

SOIL MECHANICS AND HYDRAULIC ASPECTS
OF THE CHARLESWOOD SEWAGE LAGOON

A Thesis

Presented to

the Faculty of Graduate Studies and Research
University of Manitoba

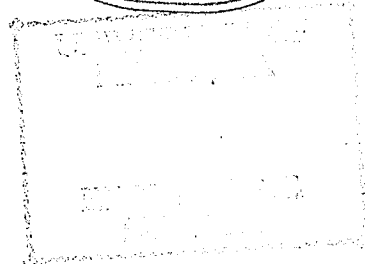
In Partial Fulfillment

of the Requirements for the Degree
Master of Science in Civil Engineering

by

Alexander G. Kuluk

1966



ABSTRACT

One of the most critical problems resulting from the operation of the Charleswood Sewage Lagoon has been the dike erosion experienced. In undertaking a research of this problem and possible causes and solutions, other soil mechanics and hydraulic aspects were also studied. This thesis presents a systematic development of the original research made and in particular deals with the dike bank revetment program developed by the author and installed by the Metropolitan Corporation of Greater Winnipeg, Waterworks and waste Disposal Division in the fall of 1965.

The performance of the installed revetment works was observed from the beginning of spring into the middle of summer of 1966. It was established that the rock rip rap was the only revetment that provided satisfactory erosion resistance. Of the other alternatives, only the wire mesh over the clay slope appeared to have sufficient merit to warrant further consideration.

The work encompassed by this thesis has constituted the initial phase of a research program which is to be continued. Numerous topics requiring additional research have been indicated and the author urges continued work along these lines.

ACKNOWLEDGEMENTS

The author is indebted to all those who have contributed or assisted in making this thesis possible.

A special thanks to Professors A. Baracos and M. Kindess, for their guidance and advice throughout the investigation.

The assistance and co-operation received from the Waterworks and Waste Disposal Division of the Metropolitan Corporation of Greater Winnipeg was greatly appreciated. In particular, the author is grateful for the help of Messrs. G.E. Burns, P.Eng. and P.M. Higgins, P.Eng.

The Manitoba Department of Health is thanked for its literature contribution to this project.

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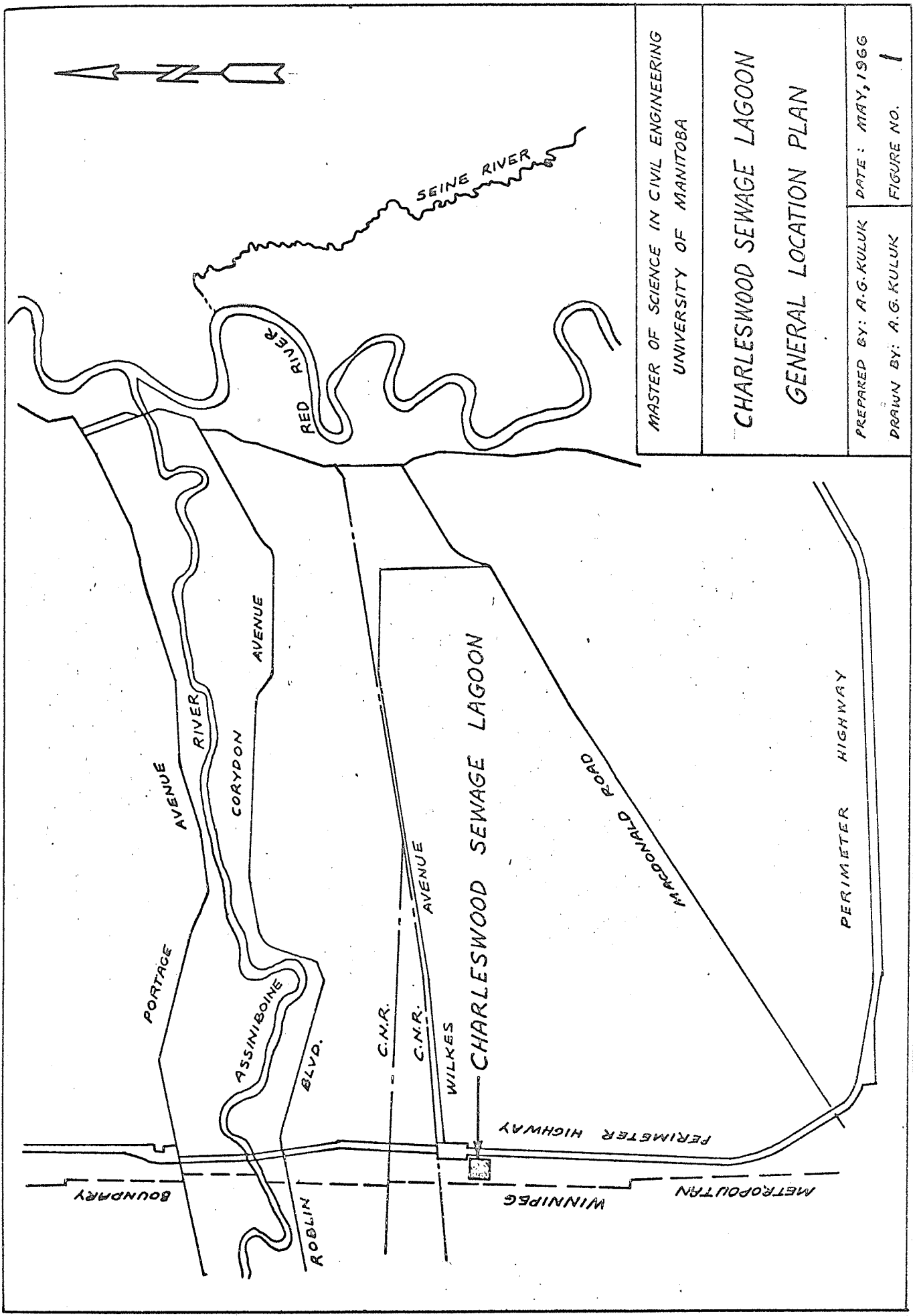
CHAPTER 1

INTRODUCTION

As an introduction of sewage lagoons into a sewage treatment improvement program, the Waterworks and Waste Disposal Division of the Metropolitan Corporation of Greater Winnipeg, had the Charleswood Sewage Lagoon built and in operation towards the end of May, 1964. Figure 1 shows the location of the lagoon. The Charleswood Sewage Lagoon occupies about 300 acres and is the first sewage lagoon in the Greater Winnipeg area. Depending on the successful performance of this lagoon is a tentative sewage treatment program which could include a second lagoon of about 1,000 acres, which would be the largest in North America.

The Charleswood Sewage Lagoon was designed to serve as a research project and yet operate as an efficient lagoon. As Figure 2 indicates, it is composed of three primary units of 45 acres each and two secondary units of 62.5 acres each. Each unit has four different interior side slopes which vary from one on six (one vertical on six horizontal slope), one on seven, one on eight, and one on nine. The exterior side slope is constant at one on five. Generally the dikes are about eight feet above the natural ground level.

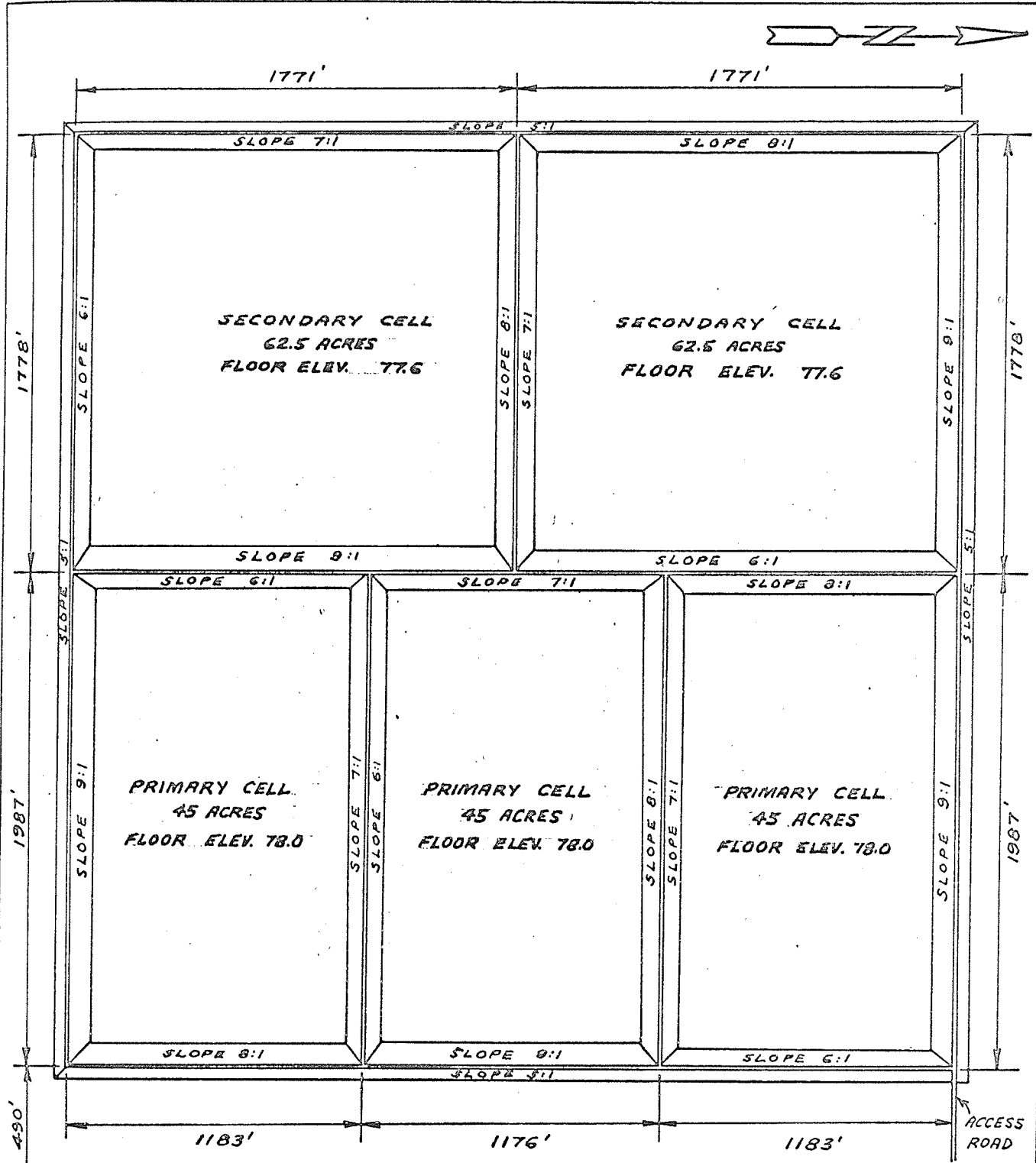
In the fall of 1964, the author was introduced to this project. At that time work had been initiated by Professor



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CHARLESWOOD SEWAGE LAGOON
GENERAL LOCATION PLAN

PREPARED BY: A.G. KULUK	DATE: MAY, 1966
DRAWN BY: A.G. KULUK	FIGURE NO. 1



PERIMETER HIGHWAY

SCALE : 1 IN. = 500 FT.

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CHARLESWOOD SEWAGE LAGOON
LAYOUT DETAIL

PREPARED BY : A.G. KULUK
DRAWN BY : A.G. KULUK

DATE : MAY, 1966
FIGURE NO. 2

4

A. Baracos of the University of Manitoba towards a program of recording seepage and soil density by means of neutron moisture and density equipment. Another planned project was the dike erosion study. As part of the above program, the Waterworks and Waste Disposal Division of the Metropolitan Corporation of Greater Winnipeg had undertaken a program of periodic surveying of fixed dike control sections.

Under the direction of Professor A. Baracos, the author undertook the above mentioned studies. The program was expanded to include other soil mechanics and hydraulic aspects as the author found pertinent or related to the project. The resulting project was unique as apparently no previous research had been carried out for a similar project. Consequently it was necessary to undertake an independent study in a systematic manner. The author chose to deal with each subject concerned from primary sources which generally involved a research of all available literature. Then as enough basic knowledge was built up, the author was in a position to draw comparisons, make conclusions or suggest additional programs of study.

The following chapters are presented so that an appreciation of the development of the research can be obtained and also so that the various topics introduced can be followed in a natural sequence.

CHAPTER 2

SEWAGE LAGOON LITERATURE REVIEW

A research of available sewage lagoon literature was carried out with the aim of providing general background and history, establishing design and operational standards, in addition to delving into difficulties encountered at other lagoon sites in similar climates.

History of Sewage Lagoons¹

Sewage ponds have been utilized in Asia for centuries, but the earliest available data regarding them come from various parts of Germany where valuable crops of fish were raised in sewage ponds.

More than 50 years ago sewage stabilization by photosynthesis was utilized in the southwestern part of the U.S. The City of San Antonio employed this principle in 1901. The use of what are known as oxidation ponds to obtain secondary treatment of sewage appeared in California in 1924. Until about 12 years ago, the main application of such ponds was for the final treatment of effluent before disposal.

The City of Fessenden in North Dakota, has discharged raw sewage into a slough west of the city since 1928. This slough has been diked, and to date has provided adequate treatment. However, the first deliberately designed lagoon in North

1. T. Lackie, "Application of Sewage Lagoons for Industrial Wastes", (Paper Presented at 9th Sanitary Inspectors Conference, Winnipeg, Manitoba, March, 1959), p. 47.

Dakota was constructed at Maddock in 1949, and as a result of observations of it, the North Dakota State Health Department endorsed the use of similar facilities for many municipalities in that state. In 1955, the U.S. Public Health Service started a survey of lagoons in North and South Dakota and concluded that they were an effective and economical answer to treatment and disposal problems. By 1957 there were about 140 lagoons in the Missouri basin, with more than 70 of these being in the Dakotas.

The first lagoon in Manitoba was constructed at Boissevain in 1954. By January, 1959, there were 22 communities served by lagoons in Manitoba, and indications are that this will remain the standard form of treatment for communities in the southern part of the Province. The previous largest lagoon in Manitoba was a 160 acre lagoon at Portage la Prairie, built to handle the domestic wastes from the city, plus those from the Campbell Soup Plant.

Definition of Sewage Lagoons²

During the past few years the terms sewage lagoons, oxidation ponds, and stabilization ponds have often been used interchangeably. This practice has led to some confusion.

About 13 years ago, when the State Health Department in North Dakota became interested in the use of open ponds

2. E.J. Cole and B.R. Stanley, "Anaerobic Stabilization Ponds for Saskatoon", (Paper Presented to the Western Canada Water and Sewage Conference, Sept. 1961), p.p.75-76.

for the treatment of raw sewage, such ponds became generally known as sewage lagoons. The term was normally reserved for ponds used for treating raw sewage. At the same time, and previously, there were, of course, many ponds which were used to treat either primary or secondary sewage effluent. These ponds were normally called oxidation ponds.

When the U.S. Public Health Service became interested in the examination of lagoons in North and South Dakota, the confusion in terms was recognized and the term waste stabilization pond was brought into use.

Towne and Davis define a waste stabilization pond as a structure specifically designed to treat liquid organic wastes by biological, chemical and physical processes commonly referred to as natural self-purification.

In Manitoba, the term sewage lagoon is in common usage. The definition used is according to the United States Public Health Service, "A sewage lagoon is an excavation designed and constructed to receive raw or pretreated domestic sewage and some organic industrial wastes; in which stabilization is accomplished by several natural self-purification phenomena. These phenomena include sedimentation, decomposition, and bacterial-algal interaction".

Engineering and Design of Sewage Lagoons

A review of literature from the standpoint of the items below was prepared.

- (1) preliminary investigation
- (2) engineering design
- (3) construction
- (4) performance

A thorough study of the first three items has been made by the Manitoba Department of Health. As a result of information on sewage lagoons installed in Manitoba, and of comparisons with the standards in use throughout Canada and the United States, the Manitoba Department of Health has established standards which have been compiled in a report entitled "The Suggested Standards for Sewage Lagoons for the Use of Consulting Engineers Submitting Reports to Manitoba Department of Health".³

The following information has been taken from the above report.

Manitoba Department of Health Design Standards

(1) Engineer's Report

The engineer's report shall contain pertinent information on location, geology, soil conditions, area for expansion, and any other factors that will affect the feasibility and acceptability of the proposed treatment.

The following information must be submitted in addition to that required in the preliminary report.

3. Manitoba Department of Health, Suggested Standards for Sewage Lagoons for the Use of Consulting Engineers Submitting Reports to Manitoba Department of Health, (March, 1964), pp.1-3.

Supplementary Field Survey Data

- (a) The location and direction of all residences, commercial development, and water supplies within fifteen hundred (1500) feet of the proposed lagoons.
- (b) Soil borings to determine surface and subsurface soil characteristics of the immediate area and their effect on the construction and operation of lagoons located on the site.
- (c) Data demonstrating anticipated percolation rates at the elevation of the proposed lagoon bottom.
- (d) A description, including maps showing elevations and contours, of the site and adjacent area suitable for expansion.

(2) Basis of Design

Area:

Due consideration will be given to possible future municipal expansion and/or additional sources of wastes when the original land acquisition is made. Suitable land should be available at the site for increasing the size of the original construction.

Pond shape:

The shape of all cells should be such that there are no narrow or elongated portions. Round, square, or rectangular ponds with a length not exceeding three times the width are considered most desirable. No islands, peninsulas, or coves

should be permitted. Dikes should be rounded at corners to minimize accumulations of floating materials.

(3) Location

Distance from Habitation:

A lagoon or pond site should be as far as practicable from habitation or any area which may be built up within a reasonable future period. Lagoons should not be located any closer than one thousand (1000) feet from any center of population; individual residences may be as close as five hundred (500) feet depending on local conditions.

Prevailing Winds:

If practicable, ponds should be located so that local prevailing winds will be in the direction of uninhabited areas. Preference should be given sites which will permit an unobstructed wind sweep across the ponds, especially in the direction of the local prevailing winds.

Surface Runoff:

Location of ponds in watersheds receiving significant amounts of runoff water is discouraged unless adequate provisions are made to divert storm water around the ponds and otherwise protect pond embankments.

Ground Water Pollution:

Proximity of ponds to water supplies and other facilities subject to contamination and location in areas of porous soils and fissured rock formations should be critically evaluated to

avoid creation of health hazards or other undesirable conditions. The possibility of chemical pollution, particularly detergents, may merit appropriate consideration.

(4) Pond Construction Details

Embankments and Dikes:

- (a) Material - Embankments and dikes shall be constructed of relatively impervious materials and compacted sufficiently to form a stable structure. Vegetation should be removed from the area upon which the embankment is to be placed.
- (b) Top width or Berms - The minimum embankment top width should be eight feet to permit access of maintenance vehicles. Lesser top widths will be considered for very small installations.
- (c) Maximum Slopes - Embankment slopes should not be steeper than:
 - (i) Inner--four horizontal to one vertical
 - (ii) Outer--four horizontal to one verticalMachine mowing may require flatter slopes.
- (d) Minimum Slopes - Embankment slopes should not be flatter than:
 - (i) Inner--six horizontal to one vertical. These flatter slopes are sometimes specified for larger installations because of wave action but have the disadvantage of added shallow areas

conductive to emergent vegetation.

(ii) Outer--not applicable, except significant volumes of surface water should not enter the ponds.

- (e) Freeboard - Minimum freeboard shall be three feet except for very small installations.
- (f) Minimum Depth - The minimum normal liquid depth should be one foot.
- (g) Maximum Depth - Maximum normal liquid depth should be five feet.
- (h) Seeding - Embankments shall be seeded above the waterline. A perennial type, low growing, spreading grass that withstands erosion and can be kept mowed is most satisfactory for seeding of embankments. In general, alfalfa and other long-rooted crops should not be used in seeding, since the roots of this type plant are apt to impair the water-holding efficiency of the dikes. Additional protection for embankments (riprap) may be necessary as soil conditions and pond sizes warrant.

Pond Bottom:

- (a) Uniformity - The pond bottom should be as level as possible at all points. Finished elevations should not be more than three inches from the average elevation of the bottom. Shallow or feathering fringe areas usually result in locally unsatisfactory conditions.
- (b) Vegetation - The bottom should be cleared of vegetation

and debris. Organic material thus removed should not be used in the dike core construction. However, suitable topsoil relatively free of debris may be used as cover material on the outer slopes of the embankment.

- (c) Soil Formation - The soil formation or structure of the bottom should be relatively tight to avoid excessive liquid loss due to percolation or seepage. Soil borings and tests to determine the characteristics of surface soil and subsoil shall be made a part of preliminary surveys to select pond sites. Gravel and limestone areas must be avoided.
- (d) Percolation - The ability to maintain a satisfactory water level in the ponds or lagoons is one of the most important aspects of design. Removal of porous topsoil and proper compaction of subsoil improves the water-holding characteristics of the bottom. Removal of porous areas, as gravel or sandy pockets, and replacement with well-compacted clay or other suitable material may be indicated. Where excessive percolation is anticipated, sealing of the bottom with a clay blanket, bentonite, asphaltic coating, or other sealing material should be given consideration. Percolation should not exceed one-quarter (1/4) inch per day in the primary cell.

(5) Miscellaneous

Landscaping:

Shrubs and other arboreal growths may be used for landscaping and shielding purposes, but should be located outside, not on, the embankment.

Expropriation:

If the land required for initial lagoon construction or anticipated expansion is not owned by the municipality or community concerned, this fact should be so indicated.

Approvals Required:

Generally, approvals from the following agencies are required before construction may proceed:

- (a) Department of Health
- (b) Provincial Sanitary Control Commission
- (c) Water Control and Conservation Branch of the Department of Agriculture
- (d) Municipal Board
- (e) Central Mortgage and Housing Corporation

Some Observations on Lagoon Design and Operation

Some of the numerous reports on the observations of lagoon operation have been summarized. The significance of such reports are that they emphasize that various aspects and problems are generally common to lagoons and similar conditions which have been found to exist at the Charleswood Sewage Lagoon are not unique.

(1) W.J. Oswald on California Lagoons⁴

A great variety of levee cross-sections are to be found. The flattest levees known to the author are those at Chico, with side slopes of about 1 on 10. No unusual operational problems have resulted from these extremely flat cross-sections.

A greater degree of levee protection from wave erosion is afforded by maintaining gentle levee slopes than by heavy compaction. Rigid compaction specifications add greatly to the cost of pond construction. Wind damage is decreased by decreasing pond size but even one or two acre ponds will suffer erosion of steep, carelessly compacted banks during strong winds. Linings have been used to protect steep levees.

(2) G.J. Hopkins on Missouri Basin States⁴

Dike slopes may be influenced by the nature of the soil and the size of the installation. Inner slopes are generally designed from 1 vertical on 3 or 4 horizontal, although slopes exceeding 1 on 5 are sometimes specified for larger installations, and slopes steeper than 1 on 3 may be warranted for small installations. Flat inner slopes have the distinct disadvantage of added shallow areas conducive to emergent vegetation. Wave action is more severe for larger installations, warranting consideration of flatter inner slopes. However, observation reveals that a given installation will develop a par-

4. U.S. Department of Health, Education, and Welfare, Waste Stabilization Lagoons (August 1st-5th, 1960).

ticular slope at the water line rather independent of the slope originally provided, the constructed slope being adjusted by wave action to give a "dished" effect near the water line.

Embankments should be seeded along the inner slope to the normal water line. This minimizes erosion, facilitates weed control and permits the maintenance necessary for good appearance.

While no evidence of underground pollution affecting water supplies has been attributed to existing installations, the significance of pollution of underground waters is so great that this aspect of location merits very serious consideration. Chemical pollution, including detergents, may travel much further than bacteriological pollution in normal soil formations.

(3) H.G. Rogers on Minnesota Lagoons⁴

Operation experience has been generally satisfactory, although some problems have been experienced with erosion of the dike at water line, seepage of liquid through the dikes, and excessive growth of emergent aquatic weeds.

(4) T.A. Filipi on Nebraska Lagoons⁴

Because of problems associated with the lagoon floor, dikes and bank erosion, the writer emphasized the need of more

4. Ibid., page 15.

soil mechanics surveys before design and construction. Furthermore, tests should be made on the finished structure and adjustments made if necessary. Specifications should be made to permit the performance test to be the governing characteristic.

It should be noted that the above writer reported that due to erosion several lagoons have formed ravines 30 inches deep in the banks.

(5) R. Forges on Ohio Lagoons⁵

A freeboard of two to five feet should be maintained on the dike. The embankment should be seeded, except below the water line, to avoid slumping and eroding.

(6) Method of combating wave erosion⁶

Lining the banks of a Nebraska sewage lagoon with glass fiber mats has solved the problem of erosion of the shore line by wind and waves.

The soil at the site of the Broken Bow municipal sewage lagoon is sandy clay and lacking in chemicals necessary for plant growth, so the use of vegetation as a bank stabilizer was ruled out. The Municipal Utilities Department decided to use a layer of heavy-duty glass fiber matting to protect the banks.

Utility crews weeded and graded the banks of the lagoon.

5. Ralph Forges, "Design Criteria for Waste Stabilization Ponds", Public Works Magazine, (January, 1963).

6. Engineering News-Record, Wave Erosion is Prevented by Glass Fiber Matting, (January 7, 1965), p. 44.

Then, they dug a two inch wide and six inch deep trench about three feet above the expected water line. They rolled out a one-half inch thick, heavy density glass fiber mat and turned the top six inches into the trench. T pins, 12 inches by 18 inches, made of No. 6 iron wire, secured the rest of the mat to the ground.

The crews coated the matting with a layer of asphalt at a gallon per square yard. This was covered with about one-half inch of gravel and another layer of asphalt. They re-filled the trench with dirt, burying the top six inches of the mat to prevent undercutting by water running down the bank.

The pond was ready for use about a week after the erosion mat was installed.

Conclusions

The sewage lagoon literature review provides a general insight into the numerous related phases of sewage lagoons. There is no doubt that a study covering all of the available information is well beyond the scope of this report. What has been made clear to the author, is that there are numerous items of study that are common to all lagoons and that bear considerable research. The topics which can be studied from the standpoint of soil mechanics and hydraulics are:

- (1) Proper freeboard allowance.
- (2) Optimum dike side slopes.
- (3) Wave characteristics and effect.

(4) Bank erosion causes; remedial or preventative measures.

It should be pointed out that erosion problems appear to be common to sewage lagoons. However, because the design of a modern lagoon is generally still in a formative stage this problem has not been adequately assessed and insignificant investigation has been made of means of correcting or eliminating erosion problems.

In the following chapters considerable detail will be presented on numerous engineering aspects and the above factors will be dealt with as they apply.

CHAPTER 3

PHOTOGRAPHIC REVIEW OF NUMEROUS SOUTHERN LAGOONS

An investigation was made of problems encountered and programs undertaken at various lagoons in the southern part of Manitoba and neighbouring provinces and states. About twenty different sites were studied and from these a representative number were selected for comments.

Fortare la Prairie Lagoon (Picture 1)

The soil here is predominantly sand and silt. The grass cover has been very effective in retarding erosion. However, due to undercutting and sloughing, the banks have experienced extensive erosion.

