REMEDIAL ACTION DESIGN
at the
LOCKPORT CITY LANDFILL

LOCKPORT (C), NIAGARA (C), NEW YORK
NYSDEC SITE No. 9-32-010

CITY OF LOCKPORT, NEW YORK

Design Analysis Report

MAR 15 1994

MARCH 1994

prepared by:

URS Consultants, Inc.
282 Delaware Avenue
Buffalo, New York 14202

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DESIGN ANALYSIS REPORT

CLOSURE OF LOCKPORT CITY LANDFILL

LOCKPORT, NEW YORK

NYSDEC SITE NO. 9-32-010

SUBMITTED TO:

NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

50 WOLF ROAD

ALBANY, NEW YORK 12233

MARCH 1994

STATE OF NEW YORK
JAMES A. LANZO
No. 65478
LICENSED PROFESSIONAL ENGINEER

PREPARED BY:

URS CONSULTANTS, INC.

282 DELAWARE AVENUE

BUFFALO, NEW YORK 14202

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1.0 INTRODUCTION

1.1 Authority and Scope

On May 15, 1989, the City of Lockport entered into a Consent Order with the New York State Department of Environmental Conservation (NYSDEC) for remediation of the Lockport City Landfill site, and in December, 1992 a Record of Decision (ROD) was issued. The Consent Order and ROD govern the remedial action design for final closure of the site. On November 5, 1992, the City entered into an agreement with URS Consultants, Inc. (URS), for the design. A description of tasks to be performed under this agreement may be found in the Project Management Plan prepared by URS in March 1993. This Design Analysis Report (DAR) is submitted in partial fulfillment of that agreement.

This DAR contains 8 sections. Section 1 presents introductory information on the scope of the design and its purpose. Section 2 describes the site conditions. Section 3 discusses field investigation aspects. Section 4 describes the rationale, the cap components, and the cap design. Section 5 discusses slope stability and waste removal issues on the steep slopes. Section 6 discusses the cap components and waste removal issues for the area adjacent to the wetland. Section 7 describes surface water management. Section 8 discusses the fence installation.

This report will form the basis of the engineering design drawings and construction specifications. It will remain in a "dynamic" document through the final closure design, changing as necessary until the design proves acceptable to all parties.

1.2 Purpose and Function

The purpose of the closure design is to meet the remedial action objectives set forth in the ROD. The remedial action must be protective of public health and the environment and must meet both New York State Standards, Criteria, and Guidance (SCGs) and Federal Applicable or Relevant and Appropriate Requirements (ARARs). Based on the findings from the Remedial Investigation (RI), the following New York State SCGs are identified as applicable to the

Lockport City Landfill:

- 6 NYCRR Part 360 (action specific)
- 6 NYCRR Parts 700 - Water Quality Standards (chemical specific)
- New York state Sanitary Code, Part 5 (Department of Health maximum contaminant levels)
- Article 24 of the Environmental Conservation Law (ECL) - Freshwater Wetlands
- 6 NYCRR Part 663
- Article 15 of the ECL - Water Resources
- 6 NYCRR Part 608

The remedial action objectives, as reproduced from the ROD, are listed below:

- Prevent direct contact with on-site contaminated soil/fill
- Reduce erosion of on-site contaminated soil/fill into The Gulf
- Reduce infiltration through the landfill; this will in turn reduce migration of contaminants in the groundwater and surface water.

These objectives can be accomplished by implementing a remedy with the following major components and their respective functions:

- Landfill Cap: The landfill cap will serve to reduce the amount of infiltration through the landfill. This will in turn reduce the amount of leachate generated and reduce the migration of contaminants into the groundwater and surface water.

The landfill cap will also prevent direct human contact with on-site contaminated soil/fill.

- Excavation of Potentially Unstable Waste from Steep Embankment: Excavation of the waste material that is potentially unstable along the steep embankment and placement of that material under the landfill cap will reduce long term erosion of contaminated soil/fill into The Gulf and reduce the risk of a future slope failure of this waste.

- Minimization of Disturbance to Stable Waste on Steep Embankment: The waste material deposited along the steep embankment which is presently stable and likely to remain stable will be left in place with minimum disturbance. This approach will minimize the risk of erosion/sediment migration contaminating The Gulf as a result of construction activities and will also eliminate the risk of a slope failure resulting from disturbance of the existing vegetation cover and/or other construction activities.

- Surface Water Management: This remedial element will reduce erosion in the steep embankment by intercepting surface run-on and diverting this surface run-on to the existing wetland area.

- Fencing: Installation of fencing will protect the landfill cap against unauthorized vehicle traffic that could jeopardize the cap integrity.

2.0 SITE DESCRIPTION

The 30-acre Lockport City Landfill site is located partially in the City of Lockport and partially in the Town of Lockport, Niagara County, New York. The site is bordered by The Gulf (a creek) to the west and north, by Sutliff Rotary Park and Railroad Street on the east, and by the City Highway Garage on the south. The property is owned by the City of Lockport. The site lies on an angular indentation of the Niagara escarpment. Site topography is irregular, with the eastern and southern portions of the site being relatively flat, and the western portion having a steep slope (escarpment) leading down to The Gulf. In its northern portion the site slopes gradually downward to a swampy area adjacent to The Gulf. The Gulf flows northward along the base of the landfill, discharging into Eighteen Mile Creek approximately one mile north of the site. The Gulf is classified by NYSDEC as a Class D water body. A 3-acre (unclassified) wetland is located north of The Gulf at the toe of the landfill. A small pond may be found south of the landfill. A 36-inch concrete pipe, installed to drain a spring near Oakhurst Street, lies within the fill. The pipe's outfall is located near The Gulf in the southwestern portion of the site. An 18-inch sanitary sewer runs adjacent to the landfill in The Gulf.

