health aspects of waste disposal. The latter considerations are obviously important, especially in view of the already heavy and increasing recreational use of the south-eastern part of Conception Bay by resident and tourist boaters, sport fishermen and scuba and skin divers.

Marine Life Considerations

Qualitative and quantitative descriptions of the benthic marine life in the areas of Chamberlin's Pond and Kelligews

Point, two sites chosen as possible outfall locations, have been presented earlier. The field survey on which that report was based was conducted in February, 1974 and consequently provided limited information on seasonal changes in the marine life of the areas. Year round underwater observations since 1971 of the marine life at Broad Coves, Conception Bay (St. Phillips) by the author have provided seasonal data, particularly on the fish. Studies by Kennedy and Steele (1971), Wells (in preparation) and Van Gulpin (in preparation) also provide additional information on the biology and seasonal movements of inshore fish along the southeastern shore of Conception Bay.

The February 1974 survey established that except for lobsters, which may be relatively abundant at the Kelligrew's site, neither site has extensive populations of commercially valuable invertebrates, and it is not likely that this situation changes seasonally. Although few fish were observed during the survey, it is important to note that these areas of Conception Bay are important spawning grounds for several commercially important fish as well as a number of fish which, while of no direct commercial importance, are ecologically important.

Capelin, winter flounder and lump fish move into shallow water (less than 10 m) in large numbers in the spring and early summer to spawn. These species lay benthic eggs which remain on the bottom until they hatch, the larvae then become free swimming. Other fish which lay benthic eggs close to shore are the sea snail, radiated shanny, rock eel and shorthorned sculpin. The first two species spawn in the spring and early summer, while the latter two spawn in the winter. Since it is the egg and larval life forms of fish which are most easily effected by pollution (Parsons 1972), waste disposal should be done in such a way as not to pose a threat to these stages.

Another fish which is abundant in shallow water in this part of Conception Bay, but which would not have been seen during the February survey is the cunner. This species is active during the summer and can avoid areas of environmental stress, but during the winter it hides under rocks and in crevices and becomes completely inactive. Thus, during the winter this species is highly vulnerable to adverse environmental conditions as indicated by a mass mortality of cunners at Upper Gullies, Conception Bay in February 1973 (Green, in press). During its winter torpid state, the cunner could be adversely effected by untreated domestic sewage.

Mariculture Considerations

Although no mariculture programmes, except some experimental work being done by Dr. John Allen of Memorial University, are currently under way along the south -eastern shore of Conception Bay, it would seem inevitable that mariculture operations using either shellfish, fin fish or algae will be established in this area if water quality is maintained. The salt water ponds such as Chamberlin's Pond are of particular interest in relation to mariculture because of the possibility they provide of controlling temperature, solenity, etc. (OFY Mariculture

Potential Study 1972). The necessity of maintaining high quality water for future mariculture operations should be considered seriously. Witness the tremendous cost of providing clean water to fish processing plants in eastern Canada (Blackwood 1972).

Consideration of Physical Environmental Conditions

Several physical environmental conditions are particularly important in relation to the effect on marine organisms of discharging domestic sewage along the south-eastern shore of Conception Bay. These are listed below:

(1) the gradually sloping sea bed and the relatively shallow water several thousand feet from shore

(2) the lack of strong tidal or along shore currents

(3) the onshore prevailing wind during part of the year

(4) the inshore ice cover in winter with the possibility of complete ice cover of much of the Bay.

The net effect of these conditions is to greatly reduce the amount of mixing of the inshore water, especially during certain periods of the year, therefore, increasing the likelihood that wastes discharged inshore could adversely effect benthic marine life.

Ecological Assessment and Recommendation

Based on the nature of the marine communities, the desirability of maintaining water quality for mariculture and the fact that several physical environmental factors reduce the mixing of inshore waters, it is my opinion that the inshore discharge of effluent from domestic sewage which has not had secondary treatment would have adverse biological effects.

The discharge of effluent from primary treatment through a diffuser located several thousand feet from shore (in 60-100' of water) would likely cause no noticeable environmental changes, provided appropriate dilution was achieved, except for possible enrichment of offshore benthic and planktonic communities. This disposal method would, however, 'waste' organic matter which could otherwise be used in agriculture and mariculture operations. Further, if such disposal methods were used throughout Conception Bay, adverse biological effects would eventually occur.

Because of the limited mixing of inshore water, the effluent from secondary treatment should be discharged through a diffuser designed so as to prevent a nutrient rich 'lens' of low solenity water from forming on the sea surface. The local enrichment that might be caused by this effluent would not in my opinion have an adverse effect on the present marine communities in the area.

As far as locating treatment facilities and outfalls at Kelligrews Point and or near Chamberlins Pond, the field survey indicated that this would not disrupt or permanently change unique or especially productive or diverse marine communities.

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APPENDIX A

BENTHIC MARINE BIOTA

IN

CONCEPTION BAY SOUTH

A REPORT

BY

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INTRODUCTION

On february , and , a survey was conducted on the benthic marine biota at three sites along the southeastern shore of Conception Bay. Two of the sites were near Chamberlains Pond and the third site was located near Kelligrews (see Fig. 1). The purpose of the survey was to determine the species diversity and relative abundance of organisms at these sites as a partial basis for evaluating the environmental consequences of placing sewage outfalls at or near these locations, and to provide baseline biological data for these sites. This report presents the results of the three days of field work. Recommendations concerning the discharge of sewage into Conception Bay will be presented separately.

Methods

All subtidal observations were made by dividing at each station. One transect was made at each station using a 100 yd. plastic survey chain to measure distance so that location of any features could be plotted.

Depths were measured with a depth gauge. The direction of each transect was that plotted on the chart for each outfall on a bearing maintained with a Sucento wrist compass. Records of species present, location, depth, abundance (percent cover or relative) were written on plexiglass sheets and later transcribed. Species were collected for identification when required. Two additional transects were made at stations 1 and 2 at 200 yd. intervals in order to determine any differences from the main transect. In each case the secondary transects were so similar to the primary one that no seperate account is warranted.

SUMMARY DESCRIPTION OF THE BENTHIC BIOTA AT STATION 1, 2 AND 3.

Station 1

The intertidal region here consists of smooth mainly small (6" to 2' diameter) boulders and scattered larger boulders. These show a fair amount of abrasion and quite small amounts of Ascophyllum, Fucus and Ralfsia are the main organisms present. In the shelter of larger rocks scattered colonies of Littorina are found. The only clear separations of zones occurs low in the intertidal near the upper limit of Strongylocentrotus. The fucoids stop abruptly while the encrusting coralline algae become conspicuous (mainly Lithothamnium and Clathromorphum) and Strongylocentrotus and Littorina become more conspicuous. As depth increases, there are no more obvious zones but the composition of the communities gradually change. Near the 10' depth the corallines have become much more common than in shallow water, often forming 30 to 50% cover. At these depths Acmaea, the limpet is especially abundant with Littoria and Strongylocentrotus.

A few Desmarestia and Agarum plants may be seen as well as moderate amounts of Volsella, Buccinum, Ophiopholis and Myxicola. The effects of waves and storms diminish rapidly with depth permitting more fragile formations than in the shallow water. In 20' to 30' depth much of the bottom is covered with a bed of loose Lithothamnium balls known as Rhodoliths. These form a rocky carpet over an area which would otherwise be sandy. These balls are often hollow and provide shelter for large numbers of animals. The most common inhabitants at this station are the polychaete worms Nereis and Myxicola. the tunicates Molgula and Didenum and the ophiuroid Ophiopholis. Rocky patches at these depths are dominated by attached Lithothamnium and Clathramorphum, and some Agarum. Some fine sandy patches are first obsered here at depths near 30 feet about 450 to 500 yds. from shore. The surface of the sand was largely brown from populations of diatoms. Burrowing in the sand were large numbers of unidentified polychaetes and a few specimens of an unknown anemone.

The summer situation would differ mainly in the addition of plants and animals to the present description. The fucus zone would contain many shortlived algae including Chordaria, Acrosiphonis, Pilayella, etc. The shallow subtidal to perhaps 10' would contain beds of Chorda, Dictoysiphon and Ectocarpus. The deeper water would probably contain more algae such as Desmarestia, Polysiphonia and others. All the subtidal would contain more fish.

Station 2

The unstable nature of the loose rocks near shore and in the intertidal at this station prevents establishment of permanent communities. Calm weather would permit colonization by Ulothrix, Diatoms, Urospora, Pilayella but these would be removed by the next storm.

The subtidal zones are similar to station 1 with the addition of quite extensive sandy areas at depths of 10' and deeper. The biota is less diverse and abundant but the previous description mainly applies.

Station 3

Large (up to 10' diameter) boulders are quite common at this location and account for many differences between this station and others. The intertidal region is largely protected from ice and storm scouring and as a result, the permanent beds of fucoids are much more luxurious covering up to 30-40% of the area. Lichens, a few patches of Phymatolithon lenormandi and hydroids are some of the species associated here and not found at the previous stations.

The lowshore starts comparably to Station 1 with its corallines, urchins and Littorina but the abundance, especially algal % cover is about twice as high. Another feature in this zone is the frequent 'turf' of diatoms, Pilayella, Scytosiphon and Petalonia on the top surfaces of boulders forming a brown 'cap' fringed by grazing Littorina, Acmaea, Strongylocentrotus. Both Corallina officinalis and Phymatolithon laericgatum form a major component of the algal cover. Baccinum is much more (24X) common here than at Station 1.

The persistance of large bounders, sometimes in ridges presents a very irregular substrate with ample shelter for a wide range of organisms. The fish Ulvaria, Pholis, Gadus and Macrozoasces all sheltered in this rock. A fair number of inactive lobsters were observed in caves under rocks and they may prove to be very abundant in summer. A maximum depth of just over 25' was reached over 400 yds. from shore at this station.