Carman Lagoon (Picture 2)

Earlier erosion problems have become entirely camouflaged by rampant willow and weed growth. The growth has become unmanagable and illustrates the close control of dike vegetation required.

Grand Forks, North Dakota Lagoon (Picture 3)

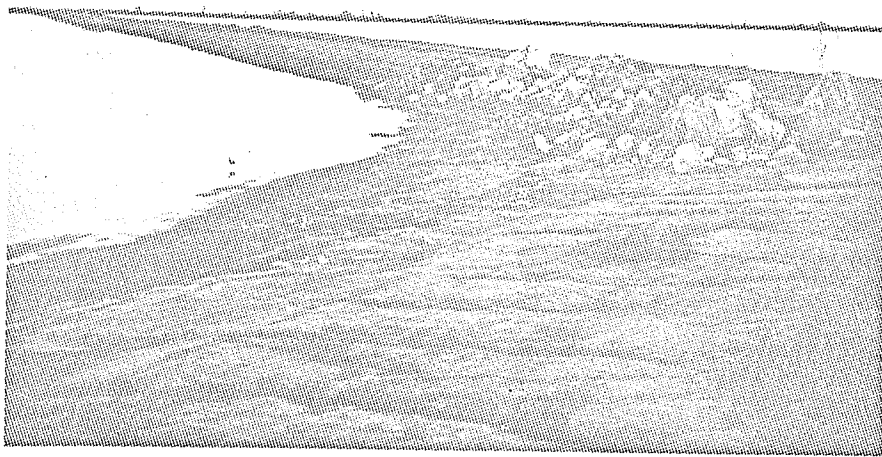
The sterilized banks have suffered rainfall runoff erosion. A dike riprapping program has been undertaken and is seen in its early stages.



Picture 1. July 15, 1969, Typical Bank Erosion
Portage in Prairie Lagoon.



Picture 2. July, 1965. Dike Bank, Grand Forks,
North Dakota.



Picture 3. July, 1965. Dike Banks. Grand Forks,
North Dakota.

Altona Lagoon (Picture 4)

The erosion problems encountered here are also significant. The photograph below illustrates the difference in wave action for a wind of about 30 miles per hour and for similar physical conditions between the secondary and tertiary cells. This is due to a difference in the concentration of the sewage effluents. This complements observations which have been made at the Charleswood Sewage Lagoon and which will be discussed later.

Winkler Lagoon (Picture 5)

To combat erosion problems the lagoon banks have been protected by various materials. This is representative of most rural lagoons. The haphazard placement is not only unsightly but it also proposes sanitary problems.

Regina, Saskatchewan Lagoon (Pictures 6 and 7)

This city has been experimenting with revetment works. Picture 6 illustrates an anchored log installation. This has not proven satisfactory to date. Picture 7 shows the use of broken sidewalk slabs as riprap works.

Charleswood Sewage Lagoon.

A complete photographic analysis of the numerous conditions encountered and recorded could complete an entire volume of work. Therefore only a small group of pertinent illustrations have been made.



Picture 4. May 18, 1965. Wave Action. Altona Lagoon.



Picture 5. May 18, 1965. Revetment Works. Winkler Lagoon.



Picture 6. Anchored Log Installation. Regina, Saskatchewan Lagoon.



Picture 7. Broken Sidewalk Slabs. Regina, Saskatchewan Lagoon.

The interior dikes were sterilized to prevent rampant vegetation growth. Picture 8 shows the effects of rainfall erosion which had occurred.

Picture 9 illustrates the ice action the dikes are subject to. The islands of earth which appear in the background are only cakes of ice covered with clay which was deposited as a result of wind erosion of the dikes.

The effect of alternate wetting and drying causes desiccation of the clay which results in granular-like particles forming. Picture 10 is an example of this effect. Considerable discussion in subsequent chapters will revolve about this behaviour.

Advanced erosion of the dike banks tends to create vertical faces at about the water line as shown on Picture 11. Cross section surveys have been taken which will be used in a later chapter to discuss this behaviour.

Prior to the installation of bank revetment works, the two cells which were to be utilized were each emptied. Picture 12 shows the test site of Primary Cell No. 1. The previous two conditions described are both very well developed here.

To supplement the earlier illustrations made (Picture 4), Picture 13 illustrates the difference in wave action between the Secondary and Primary Cells.

It was quite fortunate that during very strong winds conditions, which had gusts recorded up to about 50 miles per hour, Pictures 14 and 15 were taken which indicate the extent of the wave action.

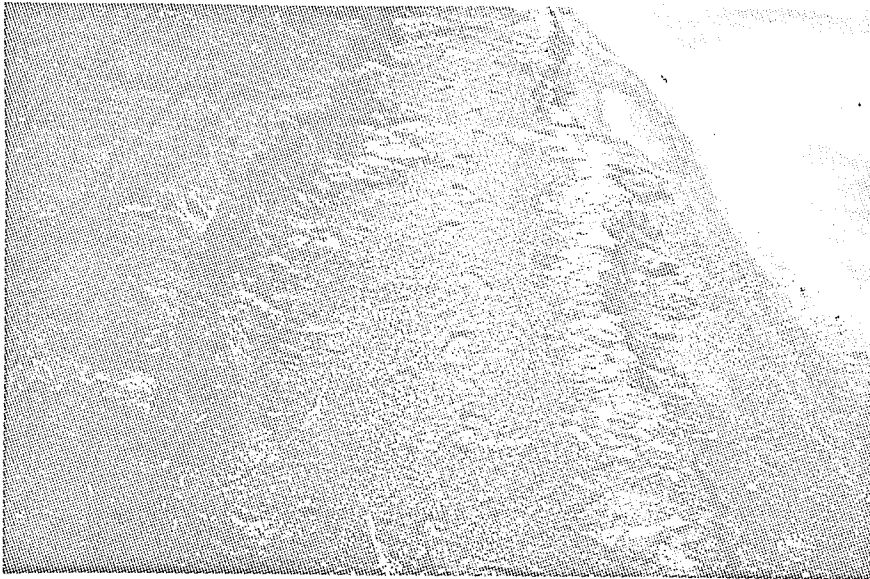


Picture 8. After Dike Sterilization some Rain-Fall Erosion.

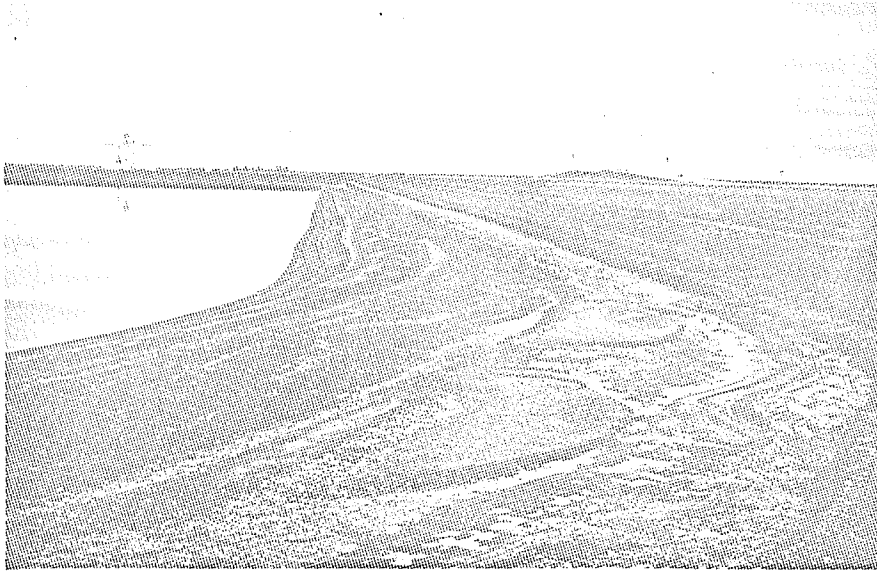


Picture 9. Ice Action and Wind Erosion.

Picture 10. Clay Desiccation Effect.



Picture 11. Advanced Erosion Steeping Effect.



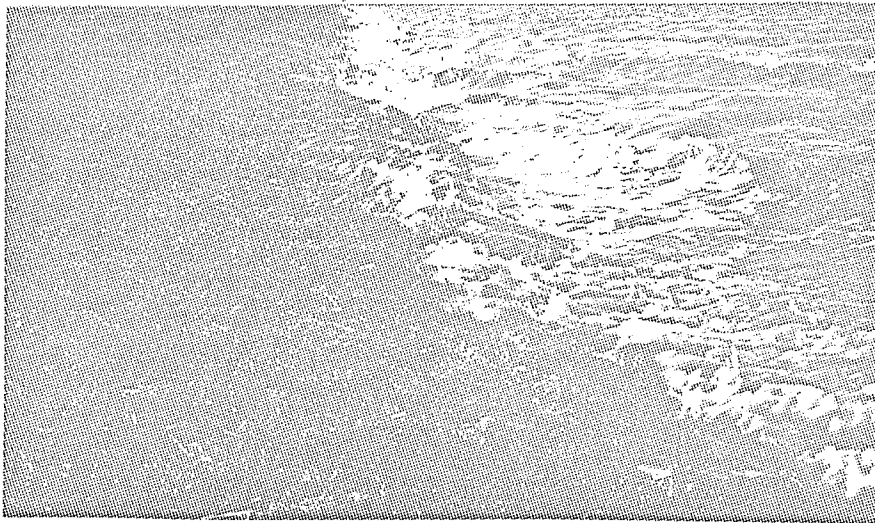
Picture 12. Typical Dike Bank Prior to Revetment Installation.



Picture 13. Wave Action Difference Secondary (left) and Primary (right) Cells.



Picture 14. Wave Action Oct., 1965. Primary
Cell No. 3.



Picture 15. Wave Action Oct., 1965. Secondary
Cell No. 2.

CHAPTER 4

SOIL MECHANICS ASPECTSPrevious Soil Investigations

A preliminary soils study for the Charleswood Sewage Lagoon was carried out in 1961 by Underwood, McLellan and Associates.¹ They collected and compiled all available soils data pertinent to the general Metropolitan Winnipeg area. The prime consideration in their study was the determination of depth of hardpan. In establishing a general pattern of hardpan depth, it was determined that the general soils structure is fairly uniform throughout the area studied.

An additional soils investigation in the Charleswood Lagoon area pointed out that the hardpan was at moderate depth, with indications that rock and shallow hardpan would be encountered in North-west St. James which could present problems for the interceptor and pump-house construction. However, since this area does not require interceptor construction until after 1981, it was felt that a detailed soils study could be made at that time to determine the extent of the problem and the most economical method of construction. The remainder of the area appeared to present no construction problems, although

1. Underwood McLellan and Associates, Winnipeg in Association with James F. MacLaren Associates, Toronto, "Report on the Sewage Facilities for the South District Metropolitan Winnipeg", (August 1961), p. 80.

possible local variations of the hardpan could be encountered in limited amounts.

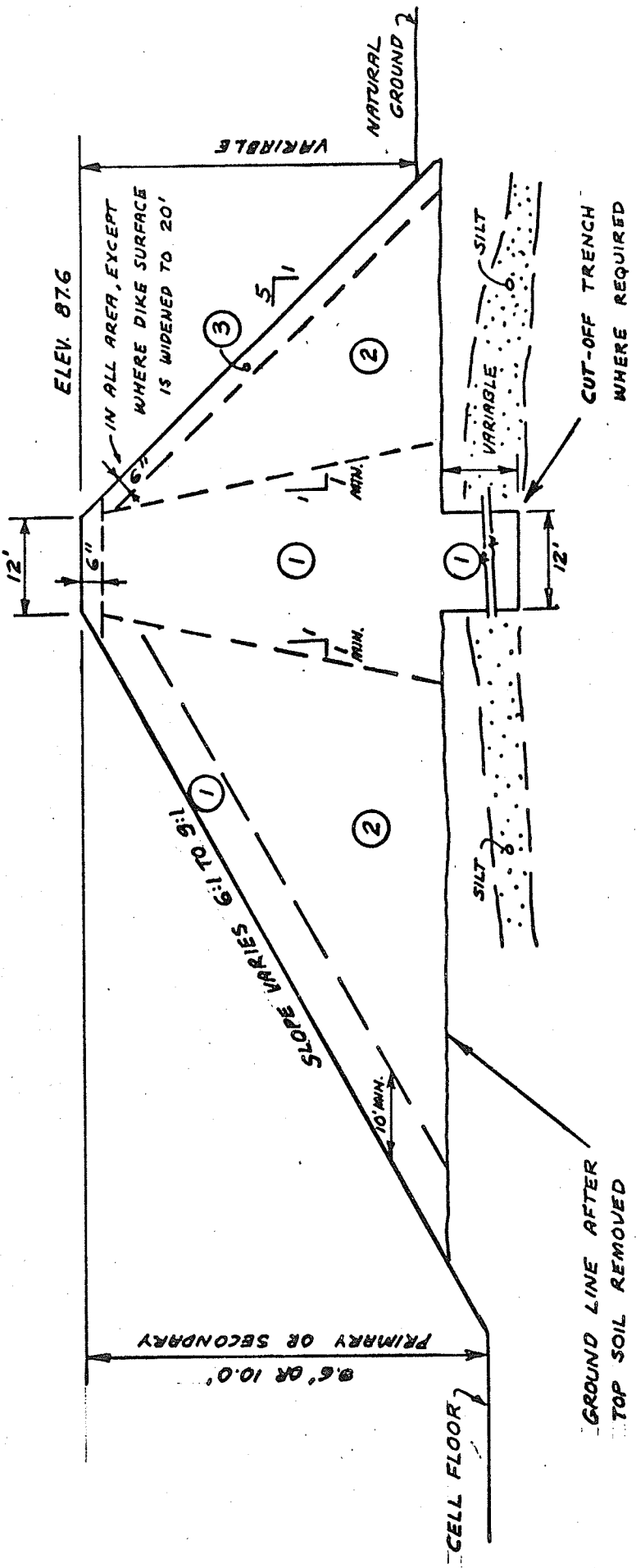
In the above report it was pointed out that no difficulty with groundwater contamination was expected because of the impermeable nature of the soils. The lagoon effluents would be pipe transported directly to the receiving stream and would present no problems.

Other miscellaneous soil testing and investigations were carried out prior to construction. The results were used only for familiarization purposes and no records were kept.

During construction, Nelson River Construction utilized hand auger holes to facilitate their construction procedure. These were not logged according to soil mechanics standards, but were simply used to inform the workmen of the silt excavation necessary.

The construction of the lagoon began with the stripping and stockpiling of the topsoil. In excavating the lagoon to design elevation, when sand or silt was encountered, it was removed and replaced with clay as indicated on Figures 3 to 5. The lagoon floor was not compacted other than that obtained due to passage of earth moving equipment. The dikes were compacted to a given density which was both field and laboratory controlled.

Figures 3 to 5 show the typical dike cross sections utilized and the dike and core fill composition. It should be noted that type 2 material, which is shown as a silt-clay mixture,



SCALES : VERT. 1 IN. = 4 FT.
 HORIZ. 1 IN. = 200 FT.

LEGEND

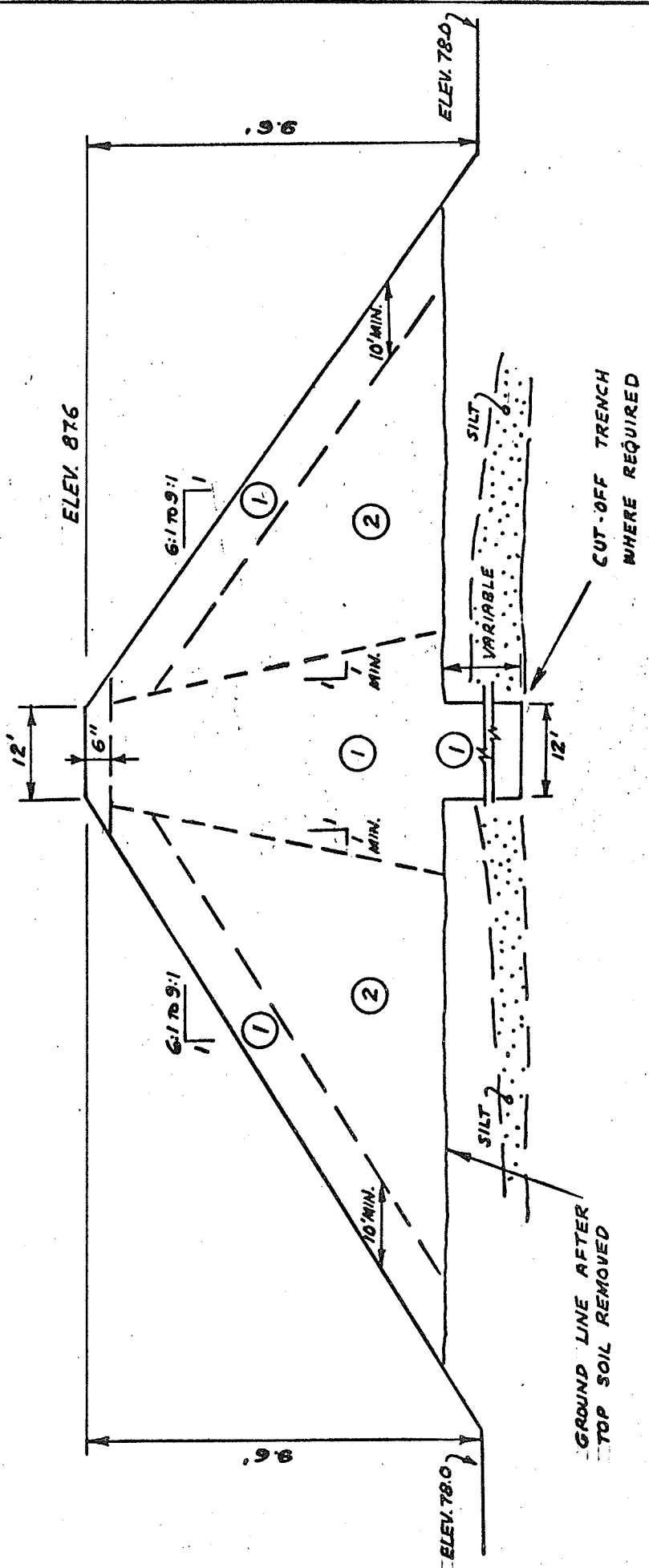
- ① CLAY
- ② SILT-CLAY MIXTURE
- ③ TOP SOIL

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CHARLESWOOD SEWAGE LAGOON
 TYPICAL EXTERIOR DIKE
 DETAIL OF CORE AND FILL PLACEMENT

PREPARED BY: A.G. KULUK
 DRAWN BY: A.G. KULUK

DATE: MAY, 1966
 FIGURE NO. 3



SCALES: VERT. 1 IN. = 4 FT.
 HORIZ. 1 IN. = 200 FT.

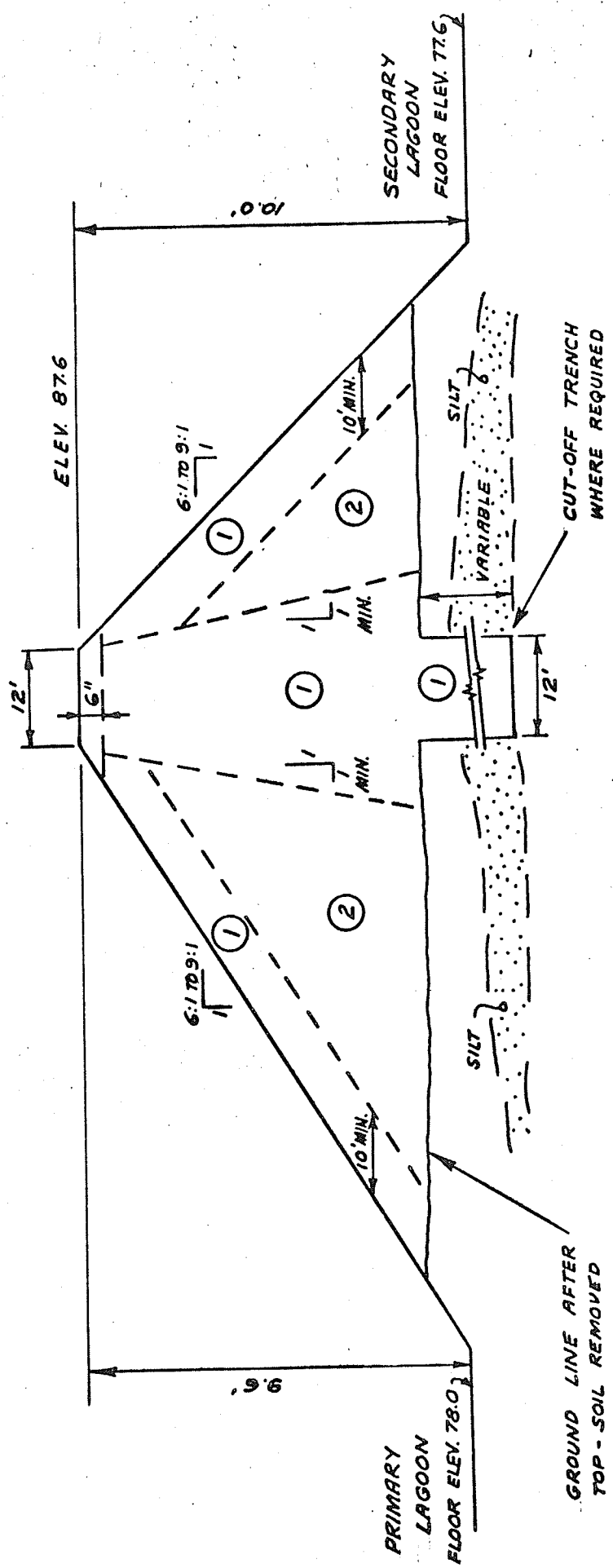
- LEGEND
- ① CLAY
 - ② SILT-CLAY MIXTURE
 - ③ TOP SOIL

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CHARLESWOOD SEWAGE LAGOON
 TYPICAL PRIMARY-PRIMARY DIKE
 DETAIL OF CORE AND FILL PLACEMENT

PREPARED BY: A.G. KULUK
 DRAWN BY: A.G. KULUK

DATE: MAY, 1966
 FIGURE NO. 4



SCALES : VERT. 1 IN. = 4 FT.
 HORIZ. 1 IN. = 200 FT.

- LEGEND
- ① CLAY
 - ② SILT - CLAY MIXTURE
 - ③ TOP SOIL

MASTER OF SCIENCE IN CIVIL ENGINEERING UNIVERSITY OF MANITOBA	
CHARLESWOOD SEWAGE LAGOON TYPICAL PRIMARY-SECONDARY DIKE DETAIL OF CORE AND FILL PLACEMENT	
PREPARED BY: A.G. KULUK	DATE: MAY, 1966
DRAWN BY: A.G. KULUK	FIGURE NO. 5

GROUND LINE AFTER
 TOP - SOIL REMOVED

CUT-OFF TRENCH
 WHERE REQUIRED

after construction was found to be predominantly clay due to the absence of anticipated silt quantities in the borrow.

One well was drilled on May 14, 1964 at the Charleswood Sewage Lagoon. It was located at the center of the sewage pumping station. The drillers' report revealed the following log.²

TABLE No. 1

DRILLERS WELL LOG

Well Log Description	From Elevation	To Elevation
(Lagoon Bottom)		78.0
Top of Concrete Slab	72.5	69.5
Red Clay Till	69.5	67.0
Grey Clay Till	67.0	59.0
Weathered Limestone	59.0	54.5
Fractured Limestone (clay in cracks)	54.5	27.5
Red Clay Shale	27.5	0.5
Limestone	0.5	

Note: Undetermined quantity of water at elevation 51.0.

Soil Investigations Carried Out.

The in situ soil at the Charleswood Sewage Lagoon area over the depth concerned with the dikes, consists of a top few

2. Water Control and Construction Branch, Manitoba Water Well Drillers' Reports, January 1 - December 31, 1964, (Department of Agriculture and Conservation, Province of Manitoba, 1965), p. 130.

feet of organic topsoil followed by fine grained, highly plastic clays which are frequently interspersed with shallow horizontal layers of silt deposits.

To further evaluate the clays of the dikes, with a view towards having general data available for an erosion analysis, certain laboratory tests were conducted. To reduce the amount of work required, representative sites were chosen at several Primary and Secondary Cells.

Soil samples were taken from the various cells at locations corresponding to the high water level, mid slope and at the toe of slope. The samples were representative of the top foot of clay. Routine laboratory classification tests were conducted. The liquid limit varied from 79 to 64 and the plasticity index varied from 43 to 37. The flow index, which should be about the same for clays of the same origin and area, varied from 26 to 22, which is an acceptable range.

Because the above work was done in connection with the erosion study, the laboratory tests were only carried out to provide sufficient information for that approach. However, it was noted that the clays of the Primary Cells tended to have higher Atterberg Limits than those of the Secondary Cells. Because the concentration of the sewage effluent differed in these cells, an interesting but involved study would be to investigate the effect of the sewage effluent on the clay properties.

The Atterberg Limits obtained were plotted on the Casagrande Plasticity Chart.³ It was found that the mean line plotted as a straight line just above and slightly flatter than the A line. According to the chart classification, this is representative of inorganic clays of high plasticity (CH group). From the general properties attributed to this group, the most pertinent are:

- (a) Medium frost action.
- (b) High compressibility and expansion.
- (c) Practically impervious, drainage characteristic.

Soils of this group are commonly utilized as blankets and cores of dike sections. The most undesirable property of these clays is the high shrinking and expanding behaviour with changing moisture conditions.

Visual examinations were made of exposed dike banks which had been previously submerged by sewage effluent. It was noted that drying caused desiccation of the surface layer. This effect created a granular-like layer made up of small sized clay particles as was indicated by Picture 10, Page 28, Chapter 3. An individual clay particle was noted to be a very hard and integral unit. Just below the surface layer, which was several inches in thickness, the drying effect did not appear effective, and the clay was moist and cohesive.

When the above surfaces again became submerged, they

3. T.W. Lambe, Soil Testing for Engineers, (New York: John Wiley & Sons, Inc., 1951), p. 27.

were noted to retain their granular-like characteristics. The action of ripples or waves readily moved the clay particles. This can be explained by the fact that clay particles lack both the density and size of granular materials to be as stable. Therefore it can be deduced that a larger and more stable grain size would not be as subject to erosion.

Just how the above effect will behave over a long period is difficult to ascertain. However, the above observation will be subject to further discussion in subsequent chapters. A worthy topic of investigation would be the effects of sewage effluent on clay properties.

Another observation worthy of note was the lack of vegetation on all interior dike slopes. A grass cover above the high water level mark could reduce wind and rainfall action on the clay banks. It would serve to retain moisture and thereby assist the clay remaining intact. A good example of this can be seen in Picture 1, Page 21, Chapter 3. This may also be a means of protection below the high water level mark but it is subject to sanitary engineering interests which are beyond the scope of this report. It may be mentioned that some of the likely problems could include emergent growth of grass and weeds, mosquito breeding and trapping of floating grease which are all undesirable.

Conclusions

The background of previous soil investigations conducted was reviewed, as well as the studies made by the author. As

further work is carried out in the erosion analysis other soil properties may become required and these will be investigated and presented as needed.

One aspect which should be studied in detail is the effect of the sewage effluent on the clay properties. This study is beyond the scope of this investigation, but it could serve as another research project. Following the literature review of the erosion of soils, this effect will be further discussed in Chapter 8.

CHAPTER 5

USE OF NEUTRON MOISTURE AND
DENSITY MEASURING EQUIPMENT

The Soil Mechanics Department of the University of Manitoba has a neutron moisture and density meter with both surface and depth probes. This equipment was purchased from Nuclear Enterprises (G.B.) Limited. Its purpose is to measure moisture contents and densities of soil in the field using radioactive sources that are lowered into cased holes. Under the direction of Professor A. Baracos this equipment was utilized at the Charleswood Sewage Lagoon.

Description of Moisture and Density Measuring EquipmentNE3401(1)

The fully transistorized equipment consists of a moisture measuring probe, a density measuring probe, a portable scaler unit type NE 5011, gamma ray and neutron shields, carrying cases and a cable for depth measurement. During transit the probes are endorsed in lead-paraffin and lead shields, respectively, which give adequate protection against radiation hazard. Calibration curves are provided which enable times for a preset number of counts to be directly related to moisture content and bulk density of the material being measured.

-
1. Nuclear Enterprises (G.B.) Ltd., Moisture and Density Measuring Equipment NE3401, Instruction Manual, Sighthill, Edinburgh, Scotland, November 1962.

Density measurements are obtained with a probe containing a 5mCi Cobalt 60 radioactive source separated from a Geiger counter tube by lead and contained along with a pre-amplifier in a stainless steel tube of outside diameter 1.5 inches. With an increase in material density the proportion of gamma-rays scattered towards the detector tube increases, but the absorption also increases; the two effects tending to counteract one another. Absorption can be made to predominate by suitable spacing of the detector and source so that the gamma radiation reaching the detector decreases with the material density and is roughly inversely proportional to it.

A 5mCi radium-beryllium source (emitting fast neutrons) is placed beside a slow neutron detector (BF_3 proportional counter tube) and enclosed along with a preamplifier in a steel tube of diameter 1.5 inches to form the moisture measuring probe. The fast neutrons emitted from the radium-beryllium source are slowed down by the nuclei of hydrogen present in the material being measured and the response from the slow neutron detector is a measure of the number of slowed-down neutrons scattered back and therefore a reliable indicator of its hydrogen content. Quick accurate measurements can thus be obtained of the water content of the material. Allowances may have to be made in the measurement if the material contains bound hydrogen or if neutron absorbers are present.

The probes are connected through cable to the portable

scaler (NE 5011) which automatically registers the time taken for a preset number of counts. Calibration curves enable the moisture content to be obtained directly by relation to the time taken for a predetermined number of neutron counts and the bulk density to be obtained directly by relation to the time recorded for a predetermined number of gamma ray counts. The portable scaler supplies all necessary operating voltages for the probes.

Test Site

A test site was selected midway along the west dike of secondary cell number two. Three stainless steel tube casings of 1-3/4 inch diameter were installed perpendicular to the dike. Each was placed about 10 feet below the ground level. One tube was located on the bank above the high water level on the wet side of the dike, and the others on the dry side, one being at the toe of the dike and the other about 40 feet west of it.

Program

Originally it was decided to carry out readings at regular intervals, to enable a complete history of moisture and density conditions to be logged. However, due to equipment malfunctions and breakdowns and other problems which will be discussed, the eventual result was only an irregular system of readings over the period July 1964 to August 1965 which were of questionable accuracy. As a result the program had

to be abandoned. Although considerable time and effort was thereby lost, it did provide a certain background and experience to those involved.

Problems encountered

1. The steel casings placed were not plugged at the bottom. As a result water seeped in, which was not noted until some time after the program had begun. This was later rectified by removing the water before readings.

2. The equipment tended to give unstable readings. Some readings drifted as much as 100 percent or more. This was confirmed under carefully controlled conditions in the laboratory. "Counts" obtained for a fixed soil moisture content varied so much that no significance could be attached to their values. This was later attributed to the fact that the drift was possible due to temperature instability of the circuits.

3. It was found that electric cable connections on the probes were not water tight and even small amounts of moisture would short the circuits. Some moisture is unavoidable in ordinary field use. In particular under our climatic conditions, the access tubes in the ground are often colder which results in condensation of moisture in them. Unless the moisture is removed completely by some means such as lowering a cloth, short circuits

can be experienced.

4. Because laboratory verification tests performed to check out the given calibration curve gave poor relationships, little confidence was placed in acceptable field readings.
5. A factor causing some of the problems may have been that numerous neutron equipment operators were involved over the period the equipment was in use. If one operator was involved over this period, he may have become familiar with the peculiarities of the equipment and thereby have eliminated some of the scatter of results.

Conclusions

Because of the equipment and operational difficulties encountered, it was found that the program of moisture and density readings carried out was inadequate to provide useful results. This program will not be resumed until greater confidence can be placed in the results which can be obtained. Despite this drawback, the author has gained valuable insight into the workings of a tool of the future, which in all likelihood will form an integral part of all soil mechanics investigations.

CHAPTER 6

WAVE ACTION AND FREEBOARD

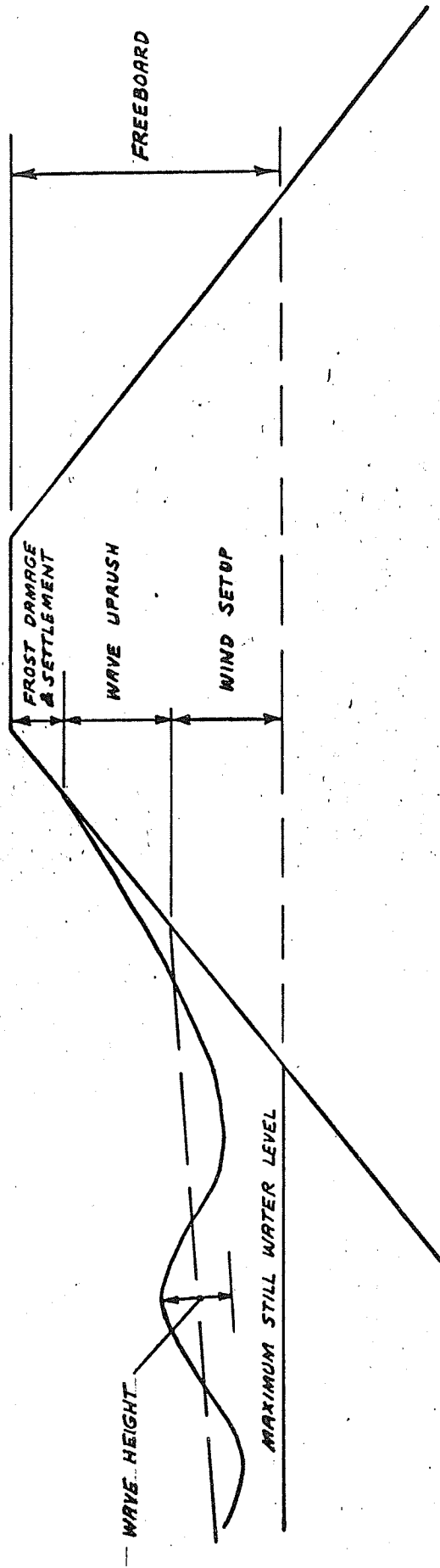
To evaluate the freeboard allowance required, a study was carried out of the wave action. Because attempts to date to maintain records of wave height and setup at the lagoon have not been successful, it was necessary to undertake a theoretical evaluation.

Freeboard Allowance Defined

Freeboard may be defined as the difference in elevation between the top of the dikes and the maximum still water elevation in the lagoon. A computed freeboard consists of allowances for wind setup, wave uprush, and frost action or settlement as shown on Figure 6.

To evaluate the amount of wave action that could occur a research of various references was made which included:

- (a) Analysis of wind records for Metropolitan Winnipeg.
- (b) Review of wind set up relations for inland reservoirs.
- (c) Review of rational procedures and generalized relationships for estimating the characteristics of wind waves generated in small inland reservoirs.
- (d) Review of analysis relevant to the development of practical relationships in estimating run-up on embankments.



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DIKE FREEBOARD
 REQUIREMENTS

PREPARED BY: A.G. KULUK
 DRAWN BY: A.G. KULUK

DATE: MAY, 1966
 FIGURE NO. 6

- (e) Study of pertinent recorded hydraulic model studies and related analysis of relevant relationships.

The following section will present the results of the above research and a comparison of the theoretical computed freeboard required to the available freeboard allowance.

Wind conditions

To select a suitable wind velocity it is necessary to analyze available wind records and to select a wind of a suitable duration. From automatic lake level recordings it has been found that a full wind setup can take place within a few hours. It follows that the wind velocity to be selected should have a duration of a few hours only.

It is generally known that the shorter a fetch is, the shorter the duration of sustained wind required. For our purposes it would not be unrealistic to select a maximum wind velocity for a wind duration of one hour. The use of the maximum recorded wind velocity for a one hour period compensates for the fact that a smaller wind duration period was not selected.

An analysis was made considering the maximum daily average winds having one hour duration for the active months (April to October) for Winnipeg at the Winnipeg Airport. Records were studied for the period 1921 inclusive to 1964. Because the fetch length is relatively the same regardless of the direction, there was no need to select a predominant wind direction. From this study a maximum wind velocity of 56 miles per hour was obtained.

For large bodies of water over land winds are usually increased by 10 to 30 percent to obtain comparable over water winds. However, for small reservoirs it has been reported that the effect of wind velocities over water surfaces is not substantially higher than velocities over land surfaces.¹

Wind Setup²

When wind blows over a body of water, a friction force is being applied to the surface of the water, in the direction of the wind. As a result of this force, the surface water will begin to move also in the direction of the wind and will begin to pile up on against the leeward shore. This will cause a return flow along the bottom of the lake from the leeward shore to the windward shore.

Theoretically it has been found that the slope of the water surface is directly proportional to the wind stress and the bottom stress, and is inversely proportional to the depth of water. It has also been found from field observations and experiments that the bottom stress is a small portion of the wind stress which is proportional to the square of the velocity. Therefore, the wind setup may be expressed by:

$$S = \frac{V^2 F}{CD}$$

where: S = wind setup above still water level (feet)

V = wind velocity (m.p.h.)

-
1. T. Saville, S.W. McClendon, and A.L. Cochran, "Freeboard Allowances for Waves in Inland Reservoirs", (Proc. A.S.C.E., Vol. 88, No. 442, Paper No. 3138, May, 1962, p. 112.
 2. Edward Kuiper, Water Resources Development, (Great Britain Butterworth and Co., 1965) pp. 187-188.

F = fetch (miles)

D = average depth (feet)

C = coefficient, usually 1,600,

however, for shallow water = 1,400

To calculate the theoretical wind setup for a typical lagoon unit it was assumed that the maximum water depth would be 8.0 feet. Therefore this was the depth considered for free-board allowance calculations.

Applying the above formula, it was found that:

$$S = \frac{V^2 F}{CD} = \frac{(56)^2 (0.3)}{(1400) (8.0)} = 0.084 \text{ feet (use 0.1 feet).}$$

Wave Characteristics

Waves are generated by winds and their characteristics are determined by the velocity of the wind, its duration, and the fetch length. It has been noted that in relatively shallow water areas, wave generation is affected by water depth. The factors used to define a wave's characteristics are height, period, and length. Wave height (H) is defined as the vertical distance between the crest of a wave and the preceding trough. The wave length (L) is defined as the horizontal distance between successive wave crests measured perpendicular to the crests. The wave period (T) is defined as the time required for a wave crest to traverse a distance equal to one wave length.

To forecast the wave characteristics for the Charles -

wood Sewage Lagoon which can be classified as a shallow water or transitional area, relationships as presented by the Beach Erosion Board were utilized.³ The results obtained were also checked with the values presented by J.C. Hufft who supplemented his study of shallow water and transitional waves with hydraulic laboratory studies.⁴

Because of the inherent variability of wind waves, the concept of "significant wave height" has been introduced. It is defined as the average of the highest one-third of the waves present. This is the concept which is currently widely accepted and which the wave height given below can be considered to correspond to.

Using a wind velocity of 56 miles per hour, a depth of 8.0 feet and a fetch of 1600 feet the following wave characteristics were calculated:

Wave height = 1.0 feet
 Wave length = 16.0 feet
 Wave period = 1.8 seconds.

Wave Uprush

When a wave reaches the toe of a sloping embankment without major modifications in characteristics, the wave will ultimately break on the embankment and run up the slope to an elevation governed by the slope, the roughness and permeability

-
3. Corps of Engineers, U.S. Army, Shore Protection Planning and Design, (Technical Report No. 4, Beach Erosion Board, 1961) pp. 25-28.
 4. John C. Hufft, "Laboratory Study of Wind Waves in Shallow Water", (Proc. A.S.C.E., Vol. 84, No. WW4, Paper No. 1765, September, 1958).

of the embankment, and the wave characteristics.

Wave uprush R, is the vertical height difference between the maximum elevation attained by wave run up on a slope and the water elevation at the toe of the slope, excluding wave action. It should be noted that for all variables kept constant, the run up increases as the slope becomes steeper.

From the research work done by K.A. Adam who supplemented his work with hydraulic model studies, results were obtained for the wave uprush corresponding to the wave characteristics previously calculated.⁵ Assuming a smooth slope characteristic, for the one vertical on six horizontal side slope the wave uprush was found to be 1.0 feet and for the one vertical on nine horizontal side slope the wave uprush was found to be 0.6 feet.

Allowance for Frost Action and Settlement

According to the Corps of Engineers of the U.S. Army, the usual settlement of a properly placed and compacted fill, exclusive of foundation settlement, will be less than one percent of the embankment height.⁶ Therefore, 0.10 feet should be assumed.

Frost action on the top of the dike tends to make the soil friable and pervious. This allows weathering to take place

-
5. Kenneth M. Adams, "A Model Study of Wave Run-up on Smooth and Rough Slopes", (Unpublished Master's Thesis, The University of Manitoba, Winnipeg, Manitoba, 1963).
 6. Corps of Engineers, U.S. Army, Earth Embankments, (Engineering and Design Manual, EM 1110-2-2300, April, 1959), p. 19.

and some allowance must be made for potential changes. Of importance is the embankment type and maintenance program. For the lagoon dikes this behaviour should not be significant.

On the basis of the above analysis an allowance of 0.5 feet should be adequate.

Theoretical Freeboard Required and Comparison

Summarising the results previously given, the total theoretical freeboard can be obtained.

TABLE NO. 2

THEORETICAL FREEBOARD REQUIRED

Description	Side Slopes 6:1 (feet)	Side Slopes 9:1 (feet)
Wind Setup	0.1	0.1
Wave Uprush	1.0	0.6
Frost Action or Settlement	0.5	0.5
Total Freeboard	1.6	1.2

For intermediate side slopes the total freeboard could be obtained by interpolation.

The actual available freeboard for a typical lagoon is approximately 2.0 feet above the assumed high water level. On the basis of the above theoretical values obtained it can be concluded that this allowance is sufficient.

It should be pointed out that if a situation had been chosen corresponding to a very rare wind velocity and having a short time duration period, it may have indicated that the freeboard allowance is inadequate. Since there is no human life involved the tendency should be to select a wind frequency that would correspond to an improbable condition rather than to a maximum possible event.

Conclusions

The theoretical freeboard allowance appraisal made, indicated that the freeboard allowance for the Charleswood Lagoon is adequate. However, as future lagoons increase in size, the type of analysis made can be repeated to insure ample freeboard.

Another very important aspect of the Charleswood Lagoon is the opportunity to undertake instrumentation and recording of actual relationships between wind and wave characteristics. In this way, rather than relying on general formulae it would be possible to develop unique equations directly pertinent to Manitoba lagoons or to shallow bodies of water.

The complication involved in making analysis of the above, is the fact that the lagoon effluent is not ordinary water. The secondary cells have a lower concentration of grease and detergent loading than do the primary cells. This has been seen to affect the wave action at certain times because there

has been little or no wave development on the primary cells whereas on the secondary cells significant wave development has occurred. These observations are substantiated by the photographs presented in Chapter 3. Therefore it appears that sewage effluent has a surface tension effect which depends upon the relative concentration of the effluent. A laboratory and field research has been begun by Metro engineers, which may lead towards firm relationships after significant data has been compiled.

The above programs discussed are definitely beyond the scope of this thesis. However, they do present a desirable field of potential research.

CHAPTER 7

BANK SLOPES

One of the most important aspects of the economy of sewage lagoons is the side slope to be utilized. As has been covered previously, this aspect is indeed subject to numerous approaches. It has been pointed out that some installations have used side slopes as steep as one on two and others are made as flat as one on ten. The Charleswood Sewage Lagoon has interior side slopes, which vary from one on six to one on nine and the exterior side slope is constant at one on five. Aside from the relative performance of each of these slopes which will be covered in a subsequent chapter, it is of importance to consider what side slope would be the optimum for local conditions. To make this study the author has taken advantage of several studies, which have been made with regard to the stability of numerous local rivers, waterways, and canals.

The common factors involved in the behaviour of lagoons and waterways include the alternate wetting and drying action, saturation of clays, wind and wave action, freezing and thawing action and ice action. The additional conditions that a waterway is subject to include the forces of flowing water at a significant velocity parallel to the banks and also the increased action of ice as carried by the above flow. It follows

therefore that a river or waterway slope is subject to more critical conditions and if anything should require a flatter side slope than that of a lagoon.

Riverbank Survey in Metropolitan Winnipeg¹

In 1960 a survey was conducted of banks along the Red and Assiniboine Rivers, mostly within Metropolitan Winnipeg, to determine at what slopes natural banks were stable, the degree of stability where possible, at what slopes natural banks failed and at what slopes such failed banks again became stable.

All banks were visually examined, classified as to stability, and cross-sections were taken of banks considered appropriate for the study. Factors affecting this stability, such as given below, were noted:

- (a) Planar alignment of the banks.
- (b) Estimated degree of top erosion.
- (c) Vegetation.
- (d) Existence of shrinkage and tension cracks.

The conclusions made from the above study were:

1. Very few stable riverbanks were found in Winnipeg and adjoining areas.
2. The "convex riverbanks" cannot be used in slope stability studies because the soils are of fluvial origin and are primarily silts.

1. A. Baracos, "The Stability of River Banks in the Metropolitan Winnipeg Area", (Proc. 14th Ann. Can. Soil Mech. Conf., A.C.S.S.H., TM 69, 1961.).

3. The majority, about eighty percent, of concave river-banks, in the highly plastic clays, between 25 feet to 40 feet high which failed in a rotational manner, became stable at slopes between 1 on 4 1/2 and 1 on 6 3/4. Naturally these failed at slopes steeper than the slopes at which they became stable.

Slopes of Drains throughout the Province of Manitoba

Considering the criteria utilized for soils similar to those found at the Charleswood Lagoon, a side slope comparable to a given drain design could be selected. For a height of dike of about eight or nine feet and a five to eight foot depth of water, a side slope of between 4 and 6 horizontal to one vertical would be selected. Field investigations have verified the fact that these side slopes are adequate from the standpoint of performance over a period of years.

Red River Floodway Slope Selection²

One of the most important considerations of the Red River Floodway was the soil mechanic's investigation to ascertain a satisfactory side slope. Professor A. Casagrande recommended a slope of one vertical to six horizontal throughout the major portion of the Floodway. Slopes of one on three were used through the Bird's Hill area because the excavation was through sand and gravel and this material has a high natural angle of repose which permits a steeper slope to be utilized.

2. J.D. Mishtak, "Soil Mechanics Aspects of the Red River Floodway", (Canadian Geotechnical Journal, Vol. 1, No. 3, July 1964).

Conclusions

On the basis of the bank slope study made in this section and also considering factors as covered in the freeboard and bank erosion sections, it appears that for physical dimensions corresponding to those at the Charleswood Sewage Lagoons, one vertical on six horizontal side slopes would be utilized. In the writer's opinion the steepest possible side slope would be one vertical on four horizontal.

A further study from this standpoint will be made in Chapter 10, when the dike cross section behaviour will be analyzed.

CHAPTER 8

BANK EROSION LITERATURE REVIEW

The nature and extent of bank erosion can be very broad, depending on the situations. Generally speaking, bank erosion of clay banks can be caused by:

(a) Wave Erosion

Waves need not be high to be erosive. Small but persistent wave wash can leave its marks on a bank.

(b) Static Erosion

Excessive erosion at the toe of a slope, which creates a steep face in the bank; an internal shear failure eventually occurs which causes sliding or sloughing of the soil.

(c) Aging

Wetting and drying have a deteriorating effect on the resistance of an embankment to erosion. This is caused by the disruptive effect of slaking on the soil structure. Freezing and thawing cycles have a similar effect, although it is not as severe. Continuous submergence in water also has an effect of making a soil less stable and more susceptible to erosion.

(d) Wind Erosion

Acting on slopes, wind dislodges the finer particles of soft formations by direct attack. Thus a cut

slope may be simply scoured back to a durable surface, or it may be progressively eroded for an infinitely long period.

(e) Rainfall Erosion

Generally areas devoid of vegetation are marked by the gulleying effect of rainfall runoff. This effect generally accelerates with time.

(f) Bank Side Slope Effect

This was discussed in detail in the previous chapter. Generally the flatter the slope the more stable the dike bank.

(g) Ice Action

Ice sheet pressure during winter or cake action during break-up can cause considerable damage.

The factors presented are general, and in fact, wave erosion and static erosion appear to be interlated with the aging effect also a factor. Therefore, a detailed study was made to evaluate the condition more distinctly and if possible to relate the bank erosion to definite soil properties.

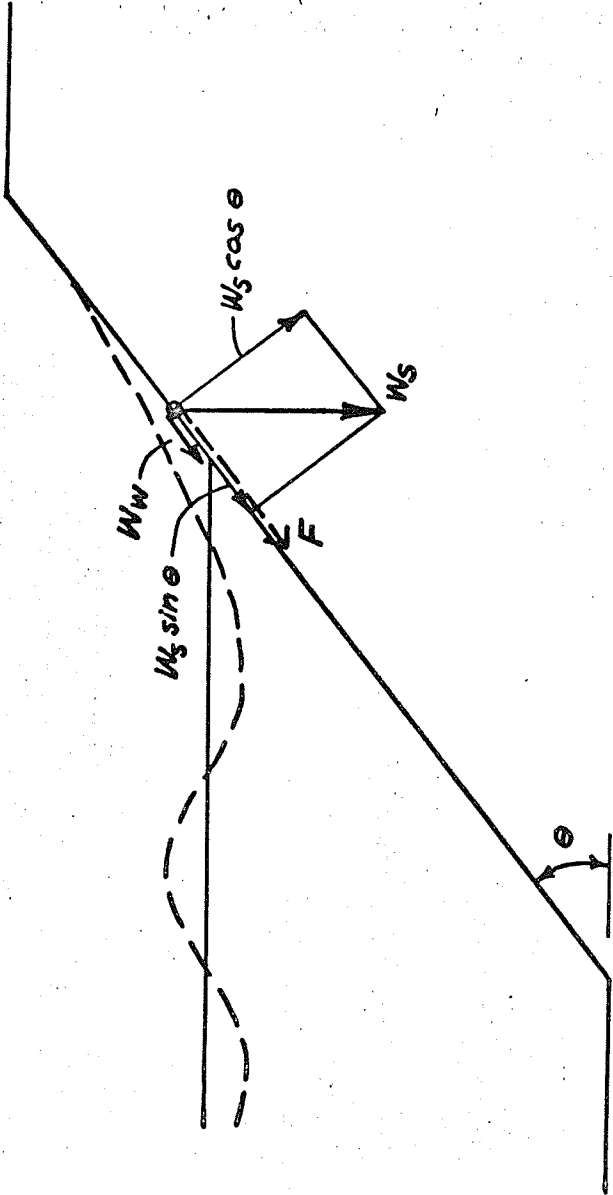
In general, the scour phenomena can be readily explained. However, there are numerous factors which apparently effect it and these are not fully understood. Following a general assesment of the phenomena, a summary of various references will be presented.

Scour on the banks takes place when the particles com-

posing the sides are acted upon by forces sufficient to cause them to move. A particle is acted upon by the force of water and also the force of gravity, which tends to make it roll or slide down this slope. This is on the assumption of a wave having uprushed and then returning down the slope, as shown on Figure 7. If the resultant due to the motion of water and the component of the force of gravity acting on the particle are large enough, the particles will move. When cohesive forces on the particle are present, the external forces acting must be sufficient to overcome this also before movement can occur. When the particles are sufficiently small they may be carried away by being taken into suspension. In addition, it is also possible for the uprush of water to cause major dislodgment, which will lead to erosion with further wave action.

It should be pointed out that the stability of side slopes of a canal in non-cohesive material involves the angle of repose of the material. This is quite different in the case of cohesive soils where apparently the cohesive force is the dominant factor and the angle of repose is not applicable. Other factors are the size and shape of soil particles, range and distribution of particle sizes and various physical and chemical properties.

To assess the details of bank erosion, a research was made of all available literature. The most pertinent references were obtained from articles on the design of channels in cohesive soils.



W_s = WEIGHT OF PARTICLE

θ = SLOPE OF BANK

W_w = WATER DISPLACEMENT FORCE

F = TOTAL FORCE ON PARTICLE

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FORCES ACTING ON A
DIKE BANK PARTICLE

PREPARED BY: A.G.KULLUK DATE: MAY, 1966

DRAWN BY: A.G.KULLUK FIGURE NO: 7

United States Bureau of Reclamation (U.S.B.R.)

This agency has carried out the most widespread and continuous research on the erosion of cohesive soils. Their work gained prominence in 1950 as a result of work done by E.W. Lane, and is still progressing to date.¹

Field and laboratory studies of erosion and tractive forces on cohesive soils have been made to establish the behaviour of soils under hydraulic forces, and physical properties of soils have been compared to hydraulic forces and the degree of soil erosion.²

The initial observations indicated that:

For fine-grained cohesive soils the plasticity characteristic appeared to be the principal property for evaluating erosion resistance.

Density has some effect on erosion resistance but not as pronounced as plasticity. For the fine-grained cohesive soils investigated, the density indicated that most of the soils were very loose. It was apparent that a loose condition would be a common occurrence for near surface soils of a canal which are subject to erosion, which indicated that little reliance should be placed on this property for erosion resistance.

1. Emory W. Lane, "Design of Stable Channels", (Trans. A.S.C.E., Vol. 120, Paper No. 2776, 1955), pp. 1234-1279.

2. United States Bureau of Reclamation, A Study of Erosion and Tractive Force Characteristics in Relation to Soil Mechanics Properties Earth Research Program. (Soils Engineering Report No. EM-643, Denver, Colorado, February 23, 1962), p. 7.