The main seasonal difference here will likely occur in the lowshore and upper intertidal. Chorda beds will be much denser than at Station 1 and will probably extend deeper. Ceramium sp. and Cystoclonium may be common in the shallow parts of the beds in addition to the species of Station 1.

The organisms obsered at the three stations are presented in Table 1 and the depth distribution and relative abundance of dominant organisms at Stations 1, 2 and 3 are shown in Figures 1, 2 and 3 respectively.

Summary of Field Survey

All of these stations are quite typical of Conception Bay in particular and much of the Newfoundland coast in general. There is a low variety of marine life and nothing unique was observed in these marine communities. Late winter is probably the worst time of the year for surveying these areas. In summer many more organisms would be evident. The most notable are probably the algae Chorda Filium, forming extensive beds with large amounts of associated Ectacarpus, Dictyosiphon and Ceramium, and the fishes. Of the latter, species such as capelin, winter flounder, cunner, rock eels and the radiated shanny would be expected to be abundant at these sites at various times of the year. In addition, such species as the lobster would be more active, and so many more would be seen.

The main differences between these stations seem substrate dependent. Station 3 with the most cover from large boulders has the greatest abundance and variety of organisms while the unstable rock and shifting sand of Section 2 has the least.

All stations, especially 1 and 2 show evidence of severe winter ice scouring to depths of at least 20 feet. Station 3 appeared to be somewhat more protected from ice damage as indicated by the relatively more developed Fucus and Astophyllum beds.

APPENDIX B

OUTFALL DIFFUSER MODEL

IN

CONCEPTION BAY

A REPORT

BY

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(1) OCEANOGRAPHIC CONDITIONS INFERRED FROM TOPOGRAPHY

Conception Bay lies at the eastern extremity of North America, at the southern edge of the Labrador sea and is exposed directly to N.E. winds blowing off the ocean, while receiving only little protection from S.E. to S.W. winds. The mean wind speed at neighboring St. John's is a high 6 $msec^{-1}$ or so in the summer and 8 $msec^{-1}$ in the winter, calm days being quite infrequent. The prevailing winds are generally westerlies, or W.S.W. in summer.

The axis of the bay is oriented almost N-S., its length being about 50 km., width 25 km. At the head of the bay lies Holyrood, where tidal observations are available and show a modest tidal range of the order of 1 m. The depth distribution in the bay is characterized by a deep trench situated to the west of Bell Island, the depth of which is in excess of 200 metres. To the south and east of Bell Island the bay is shallower. It is in this part, some 15 km. north of Holyrood that the proposed outfall sites are located. Given the small tidal range, the fact that the outfall sites are close to the head of the bay, and the existence of a deep trench further west, it is certain that tidal currents off Kelligrews and Manuels are quite weak, of order 2 cm sec^{-1}. On the other hand, in these shallow coastal waters wind-driven currents should be quite strong, noting in particular the almost continuous action of moderate to strong winds. The typical speed of wind-driven coastal currents under the circumstances prevailing in Conception Bay should be at least 30 cm sec^{-1}, possibly 50 cm sec^{-1}, and they should completely dominate tidal motions.

During the summer, stratification is certain to develop in the deeper parts of the bay, with a seasonal thermocline becoming established at 20 - 30 metres depth. This has no direct effect on the shallow coastal waters off the proposed outfall sites which are likely to remain well mixed to the bottom, but indirectly it contributes to the strength of coastal currents, by the "coastal jet" mechanism (inflow of warm water into the coastal zone, see Csanady, 1972). In the winter the shallow waters come to be ice covered, but continuous ice cover presumably affects only a small portion of the bay, most of it still being exposed to the influence of winds. Under the ice, near-shore currents should be much slower than if the wind was directly acting on the surface.

Wind-driven nearshore currents flow predominantly parallel to the shore, north or south according to the direction of the last major wind impulse. However, winds having a component normal to shore also set up a circulation in a section perpendicular to shore, with the top layers flowing one way, and return flow occurring at depth. In particular, the frequent west - southwest winds are certain to produce onshore drift of the surface layers off Kelligrews and Manuels, compensated for by offshore flow at depth. In a strong, well established shore-parallel current this pattern of crossflow merely enhances mixing in an onshore direction, but if the depth-average current is weak or nonexistent, the onshore-offshore motions may become of critical importance in determining the fate of sewage released offshore.

Quantitatively, the following estimates may be made. A wind of speed V cm sec^{-1} exerts on the underlying water surface a stress of

$$\tau = c_d \rho V^2 \tag{1}$$

where ρ is air density and c_d is a drag-coefficient, the value of which lies between 1.5×10^{-3} and 2.5×10^{-3}, higher values applying in lighter winds (Wu, 1969). This gives a stress of about 1 dyne cm^{-2} in a 6 $msec^{-1}$ wind, 10 dynes cm^{-2} in a 20 $msec^{-1}$ (40 knot) gale. In the absence of other influences the wind accelerates the water, imparting to it momentum at the rate τ per unit surface area. Thus the average velocity produced by a wind of duration t in depth h is

$$V = \frac{\tau t}{\rho h} \tag{2}$$

In shallow water high current speeds may therefore be expected (Csanady, 1973). A stress of 1 dyne cm^{-2}, exerted for 10^4 seconds (2.8 hours) on a water column 10 metres deep should produce according to Eq. (2) a current of velocity 10 cm sec^{-1}.

At much higher velocities bottom friction interferes with the further acceleration of the water. The magnitude of the shear stress at the bottom is also given by Eq. (1), but now with ρ = water density, V = v = current speed, and a drag coefficient of 2×10^{-3} to 3×10^{-3} (Bowden et al, 1959). Thus a surface stress of 1 dyne cm^{-2} exerted by the wind can be completely balanced by the bottom stress, if the current velocity is around 20 cm sec^{-1}. The velocity determining bottom stress is what occurs close to the bottom, say 2 or 3 metres above the bottom. The surface layers tend to move rather faster than this, hence the "typical" velocity estimates quoted before. In greater depths or above a thermocline the surface layers can move much faster than the bottom ones.

The effect of wind in creating velocity *differences* in a water column is particularly clear in the case of wind blowing perpendicular to shore. A pressure gradient in this case becomes established balancing the wind (the surface level of the water rises slightly at the shore). The surface layers move with the wind onshore, but the net onshore flow must be zero, and a return current becomes established below the surface (Baines and Knapp, 1965). The typical velocity of the surface is under such circumstances of order $20\sqrt{\tau/\rho}$, which at a stress of 1 dyne cm^{-2} is 20 cm sec^{-1}, while the return flow is much slower, of order 2 cm sec^{-1}. The latter occupies most of the depth, the onshore drift being confined to the top 20% or so of the water column. Under actual field conditions such onshore-offshore drift may be expected to be superimposed on a longshore current under most circumstances. Expected wind effects in the shallow water off a coast are schematically illustrated in Fig. (1).

(2) JET MIXING OF SEWAGE WITH SEAWATER

As the sewage is released from a pipe or a port into the sea, it forms a buoyant jet which tends to rise to the surface ("boil", in common jargon). In this jet-phase the sewage also entrains seawater and mixes with the latter. At release the density deficiency of sewage versus seawater is about 30 parts in 1,000: this reduces progressively as the sewage is diluted. Moderately diluted sewage is still quite buoyant and it tends to form "lenses" or "bubbles" on the sea surface, separated from the seawater below by a "pycnocline" or density-front. The arrangement of light water on top of heavy is quite stable and tends to suppress turbulence and vertical mixing. Thus lenses or bubbles of sewage, once formed, tend to retain their identity for long periods. This is undesirable, and the function of properly designed outfalls is to prevent the formation of stable sewage pools. This can be accomplished by an adequate degree of mixing of the sewage with seawater while still in the jet phase.

Experience off the California coast has indicated that sewage pools break down readily when their dilution is 200 or more (200 parts of seawater to 1 part of sewage). When the winds are stronger, a lower initial dilution is likely to be adequate, an initial dilution of 100 being presumably quite acceptable in Newfoundland waters. The dilution at the surface, at the end of the jet phase, i.e. after a sewage jet has boiled to the surface, is mainly a function of water depth to port diameter ratio h/d, with the densimetric Froude number* having a less pronounced effect (Rawn et al, 1960). An initial dilution of 100 can be accomplished by a depth to port diameter ratio h/d of approximately 80 at low Froude numbers, (order one) and by a slightly higher h/d at higher Froude numbers (order 10). A ratio of h/d = 100 therefore seems a satisfactory design choice, although a higher value would give greater assurances that initial mixing is indeed satisfactory under all circumstances.

Another important design consideration is to provide sufficient seawater necessary for the initial dilution process. The maximum sewage release rate of the outfalls contemplated is 1680 USGPM = 106 liters sec^{-1} $\simeq 0.1\ m^3\ sec^{-1}$ of seawater. Hundred fold dilution therefore requires $10m^3\ sec^{-1}$ of seawater. Given a current velocity of 10 cm sec^{-1}, this much water flows through a section of 10 metres square. In 10 metres depth, the arrangement shown in Fig. (2) should be quite satisfactory. Note that this is somewhat different from my preliminary recommendations, and follows more critical assessment of the initial dilution process. Y-diffusers of this kind have been found to work

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* The Froude number (internal) is defined as $Vj/\sqrt{gh\Delta\rho/\rho}$, where Vj is jet velocity, $\Delta\rho/\rho$ = fractional density defect.

quite satisfactorily elsewhere. The larger spread in the longshore direction is desirable to allow for the possibility of particularly slow currents. If the water depth over the diffuser (at low tide) is less than 10 metres (33 ft), the size of the diffuser manifold should be scaled up to provide the necessary large cross section.

It does not seem to be possible to trade off the required degree of initial dilution against any other factor, such as even secondary treatment. Should the sewage, whether treated or untreated, be released without proper initial dilution, it would form a readily observable plume. Even if quite innocuous, such a plume can certainly create public relations problems.

The initial dilution problem is discussed exhaustively by Wiegel (1964), chapter 16.