In recent years, studies have been in progress to establish better design criteria for earth-lined and unlined canals constructed in fine, cohesive soils. To continue these studies a recirculating flume was constructed and tests conducted on several eight inch-diameter by 20.3 centimeter high soil samples.³ Soil density, soil moisture content, and temperature of flowing water in the flume were well controlled. Samples were tested by gradually increasing the boundary shear acting on the sample until shear became critical and the sample began to erode. Tests indicated that the time increment used for increasing the boundary shear on the soil was not critical within the range used and that minor increases in temperature of the flowing water had little effect on the discharge necessary to produce erosion. For the soil tested, the boundary shear required to erode the soil was a function of the moisture content at which the soil was compacted. However, the tests were inconclusive regarding the effect of the soil densities.

E.M. Laurson (1956)⁴

Little is known concerning the resistance to movement

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3. United States Bureau of Reclamation, Canal Erosion and Tractive Force Study - Analysis of Data from a Boundary Shear Flume. (Hydraulic Report No. Hyd 532, Denver, Colorado, December 12, 1963), p. 1.
 4. Emmett M. Laurson, "Sediment-Transport Mechanics in Stable Channel Design," (Trans. A.S.C.E., Paper No. 2018, 1956), p. 202.

of clays except that some clays can withstand very high boundary shear. One might speculate that resistance to boundary shear should be related to the internal shearing characteristics of the clay mass in an undisturbed saturated condition. For naturally deposited material, which is not likely to be completely homogeneous, the weakest part of the mass would govern. Erosion of seams or pockets of weak material would expose the adjacent stronger material to greater forces.

P.W. Terrell and W.M. Borland (1956)⁵

It has been noticed that certain canals scour, whereas others under very similar conditions do not. These conditions were similar as to width, depth, side slopes, velocities, reasonable freedom from sediment load, mechanical analysis, and plasticity index of soils. Because these factors do not account for the dissimilarity of results, soil samples were taken for chemical examination. The examination may show that an ion exchange between water and soil hydration of soil material provides a binder in some localities. Should this prove a dependable condition, future evaluation of a proposed section should include whatever advantage can be taken of such benefits.

The allowable tractive forces for ephemeral streams in cohesive soils are higher than perennial streams in the same soil

5. Pete W. Terrell and W.M. Borland, "Design of Stable Canals and Channels in Erodible Material," (Proc. A.S.C.E., Vol. 32, No. HY1, Paper No. 880, February, 1956), p. 5, 15.

Vegetation is effective in increasing the allowable tractive force on the banks. However, vegetation will not grow in the perennially inundated parts of the channel unless velocities are exceedingly low.

I.S. Dunn (1959)⁶

The past experience in canal design has been to accumulate data, mostly from experience, and to relate the stability of a canal in a specified material to the maximum permissible average velocity. The soil materials in this relationship have been described by such terms as silty-clay, hardpan, etc. These terms are very general and are not adequate to fully describe the properties of the soil with respect to their resistance to erosive forces.

The effect of soil properties on tractive resistance was investigated by the writer by measuring the hydraulic shear stresses necessary to cause erosion of soil samples in which the cohesion was varied by applying different degrees of preconsolidation. A vane borer was used to measure the shear strength of the soil.

The author's conclusion was that good predictions of allowable tractive forces for soils with a plasticity index between 5 and 16 could be made and that a prediction based on per cent of silt and clay in the soil was also quite accurate.

6. Irving S. Dunn, "Tractive Resistance of Cohesive Channels", (Proc. A.S.E.E., Vol. 85, No. SM3, Paper No. 2062, June, 1959), pp. 1-24.

E.T. Smerdon and R.P. Beasley (1959)⁷

The study made investigated a number of cohesive soils, both in the laboratory and in a hydraulic flume, to determine if the critical force could be correlated to the physical properties of the soil. The data from the hydraulic and physical tests on the soils were analyzed statistically to determine the significance of the apparent correlation between critical tractive force and pertinent soil properties. For the soils tested, the critical tractive force was found to be well correlated with each of the following soil properties:

- (1) Plasticity index.
- (2) Dispersion ratio.
- (3) Mean particle size.
- (4) Per cent clay.
- (5) A less significant correlation existed between critical tractive force and the phi-mean particle size.

All the soil samples were taken in the early spring when the soil was dry enough for a truck to be moved into the field. However, the moisture content and the degree of aggregation of the soils varied considerably. The moisture content at the time of the testing in the hydraulic flume was essentially the same, since the soils were wetted prior to each test. However, the degree of aggregation of the soil likely affected

7. E.T. Smerdon and R.P. Beasley, "The Tractive Force Theory Applied to Stability of Open Channels in Cohesive Soils," (Research Bulletin 715, The University of Missouri, Columbia, Missouri, October, 1959), 36 pp.

the tests, since highly aggregated soils tended to erode as aggregates rather than by dispersing into smaller units during the erosion process. It is not known what effect the moisture content of the soils at sampling time had on the results.

Another factor which may have had an effect was the severity of the winter prior to testing. The amount of alternate freezing and thawing that a soil is subjected to affects the stability of the soil aggregates. Therefore, since the soils were sampled from different parts of the state of Missouri and were exposed to different degrees of freezing and thawing, the climate of the previous winter may have been a factor.

S.R. Miller (1960)³

When the channel bed and banks contain clay the property of cohesiveness becomes the key to stability. At present there is no completely reliable means of evaluating or establishing cohesiveness in terms of the ability of the particular material to withstand the imposed erosive forces. It is known that, in general, the more cohesive the material the greater its ability to resist scour. Considering particle size as a measure of cohesion, we find that the ability of the material to resist scouring velocities increases with decreasing particle size below the silt range. But particle size alone is not a satisfactory measure of the cohesive properties of resisting ability

³ S. R. Miller, "A Critique on Stable Channels in Cohesive Materials and a Research Proposal," (Research Report No. 333, Agricultural Research Service, U.S.D.A., August 18, 1963), 52 pp.

of a soil. Further complicating the situation is the effect of vegetation on stability of a channel bank. The ability of vegetation to become established is related to the organic qualities of the material which generally infers that cohesive type properties must be present. There are still other factors that complicate the evaluation of stability of cohesive materials including an increase or decrease in resistance properties with wetting, the effects of alternate wetting and drying, and freezing and thawing.

A new and far reaching examination of the problem of stability in cohesive materials should be taken. The purpose of this report is to critically examine the more common and widely used and accepted procedures or expressions relating to resistance of cohesive materials to the scour action of flowing water and then to propose research investigations that, if pursued, should improve the existing procedures, result in new evaluation methods, and lead to establishment of criteria more accurate and useful to the design engineer.

E.M. Flaxman (1963)⁹

From an assessment of work previously done numerous comments were made. Observation and studies such as those of A.W. Skempton and K. Terzaghi, pointed toward the mineralogical

9. Elliot M. Flaxman, "Channel Stability in Undisturbed Cohesive Soils," (Proc. A.S.C.E., Vol. 89, No. HY2, Paper No. 3462, March, 1963), pp. 87-96.

chemical and physical conditioning that cohesive soils undergo in the process of deposition and aging. F.P. Trask and J.E.H. Close have related the shear strength of various clays and clay sand mixtures with water content over a considerable range. Their investigations demonstrated that substantial losses in strength occur with increases in moisture content.

The author pointed out that a relationship seems to exist between permeability and erosion resistance of soils. He also made observations of cohesive soils losing their cohesiveness after saturation.

E.H. Grissinger and L.E. Amussen (1963)¹⁰

Cohesive materials derive their stability and resistance to erosion primarily through their electrochemical properties. For naturally occurring cohesive channel materials, these forces originate primarily in the clay fraction. The magnitudes of these interparticle forces vary with the state of hydration and time for any particular clay. The critical hydraulic conditions for stability of a given cohesive soil are variable and dependent on the short period intimacy of the channel boundary with water.

On the basis of tests performed on relatively dry and wet samples it was found that an increasing rate of erodibility was dependent on the degree of wetting of the cohesive material.

10. (Reference 9.) Discussion, (Proc. A.S.C.E., Vol. 89, No. HY6, Paper No. 3708, November, 1963), p. 259.

The initially dry, cohesive materials are relatively stable, probably due to interparticle friction and minute moisture films of high surface tension. During the erosive event, the cohesive materials absorb water, the interparticle stresses are apparently relieved by the free water, and the rate of erosion is increased.

It stands to reason if a cohesive soil is continually saturated, then it will have a reduced resistance due to factors as discussed, and it may be assumed that it takes on the inherent stability of non-cohesive materials which are a function of size, specific gravity, shape, and surface texture of the individual particles.

D. Berghager and C.C. Ladd (1964)¹¹

A review of previous work indicated that most of the previous investigations did not adequately control the engineering properties of the clays being eroded and/or did not measure relevant soil properties, especially regarding strength characteristics, in establishing correlations between soil properties and erosion resistance.

The problem is complex because it demands advanced knowledge both from the fields of soil mechanics and hydraulics. Recognizing the need for a concentrated study of the erosion of cohesive soils by both hydraulic and soil engineers

11. Dirk Berghager and Charles C. Ladd, "Erosion of Cohesive Soils," (Research Report R64-1, Massachusetts Institute of Technology, Cambridge 39, Massachusetts, January, 1964), 24 pp.

the Soils and Water Resources Divisions, Department of Civil Engineering at the Massachusetts Institute of Technology, are jointly carrying out a research program in this direction.

It is clear that many of the previous investigations have been hampered by the fact that both hydraulic and soil engineering were not used in research on the erosion of cohesive soils. Soil Engineering is not sufficiently developed to predict the behaviour of a cohesive soil under the conditions relevant to erosion by flowing water. For example, at the soil water interface, the effective stress is zero, but the literature in soil engineering has almost no data on strength at very low normal stresses. Thus the research program at M.I.T. was devised to:

- (1) Observe and define erosion through means of flume tests and thereby to determine the relevant parameters.
- (2) Run shear tests on soil at almost zero normal stress in order to help predict the behaviour of soil when exposed to action by running water.

Some results as were reported are:

Scour seemed to consist of a random plucking of small, flat chunks of soil. The erosion rate seemed to increase with increasing tractive force.

In samples allowed to settle from suspension and which have not been consolidated, scour seemed to consist of a more continuous detachment of pieces too small to be distinguishable.

At the end of the testing it was observed that scour had formed a rough soil surface, which seemed harder and stronger than the original surface.

A feeling was gained for how scour takes place and for the magnitudes involved.

The first samples tested eroded at an approximately steady rate regardless of the applied shear stresses. Small chunks of soil were carried away even at the smallest velocities. It was felt that this was due to slaking. Slaking is the progressive breakup of a soil when immersed in water. However, for advanced testing, the sample was consolidated under vacuum so that no air would be present. Thus the release of air upon wetting did not occur and the detachment of soil pieces did not occur.

There is a great deal of valuable information in this publication and future investigations modelled on this approach should provide valuable insight into the detailed behaviour of cohesive soils.

A.S.C.E. Task Force (1964)¹²

Stated broadly, the solution of stable channel problems involves the deciphering of the laws of nature and their checks and balances which cause channels to perform in the great variety of fashions which have been observed. One cannot help but marvel at how intricate and delicate some of these influ-

12. Daryl B. Simons, Panel Chairman, Panel No. 3. "Stable Channels," (Proc. A.S.C.E., Vol. 90, No. 124, December, 1964, Part 1 of 2 Parts), pp. 50-62.

ences are in nature by observing the variety of bed forms, channel dimensions and flow characteristics that occur from place to place, and from time to time. These can presently be explained only in the broadest qualitative terms. With the present state of knowledge only one or a few variables can be studied at a time in an attempt to better understand their individual effects on the performance of a channel.

One of the largest voids in design criteria exists where cohesive materials are involved. We do not know the magnitude of the forces necessary to cause scour (which varies with degree of wetting), we do not know which physical and chemical properties, and we do not have instruments or techniques fully developed for performing comprehensive measurements in the laboratory and field. Present and past methods of describing cohesive materials should be carefully reviewed including the plastic index, texture test, chemical compositions and characteristics of the kind determined in a device such as the USBR shear tank. The possibility of developing shear vanes should be pursued. Such research could be carried out in the field on both large and small channels. With the development of a suitable field instrument to measure shear strength, it is also essential to develop an instrument capable of measuring shear stress on the banks of cohesive channels. Concurrently, it is essential to continue the search for parameters or variables which express the resistance to flow of cohesive banks. Perhaps instruments can be designed with

which bank resistance can be directly measured.

Methods for determining the stable slope of various soil materials when subjected to wave action requires study. Relationships between various exposures, climate, site conditions, and the size of the waves generated need investigation.

Emmanuel Partheniades¹³

In general, the work in the erosion field has been concentrated in the correlation of some critical velocity or critical shear stress to the fundamental clay properties. Some of the correlations are based on small scale scour tests, but usually the definitions made are arbitrary and entirely based on visual observation and the judgement of the observer.

A straight rectangular flume was used with recirculating water at ocean salinity. A special kind of clay commonly known as San Francisco bay mud was used as bed material. The developed erosion patterns together with chemical and mechanical changes of the bed surface during the process of erosion and deposition were carefully observed and described. Cementing agents dissolved in the water and suspended sand were found to cause significant changes in the properties of the bed material and resistance to erosion.

13. Emmanuel, Partheniades, "Erosion and Deposition of Cohesive Soils," (Proc. A.S.C.E., Vol. 91, No. HY1, Paper No. 4204, January, 1965), pp. 105-139.

A.G. Altschaeffl¹⁴

The erodibility of a certain clay to a given mechanical influence is greatest in the case of high content of a dispersing agent in water. The liability of a clay to become dispersed depends on its type, content of humic gels, salinity, etc. It would appear that the properties of the water play a strong role in the resistance a clay offers to erosion. As the work done by E. Partheniades were conducted with water at ocean salinity, the influence of the water quality on erosion rate could not be determined.

The foregoing suggests that susceptibility to erosion is not as strongly influenced by the state of compaction as was previously believed. In fact, it may be that erosion of clay, whether in the bed of a stream or at the sides of a crack that has developed in a compacted earth dam, can be severe, regardless of the state of soil compaction, if the proper fluid environment is present.

Conclusions

The literature research presented gives an outline of the various work which has been carried out in the field of soil erosion. As the numerous references have indicated, there are numerous relevant soil properties to be considered, but there appears to be no single definite approach.

14. (Reference 13) Discussion, (proc. A.S.C.E., Vol. 91, No. HY5, Paper No. 4404, September, 1965), pp. 301-308.

The author believes that the best manner in which to undertake a given erosion problem is for the designer to become familiar with all the literature available from the soil mechanics and hydraulic engineering standpoints, review the behaviour of similar projects, and after studying the particular project in detail, apply his personal knowledge and judgement. This is the approach that the author has utilized for the Charleswood Sewage Lagoon erosion problem.

It had been suggested that a hydraulic model study could be used to investigate this project. In numerous reported cases where models have been utilized, the research work was well enough advanced so that the engineers could isolate their model to only several variables. In this case there are many different factors involved. In addition, since the project itself could be studied for a period of several years, there appeared to be no need to attempt to duplicate the behaviour of the prototype in a laboratory model.

From the author's viewpoint there appeared to be no need to undertake detailed soil testing because there was nothing to correlate it with and as a result little could be gained. General soil classification tests were conducted which uncovered a horizon of research from the standpoint of soil mechanics, but no substantial evidence was found to apply to the erosion analysis.

Basically the erosion problem can be attested to several important factors:

- (1) Because of continuous saturation, alternate wetting and drying, freezing and thawing, the surface layer of the clays of the lagoon banks have not remained as the highly cohesive clays that were installed. Loss of cohesion due to these "slaking" and desiccation factors, has resulted in these clays now corresponding closer to a relatively uniform, fine grained, non-cohesive soil. The larger the grain size and the better the gradation, the more erosion resistant a given non-cohesive soil will be. Therefore, it can be seen why water waves can easily erode the lagoon banks.
- (2) Ice action during break-up causes considerable damage. Little can be done about this unless steps such as mechanical means of air bubble injection to prevent ice formation or bank revetments which includes schemes such as floating chained logs are utilized. However, the relative worth of such steps would have to be established.

These points illustrate why basically erosion should be expected at the lagoon banks. One can try to accept this natural phenomenon and undertake periodic reshaping and maintenance as required, until such time as it can be ascertained whether or not the erosion behaviour has become stabilized or if it is indeed accelerating with time. As an alternative to just sitting back and waiting an installation of economical revetment measures can be made and their relative worth as-

certained. This approach has been undertaken, and subsequent chapters will present an overall development program of re-vegetation works.

One other approach which should be mentioned is that of chemical stabilisation. Because of the conflict with sanitary engineering interests, which of course are the prime purpose of sewage lagoons, there was no investigation undertaken from that standpoint.

One factor which has not been dealt with in the study made is the surface tension factor which was discussed in Chapter 6. The surface tension of the sewage effluent appears to vary with the degree of concentration of grease and detergents. Because pore water within clays is the basis of the interparticle stresses which creates cohesion, the sewage effluent could affect the cohesion of clays, as well as other properties. This could be investigated but it would require very delicate and detailed studies. This is beyond the scope of this thesis. However, a research of this phenomenon could well lead to unique conclusions regarding the effect of sewage effluent on soil properties and erosion.

CHAPTER 9

DIKE BANK EROSION DISCUSSURE

Because the most important aspect of the Charleswood Sewage Lagoon is the erosion problem, it was decided that special detailed work with respect to it was required. Therefore the next step was to analyze all of the available data on the actual dikes. Before this is presented, it would be wise to ponder over the various factors with respect to erosion that have been discussed in previous chapters.

A general review of sewage lagoon literature (Chapter 2) and a photographic review of numerous lagoons in the vicinity of Manitoba (Chapter 3) indicated that erosion problems are common to sewage lagoons. Some types of erosion problems and forms of remedial or corrective measures were outlined and illustrated.

Both visual observations (Chapter 4) and literature reviews (Chapter 7) indicated a phenomenon which involves a clay's loss of cohesion when for a significant time duration it is subjected to alternate wetting and drying, freezing and thawing, or continuous saturation. Under such conditions it was pointed out that the surface layer of clay of the dikes becomes equivalent to relatively uniform, fine grained, non-cohesive soil. For such soils it is known that the larger the grain size and the better the gradation, the more erosion

resistant a given soil will be.

From the standpoint of clay dike slopes, it was concluded in Chapter 6 that for this locality, the side slope that would be selected for the given physical dimensions would be one on six. It was also suggested that one on four side slopes would be the steepest possible.

Although little was covered from the aspect of wave properties and erosion, Chapter 5 dealt with the wave properties in detail and also introduced theoretical calculations of wave properties for a given wind condition.

Upon recapitulating the above factors it may be asked if an overall relationship exists. By applying the reasoning that the cohesionless clay simulates a granular type of soil it was found that an interesting analogy could be applied. A certain background must be presented before this analogy can be made evident.

Where bank erosion has been experienced immediately below hydraulic drop structures, investigations conducted have indicated that it has been caused by wave wash.¹ Eddy action has often been termed the cause. However, from both model and prototype behaviour it was observed that the velocity of the eddy between the discharging flow and the canal bank was low whereas the wave action was pronounced. From

1. P.F.R.A., Hydraulic Design of Chute Drop Structures, (Design Bulletin No. 4, Canada Department of Agriculture, Regina, Saskatchewan, December, 1954.) pp. 10 and 11.

model studies the wave action source was determined. The hydraulic jump in the stilling basin, is a violent phenomenon given to rapid pulsations whereby a "slug" of flow will alternately surge strongly out of the basin one second and be much weaker the next. This surging action sets up waves in the downstream canal by the same principle as a wave generating machine in a laboratory, except that the wave frequency and amplitude induced by the jump are not a constant. The point of maximum attack on the basin is a function of several times the sequent depth beyond the end of the basin.

In areas where it has been necessary to protect the banks of the canal, studies have been made to evaluate the size of riprap required. Iribarren has shown on beach erosion work, that the following relationship exists:²

$$D_m = \frac{K A}{\cos a - \sin a}$$

where: D_m = equivalent diameter of the stone

A = height of waves

a = slope of bank

K = empirical factor which takes into account the wave properties, specific gravity of the stone, shape of the stone, and factor of safety.

2. R. Iribarren Canvanilles and C.W.Y. Clano, "Generalization of the Formula for Calculation of Rock Fill Dikes and Verification of its Coefficients," (Translated in the Bulletin of the Beach Erosion Board, Vol. 5, 1951), p. 4.

Drawing from the earlier statement that a cohesionless clay surface simulates a uniform, fine grained, non-cohesive material, it can be seen that the Iribarren formula could be applied to arrive at an equivalent diameter of stable material required.

For numerous reasons it is not possible to simply apply this formula to obtain a diameter of material required. Firstly, K values are only available for relatively large diameter stone, which is definitely beyond the range of the Charleswood Sewage Lagoon conditions. Secondly, the wave properties to apply are difficult to access. A peak wave condition cannot be applied because of its infrequent occurrence. Again the common wave condition is difficult to access, in addition to fact that it is affected by the sewage effluent. Also at this stage the side slope is still a variable.

It is felt that a study of normal wave conditions acting on a slope between one on six and one on four would result in a particle diameter corresponding to a medium to coarse grained sand. However, the actual problem is complicated by other factors such as ice action and fluctuating water levels. The above analogy does serve to illustrate how one could approach the problem from a standpoint of wave properties, slope of bank and particle size.

Some time after this analogy was prepared by the author he found that studies from the same approach as mentioned

in the previous paragraph had been undertaken. Although no report is available at this time it was learned that the Ontario Water Resources Commission had performed investigations during 1965 which involved the effects of waves on a particular sand for side slopes varying from one on two to one on six.

At the time that a firm program had to be set up with respect to bank revetment works, the previous comments made were not in the same order and furthermore it was decided to have a more general program than one strictly from stable particle diameter versus wave conditions. For these reasons no intensive study from that viewpoint was undertaken by the author.

CHAPTER 10

ANALYSIS OF LAGOON LOADING HISTORY AND DIKECROSS SECTION BEHAVIOR

Since the original operation of the lagoons in May 1965, a record has been maintained of the water levels, and also periodic dike cross sections have been taken. An analysis of this information was made for the period covering May 1965 to July 1965 and overall conclusions prepared. It should be understood that term water actually denotes sewage effluent.

Cross Section Analysis

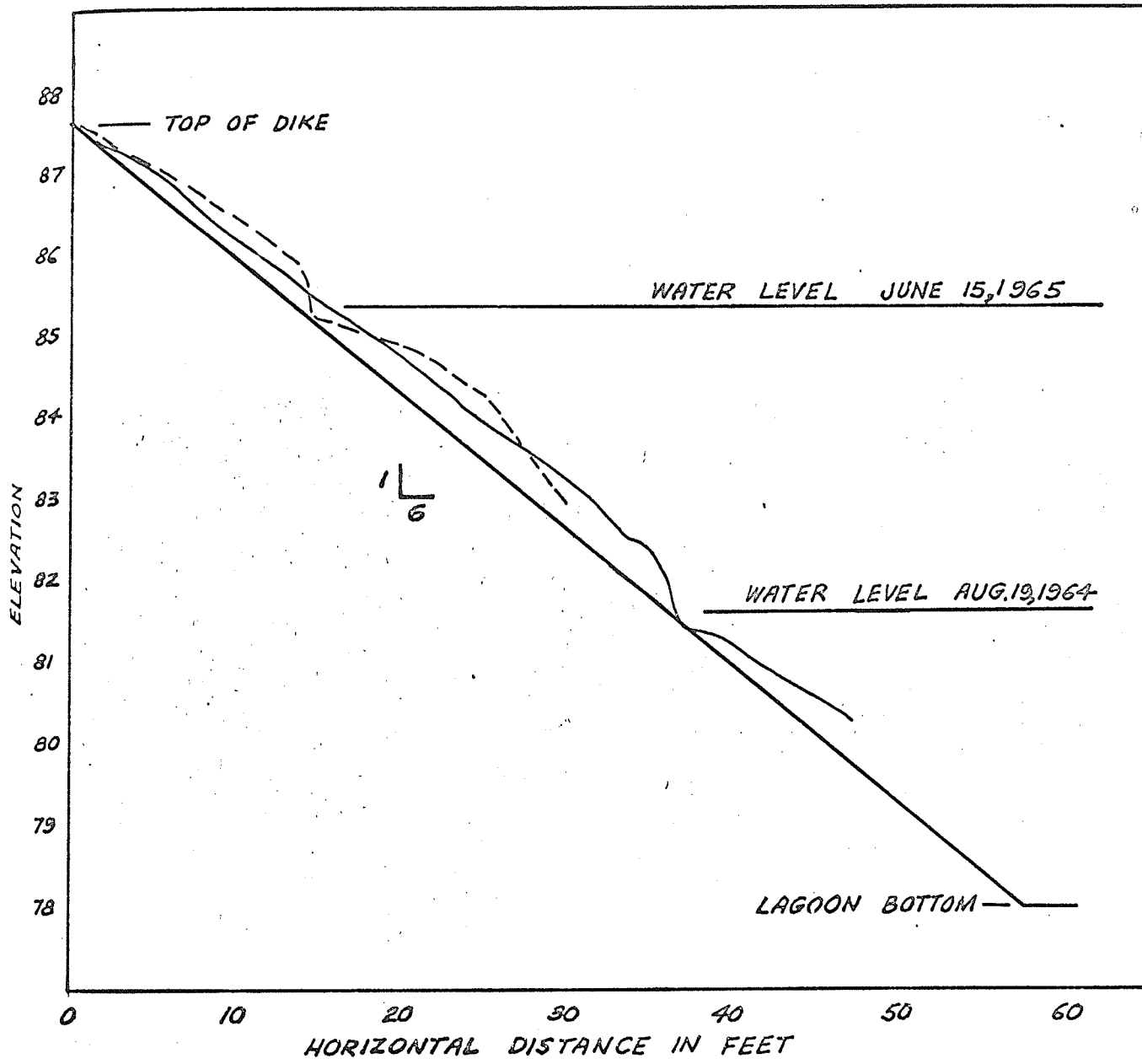
Each lagoon has numerous fixed dike control cross sections. Cross sections were available for the following dates:

Primary Cell No. 1	August 19th, 1964 June 14 and 15th, 1965
Primary Cell No. 2	September 3 and 8th, 1964 June 15th, 1965
Primary Cell No. 3	September 3 and 8th, 1964 June 14 and 15th, 1965
Secondary Cell No. 1	August 19th, September 1 and 2nd, October 7th, 1964 June 14th, 1965
Secondary Cell No. 2	August 14, 17, and 18th, 1964 November 4 and 5th, 1964 June 14th, 1965

From a study of these cross sections the following observations were made:

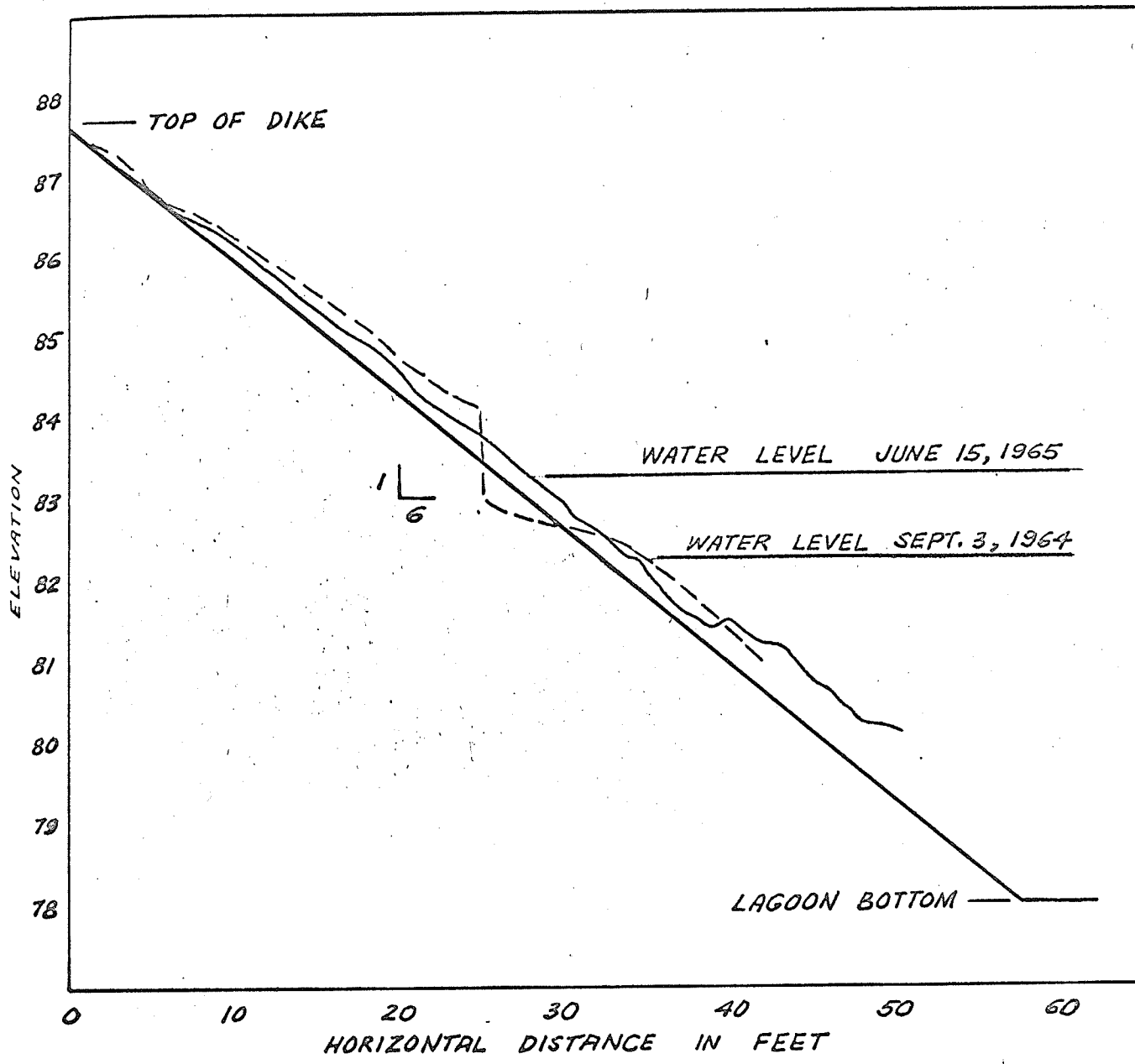
- 1) Generally, the dike cross sections have expanded since their original construction.
- 2) Little unique behaviour can be noted as the dikes change from one vertical on six horizontal side slopes, to one vertical on nine horizontal side slopes.
- 3) The erosion experienced is generally one of the two types:
 - (a) Troughing effect at water level.
 - (b) Vertical face created at water level, which is actually an advanced condition of (a).
- 4) The relative overall degree of erosion was as follows:
 - (a) Primary Cell Number 2 has experienced the greatest erosion.
 - (b) Primary Cell Numbers 1 and 3 have had about an equal amount of erosion, which is notably less than that of (a).
 - (c) Secondary Cell Numbers 1 and 2 have behaved in similar fashions, which constitutes the lightest erosion.
- 5) At some cross sections taken, the surveys were not extended far enough below the water level to provide useful results.

To provide a visual comparison, Figures 8 to 12 were prepared to illustrate typical cross section behaviour.



LEGEND
 — AS DESIGNED
 — AUGUST 19, 1964
 - - - JUNE 15, 1965

MASTER OF SCIENCE IN CIVIL ENGINEERING UNIVERSITY OF MANITOBA	
TYPICAL CROSS SECTION OF INTERIOR DIKE BANK PRIMARY CELL NUMBER 1	
PREPARED BY: A.G.KULUK	DATE: AUGUST, 1965
DRAWN BY: A.G.KULUK	FIGURE NO. 8



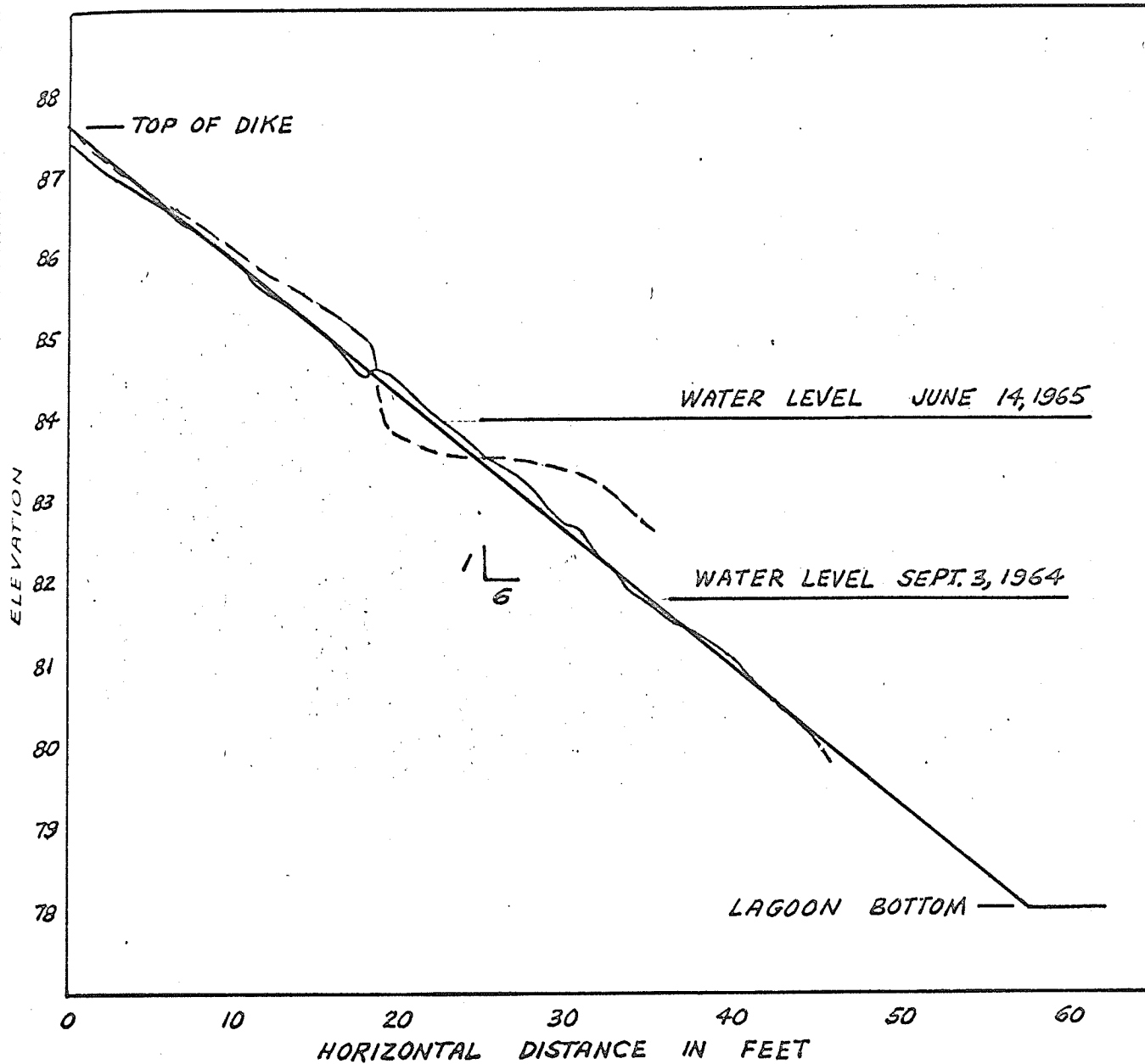
LEGEND

—— AS DESIGNED

—— SEPTEMBER 3, 1964

--- JUNE 15, 1965

<p>MASTER OF SCIENCE IN CIVIL ENGINEERING UNIVERSITY OF MANITOBA</p>	
<p>TYPICAL CROSS SECTION OF INTERIOR DIKE BANK PRIMARY CELL NUMBER 2</p>	
<p>PREPARED BY: A.G.KULUK</p>	<p>DATE: AUGUST, 1965</p>
<p>DRAWN BY: A.G.KULUK</p>	<p>FIGURE NO. 9</p>



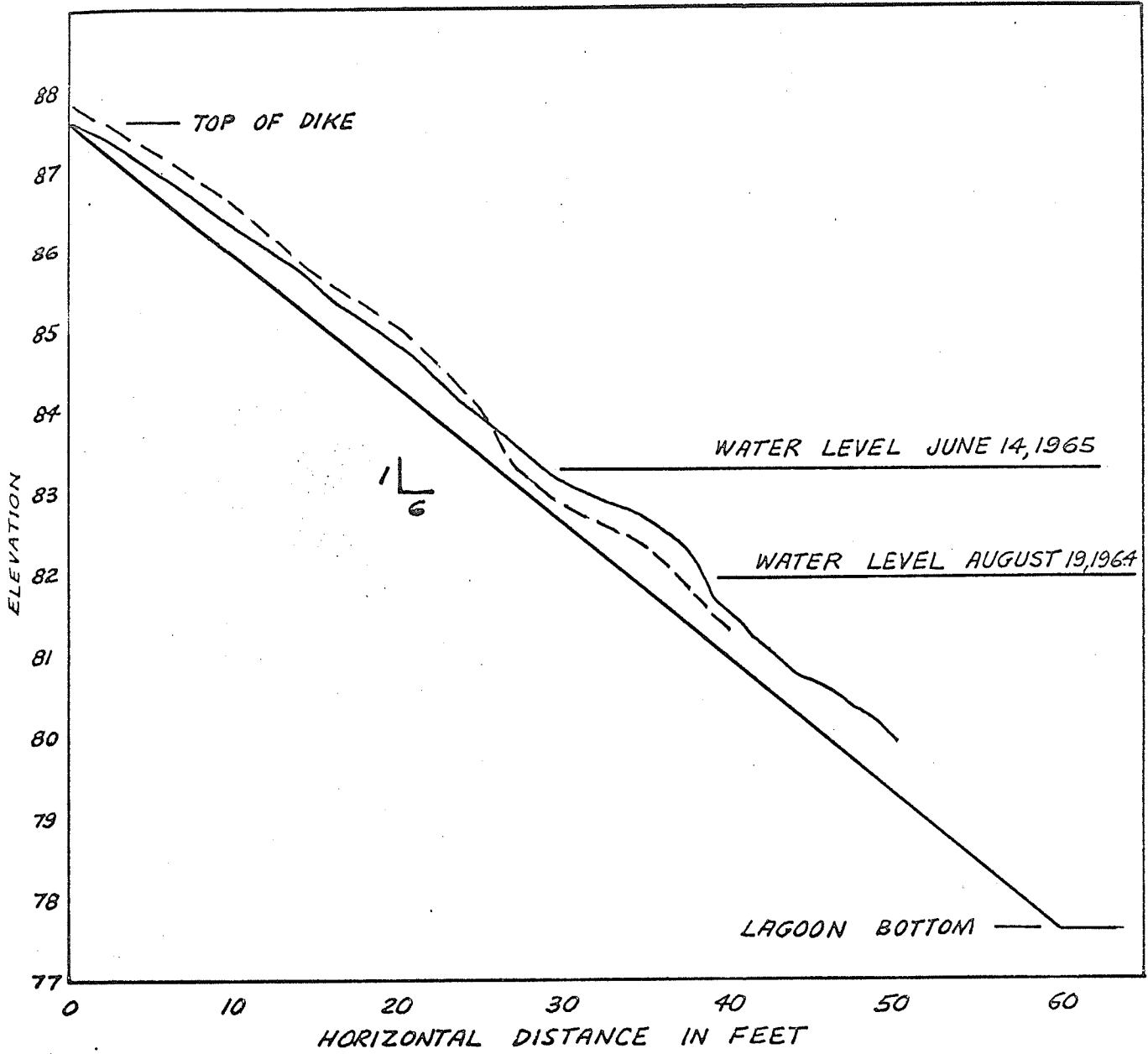
LEGEND

———— AS DESIGNED

———— SEPTEMBER 3, 1964

----- JUNE 14, 1965

MASTER OF SCIENCE IN CIVIL ENGINEERING UNIVERSITY OF MANITOBA	
TYPICAL CROSS SECTION OF INTERIOR DIKE BANK PRIMARY CELL NUMBER 3	
PREPARED BY: A.G.KULUK	DATE: AUGUST, 1965
DRAWN BY: A.G.KULUK	FIGURE NO. 10



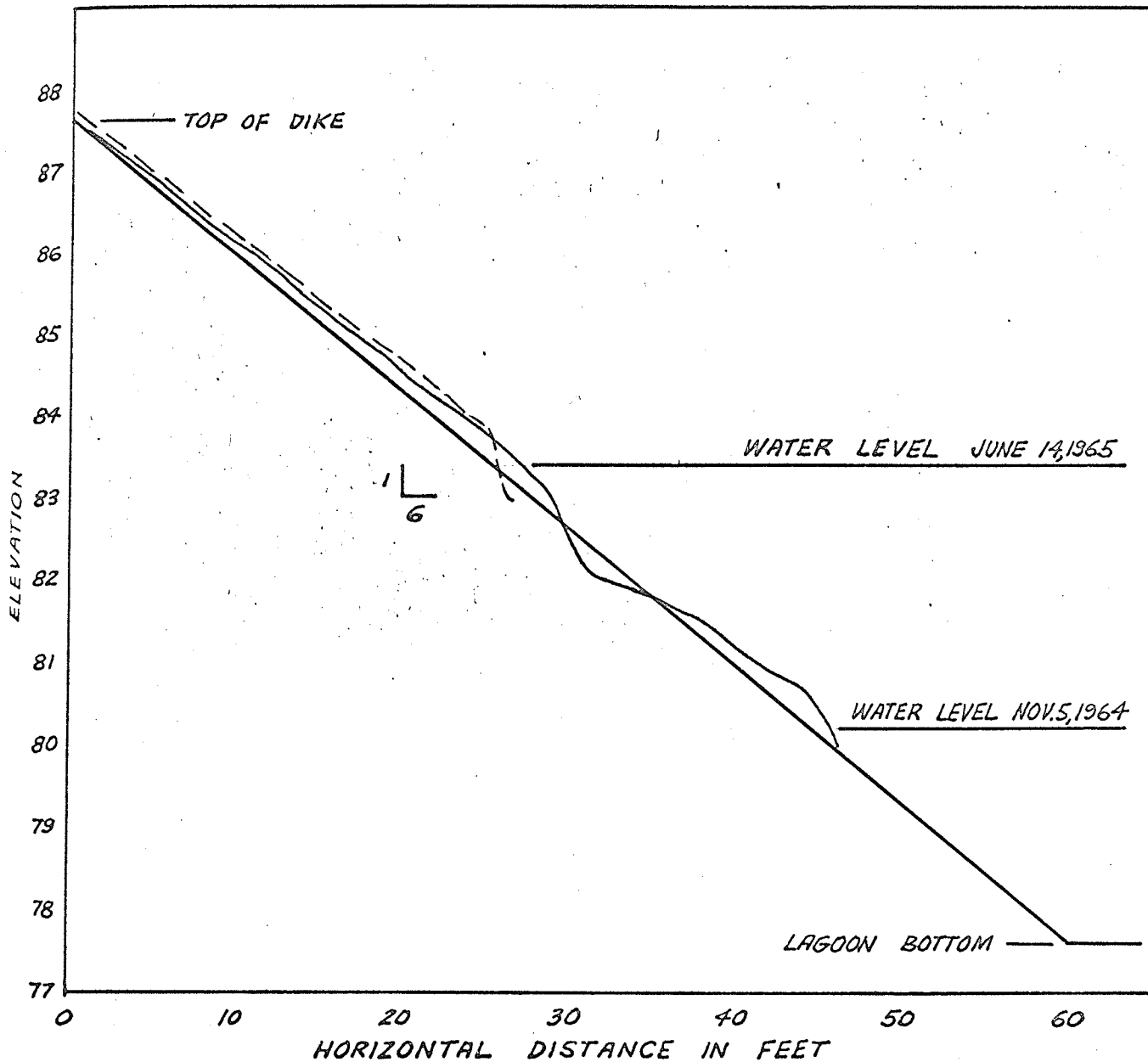
LEGEND

- AS DESIGNED
- AUGUST 19, 1964
- JUNE 14, 1965

MASTER OF SCIENCE IN CIVIL ENGINEERING
UNIVERSITY OF MANITOBA

TYPICAL CROSS SECTION
OF INTERIOR DIKE BANK
SECONDARY CELL NUMBER 1

PREPARED BY: A.G.KULUK	DATE: AUGUST, 1965
DRAWN BY: A.G.KULUK	FIGURE NO. 11



LEGEND

- AS DESIGNED
 — NOVEMBER 5, 1964
 - - - JUNE 14, 1965

MASTER OF SCIENCE IN CIVIL ENGINEERING
UNIVERSITY OF MANITOBA

TYPICAL CROSS SECTION
OF INTERIOR DIKE BANK
SECONDARY CELL NUMBER 2

PREPARED BY: A.G.KULUK

DATE: AUGUST, 1965

DRAWN BY: A.G.KULUK

FIGURE NO. 12

To emphasize the fact that the different side slopes have behaved in similar fashions, Figure 13 was prepared which compares the behaviour of a typical one on six side slope to a typical one on nine side slope.

Lagoon Loading History

Lagoon loading data was analyzed for the following periods:

Primary Cell No. 1	May 25th, 1964 to July 1st, 1965
Primary Cell No. 2	May 25th, 1964 to July 30th, 1965
Primary Cell No. 3	May 25th, 1964 to July 1st, 1965
Secondary Cell No. 1	May 25th, 1964 to July 30th, 1965
Secondary Cell No. 2	August 20th, 1964 to July 1st, 1965

The above cell loading history was plotted as shown on Figure 14.

From a study of above figures, comparing the primary cells to secondary cells, the following was deduced:

- (1) Primary cells were generally subjected to a condition of high water level for long periods. In particular, cell number 3 has had the highest water level.
- (2) Secondary cells were subjected to conditions of irregular loading similar to the teeth of a rip saw. Cell No. 2 has had a more regular pattern of loading.
- (3) Primary cells have had the highest water levels at break-up and therefore have been subjected to the greatest ice action.

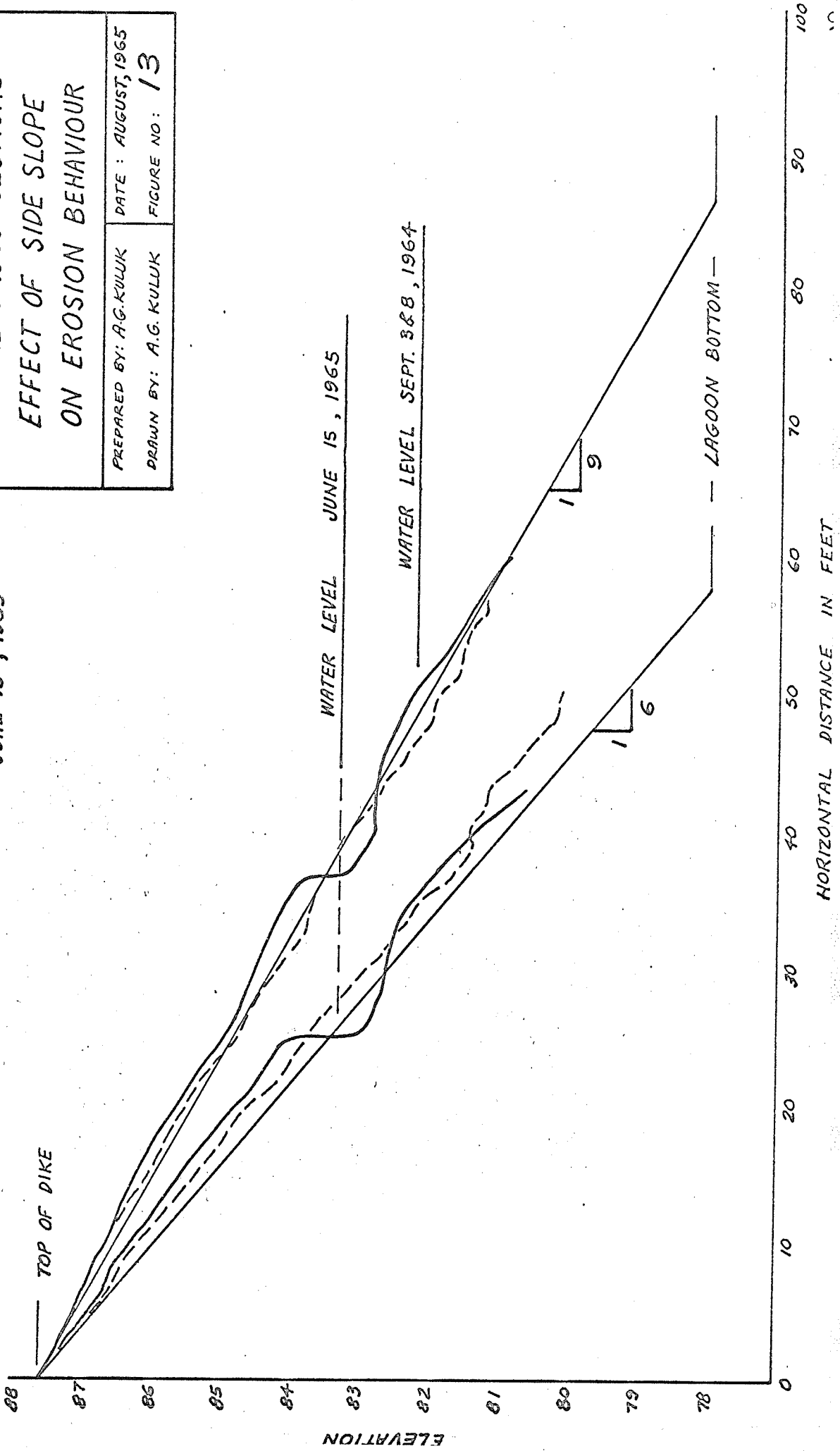
LEGEND

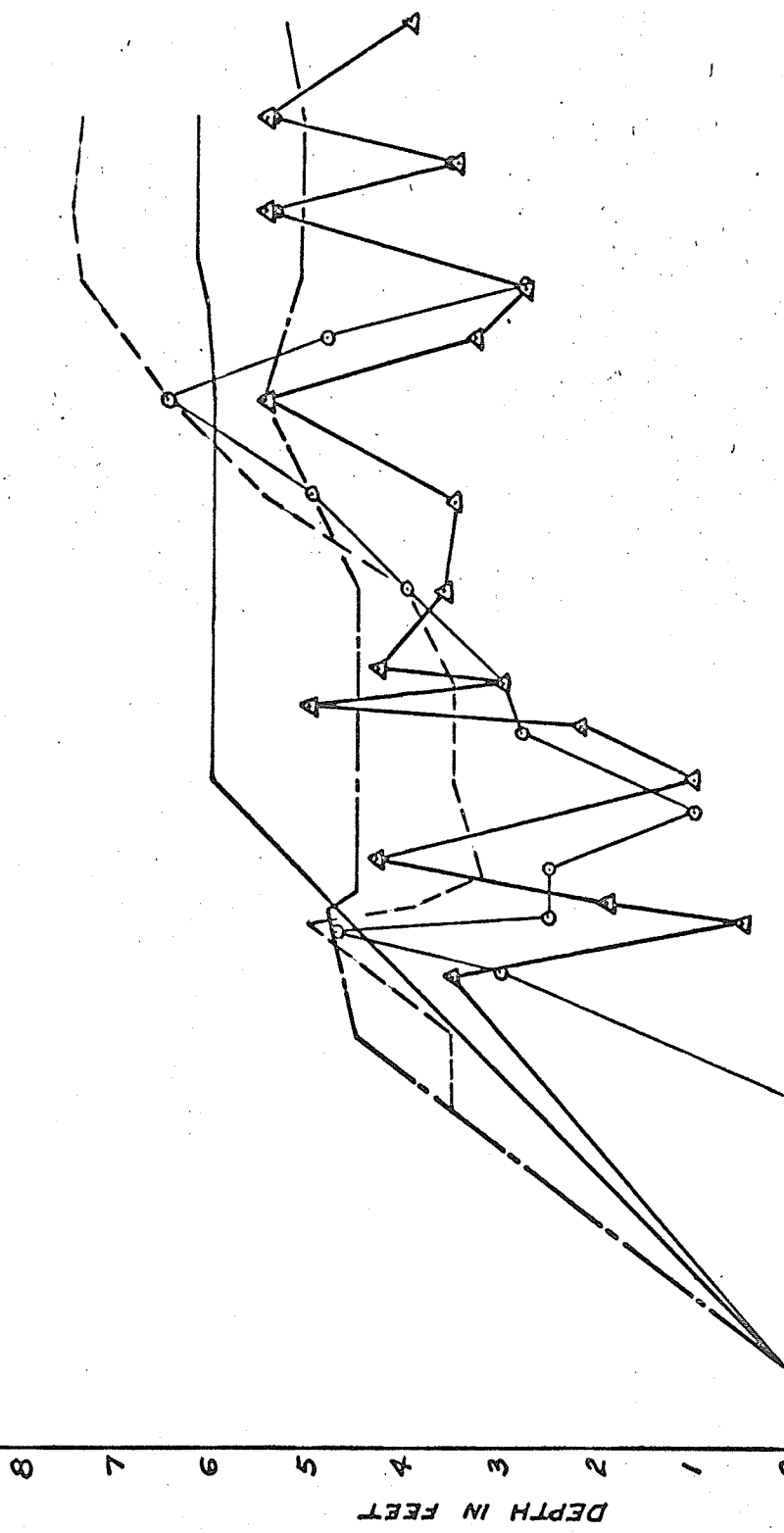
— AS DESIGNED

- - - SEPT. 3&8, 1964

— JUNE 15, 1965

MASTER OF SCIENCE IN CIVIL ENGINEERING UNIVERSITY OF MANITOBA
TYPICAL CROSS SECTIONS EFFECT OF SIDE SLOPE ON EROSION BEHAVIOUR
PREPARED BY: A.G. KULUK DRAWN BY: A.G. KULUK
DATE: AUGUST, 1965 FIGURE NO: 13





MAY JUNE JULY AUG. SEPT. OCT. NOV. DEC. JAN. FEB. MARCH APRIL MAY JUNE JULY AUG.
1964 1965

LEGEND

- PRIMARY CELL NUMBER 1
- .-.- PRIMARY CELL NUMBER 2
- PRIMARY CELL NUMBER 3
- △— SECONDARY CELL NUMBER 1
- SECONDARY CELL NUMBER 2

MASTER OF SCIENCE IN CIVIL ENGINEERING UNIVERSITY OF MANITOBA	
CHARLESWOOD SEWAGE LAGOON CELL LOADING HISTORY	
PREPARED BY: A.G.KULLUK	DATE: AUGUST, 1965
DRAWN BY: A.G.KULLUK	FIGURE NO. 14

Analysis of Lagoon Loading History and Cross Section Behaviour

Combining the lagoon loading history analysis and the cross section analysis, a study was made of the common factors involved with the aim of correlating the erosion experienced.

One factor indicated was that apparently the irregular loading of the secondary lagoons did not significantly effect the erosion experienced.

A study of the water level at ice break-up yielded:

TABLE No. 3
APRIL 1965 WATER DEPTHS

Cell	April 1965 Depth in feet
Primary Cell No. 1	7.2
Primary Cell No. 2	5.2
Primary Cell No. 3	6.1
Secondary Cell No. 1	3.1
Secondary Cell No. 2	3.6

Evidently the secondary lagoons were subjected to less ice action during break-up. Since they have had almost similar water depths with the more critical type of loading, which is the rapid drawdown condition, this factor should be considered as the major cause of the erosion at the primary cells.

Since each lagoon has had significant erosion and in particular because the erosion at spring break-up has been proven important, it was decided to implement an experimental

program of revetment works which would control or prevent this behaviour. By mutual agreement it was decided that the author would recommend the experimental program and the Waterworks and Waste Disposal Division of Metro would carry out the installation.

The following chapter will cover the details of this arrangement and outline the proposed program.

CHAPTER 11

BANK REVETMENT WORKS PROGRAM

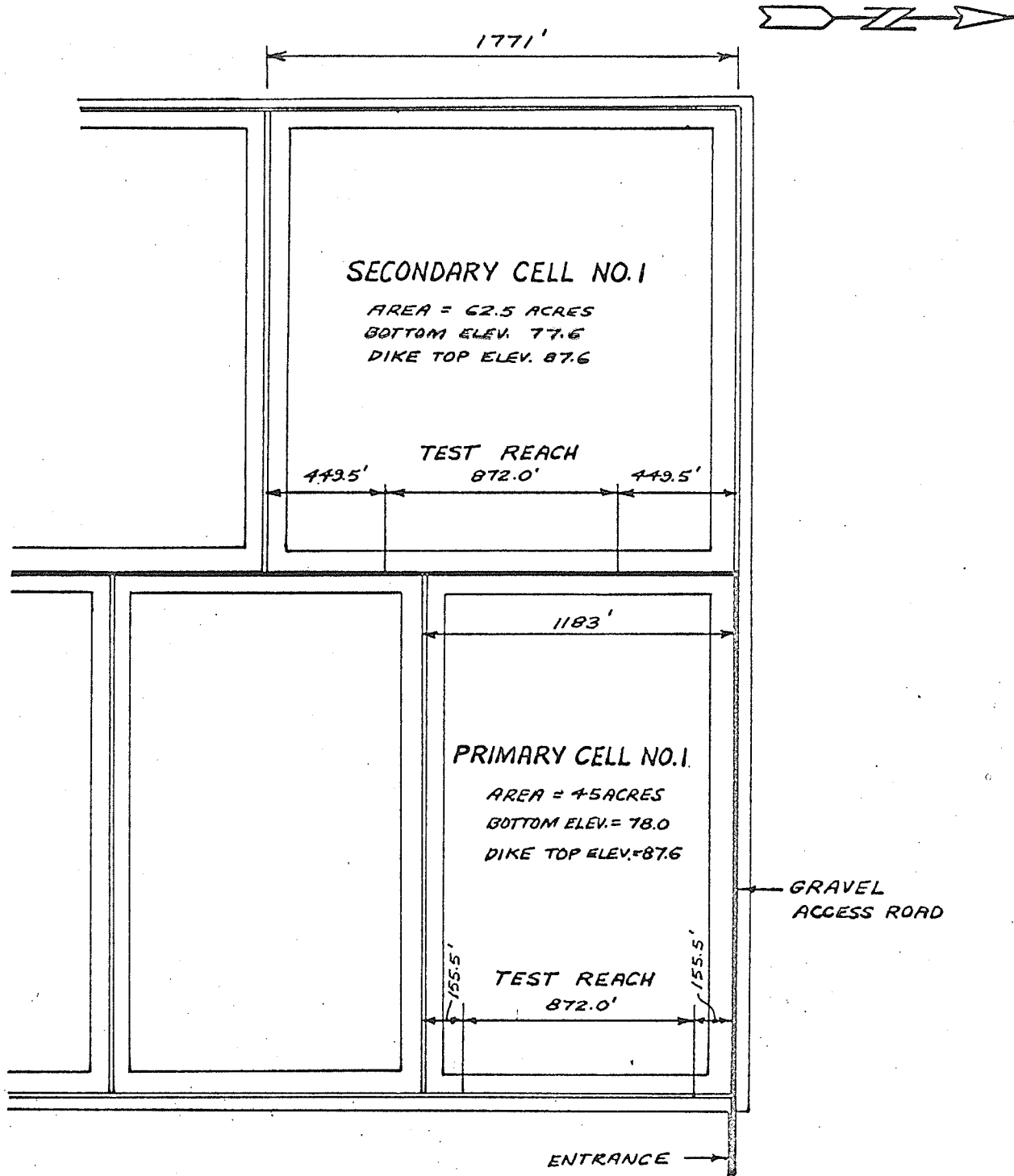
The behaviour to date does not appear to be serious enough to warrant an all out program of expensive bank erosion protection. However, the significant erosion which has occurred over the two years of operation does necessitate the investigation of inexpensive revetment works to evaluate their effect on future operation. Such an installation would serve to establish the best form of protection in addition to providing a comparative cost of the alternative schemes.

Original Revetment Works Program Proposed

The first step taken towards establishing a potential program was to determine test areas. From the study of the cell behaviour as outlined in chapter 10, it was decided that one representative primary and one representative secondary cell should be used. In selecting the cells to be used, the test area accessibility governed, and therefore primary cell number 1 and secondary cell number 1 were chosen.

The next step was to select a test section from each cell. The areas picked were such that they would have similar fetches, be aligned in the same direction, and have similar original side slopes. Figure 15 shows the test reach locations.

With the experimental sections established, the next



SECONDARY CELL NO.1

AREA = 62.5 ACRES
 BOTTOM ELEV. 77.6
 DIKE TOP ELEV. 87.6

TEST REACH

449.5' 872.0' 449.5'

PRIMARY CELL NO.1

AREA = 4.5 ACRES
 BOTTOM ELEV. = 78.0
 DIKE TOP ELEV. = 87.6

TEST REACH

155.5' 872.0' 155.5'

GRAVEL ACCESS ROAD

ENTRANCE

SCALE : 1" = 600'

MASTER OF SCIENCE IN CIVIL ENGINEERING UNIVERSITY OF MANITOBA	
CHARLESWOOD SEWAGE LAGOON TEST REACH LOCATIONS ORIGINAL PROPOSAL	
PREPARED BY: A.G.KULUK	DATE: AUGUST, 1965
DRAWN BY: A.G.KULUK	FIGURE NO. 15

phase was to set up a revetment works program to be in agreement with the physical features of the lagoon. The original program set up was based on the following considerations:

1) To establish optimum side slopes.

The performances of one on six and one on nine side slopes, which correspond to the extremes used in the lagoons, in addition to one on four side slopes, were to be studied and compared. It was felt, however, that slopes flatter than one on six were not required and the aspect of eliminating it from the test program should be considered.

2) The general scheme of test sections.

A series of test reaches with buffer zones or transitions were decided upon. These are to provide a separation of individual test sections and also to give a running comparison with the installed sections.

3) The design water level was set at 6 foot depth for each cell.

The original revetment works program was laid out as shown on Figure No. 15. The suggested test sections of revetment works were chosen after an intensive study of available literature.

Type A - Rebuilt Clay Slope

The purpose of this section was to provide a natural clay slope cross section that could be used to provide a measure of comparison to the installed sections.

Type B - Rebuilt Clay Slope with Chained and Anchored Telephone Poles Installed at Toe of Slope (Figure 16)

In various areas throughout Manitoba chained telephone poles or logs have been installed to protect river banks from wave action. Generally the arrangement consists of a series of logs chained together forming a floating boom which is anchored at intervals, by piles or cabled to the bank.

Type C - Granular Layer (Figures 17 and 18)

From experience with numerous reported installations, the general procedure is to place about a six inch blanket of graded filter material on a dressed slope, followed by about a twelve inch layer of quarry stone. Both blankets extend from the design water surface to the toe of the slope.

Of special interest are the reported surface wave erosion tests performed in a hydraulic flume at the Bureau of Reclamation in Denver, Colorado, on materials shipped from Yakima Project, Washington.¹ The results of those experiments established the most stable cover blanket for the above given location. Because the physical features involved correspond closely to those encountered on this project, the curve was reproduced as shown on Figure 18, and it was suggested that a gradation be obtained as close as possible.

Type D - Granular Layer and Wire Mesh (Figure 19)

From a study of numerous reported installations, the

1. E.J. Carlson, "Gravel Blanket Required to Prevent Wave Erosion," (Proc. A.S.C.E., Volume 85, No. HY5, Paper No. 2021, May, 1959)

ORIGINAL PROPOSAL
TYPE B

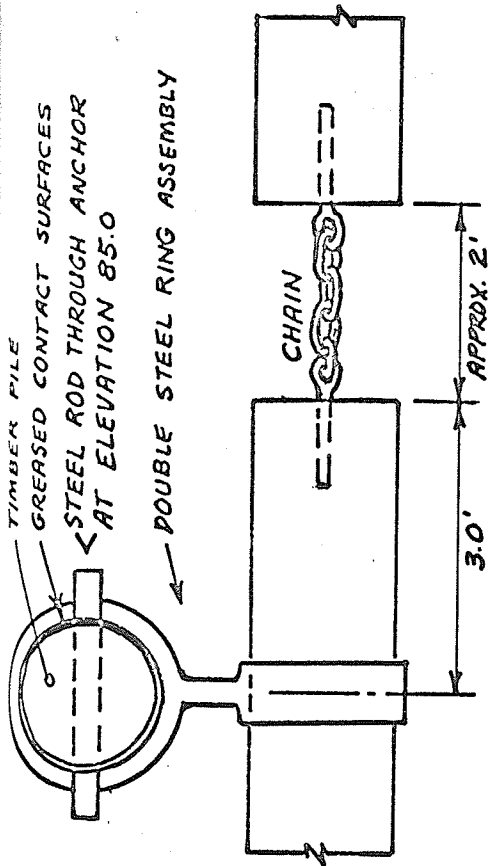
BANK REVETMENT WORKS

PREPARED BY: A.G. KULUK

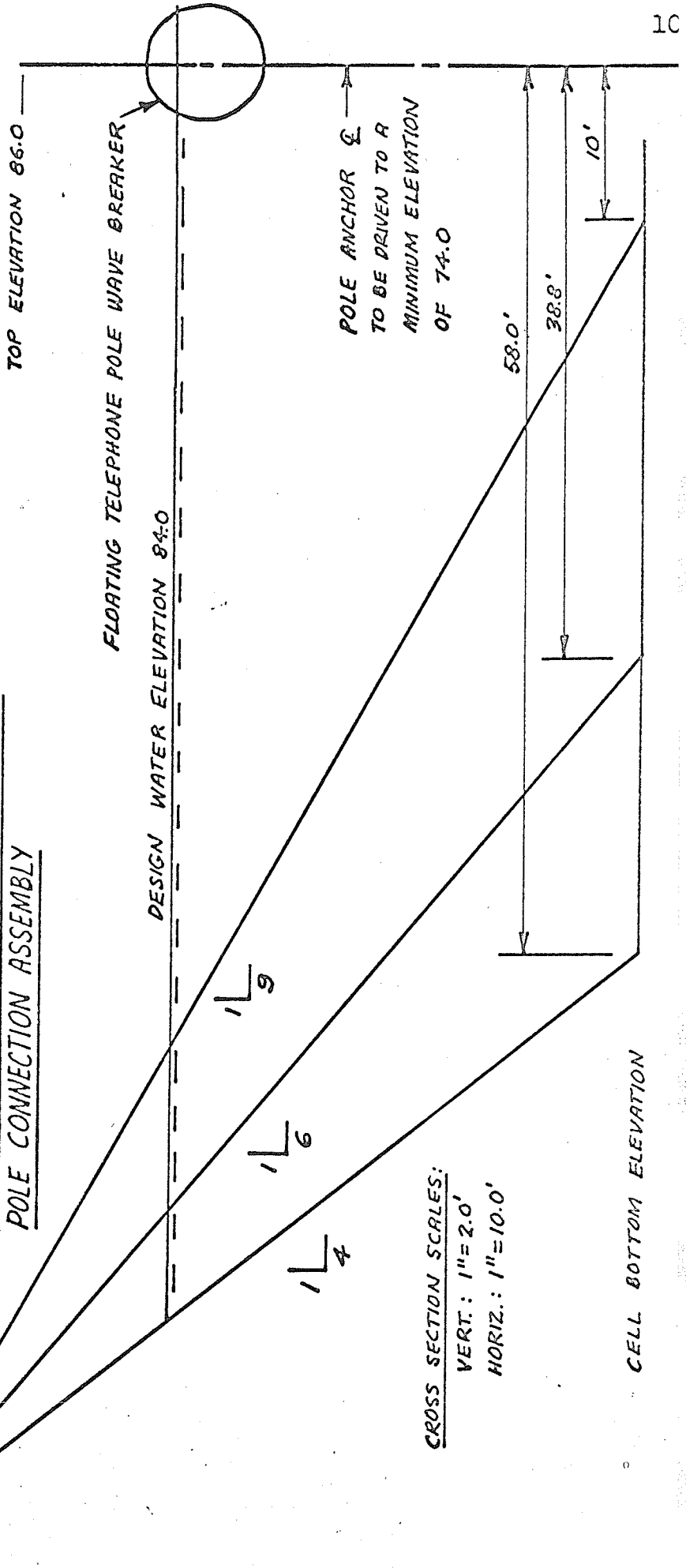
DATE: SEPT., 1965

DRAWN BY: A.G. KULUK

FIGURE NO: 16



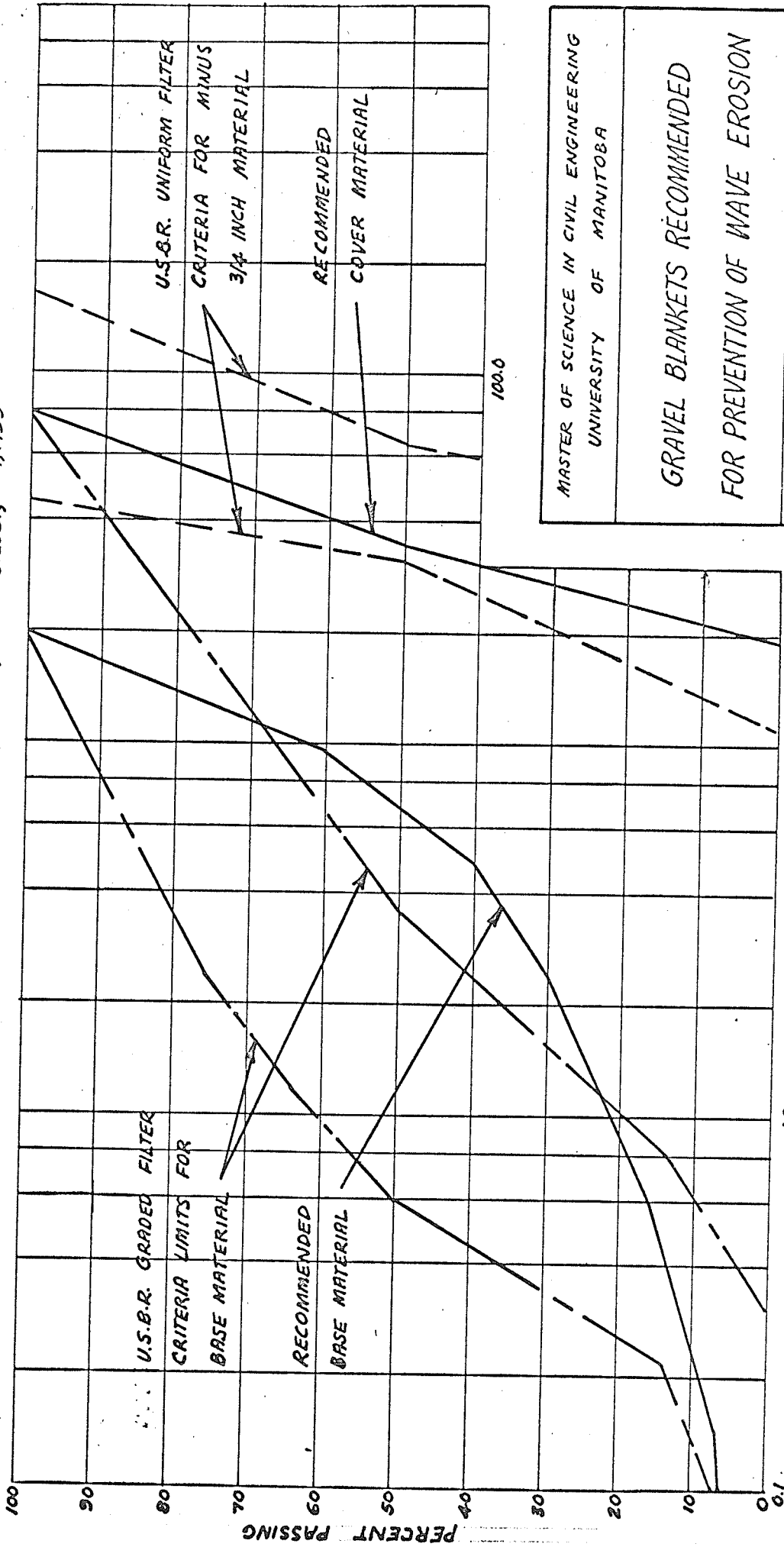
DETAIL PLAN OF PILE ANCHOR TO TELEPHONE
POLE CONNECTION ASSEMBLY



CROSS SECTION SCALES:
VERT.: 1"=2.0'
HORIZ.: 1"=10.0'

CELL BOTTOM ELEVATION

THIS GRAPH WAS REPRODUCED FROM FIG. 18, PAGE 135 OF
 GRAVEL BLANKET REQUIRED TO PREVENT WAVE EROSION, BY:
 E.-J. CARLSON, PROC. A.S.C.E., VOL. 85, NO. HY5, PAPER NO. 2021, MAY, 1959

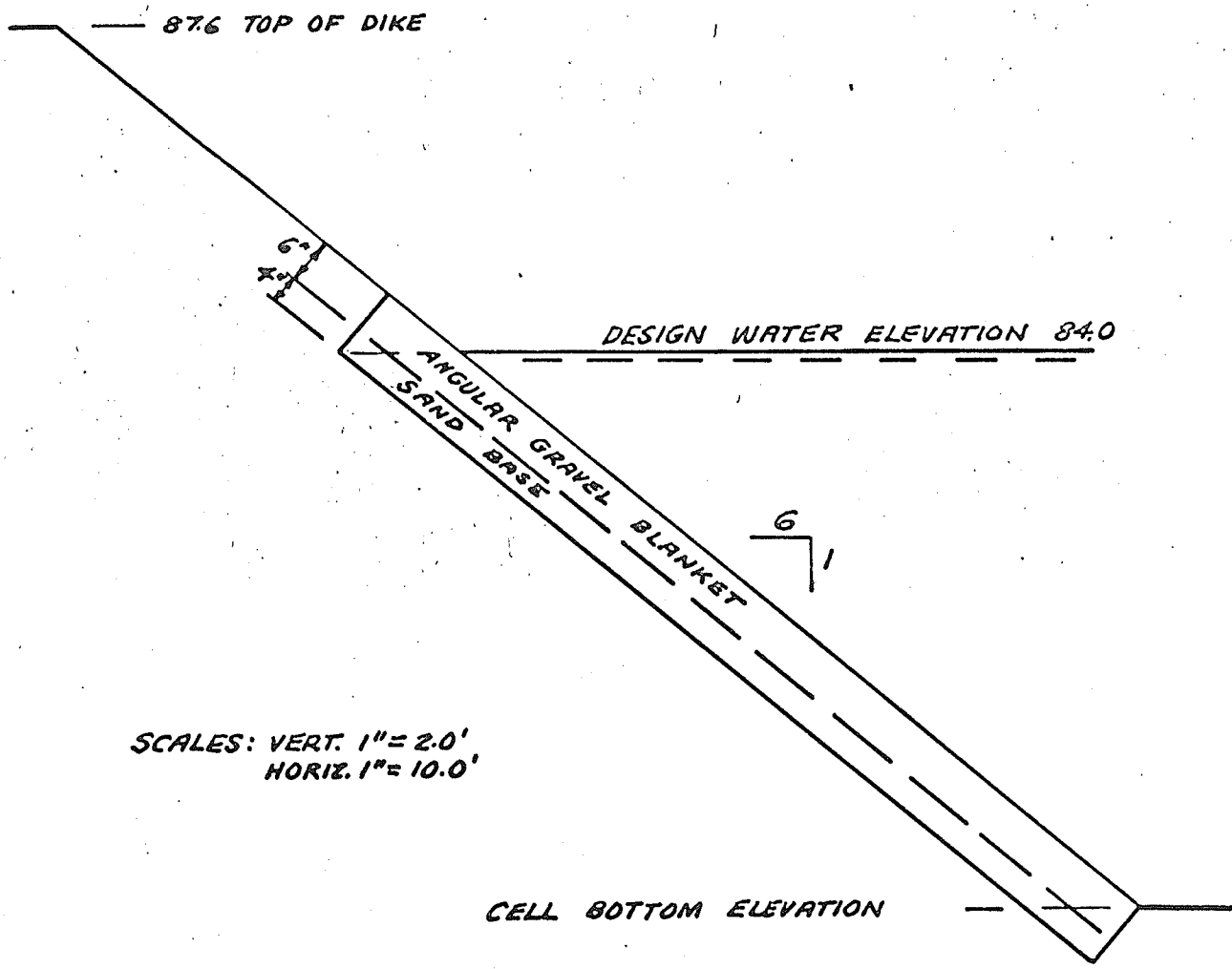


MASTER OF SCIENCE IN CIVIL ENGINEERING
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**GRAVEL BLANKETS RECOMMENDED
 FOR PREVENTION OF WAVE EROSION**

PREPARED BY: A.G. KULUK DATE: SEPT, 1965
 DRAWN BY: A.G. KULUK FIGURE NO. 17

DIAMETER OF PARTICLE IN MILLIMETERS



SCALES: VERT. 1" = 2.0'
 HORIZ. 1" = 10.0'

NOTES :

1. FOR OPTIMUM GRADATION
SEE FIGURE NO. 17
2. BASE AND BLANKET TO BE
PARALLEL TO EACH SLOPE

MASTER OF SCIENCE IN CIVIL ENGINEERING UNIVERSITY OF MANITOBA	
ORIGINAL PROPOSAL TYPE C BANK REVETMENT WORKS	
PREPARED BY: A.G.KULUK	DATE: SEPT., 1965
DRAWN BY: A.G.KULUK	FIGURE NO. 18

ORIGINAL PROPOSAL
TYPE E
BANK REVETMENT WORKS

PREPARED BY: A.G. KULUK DATE: SEPT, 1965
DRAWN BY: A.G. KULUK FIGURE NO. 19

NOTES:

1. SUGGESTED WOVEN WIRE MESH
HOT GALVANIZED STEEL
ASHDOWN'S STYLE 1036-33
2. WIRE MESH TO BE PLACED
PARALLEL TO EACH SLOPE.

TOP OF DIKE 876

ELEVATION 86.0

CROSS SECTION SCALES:

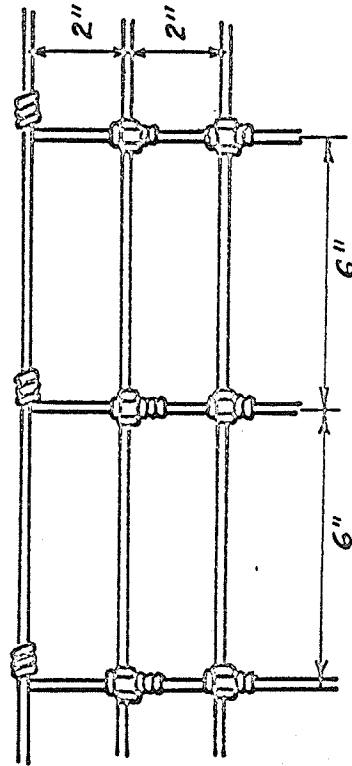
VERT. 1" = 2.0'
HORIZ. 1" = 10.0'

DESIGN WATER ELEVATION 84.0

STEEL TIES SPACED AT 10' ALONG SLOPE & 40' TRANSVERSELY

WOVEN WIRE MESH
LAID ON SLOPE SURFACE

CELL BOTTOM



DETAIL PLAN OF WOVEN WIRE MESH

reported work done on the Russian River in California, offered the best reference.² A gravel layer was placed on a dressed slope and then wire mesh was laid over this blanket and anchored at the top and bottom. Under operation it was noted that the wire mesh was not in complete contact with the gravel blanket and this was permitting movement of some of the gravel down the slope. This condition was later remedied by driving hooked rods at intervals through the mesh to create closer contact with the gravel blanket.

Type E - Wire Mesh over Rebuilt Clay Slope

Because installations of wire mesh on original clay banks have been employed successfully in various areas, this also became an experimental section.