(3) STRAIGHT PLUME DIFFUSION MODEL

Effluent released into coastal waters travels most of the time parallel to the shore for long distances. Experience in wind-dominated coastal waters has shown this to be the case. (Csanady, 1970): a long nearshore plume forms, in a direction determined by the dominant wind impulses of the immediate past. As weather cycles have a typical period of some 100 hours, also currents typically reverse direction once every few days. During reversal, flow conditions are generally chaotic (and can be critical from the point of view of effluent dispersal, as we shall discuss later) but the net total effect is usually to remove the previously contaminated water from the shore zone and mix it efficiently with the much larger water masses of the open sea. Thus after each such current reversal episode a new effluent plume may be assumed to form. The total length of each such plume is in this manner limited by the persistence of currents in one direction, which is as we just said, of the order of 100 hours. The typical current excursion during a period of this order is 20 cm sec^{-1} times 100 hours or a distance of the order of 60 km. In Conception Bay a northward displacement alongshore of this magnitude will take the effluent out of the Bay altogether. It is not to be expected that such a near-shore plume enters the next bay along the coast to the east, because the flow is likely to separate from the shore somewhere near the mouth of the bay. What one may define as the "influence zone" of the plumes released at the proposed sites is therefore essentially the entire eastern side of the bayshore. When the winds drive the flow toward the head of the bay at Holyrood, the coastal currents may be expected to have a stagnation point in that neighborhood, where effluent would flow out to sea, or rather toward the deep central trench of Conception Bay. Southward plumes would therefore have a maximum length of some 15 km, northward plumes about 30 km.

A conventional diffusion model may be applied to such a shore-parallel plume. If the centerline of the initially released plume is at a distance b from the shoreline, one would expect to approximate field conditions by assuming that the plume distributes itself about a centerline parallel to the shore at approximately the same distance b, growing laterally as it travels downstream. A complication is, however, that observed plumes meander about such a mean position in an irregular way, by a substantial fraction of the total distance from shore to plume. In calculating shoreline concentrations of effluent it is therefore safer to assume that the centerline is some fraction of b, say Y = 0.6 b, from the shore. One may look at it also as if eddies of size 0.4 times b were bringing the plume shoreward by this distance on at least some occasions.

A conventional, "Gaussian" diffusion model may now be constructed by centering a straight plume at the distance 0.6 b from shore. Diffusion models of this kind have been used for some time in dealing with air pollution problems (Sutton, 1953; Csanady, 1973). The same kind of model was applied to sewage plumes by Brooks (1960). Our main interest is in calculating shoreline concentrations. Brooks' model may be used for this purpose after adding a mirror image term to satisfy continuity ("reflection" of the plume by the shoreline). Brooks has also recommended the use of a variable eddy diffusivity. Subsequent experimental work in shallow coastal waters (Foxworthy et al, 1966; Csanady, 1970) has shown this not to be necessary, a constant diffusivity model being more realistic. A good "typical" value of horizontal diffusivity in such situations was found to be $K = 1000\ cm^2\ sec^{-1}$. The finite size of the initial sewage cloud is well modelled by a line source of length 2w perpendicular to the current. The current is assumed to be shore-parallel, and of constant speed U. Fig. (3) illustrates this diffusion model.

Important parameters determining the concentration of sewage χ (parts of sewage to parts of seawater) are the concentration χ_o after initial dilution, the current speed U and diffusivity K we already mentioned. In assessing the effects of sewage pollution also the die-off rate of bacteria must be considered. If the bacterial count is ϕ bacteria per hundred milliliters, the rate of die-off can be represented by

$$\frac{\partial \phi}{\partial t} = -k\phi \qquad (3)$$

where k is a decay constant of dimension, time^{-1}, its reciprocal being a decay time-scale. Data summarized by Orlob (see Wiegel, 1964) show that k^{-1} is generally of order 10 hours, $k = 0.3 \times 10^{-4}$ sec^{-1}. However, the variability in k is rather large and in biologically productive waters bacterial decay is generally fast. The waters around Newfoundland are known to be quite productive and one would expect a generally high decay factor. However, it is uncertain whether such high productivity in fact characterizes the shallow waters between Bell Island and the proposed outfall sites.

The concentration field of a finite line source as per Fig. 3 and a Gaussian model is, as may be deduced from formulae in standard texts,

$$\chi = \chi_o \left[\text{erf}\left(\frac{w + y}{\sqrt{2}\sigma}\right) + \text{erf}\left(\frac{w - y}{\sqrt{2}\sigma}\right) + \text{erf}\left(\frac{w + y + 2Y}{\sqrt{2}\sigma}\right) + \text{erf}\left(\frac{w - y - 2Y}{\sqrt{2}\sigma}\right) \right] \quad (4)$$

were erf ( ) is the error function, and σ is the standard deviation given by

$$\sigma^2 = 2K \frac{x}{U} \quad (5)$$

so that σ grows with distance from the source as $x^{\frac{1}{2}}$. Of greatest interest are concentrations at shore, $y = -Y$, which are

$$\chi_s = \chi_o \left[\text{erf}\left(\frac{w - Y}{\sqrt{2}\sigma}\right) + \text{erf}\left(\frac{w + Y}{\sqrt{2}\sigma}\right) \right] \quad (6)$$

This gives very low concentrations at small distances from the source, the physical reason being that the plume has to grow laterally before its effects can be observed at the shore. At larger distances (where σ is comparable to Y), and for the usual small w/Y ratios, Eq. (6) can be adequately approximated by

$$\chi_s \approx \chi_o \sqrt{\frac{2}{\pi}} \frac{w}{\sigma} \exp\left(-\frac{Y^2}{2\sigma^2}\right) \quad (7)$$

$$(w << Y)$$

This may also be stated in terms of the source-strength of sewage, Q, for which the initial balance holds:

$$Q = whU\chi_o \quad (8)$$

Substituted into Eq. (7) this yields

$$\chi_s = \frac{Q}{hUY} \psi\left(\frac{\sigma}{Y}\right) \quad (9)$$

where we have introduced the auxiliary function $\psi(\ )$:

$$\psi\left(\frac{\sigma}{Y}\right) = \sqrt{\frac{2}{\pi}}\ \frac{Y}{\sigma}\ \exp\left(-\frac{Y^2}{2\sigma^2}\right) \tag{10}$$

The function $\psi(\ )$ is shown in Fig. (4). Physically, it represents nondimensional shore concentration versus nondimensional plume width. Its maximum occurs where $\sigma = Y$, and is $\psi_m = 0.479$. To relate the corresponding value of σ (=Y) to distance alongshore, Eq. (5) has to be used, which involves the diffusivity K and the current speed U. Note that the rather uncertain estimate one can make of K only affects where the maximum shore concentration occurs, not its value. Furthermore, the curve $\psi(\ )$ is seen to be a flat one near its maximum, so that concentrations at shore should remain close to the maximum for a long stretch of the shore.

If now the available shoreline is long enough for the plume to diffuse to the shore (i.e. if ψ_m^{-1} is reached somewhere within the range of the longshore plumes), the maximum shore concentration is:

$$\chi_m = \chi_o \frac{w}{Y} \psi_m = 0.479 \frac{w}{Y} \chi_o \tag{11}$$

The dilution of the sewage field through horizontal diffusion is at this point:

$$\frac{\chi_o}{\chi_m} = \frac{Y}{w}\psi_m^{-1} \simeq 2\frac{Y}{w} \tag{12}$$

showing the importance of releasing the sewage sufficiently far from shore. If, for example, w = 5 metres, Y = 400 metres, diffusion will dilute the sewage-seawater mixture released by a factor of 160 before reaching shore. This is of course subsequent to an initial dilution of 100 or so, giving a total dilution of sewage by a factor of 1.6×10^4. With a release distance b = 400 metres, if we allow for meandering by taking Y = 240 metres, the dilution through diffusion will only be about 100, or a total of 10^4.

We can also examine how far from the release the diffusing plume reaches the shore. Fig. (4) shows that ψ begins to rise steeply where $\sigma/Y = 0.4$. This gives, with Eq. (5),

$$\chi = \frac{U\ (0.4\ Y)^2}{2K} = 4.6\ \text{km}$$

using $U = 10\ \text{cm sec}^{-1}$, $K = 1000\ \text{cm}^2\ \text{sec}^{-1}$ and Y = 240 metres. At four times this distance the maximum value of ψ is pretty well reached, so that the location of ψ_m is barely within the reach of the sewage plumes expected to develop in Conception Bay.

The above model refers to the diffusion of conservative substances. Bacteria contained in the sewage behave differently, because they decay, as we have already seen. Taking the decay law, Eq. (3), into account, we find similar formulae for the bacterial count ϕ as we found for the concentration χ, except that we have to replace χ_o by $\phi_o \exp(-kx/U)$, where ϕ_o is bacterial count after initial dilution. At shore specifically the bacterial count will vary as

$$\phi = e^{-kx/U}\ \frac{w}{Y}\ \psi\ \phi_o \tag{13}$$

In other words, the extra reduction of bacterial count through die-off is by the factor exp (kx/U). Using $k = 0.3 \times 10^{-4}\ sec^{-1}$, $U = 10\ cm\ sec^{-1}$, this factor is:

at x =	5 km	7.5 km	10 km	15 km
$e^{kx/U}$	4.48	9.49	20.09	90
ϕ/ϕ	1926	2118	2087	9351

The last line gives the reduction in bacterial count due to diffusion <u>and</u> die-off. With an initial dilution of 100 the total reduction in bacteria would be by a factor of 2×10^5. Given a bacterial count in the sewage of 2×10^6 (which is sometimes assumed in such calculations) we find predicted maximum shore concentrations of 10 bacteria per 100 ml, which is quite acceptable.

(4) <u>ONSHORE WIND MODEL</u>

The straight plume model may be regarded as describing conditions expected to occur 80 or 90% of the time. In this light it is indeed necessary that predicted bacterial counts on shore be low: many ordinances require long term means of coliform bacteria as low as 10 bacteria per 100 ml. It is generally recognized that occasionally much higher concentrations will occur. Practical experience has shown that the critical conditions are generally associated with strong onshore winds (Rawn et al, 1960; Stewart, 1973). Treatment plants often resort to chlorination during periods of onshore winds. A quantitative model has so far not appeared in the literature for the assessment of such critical conditions. It is, however, not difficult to construct such a model on the basis of conventional diffusion theory.

An onshore wind establishes a "setup", or slight increase of elevation at shore, in such a way that the "effective" gravity, surface slope times gravitational acceleration g, balances the wind stress τ. The water does not remain stagnant, however, but moves with the wind shoreward in the surface layer, while a compensating return flow occurs at depth. The velocity distribution in such wind-drift has already been illustrated schematically in Fig. (1a).

As the initial sewage-seawater mixture is released into such a flow field, the deep layers travel shoreward, the bottom layers seaward. However, vertical mixing distributes the sewage at all distances from shore over the available depth, both shoreward and seaward of the release area. The net effect is very similar to diffusion in a direction perpendicular to shore. Indeed horizontal diffusion in the sea is usually produced by essentially the same mean-flow vertical mixing interaction. The mechanism of this kind of accelerated diffusion is clearly discussed in a recent article by Fisher (1973).

It would be possible to calculate the effective diffusivity associated with the wind-drift velocity profile shown in Fig. 5. This, however, would require some theoretical development and preferably experimental verification. Here we shall simply use an increased value of diffusivity, suggested by experimental evidence in the Great Lakes, obtained on occasions of non-uniform currents (Csanady, 1970). A value of $K = 2000\ cm^2\ sec^{-1}$ seems appropriate, although it must be admitted that this is a rather uncertain estimate.

To assess the most unfavorable conditions, we consider the case with no longshore current at all, but effective diffusion normal to shore, caused by onshore winds. Such conditions can clearly last only for a limited period T, perhaps 10^4 to 10^5 seconds (2.8 or 28 hours). To model such an unfavorable episode, we consider the one-dimensional diffusion problem of a line source parallel to shore beginning to operate at time $t = 0$. The shore, located at $y = -Y$ begins to receive significant quantities of effluent (½ the initial concentration χ_0) when σ is equal to Y, where

$$\sigma^2 = 2Kt \tag{14}$$

Given $\sigma = Y = 400$ metres, $K = 2000\ cm^2\ sec^{-1}$ we find $t = 4 \times 10^5$ seconds, or rather longer than the expected duration of the most unfavorable conditions. Even a considerably larger effective K would not change this conclusion. Note, however, that t varies as Y^2, and halving Y would reduce t to 10^5 seconds, in which case ½ the initial concentration could be observable at shore during the longest stagnation periods.

If the unfavorable conditions persisted for long enough, our simple line source model would predict that the initial sewage-seawater concentration would be reached at shore. This holds for a <u>conservative</u> diffusing substance. For a <u>decaying</u>

substance, such as bacteria, the decay being governed by Eq. (3), the maximum shore concentration would be only

$$\chi_s = \frac{\chi_o}{\cosh\,(Y/\sqrt{K/k})} \tag{15}$$

This may be shown simply from the classical diffusion equation with a decay term. Given $K = 2000\ \mathrm{cm}^2\ \mathrm{sec}^{-1}$, $k = 0.3 \times 10^{-4}\ \mathrm{sec}^{-1}$, $Y = 400$ metres, we find $\chi_o/\chi_s = 134$, or quite a substantial reduction in bacterial count through die-off alone. With a bacterial count of 2×10^6 in the sewage, an initial dilution of 100, and the above reduction due to die-off we find a maximum shoreline count of 150 which is quite acceptable for a maximum reading. Note, however, that neither the diffusivity K, not the die-off rate k are reliable figures, and that both of these occur in the exponent of an exponential function.

(5) CONCLUSIONS

The most important points for the design of the proposed sewage outfall may be summarized as follows.

(1) To achieve high, controlled initial dilution the sewage must be released in sufficiently deep water, through a number of ports, each of which is no larger than 1/100 of the depth.

(2) Large quantities of seawater must be readily available for the initial dilution. This requires that the ports be distributed over a sufficiently large area of the seabed.

(3) Under average conditions high further dilution may be

accomplished (by the time sewage reaches the shores) by releasing the sewage sufficiently far from shore at a distance many times the size of the initial cloud.

(4) The same design measure (release far from shore) will also keep shore concentrations within reasonable bounds, at times of the most unfavorable conditions, which occur with onshore winds.

A suitable combination of design parameters is given in Fig. 2 of this report.

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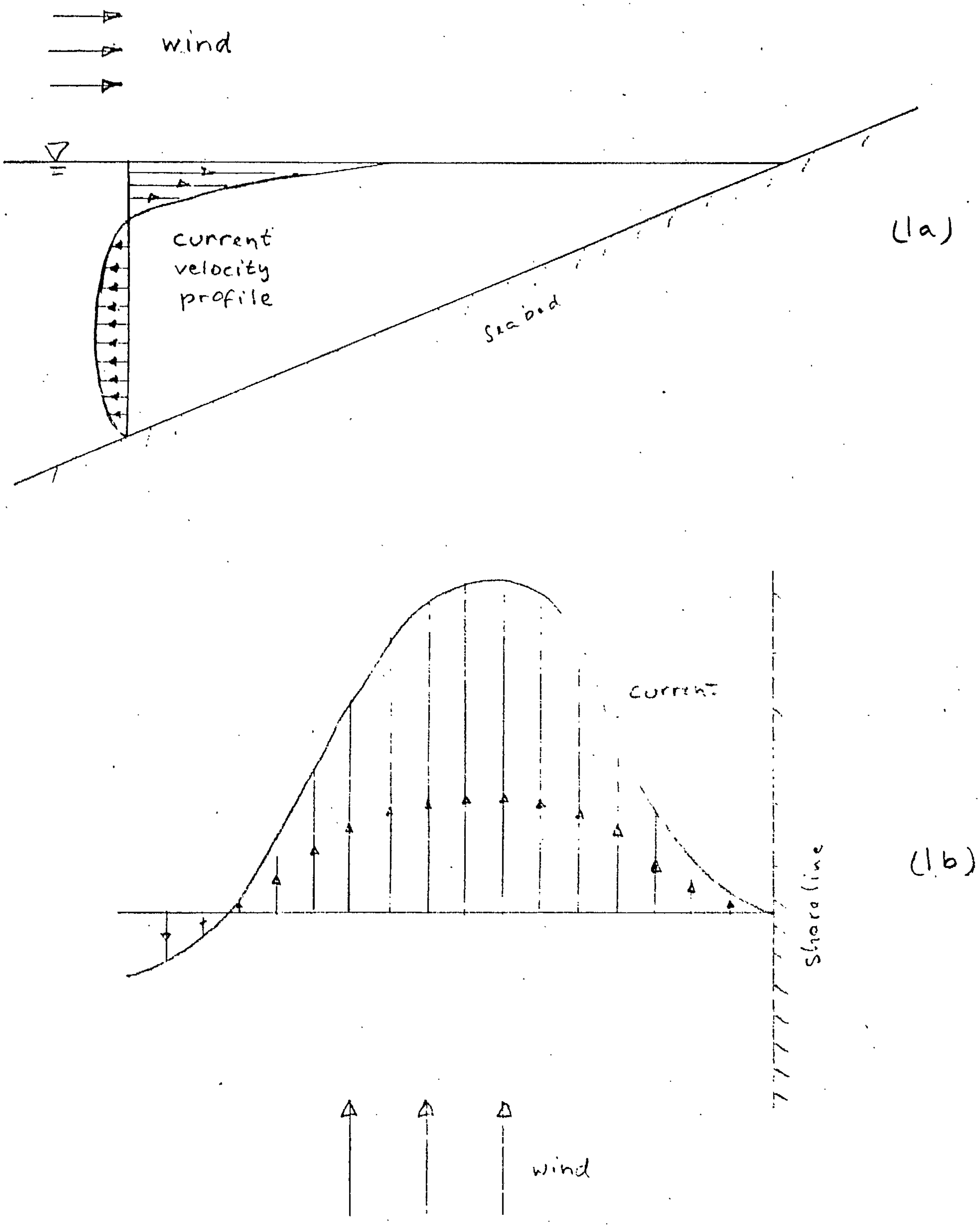

Fig. 1. Effects of onshore (1a) and longshore (1b) winds on currents in coastal zone.