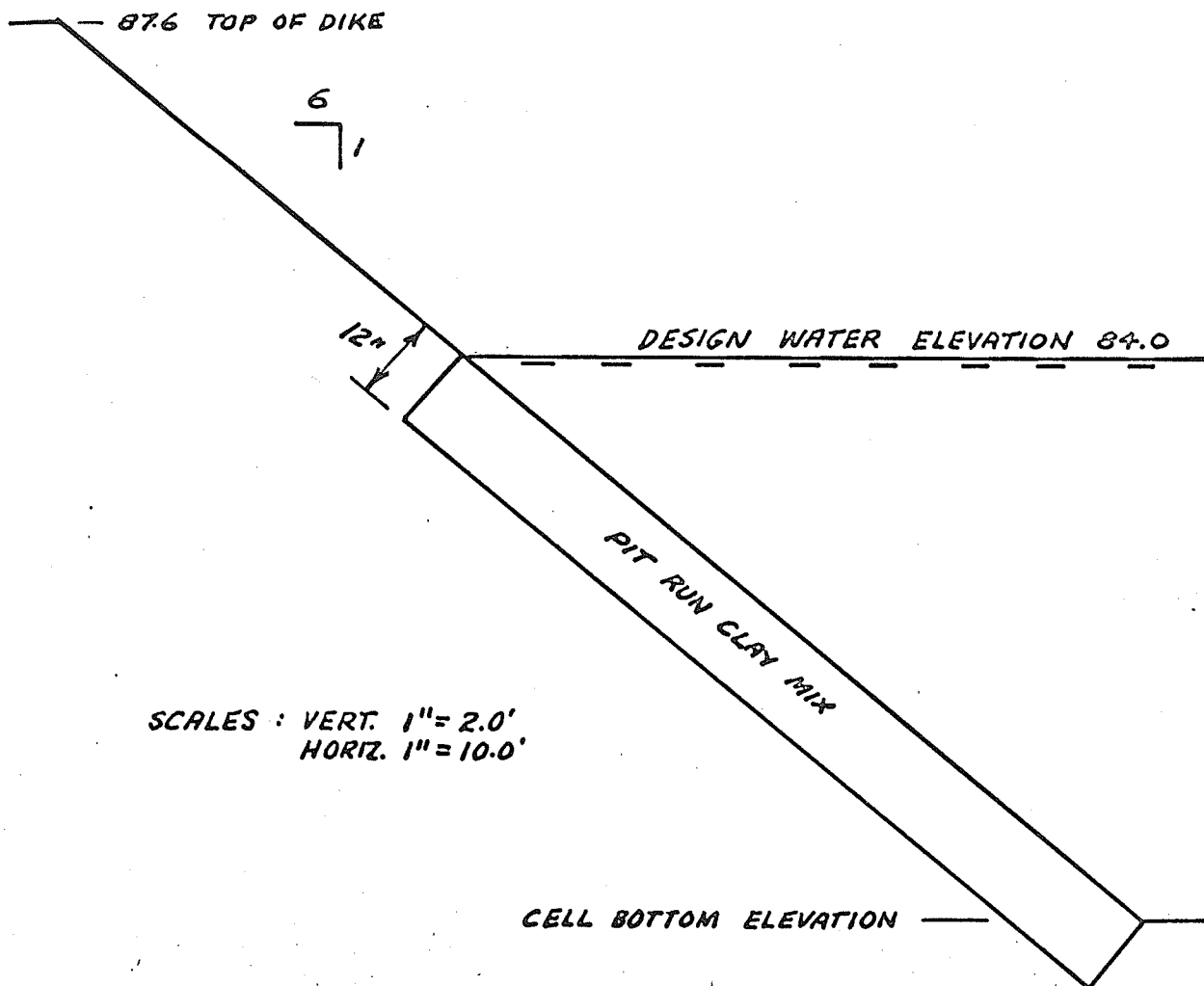
Type F - Pit Run Clay Mix (Figure 20)

Pure clay shrinks and cracks when it is dry. A small amount of sand and gravel in the clay can substantially reduce these occurrences without destroying the toughness of the clay. Since this material is readily available in Metropolitan Winnipeg, this scheme is very practical.

Type G - Riprap Placed over Operating Range (Figure 21)

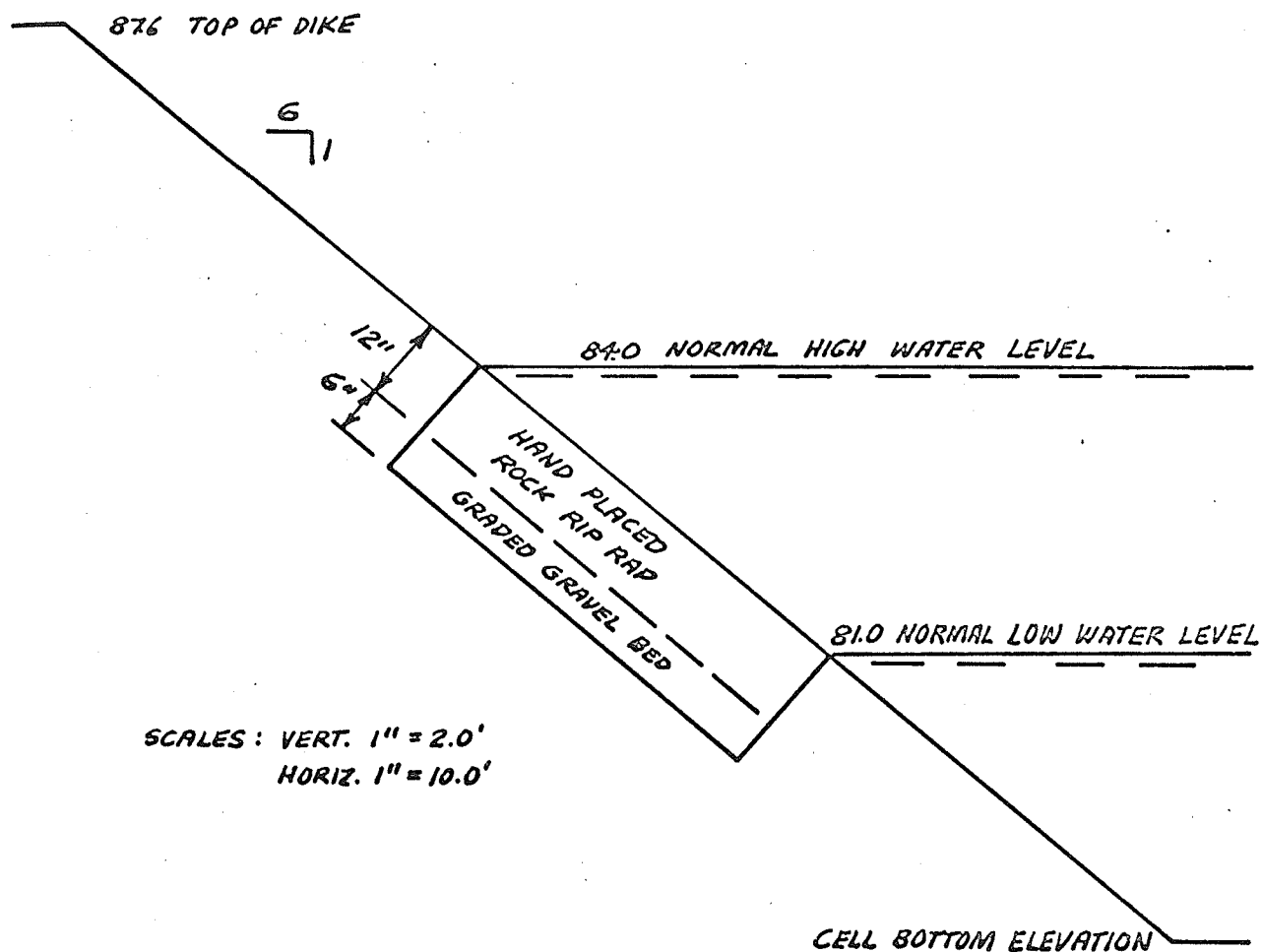
Experience with local projects indicates that rock riprap should vary in size from 6 inches to 12 inches in a layer

2. I.H. Steinberg, "Russian River Channel Works," (Proc. A.S.C.E., Vol. 86, No. WW4, Paper No. 2647, November, 1960), p. 31.



SCALES : VERT. 1" = 2.0'
 HORIZ. 1" = 10.0'

MASTER OF SCIENCE IN CIVIL ENGINEERING UNIVERSITY OF MANITOBA	
ORIGINAL PROPOSAL TYPE F BANK REVETMENT WORKS	
PREPARED BY: A.G. KULUK	DATE: SEPT., 1965
DRAWN BY: A.G. KULUK	FIGURE NO. 20



NOTES :

1. ROCK RIP RAP SHOULD VARY IN SIZE FROM 3 IN. TO 9 IN. OVER A LAYER 12 IN. THICK AND ARRANGED SO THAT THERE ARE NO LARGE VOIDS EXPOSED BETWEEN THE ROCKS.
2. GRAVEL AND RIP RAP LAYERS TO BE PARALLEL TO EACH SLOPE.

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ORIGINAL PROPOSAL
TYPE G
BANK REVETMENT WORKS

PREPARED BY: A.G. KULUK

DATE: SEPT., 1965

DRAWN BY: A.G. KULUK

FIGURE NO. 21

about 18 inches thick, and arranged so that there are no large voids exposed between the stones. About a 6 inch layer of coarse bedding material should be provided under the riprap; the reason being that riprap failures are generally due to erosion of the material beneath the rock.

Because the project involved is smaller in magnitude than local projects, the size of rock was reduced to be 3 to 9 inches and the layer reduced to 9 inches.

Due to the cost involved, it was decided that this approach would be feasible only if the protection would be limited to provide protection over the water level operating range.

Type H - Cellular Concrete Block Revetment

Experimental work in western New York has indicated that revetment of specially designed cellular concrete revetment has been very effective against erosive forces at the surface of banks.³ In comparison to riprap this installation has proven far superior. However, unless these blocks are mass-produced in great quantities the cost locally is prohibitive. Because the cost factor was not ascertained at the time of the program layout this remained a potential scheme.

3. D.A. Parsons and R.P. Apmann, "Cellular Concrete Block Revetment," (Proc. A.S.C.E., Volume 91, No. 582, Paper No. 4311, May, 1965), p. 27.

Modified Revetment Works Program

The original proposal as discussed previously was presented by the author at a general meeting between several representatives of the Waterworks and Waste Disposal Division of Metro and several University of Manitoba professors. As a result of the meeting the following changes were made:

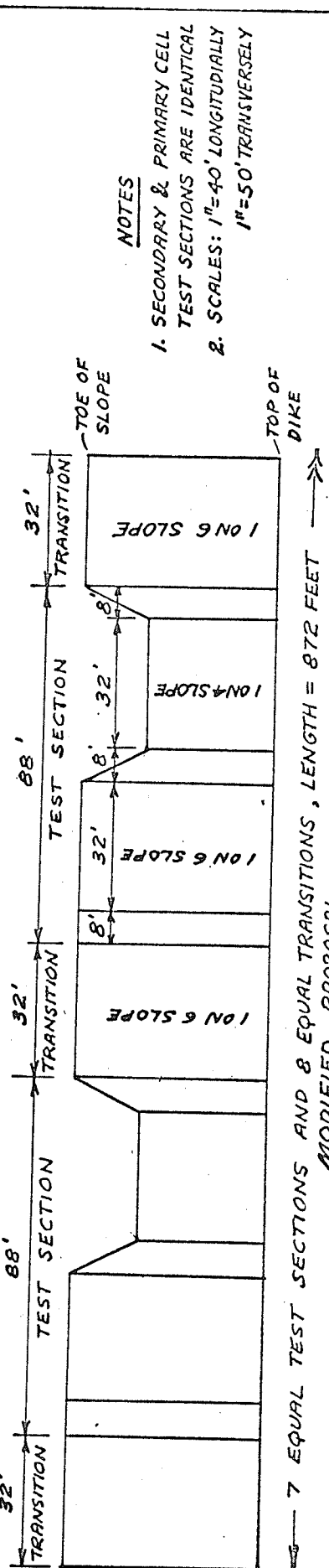
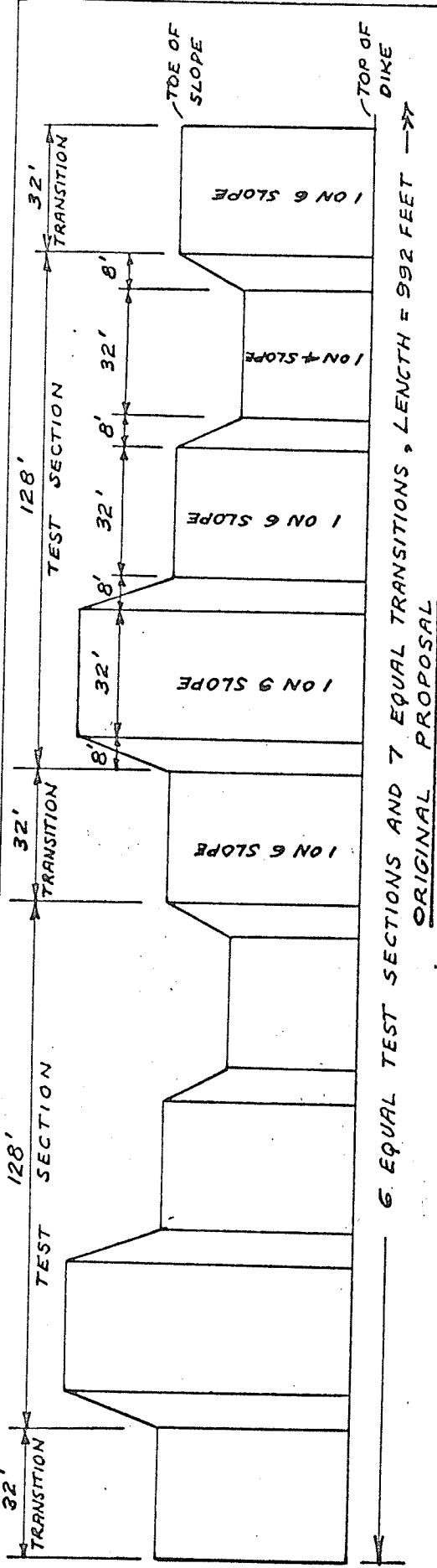
(1) The 1 on 9 slope proposal was eliminated from further consideration. In addition to the facts presented why slopes flatter than 1 on 6 were unnecessary, Metro studies indicated that any slope flatter than 1 on 6 would be uneconomical for future installations. Therefore the program was reduced to a comparison of 1 on 6 to 1 on 4 side slopes only.

(2) It was also suggested that the design water depth should be increased by 2.0 feet to 8.0 feet. This would permit lagoon operation up to maximum water level if required.

The overall layout is shown on Figure 22.

Installed Revetment Works

After the Modified Proposal was prepared and presented, numerous unexpected difficulties arose which jeopardized the installation of the program. Due to an unusually wet latter summer and fall, construction equipment and labor forces were extremely difficult to obtain. With these conditions prevailing, in September, 1965, it was decided that an installation of the complete Modified Proposal was not possible. For these reasons the installed program as shown on Figure 22 was reduced



NOTES

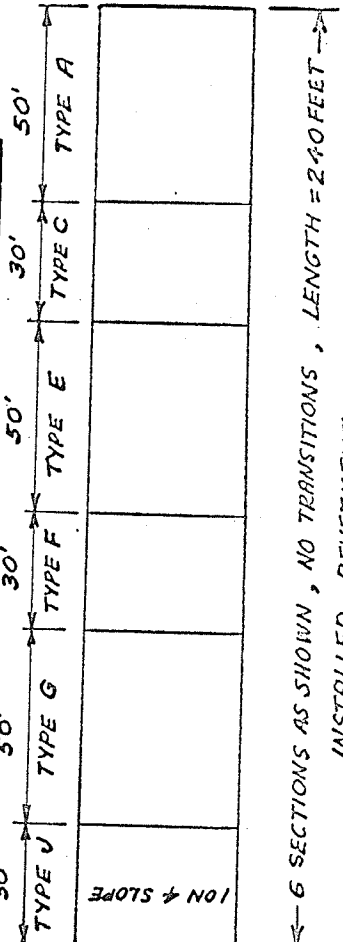
1. SECONDARY & PRIMARY CELL TEST SECTIONS ARE IDENTICAL
2. SCALES: 1" = 40' LONGITUDinally
1" = 50' TRANSVERSELY

MASTER OF SCIENCE IN CIVIL ENGINEERING
UNIVERSITY OF MANITOBA

CHARLESWOOD SEWAGE LAGOON.
DEVELOPMENT OF PROGRAM
OF BANK REVETMENT WORKS

PREPARED BY: A.G.KULUK
DATE: OCTOBER, 1965

DRAWN BY: A.G.KULUK
FIGURE NO. 22



substantially from that proposed originally.

The most important change made was that the program was limited to an installation of sections at slopes of 1 on 4. The slopes were constructed by cutting back the existing 1 on 6 slopes.

Because the lagoon bottom did not have ample time to dry properly, it was not possible to install type B, which was the telephone pole and pile installation. Other changes made are discussed below.

Figures numbered 23 to 27 illustrate the final cross sections installed. It should be noted that the alphabetic symbols used for the various types were kept the same as those used in the original proposal.

Type A - Rebuilt Clay Slope

This remained the same as was previously discussed.

Type C - Granular Layer (Figure 23)

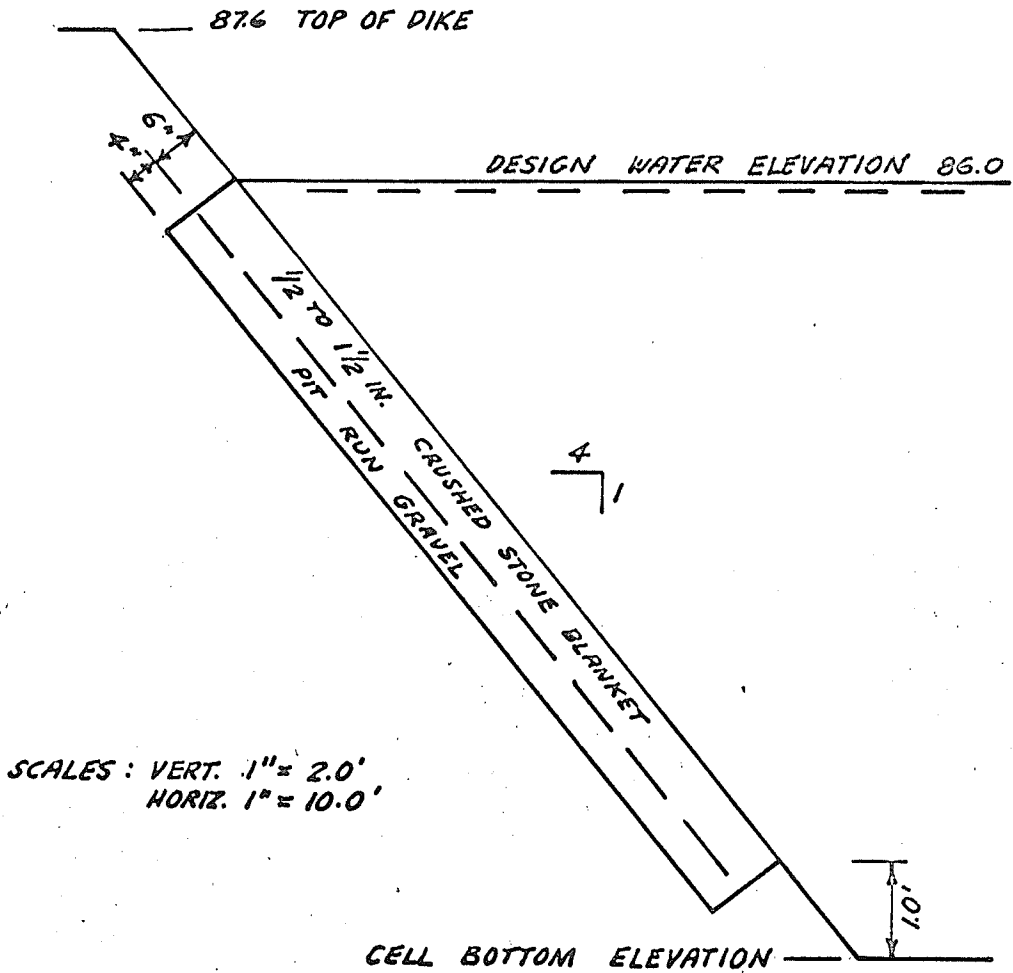
Easily acquired local materials were utilized. Pit run gravel was used as a base overlaid by $\frac{1}{2}$ to $1\frac{1}{2}$ inch diameter crushed stone.

Type E - Wire Mesh over a Rebuilt Clay Slope (Figure 24)

Because 2 inch by 6 inch mesh was not available, 4 inch square wire mesh was installed. The anchors were increased from 12 to 18 inches to insure better anchorage.

Type F - Pit Run Clay Mix (Figure 25)

This remained the same as was previously discussed.



SCALES : VERT. 1" = 2.0'
 HORIZ. 1" = 10.0'

MASTER OF SCIENCE IN CIVIL ENGINEERING UNIVERSITY OF MANITOBA	
INSTALLED PROGRAM TYPE C BANK REVETMENT WORKS	
PREPARED BY : A.G. KULUK	DATE : OCTOBER, 1965
DRAWN BY : A.G. KULUK	FIGURE NO. 23

MASTER OF SCIENCE IN CIVIL ENGINEERING
UNIVERSITY OF MANITOBA

INSTALLED PROGRAM
TYPE E
BANK REVETMENT WORKS

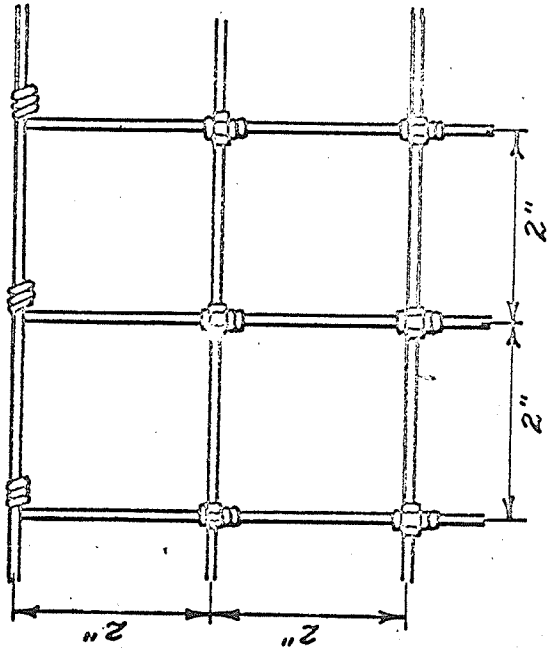
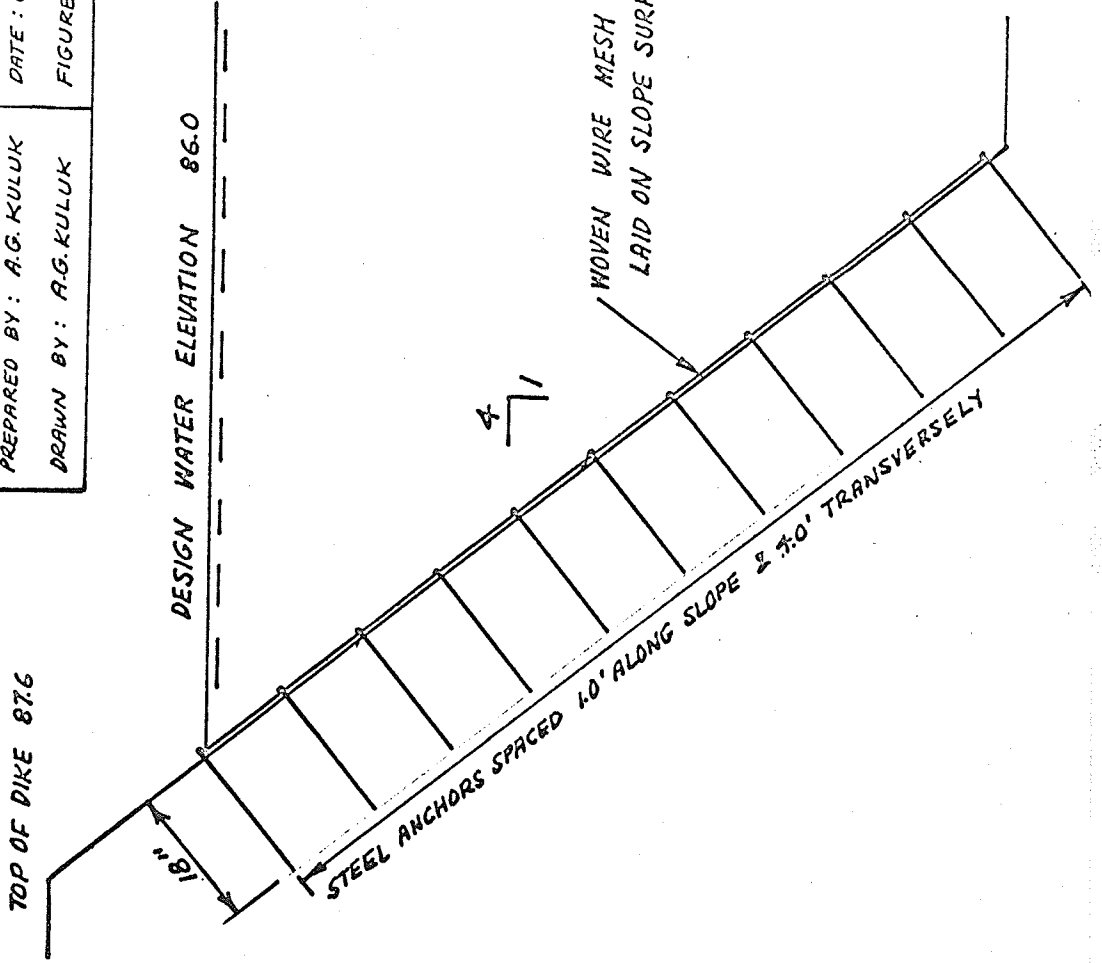
PREPARED BY: A.G. KULUK
DRAWN BY: A.G. KULUK