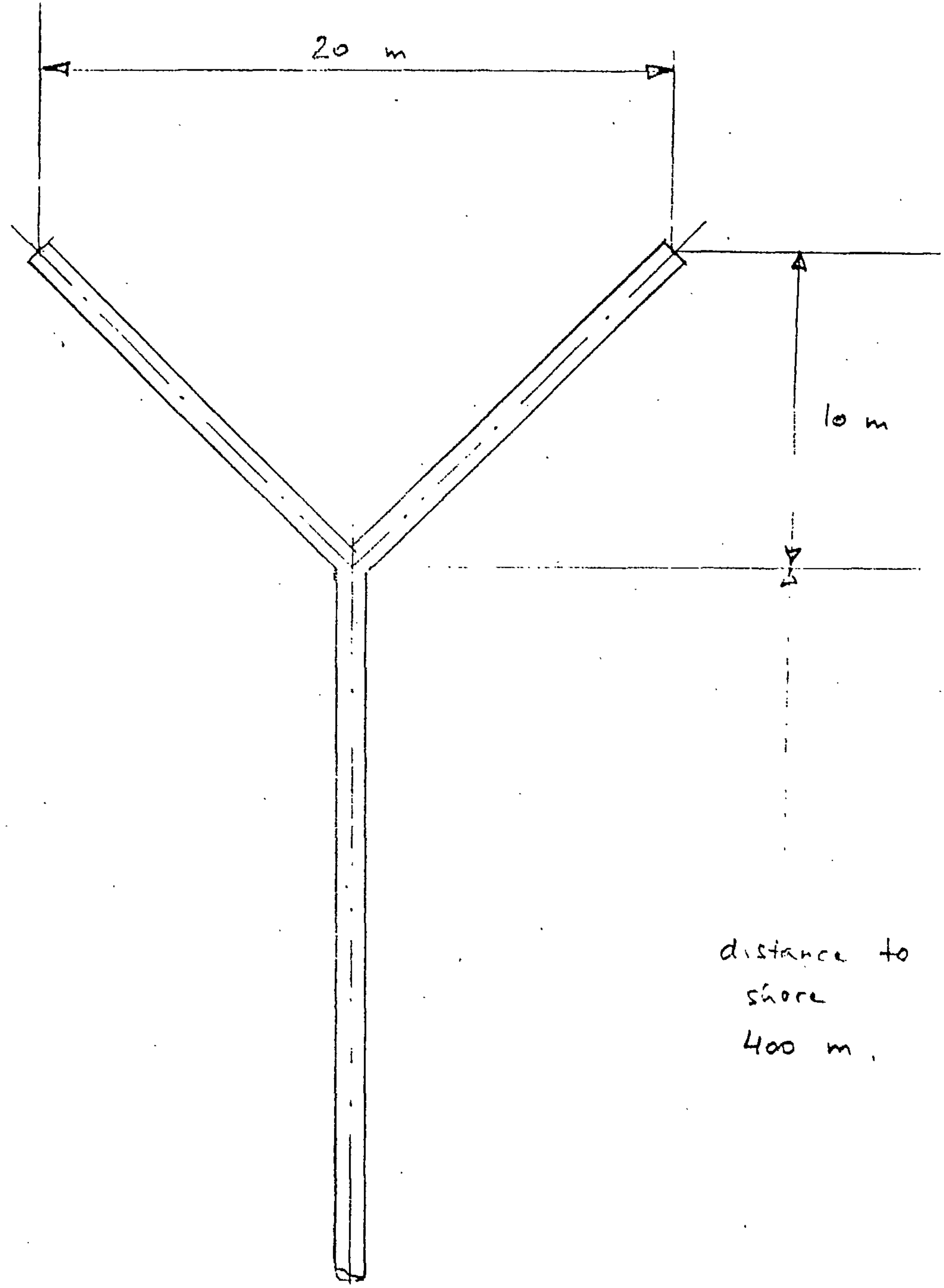

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Fig. 2. Common diffuser arrangement in plan, dimensions as suggested for Conception Bay. Ports are arranged on Y-legs, 16 3" diameter ports being adequate.

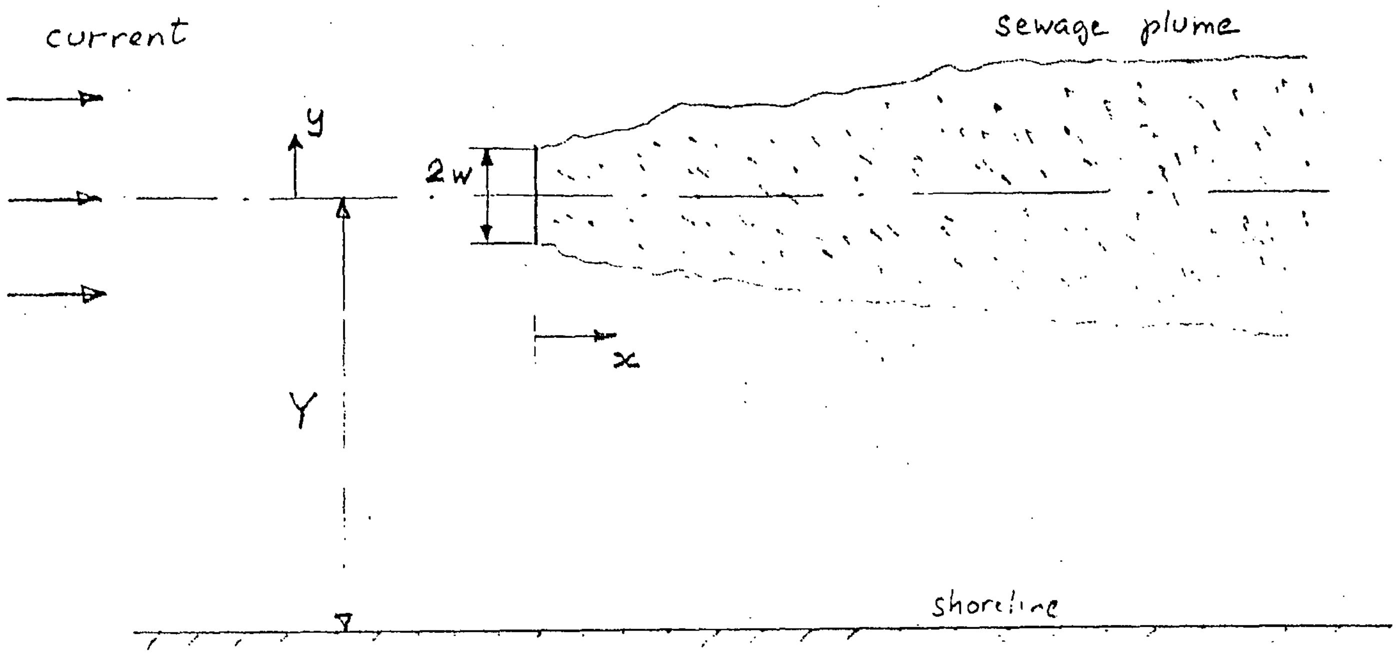

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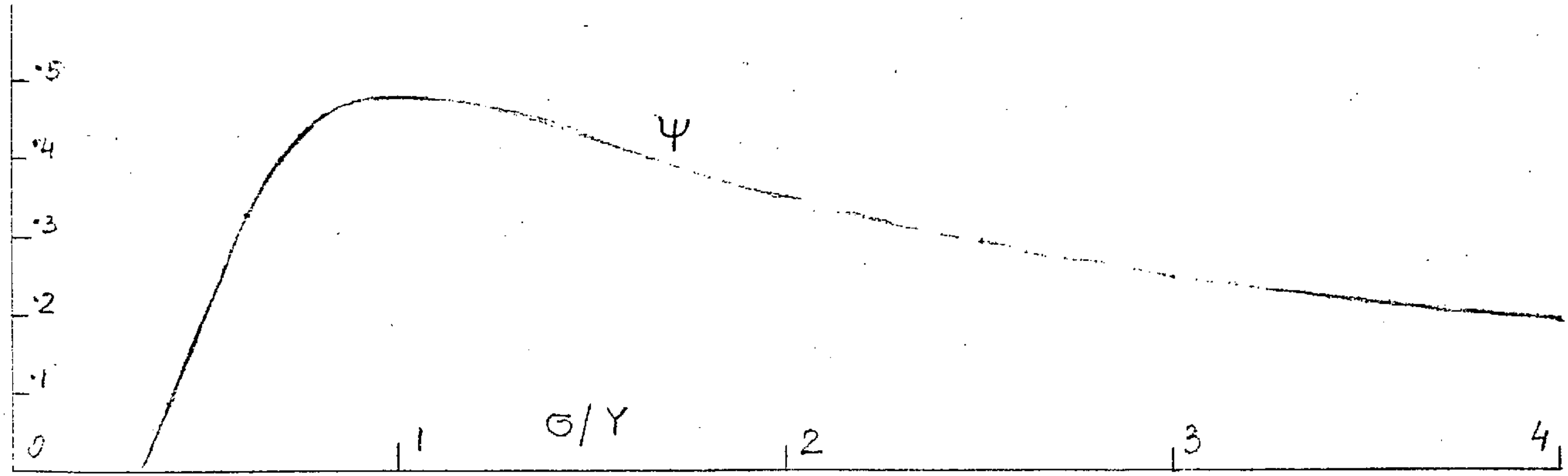
Fig 3. Model of straight sewage plume in uniform longshore current.

4



[redacted] [redacted] Fi[redacted] 4.[redacted]ond[redacted]ens[redacted]al [redacted]re[redacted]nce[redacted]at[redacted] of[redacted]ns[redacted]ati[redacted]su[redacted]anc[redacted] [redacted] [redacted] [redacted] [redacted]

APPENDIX C

ASSESSMENT OF ECOLOGICAL IMPACT

IN

CONCEPTION BAY SOUTH

A REPORT

BY

DR. J. M. GREEN

MEMORIAL UNIVERSITY OF NEWFOUNDLAND

FENCO

INTRODUCTION

The purpose of this report is to provide a preliminary assessment of the ecological impact of discharging sewage of varying degrees of treatment into Conception Bay. It is assumed that the wastes referred to here are domestic sewage as defined in "Environmental Management Regulations 1972" issued by the Province of Newfoundland. It should also be noted that the report considers only the effects of effluents on the marine biota and does not consider aesthetic and human health aspects of waste disposal. The latter considerations are obviously important, especially in view of the already heavy and increasing recreational use of the south-eastern part of Conception Bay by resident and tourist boaters, sport fishermen and scuba and skin divers.

Marine Life Considerations

Qualitative and quantitative descriptions of the benthic marine life in the areas of Chamberlin's Pond and Kelligews

Point, two sites chosen as possible outfall locations, have been presented earlier. The field survey on which that report was based was conducted in February, 1974 and consequently provided limited information on seasonal changes in the marine life of the areas. Year round underwater observations since 1971 of the marine life at Broad Coves, Conception Bay (St. Phillips) by the author have provided seasonal data, particularly on the fish. Studies by Kennedy and Steele (1971), Wells (in preparation) and Van Gulpin (in preparation) also provide additional information on the biology and seasonal movements of inshore fish along the southeastern shore of Conception Bay.

The February 1974 survey established that except for lobsters, which may be relatively abundant at the Kelligrew's site, neither site has extensive populations of commercially valuable invertebrates, and it is not likely that this situation changes seasonally. Although few fish were observed during the survey, it is important to note that these areas of Conception Bay are important spawning grounds for several commercially important fish as well as a number of fish which, while of no direct commercial importance, are ecologically important.

Capelin, winter flounder and lump fish move into shallow water (less than 10 m) in large numbers in the spring and early summer to spawn. These species lay benthic eggs which remain on the bottom until they hatch, the larvae then become free swimming. Other fish which lay benthic eggs close to shore are the sea snail, radiated shanny, rock eel and shorthorned sculpin. The first two species spawn in the spring and early summer, while the latter two spawn in the winter. Since it is the egg and larval life forms of fish which are most easily effected by pollution (Parsons 1972), waste disposal should be done in such a way as not to pose a threat to these stages.

Another fish which is abundant in shallow water in this part of Conception Bay, but which would not have been seen during the February survey is the cunner. This species is active during the summer and can avoid areas of environmental stress, but during the winter it hides under rocks and in crevices and becomes completely inactive. Thus, during the winter this species is highly vulnerable to adverse environmental conditions as indicated by a mass mortality of cunners at Upper Gullies, Conception Bay in February 1973 (Green, in press). During its winter torped state, the cunner could be adversely effected by untreated domestic sewage.

Mariculture Considerations

Although no mariculture programmes, except some experimental work being done by Dr. John Allen of Memorial University, are currently under way along the south -eastern shore of Conception Bay, it would seem inevitable that mariculture operations using either shellfish, fin fish or algae will be established in this area if water quality is maintained. The salt water ponds such as Chamberlin's Pond are of particular interest in relation to mariculture because of the possibility they provide of controlling temperature, solenity, etc. (OFY Mariculture

Potential Study 1972). The necessity of maintaining high quality water for future mariculture operations should be considered seriously. Witness the tremendous cost of providing clean water to fish processing plants in eastern Canada (Blackwood 1972).

Consideration of Physical Environmental Conditions

Several physical environmental conditions are particularly important in relation to the effect on marine organisms of discharging domestic sewage along the south-eastern shore of Conception Bay. These are listed below:

(1) the gradually sloping sea bed and the relatively shallow water several thousand feet from shore

(2) the lack of strong tidal or along shore currents

(3) the onshore prevailing wind during part of the year

(4) the inshore ice cover in winter with the possibility of complete ice cover of much of the Bay.

The net effect of these conditions is to greatly reduce the amount of mixing of the inshore water, especially during certain periods of the year, therefore, increasing the likelihood that wastes discharged inshore could adversely effect benthic marine life.

Ecological Assessment and Recommendation

Based on the nature of the marine communities, the desirability of maintaining water quality for mariculture and the fact that several physical environmental factors reduce the mixing of inshore waters, it is my opinion that the inshore discharge of effluent from domestic sewage which has not had secondary treatment would have adverse biological effects.

The discharge of effluent from primary treatment through a diffuser located several thousand feet from shore (in 60-100' of water) would likely cause no noticeable environmental changes, provided appropriate dilution was achieved, except for possible enrichment of offshore benthic and planktonic communities. This disposal method would, however, 'waste' organic matter which could otherwise be used in agriculture and mariculture operations. Further, if such disposal methods were used throughout Conception Bay, adverse biological effects would eventually occur.

Because of the limited mixing of inshore water, the effluent from secondary treatment should be discharged through a diffuser designed so as to prevent a nutrient rich 'lens' of low salinity water from forming on the sea surface. The local enrichment that might be caused by this effluent would not in my opinion have an adverse effect on the present marine communities in the area.

As far as locating treatment facilities and outfalls at Kelligrews Point and or near Chamberlins Pond, the field survey indicated that this would not disrupt or permanently change unique or especially productive or diverse marine communities.

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A B B R E V I A T I O N S

acre-foot	acre-ft
billion gallons	bil gal
billion gallons per day	bgd
biochemical oxygen demand	BOD
brake horsepower	bhp
capita	cap or c
chemical oxygen demand	COD
cubic	cu
cubic feet per day	cfd
cubic feet per hour	cfh
cubic feet per minute	cfm
cubic feet per second	cfs
cubic foot (feet)	cu ft.
cubic yard	cu yd
degree(s)	deg
degree(s) Centigrade (Clsius)	^{o}C
degree(s) Fahrenheit	^{o}F
diameter	dia
dissolved oxygen	DO
dissolved solids	DS
feet	ft
feet per second	fps
foot	ft

gallon(s) (Imperial)	gal
gallon(s) per capita per day	gpcd
gallon(s) per day	gpd
gallons per day per capita	gpd/cap
gallon(s) per hour	gph
gallon(s) per minute	gpm
gallon(s) per minute per square foot	gpm/sq ft
hour (s)	hr
inch (es)	in.
liters	l
mean sea level	MSL
meter (s)	m
mile (s)	mi
mile (s) per hour	mph
milligram (s)	mg
milligrams per liter	mg/l
minimum	min.
minute (s)	min
mixed liquor suspended solids	MLSS
most probable number	MPN
part (s) per billion	ppb= g/l
part (s) per million	ppm* = mg/l**
part (s) per thousand	ppt

* For gases

** For aqueous solutions; for non-aqueous solutions, correct for density.

pound (s)	lb
pound (s) per day per acre	lb/day/acre
pound (s) per square foot	psf
pound (s) per square inch	psi
pound (s) per square inch absolute	psia
pound (s) per thousand cubic feet	lb/1,000 cu ft
revolution (s) per minute	rpm
revolution (s) per second	rps
second (s)	sec
second feet (cubic feet per second)	cfs
side water depth	SWD
sludge density index	SDI
sludge volume index	SVI
solution	soln
specific gravity	sp gr
square	sq
square foot (feet)	sq ft
square inch (es)	sq. in.
square yard (s)	sq yd
suspended solids	SS
time	t
total oxygen demand	TOD
total solids	TS
total suspended solids	TSS
total volatile solids	TVS

ADDENDUM SHEET

Explaining Minor Report Drawing Omissions

All drawing base maps (of various scales) originate with the Department of Mines and Surveys, Canada.

Drawing No. 1 Sources.	-	Existing land use by Provincial Planning Office, Department of Municipal Affairs and Housing.
	-	Study Area Boundaries in accordance with terms of reference and conditioned by subsequent suggestions from the Provincial Planning Office and detailed study examination of topographic considerations.
Drawing No. 2 Sources	-	FENCO survey combined with local knowledge and data supplied by Provincial Fisheries Department.
	-	Health hazardous areas will be identified and categorized later in the final submission.
Drawing No. 5	-	The population, flows BOD and suspended solids are all projected design figures for the year 1995.
Drawing No. 6	-	The three (thick) lines separate parts to be seen on detail plans 6 - 1 to 6 - 4, inclusive.
	-	Dotted lines show sewerage areas which can be seen on Drawing No. 5.
Drawing No. 7	-	Phase I is based on the present land use plan. Phase II will be based on a future land use plan under preparation by Provincial Planning Office.
	-	The Packages refer to the areas which can be constructed separately but a sequence will have to be adopted, as shown, in order to fully utilize treatment and trunk main pipe facilities. The relative importance of population density was also taken into account. This consideration also applied in locating the treatment site. Package C_1, though quite densely populated, poses difficult conditions and hence may be more expensive to construct sewer systems. The portion of C_I, on northern side of the Manuels river, which is shown as C_I should be designated as B_{II}. Letters I and II denote respective treatment plant.

volatile solids	VS
volatile suspended solids	VSS
volume	vol
weight	wt
yard (s)	yd
year (s)	yr

COLLECTION AND DISPOSAL STUDY

CONCEPTION BAY SOUTH AREA

DRAFT

FENCO

Foundation of Canada Engineering Corporation Limited

LEGEND

LAND USE

- RESIDENTIAL
- INSTITUTIONAL
- INDUSTRIAL
- RECREATIONAL
- ADMINISTRATIVE BOUNDARIES

SCALES

25,000' 12,500'

1" = 25,000'

SEWAGE COLLECTION AND DISPOSAL STUDY
CONCEPTION BAY SOUTH AREA

ADMINSTRATIVE BOUNDRIES AND LAND USE

D.R.E.E. PROJECT NO. 3·2 FENCO PROJECT NO. I-6177-I

FENCO

Foundation of Canada Engineering Corporation Limited

DWG. NO. 1 REV.

604-73

LEGEND

P C P = POLLUTION CONTROL PLANT

SCALES

25,000'

1" = 25,000'