DATE: OCTOBER, 1965
FIGURE NO. 24

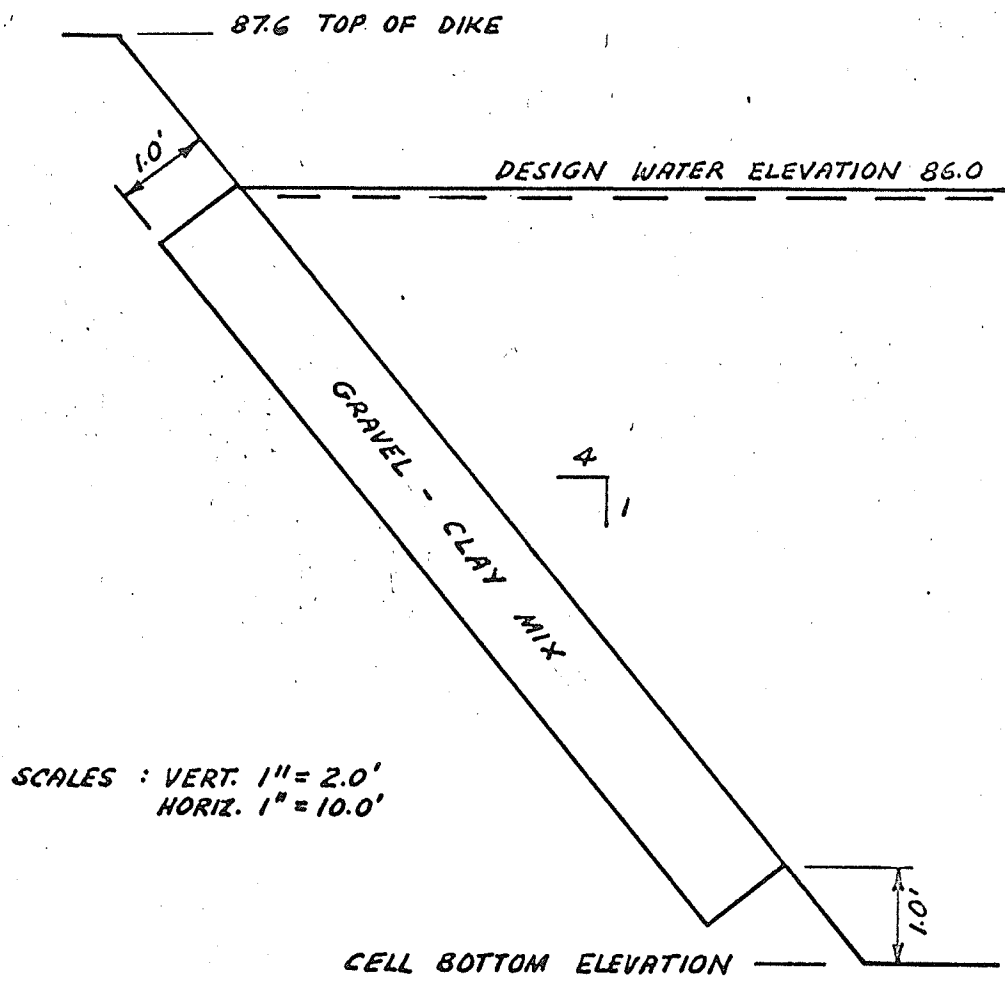
NOTES

1. WOVEN WIRE MESH USED WAS
HOT GALVANIZED STEEL
ASHBURN'S STYLE 1036-33

SCALES: VERT. 1"=2.0'
HORIZ. 1"=10.0'



DETAIL PLAN OF WOVEN WIRE MESH



MASTER OF SCIENCE IN CIVIL ENGINEERING UNIVERSITY OF MANITOBA	
INSTALLED PROGRAM TYPE F BANK REVETMENT WORKS	
PREPARED BY: A.G.KULUK	DATE: OCTOBER, 1965
DRAWN BY: A.G.KULUK	FIGURE NO. 25

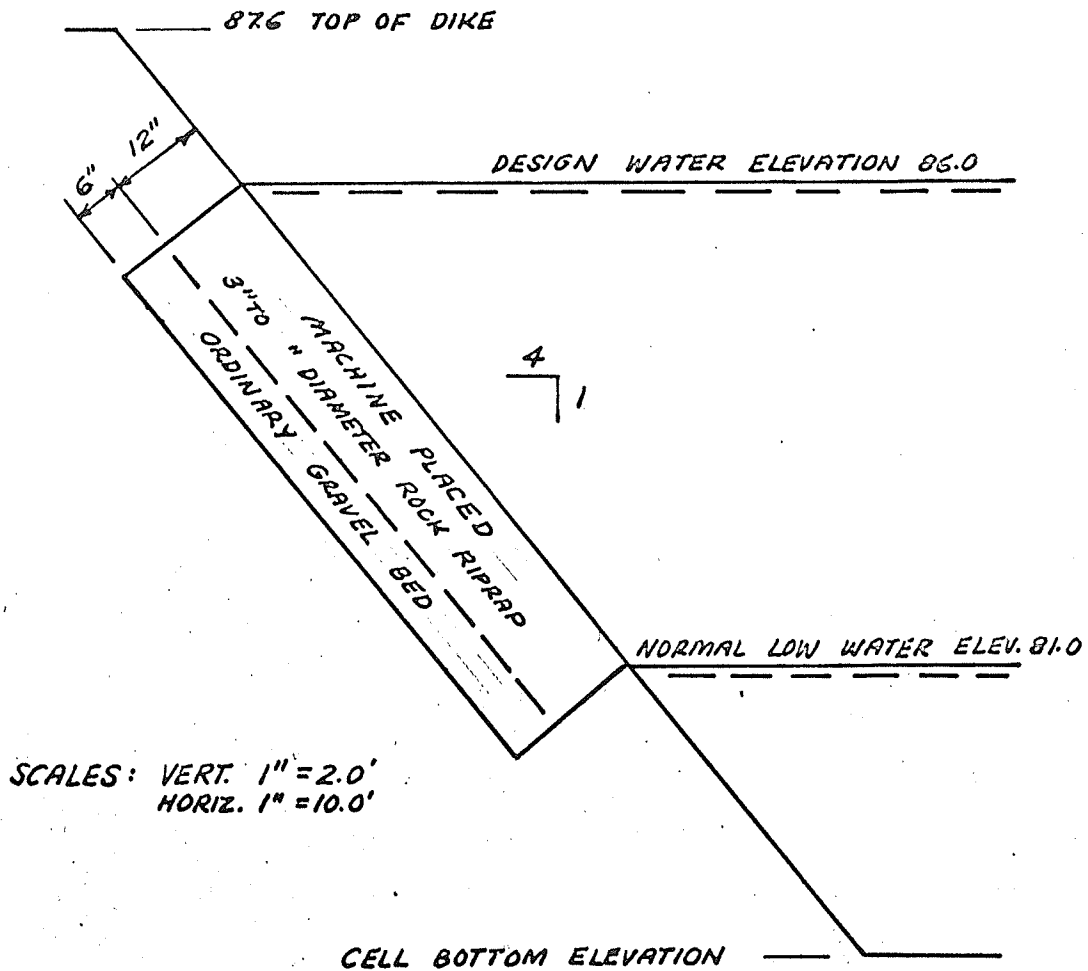
Type C - Riprap Placed over Operating Range (Figure 26)

Again easily acquired local materials were utilized. The bed was composed of ordinary gravel bed and overlaid by machine placed riprap, varying in diameter from three to six inches.

Type D - Unshrinkable Fill (Figure 27)

This test section was not suggested by the author because it was felt that the cost would be prohibitive. It was installed as a result of work by Metro Waterworks and Waste Disposal engineers. The author would like to point out that a superior installation would have been obtained if a granular base had been utilized.

The unshrinkable fill was transit mixed and contained about $\frac{1}{2}$ bag of cement per cubic yard. The concrete was simply dumped on the prepared clay slope and leveled by a small scraper.



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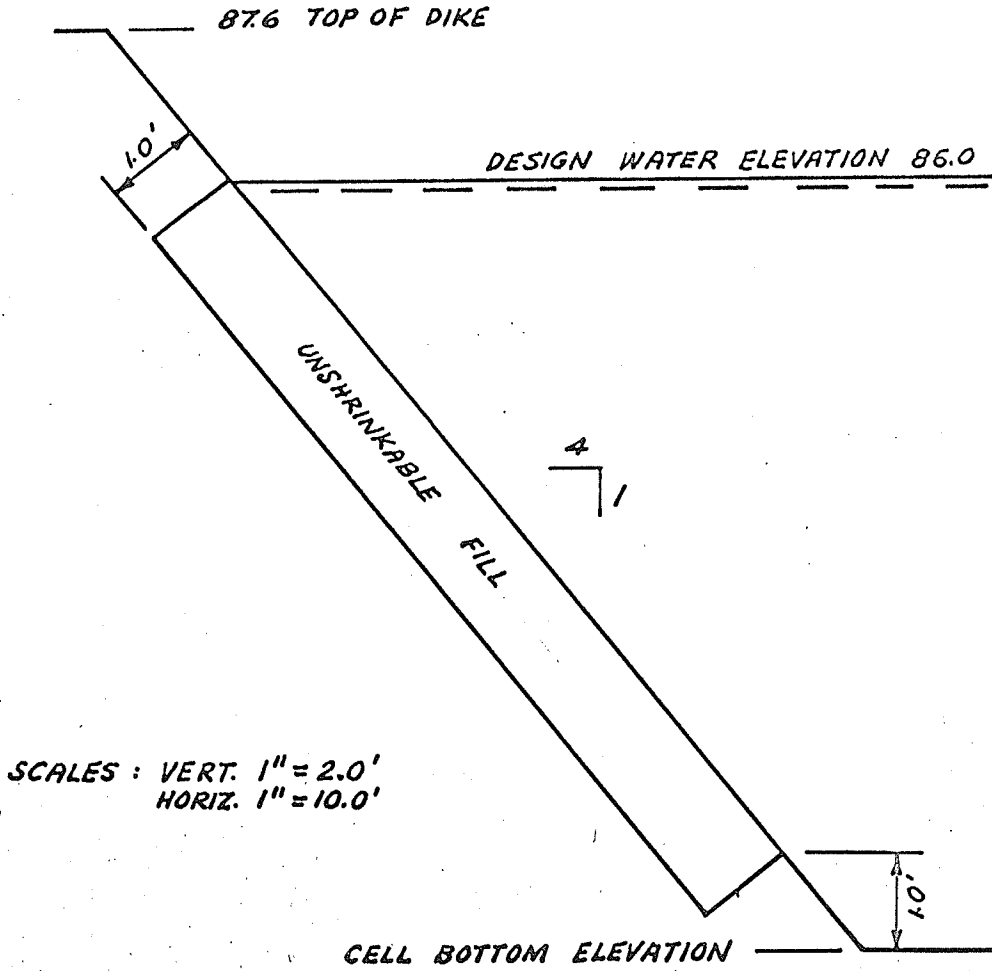
INSTALLED PROGRAM
TYPE G
BANK REVETMENT WORKS

PREPARED BY: A.G. KULUK

DATE: OCTOBER, 1965

DRAWN BY: A.G. KULUK

FIGURE NO. 26



MASTER OF SCIENCE IN CIVIL ENGINEERING UNIVERSITY OF MANITOBA	
INSTALLED PROGRAM TYPE J BANK REVETMENT WORKS	
PREPARED BY: A.G. KULUK	DATE: OCTOBER, 1965
DRAWN BY: A.G. KULUK	FIGURE NO. 27

CHAPTER 12

PRELIMINARY OBSERVATIONS ON BANK REVETMENT BEHAVIOUR

The final stage of research work undertaken by the author was a preliminary evaluation of the behaviour of the installed revetment works at the Primary No. 1 and Secondary No. 1 Cells as were previously described in Chapter 11. The purpose of this work was to follow up on the test sections designed and which were installed by the Waterworks and Waste Disposal Division of the Metropolitan Corporation of Greater Winnipeg to establish the revetment performances after the spring and summer operation with a view towards providing recommendations.

Due to time limitations, the work done was based on visual and photographic evaluations. Due to adverse conditions during and following construction, no useful photographs were obtained at that time. In subsequent work to be carried out by others, field surveys will be utilized to further evaluate the performance.

From the observations made in the spring of 1966, when the cells were relatively empty, it was concluded that both of the test sites were in similar condition so that their behaviour could be reduced to a discussion dealing with each revetment type. Whenever either cell experienced unusual behaviour, a note to that effect is made.

Observations - May, 1966

Photographs are used to supplement the observations made. The revetment schemes are discussed beginning at the north end and progress consecutively southward.

Type a - Rebuilt Clay Slope

Minor erosion was experienced from rainfall runoff.

Type C - Granular Layer

This was in excellent condition.

Type E - Wire Mesh Over Rebuilt Clay Slopes

The wire mesh did not appear to be fully effective as in numerous areas the clay was scoured out from under the wire mesh as shown in Photograph number 5.

Type F - Pit Run Clay Mix

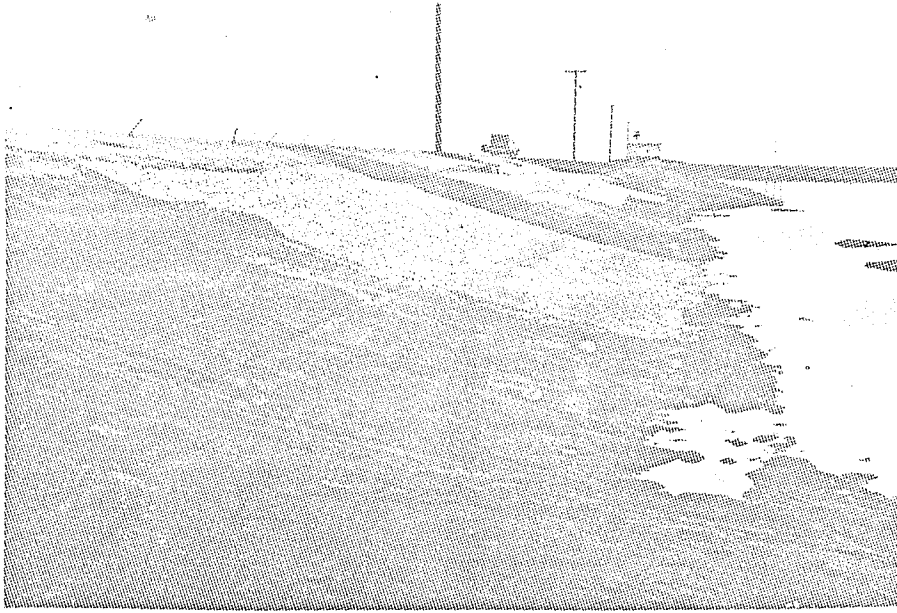
Minor erosion was experienced from rainfall runoff.

Type G - Rock Rip Rap Over Operating Range

In excellent condition, except that in Primary Cell No.1, where the water depth was greater, vegetation could be seen growing in the spaces between the rocks.

Type J - Unshrinkable Fill

The surface of the fill was loose and the layer did not appear to be integral. Photographs numbered 2 and 4 illustrate these points.



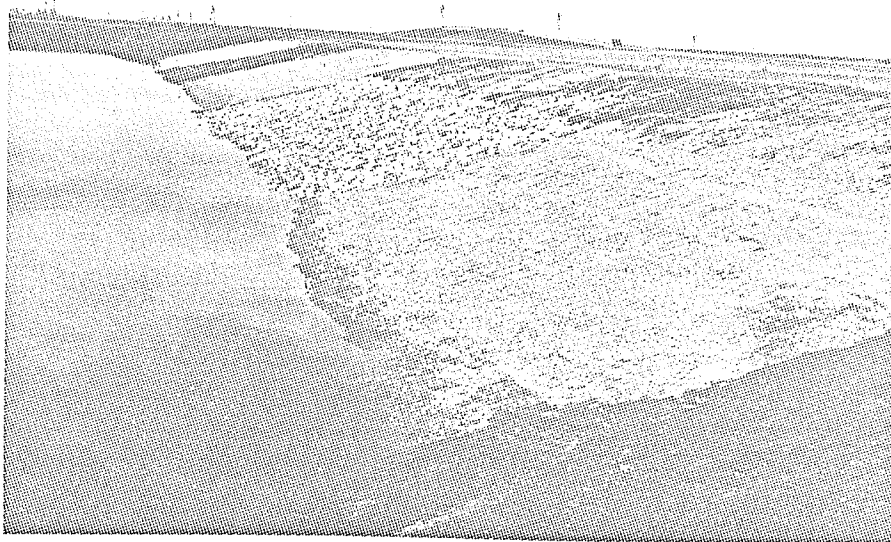
Photograph No. 1 - At North End of Secondary
Cell No.1 Looking Southward.



Photograph No. 2 - At South End of Secondary
Cell No.1 Looking Northward.



Photograph No.3 - At North End of Primary
Cell No.1 Looking Southward.



Photograph No.4 - At South End of Primary Cell
No.1 Looking Northward.



Photograph No. 5 - Wire Mesh Over Rebuilt Clay Slope.



Photograph No. 6 - Pit Run Clay Mix.

Conclusions.

Little could be concluded in the spring of 1966. It appeared that the granular layer and rock rip rap were extremely stable and erosion resistant as had been anticipated. The wire mesh over the clay bank did not seem very effective. The remaining sections had all experienced some degree of erosion. The following mid summer observations will provide more conclusive results.

Final Observations July, 1966

The behaviour of the revetment works was analyzed with the cells loaded. Photographs were taken towards the end of July to supplement the discussion. The revetment schemes are discussed beginning at the north end and progress consecutively southward.

Type A - Rebuilt Clay Slope

Serious erosion due to wave action at the water level has been experienced. The extent of erosion experienced over the short period of time that these slopes have been in operation indicates that the 1 on 4 slope is more erosive over a short term period than flatter slopes such as 1 on 6 to 1 on 9. Whether or not over a long term period the net effect would be significantly different can not be ascertained at this time.

Type C - Granular Layer

The degree of erosion experienced by this revetment type surprised the author as it was not anticipated. Photograph number 10 illustrates the step-like erosion effect as compared to the more vertical face formed by erosion at the rebuilt clay slope. It was noted that the granular layer eroded to a greater extent at the secondary lagoon.

Type E - Wire Mesh Over Rebuilt Clay Slope

This has been considerably more effective than early indications gave. As photographs numbered 7 to 9 indicate, it has been more effective than both the granular layer and pit-run clay mix revetments. Therefore, it is felt that further application of this principle should be utilized.

Type F - Pit Run Clay Mix

As shown on photograph number 11, this type has experienced heavy erosion and does not appear to have any merit of further consideration.

Type G - Rock Rip Rap Over Operating Range

This has remained in excellent condition as attested to by photographs 7, 11 and 12. The one disadvantage appears to be that it is conducive to emergent vegetation.

Conclusions

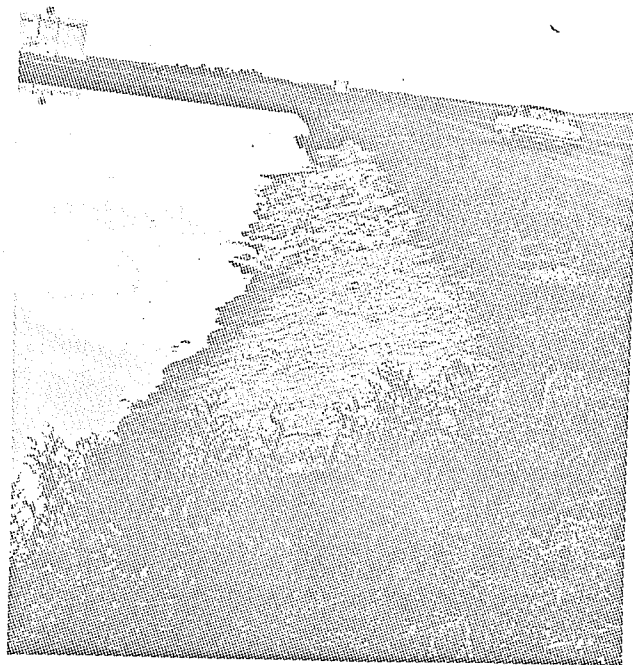
Without entering into economical and sanitary aspects,



Photograph No. 7 - At North End
of Secondary Cell No. 1 looking
Southward.



Photograph No. 8 - At South End
of Secondary Cell No. 1 looking
Northward.



Photograph No. 9 - At South End
of Primary Cell No. 1 looking
Northward.



Photograph No.10 - Granular Layer.
Note: Vertical Face Eroded at Rebuilt Clay
Slope Just North of the Granular
Layer.



Photograph No.11 - Pit Run Clay Mix.
Note: Wire Mesh Over Clay Slope Just North of
the Pit Run Clay Mix.



Photograph No. 12 - Unbrinkable
Fill.

the best behaviour by far has been obtained from the rock rip rap revetment. The unshrinkable fill has begun to deteriorate and its usefulness will probably not be effective much longer. The wire mesh over the clay slope has stood up much better than expected and it definitely bears further consideration. A stronger and finer spaced mesh placed over a seeded slope could form a very effective surface. Perhaps a material such as a fine mesh jute or a plastic mesh could be utilized instead of the wire mesh. Both the granular layer and the pit run clay mix revetments have been unsatisfactory.

The above conclusions must be supplemented at a later date by some further evaluations. The cells should be drawn down so that the complete cross sections can be examined. Also a greater period of time is required in order that the behaviour with time can be established.

CHAPTER 13

RECOMMENDATIONS FOR FUTURE RESEARCH

The work in this thesis has encompassed the fields of soil mechanics, hydraulics and sanitary engineering. Accordingly it has not been possible to devote sufficient detail and time to certain subjects. In addition, because this thesis has dealt primarily in the fields of soil mechanics and hydraulics, numerous associated sanitary engineering aspects have not been undertaken. On the basis of the work that has been carried out, the following topics for continued research are suggested:

(1) Continued studies of the behaviour of the dikes and the dike revetment works installed at the Charleswood Sewage Lagoon.

At some later date, the cells should be drawn down and the overall behaviour of the slopes evaluated. At that time also additional schemes could be installed. In chapter 12, it was suggested that the wire mesh over the clay slope should be investigated from the standpoint of a smaller mesh, different mesh material and seeding the area.

(2) Further literature review and assessment of current research being undertaken in the field of erosion of cohesive soil. Work towards obtaining a standard laboratory or field test should be considered.

(3) Advanced research in clay mineralogy with an eye towards establishing the effect on clay interparticle stresses by factors such as sewage effluent instead of ordinary water being present and the sewage effluent pH varying considerably within each cell. In particular, a detailed study of the exchangeable cations of the clay in the primary and secondary lagoons would be in order.

(4) An investigation of wave phenomena for small bodies of water in the general size range of the Charleswood Sewage Lagoons should be made. Establishing relationships for these and also evaluating the effects that can be caused by the differences in surface tension of ordinary water and water containing various concentrations of sewage effluent would be a worthy investigation.

APPENDIX ACLAY MINERALOGY CONCEPTS

In previous chapters, numerous references were made to certain clay properties which involved aspects of the clay structure and forces acting between particles. These concepts are covered in the field of clay mineralogy which is a very broad and involved subject. As a result it was decided to include this appendix so that additional information would be available to compensate for background detail omitted when various concepts were introduced. This section is not intended to be a thorough discourse on clay mineralogy, but rather it serves to elaborate on several important concepts.

When a particle has a specific surface large enough to cause the electrical forces to dominate the mass forces, it is termed a "colloid".¹ The approximate size range of colloids is one micron to one millimicron (10^6). Clay-size soil particles are thus generally in the colloidal range.

Clay particles carry a net negative charge which is balanced by exchangeable cations. When the clay is dry, the cations cluster at the clay surfaces to neutralize the particles. When a clay is placed in water the cations plus a smaller

1. T.W. Lambe, "The Structure of Compacted Clays," (Proc. A.S.T.M., Vol. 34, No. SM2, Paper No. 1654, May, 1958), pp. 4 and 7.

number of anions swarm around the colloid. The swarm of counterions is called the double layer. The counterions are also called exchangeable because they can be replaced. The Gouy-Chapman Theory has shown that the potential distance curve or double layer, varies with characteristics of the dispersion medium.

Since the clay particles carry net negative charges, they electrostatically repel each other. This repulsion between two adjacent particles becomes effective when the particles approach each other close enough for the double layers to overlap. Colloid chemists have developed theoretical expressions for the electrical repulsion between colloids as a function of the distance between them. These repulsive forces are also known as Helmholtz or Zeta Forces.

Secondary valence attractive forces or Van der Waals Forces also act between particles. Combining the repulsive and attractive potential expressions, chemists have developed equations relating the total potential energy to interparticle spacing for both spherical and plate colloids. These equations show that if the potential energy is reduced when adjacent particles approach each other, they will so approach and flocculate or tend to form aggregates. If the energy of the system increases when the particles approach each other, they will move apart or disperse. Changes in the variables of the colloidal system can cause aggregation or dispersion.

Among the variables in a soil-water system which can affect colloidal stability are:

1. Electrolyte Concentration.
2. Ion Valence.
3. Dielectric Constant.
4. Temperature.
5. Size of Hydrated Ion.
6. pH.
7. Anion Absorption.

The Gouy-Chapman Theory has shown, in general terms, that a tendency towards flocculation is usually caused by:

Increasing: Electrolyte Concentration.

Ion Valence.

Temperature.

Decreasing: Dielectric Constant.

Size of Hydrated Ion.

pH.

Anion Absorption.

The influence of the first four variables follows from the Gouy-Chapman Theory of the diffuse double layer, since a decrease in the double layer thickness reduces the electrical repulsion, which in turn causes a tendency towards flocculation.

The smaller an ion plus its "shell" of hydration water, the closer it can approach the colloidal surface; thus the smaller the hydrated ion, the smaller the double layer and the more likely its flocculation.

The pH of the pore fluid affects the net negative charge on a soil particle by altering the extent of dissociation of OH groups on the edges of the particle. High pH encourages

the dissociation and increases the net charge, thus expanding the double layer; low pH does the reverse. Lowering the pH, therefore, tends to cause flocculation and raising it tends to cause dispersion.

The adsorption of anions, especially polyvalent anions, increases the charge on the particles and thereby tends to cause dispersion.

From the previous work presented it can be seen why it is not possible to simply study one variable without establishing the behaviour or effects of the others. In particular it would not be possible to undertake detailed investigations of the pH change only without evaluating the changes in other factors as well. To undertake a study of this type would constitute a major research project. For this reason such a study was not made.

Using the theory previously outlined, it is now possible to introduce the behaviour of cohesion and shrinkage.² The charged clay surfaces together with their associated exchangeable ions react with water molecules and the water molecules become orientated in the strong electric field near the charged surfaces. It is the resulting layers of orientated water molecules which give the characteristic properties of plasticity, cohesion and shrinkage of clays.

2. The United States Department of Agriculture, Soil: the 1957 Yearbook of Agriculture, (Washington, D.C.: The U.S. Government Printing Office, 1957.).

Water molecules, being highly dipolar are strongly adsorbed and orientated on the negatively charged surfaces of clay particles. The water itself exhibits strong cohesion, as shown by its high surface tension. Therefore, as a result of drying, adjacent soil particles are drawn together with considerable force and the soil develops greater cohesion and mechanical strength. Orientation of the particles into a tighter state of packing results in further shortening and strengthening of the water bonds between the particles. This results in shrinkage of the soil. Sufficient drying causes a crust to form which begins to crack up as a result of shrinkage. Although the cohesion existing between groups of particles is at a maximum, the cohesion of the surface layer no longer exists. Therefore, this phenomenon must be appreciated when considering previous statements made in earlier chapters regarding the loss of cohesion of the clay banks upon drying, because it was this latter condition implied.

Soil loses its cohesion and becomes plastic in the presence of increasing amounts of water. At a very high moisture content, soil loses its cohesion strength and its plastic properties and approaches a fluid in its mechanical properties. In this condition, the interparticle stresses are so small that the effects of the oriented water molecules on the surfaces cannot influence the behaviour of the particles.

In summary, water has an important effect on the properties of clay. A great deal remains to be determined about clay

water, especially the magnitudes of the pressures existing in the water and the effects of these pressures on the clay. In the average natural clay encountered by an engineer, essentially all of the water is within the double layer. Any classification of the clay water is somewhat arbitrary and will present difficulties because the nature of soil water depends on variable properties of the soil-water system.

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