SEWAGE COLLECTION AND DISPOSAL STUDY
CONCEPTION BAY SOUTH AREA

MARINE LIFE

D.R.E.E. PROJECT NO. 3-2 | FENCO PROJECT NO. I-6177-I

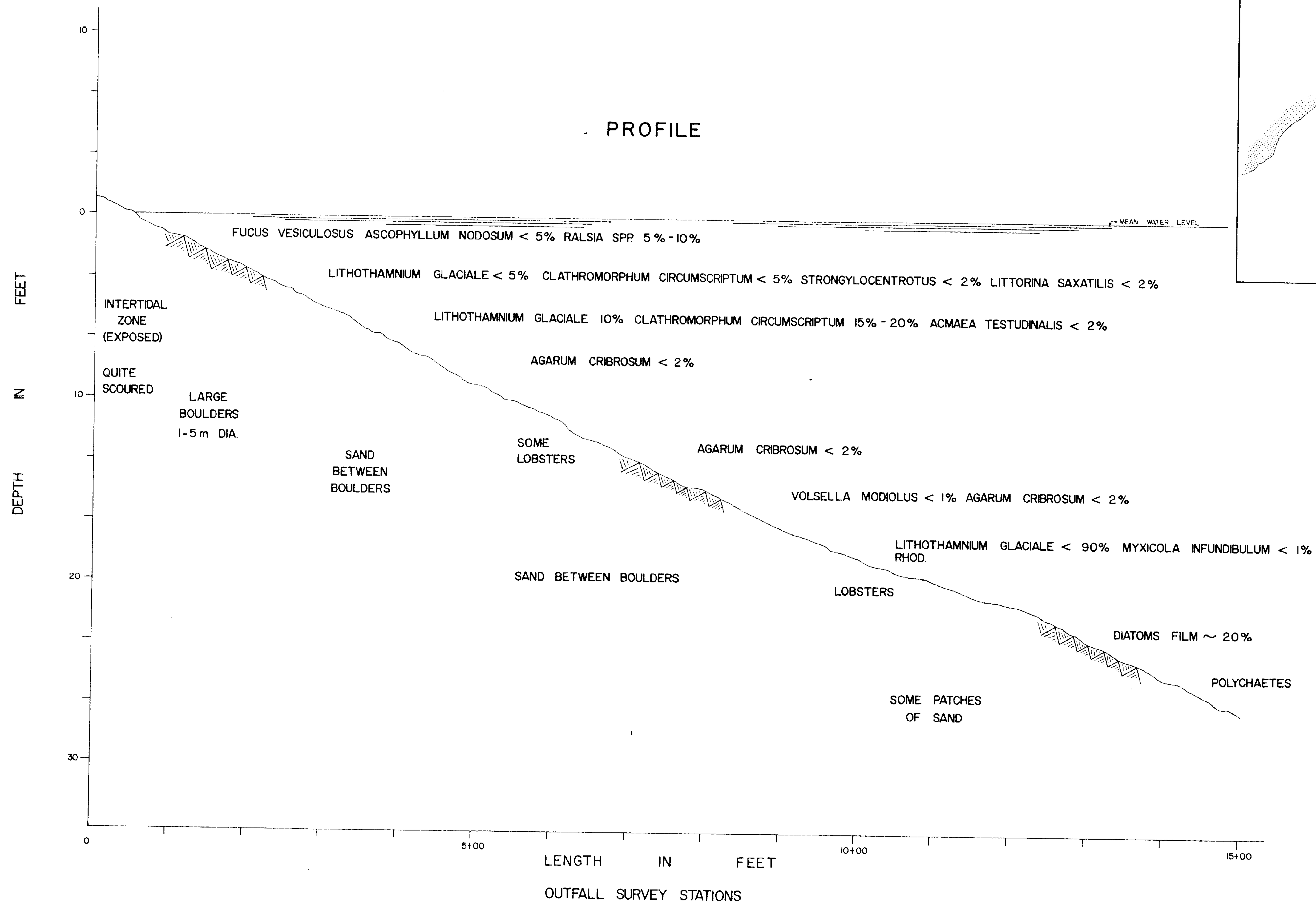
FENCO

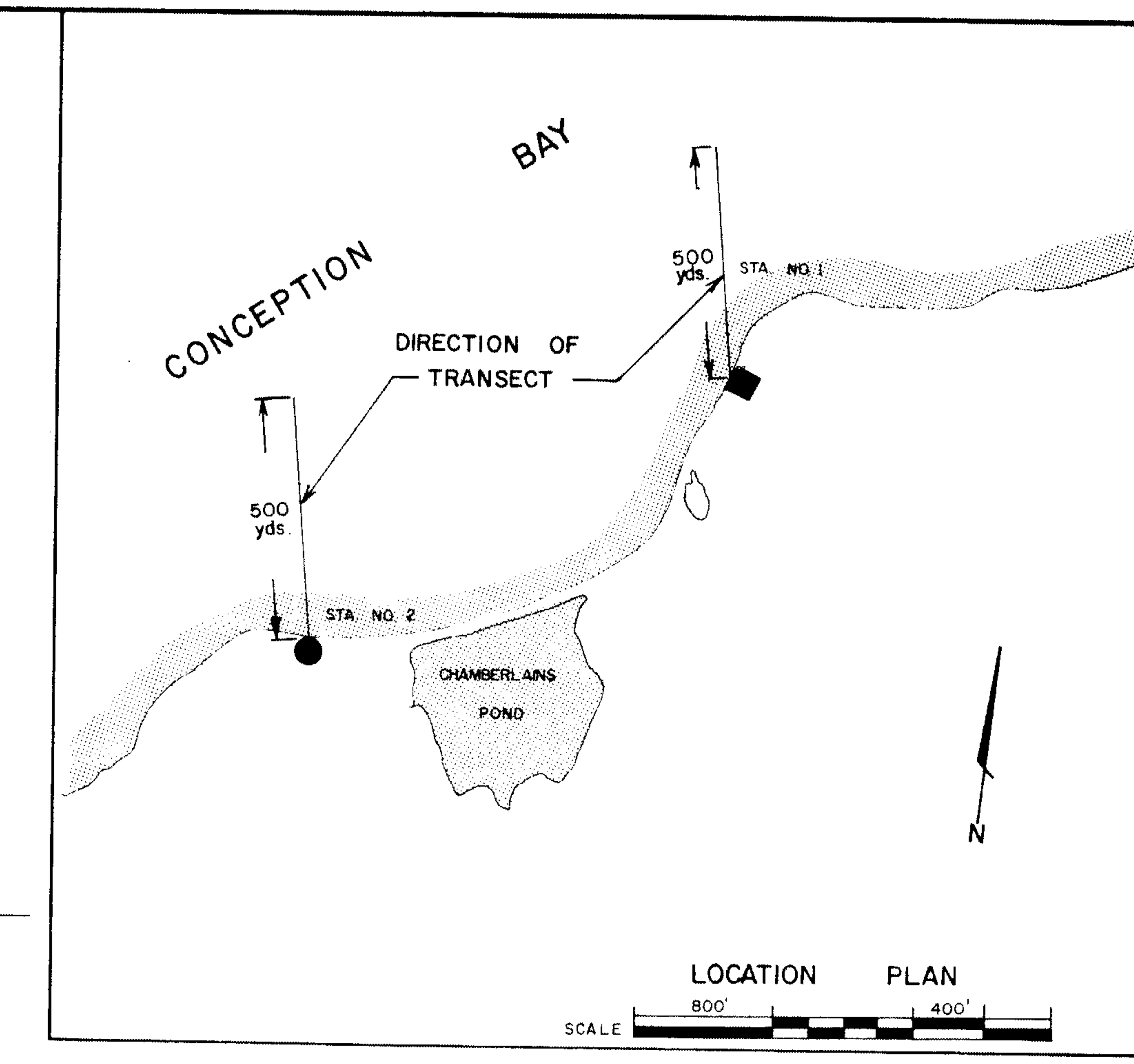
Foundation of Canada Engineering Corporation Limited

DWG NO 2

REV

604-73





STATION Nº 1 & 2

DRAFT

DRAFT

FENCO PROJECT	NO. I - 6177 - 1
D.R.E.E. PROJECT	NO. 3 - 2

PROJECT TITLE

SEWAGE COLLECTION AND DISPOSAL STUDY

CONCEPTION BAY SOUTH AREA

DRAWING TITLE

SPECIES DIVERSITY AND RELATIVE ABUNDANCE OF MARINE BOTTOM ORGANISMS

FENCO

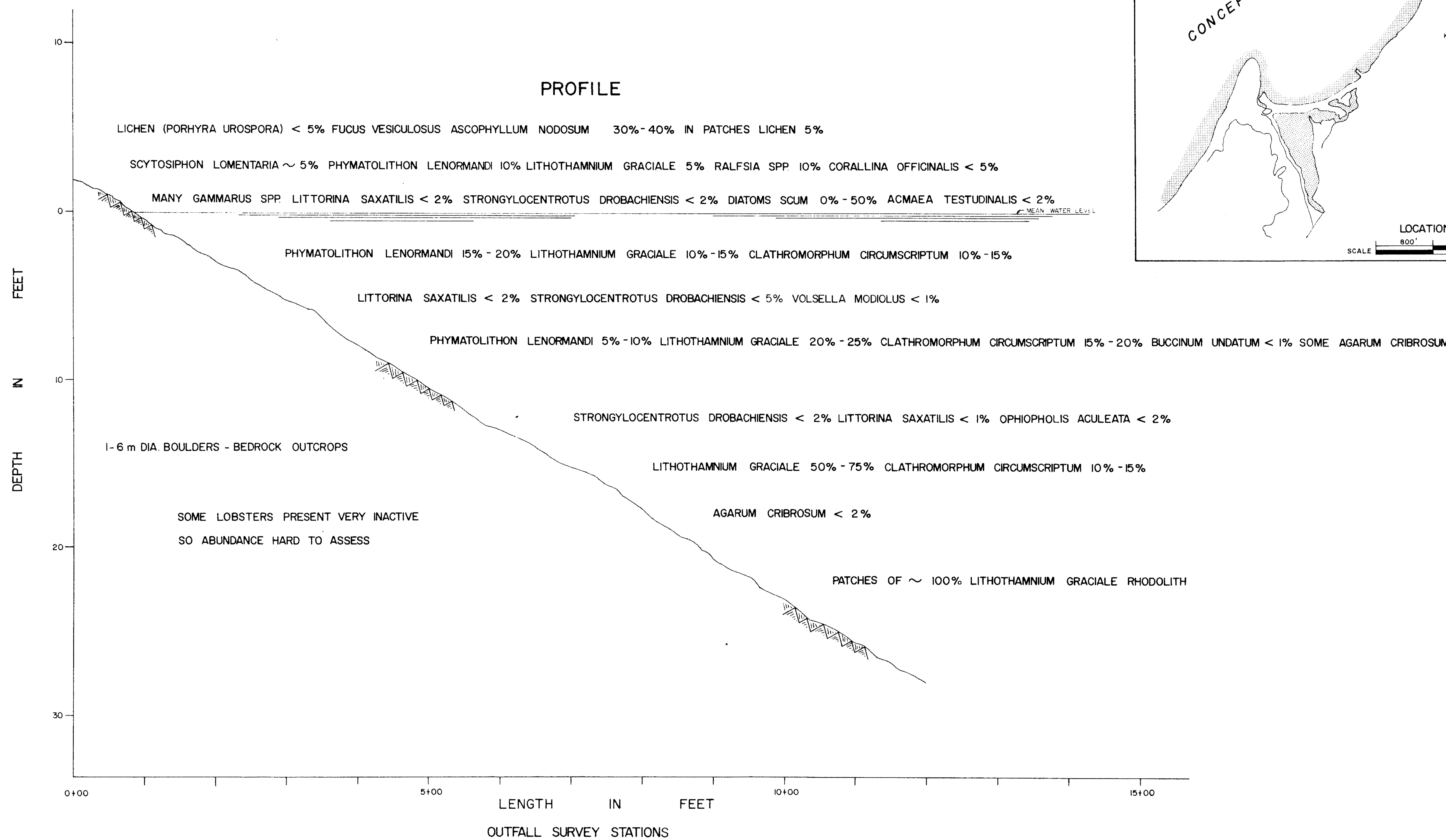
Foundation of Canada Engineering Corporation Limited

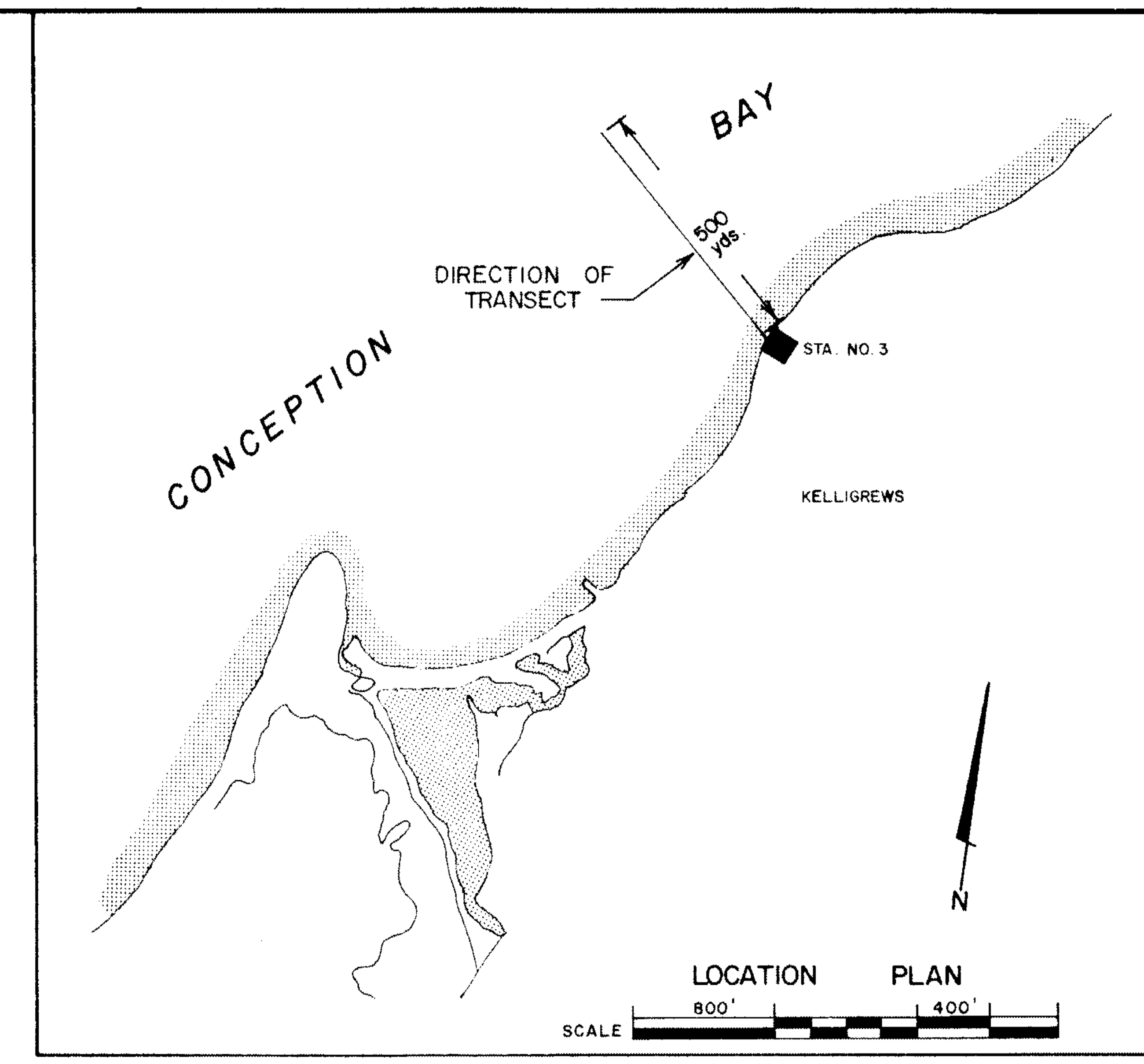
SCALE HORIZ. 0 100 200

VERT. 0 10

DRAWING NUMBER 3

REV.





STATION Nº 3

DRAFT

FENCO PROJECT	NO. I - 6177-1
D.R.E.E. PROJECT	NO. 3-2

PROJECT TITLE
SEWAGE COLLECTION AND DISPOSAL STUDY
CONCEPTION BAY SOUTH AREA

DRAWING TITLE
SPECIES DIVERSITY AND RELATIVE ABUNDANCE OF MARINE BOTTOM ORGANISMS

FENCO
Foundation of Canada Engineering Corporation Limited

SCALE HORIZ. 0 100 200
VERT. 0 10

DRAWING NUMBER 4 REV

NOTES :

ALL FLOWS ARE AVERAGE
ALL GALLONS ARE IMPERIAL
P C P = POLLUTION CONTROL PLANT
BOD = BIOCHEMICAL OXYGEN DEMAND
S S = SUSPENDED SOLIDS

SCALES
1" = 25,000'

SEWAGE COLLECTION AND DISPOSAL STUDY
CONCEPTION BAY SOUTH AREA

SEWERAGE AREAS

D.R.E.E. PROJECT NO. 3-2 FENCO PROJECT NO. I-6177-1

FENCO
Foundation of Canada Engineering Corporation Limited

DWG. NO. 5

REV.

604-73

DRAFT

LEGEND

PUMP STATION	●
POLLUTION CONTROL PLANT	▨
GRAVITY SEWER	——
FORCEMAIN	-----
SEWERAGE AREAS	1-17

SCALES

25,000' 12,500'

1" = 25,000'

SEWAGE COLLECTION AND DISPOSAL STUDY
CONCEPTION BAY SOUTH AREA

KEY PLAN

FENCO

Foundation of Canada Engineering Corporation Limited

D.R.E.E. PROJECT NO. 3·2 FENCO PROJECT NO. I-6177·1

DWG. NO. 6 REV

604-73

LEGEND

GRAVITY SEWER	
LOW PRESSURE SEWER	
FORCEMAIN	
PUMPING STATION	O
POLLUTION CONTROL PLANT	

NOTE: ALL GRAVITY SEWERS 8"Ø UNLESS OTHERWISE INDICATED

SCALES

800' 400'

1" = 800'

SEWAGE COLLECTION AND DISPOSAL STUDY
CONCEPTION BAY SOUTH AREA

SEWER SYSTEM

FENCO

Foundation of Canada Engineering Corporation Limited

D.R.E.E. PROJECT NO. 3·2 FENCO PROJECT NO. I-6177-I

DWG NO. 6-1 REV

604-73

LEGEND

GRAVITY SEWER
LOW PRESSURE SEWER
FORCEMAIN
PUMPING STATION
POLLUTION CONTROL PLANT

NOTE: ALL GRAVITY SEWERS 8"Ø UNLESS OTHERWISE INDICATED

SCALES

800' | 400'

1" = 800'

SEWAGE COLLECTION AND DISPOSAL STUDY
CONCEPTION BAY SOUTH AREA

SEWER SYSTEM

FENCO

Foundation of Canada Engineering Corporation Limited

D.R.E.E. PROJECT NO. 3·2 | FENCO PROJECT NO. I-6177-I | DWG. NO. 6-2 | REV

604-73

LEGEND

GRAVITY SEWER	
LOW PRESSURE SEWER	
FORCEMAIN	
PUMPING STATION	O
POLLUTION CONTROL PLANT	

NOTE: ALL GRAVITY SEWES 8"Ø UNLESS OTHERWISE INDICATED

SCALES

800' 400'

1" = 800'

SEWAGE COLLECTION AND DISPOSAL STUDY
CONCEPTION BAY SOUTH AREA

SEWER SYSTEM

FENCO

Foundation of Canada Engineering Corporation Limited

D.R.E.E. PROJECT NO. 3·2 FENCO PROJECT NO. I-6177·I

DWG NO 6-3 REV

604-73

LEGEND

- GRAVITY SEWER
- LOW PRESSURE SEWER
- FORCEMAIN
- PUMPING STATION
- POLLUTION CONTROL PLANT

NOTE: ALL GRAVITY SEWERS 8"Ø UNLESS OTHERWISE INDICATED

SCALES

800' 400'

1" = 800'

SEWAGE COLLECTION AND DISPOSAL STUDY
CONCEPTION BAY SOUTH AREA

SEWER SYSTEM

FENCO

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D.R.E.E. PROJECT NO. 3·2 | FENCO PROJECT NO. I·6177·1 | DWG. NO. 6-4 | REV

604-73

LEGEND

PHASE I PCP I : PACKAGES AI, BI, CI, DI, EI, FI, GI

PCP II : AII, BII, CII

PHASE II AREA WITHIN BOUNDARY NOT DESIGNATED INTO PACKAGES.

PCP = POLLUTION CONTROL PLANT

SCALES

25,000' | 12,500' |

1" = 25,000'

SEWAGE COLLECTION AND DISPOSAL STUDY
CONCEPTION BAY SOUTH AREA

PHASES & PACKAGES

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