The landfill has two fill areas, separated by the north-south running railroad tracks. The main portion of the landfill is located west of the tracks. A smaller fill area, located east of the tracks, once served as a borrow pit for cover material for the western portion of the landfill. It was later filled with refuse, covered, and graded.

The area east of the tracks has a sparse covering of small trees and brush. The western portion of the landfill, near The Gulf, is thickly covered with trees and brush. Large piles of tree limbs are scattered on the surface. The sideslopes are heavily vegetated. Numerous empty drums and other refuse items protrude from this area.

The Lockport City Landfill was operated by the City of Lockport as a municipal and industrial waste landfill from the early 1950s until 1976. It has been reported that unknown quantities of a variety of wastes were disposed of at the landfill. This included sewage sludge, wood starch contaminated with peroxide waste, keto and oxylite waste, steel barrels, plastics, glass, cardboard, and waste paper. The method of disposal reportedly consisted of trenching into the overburden, depositing and then burning the wastes, and finally covering the wastes with excavated materials each day. A small northwest-southeast trending ravine identified from aerial photographs and geophysical survey, and confirmed by borings, had been filled with wastes by 1968.

3.0 FIELD INVESTIGATION

The Lockport City Landfill has been the subject of a number of investigations, beginning in 1981. The following is a summary of past investigations and findings:

Niagara County Department of Health - March 1981: A site inspection uncovered numerous contraventions of 6 NYCRR Part 360. Among these contraventions were an orange-colored leachate entering The Gulf through the 36-inch outfall pipe protruding from the face of the landfill. Furthermore, large amounts of refuse had been placed without cover and too close to surface waters, causing leachate and runoff to enter the stream. No final cover, not even vegetative cover, had been applied to it.

NYSDEC - December 1981: During a site inspection by Region 9 personnel, three water samples and three sediment samples were collected from The Gulf. Sediment samples showed high concentrations of iron (110,000 ppb), chromium (150 ppb), copper (40 ppb), lead (640 ppb), and zinc (1,500 ppb). Low concentrations of halogenated organics were also detected in all three samples.

One water sample taken from a leachate outbreak showed concentrations of arsenic (52 ppb), iron (10 ppm), and lead (0.2 ppm) in excess of SCG values for Class D water bodies.

NYSDEC - November 1983: A Phase II investigation was carried out for NYSDEC by RECRA Research, Inc. Field work involved placement of six monitoring wells, a geophysical survey, and a soil, surface water, and groundwater sampling program. A final Hazard Ranking System score of 23.2 was obtained.

City of Lockport - April 1992: A Remedial Investigation (RI) was performed. Findings were presented in a RI report. Also included in the report was a Health Risk Assessment (HRA). The major activities included the following:

- Preparation of a topographic map of the site.
- Community Well Surveys

- Radiological Surface Survey
- Soil Gas Survey
- Surface Geophysical Survey
- Subsurface Drilling Program
- Hydrogeological Testing
- Stream Hydrology Studies
- Macrobenthic Survey

Specific findings of this RI pertinent to the design are as follows:

- A localized pocket of suspected methane gas was encountered at one of the boring locations at the depth of 16 feet during the subsurface drilling program. Sustained LEL (Lower Explosive Limit) levels of 100% were detected at this boring location for a two-hour period.

- The RI delineated the boundary of filled trenches and the approximate limits of waste fill, based upon geophysical survey results, supplemented by historical aerial photograph and soil boring evidence.

City of Lockport - July 1992: A Feasibility Study (FS) was prepared. This study formed the basis of the selected remedy presented in the R.O.D.

The data developed from the RI was deemed sufficient to proceed directly with design, without the need for an additional round of field investigation (Predesign Investigation), although field trips are anticipated during the design to provide confirmatory survey data.

4.0 LANDFILL CAP

4.1 Design Rationale

The ROD requires that the site be capped in accordance with the existing regulations for closure of a solid waste landfill, which are specified in 6 NYCRR, Part 360. This regulation will therefore be the standard for design. The goal of the design is a landfill cap which will serve as an effective, long-term barrier to surface water infiltration and which is least susceptible to damages caused by desiccation cracking, frost action, root penetration, slope stability, and erosion.

4.2 Cap Components

In conformance with Part 360 the cap will consist of the following components:

- Gas-vent risers (1 per acre) - The 6-inch diameter gas-vent risers will be installed 3 feet into the landfill (waste), and will have at least 3 feet exposure above the final grade of the cap.

- Gas-venting system - The gas-venting system can be of a 12-inch sand layer with a minimum coefficient of permeability of $1x10^{3}$ cm/sec or a HDPE drainage net sandwiched by geotextile filter fabric that provides hydraulic transmissivity equivalent to that of 12-inch sand layer (equivalent minimum transmissivity of $3x10^{-6}$ m^2/sec). The HDPE drainage net with geotextile filter fabric is preferred and included in the design based on economic considerations.

- Low permeability barrier layer - The barrier layer will be an 18-inch layer of low permeability soil ($1x10^{7}$ cm/sec or less) placed to reduce surface water infiltration into the landfill.

- Barrier protection layer - A 24-inch barrier protection layer will be placed above the low permeability barrier to protect it from cracking, frost, root penetration, and erosion.

- Topsoil - A 6-inch topsoil layer will be placed above the barrier protection layer to support vegetative growth.

- Vegetative cover

A typical section of the Part 360 cap is presented in Figure 4-1.

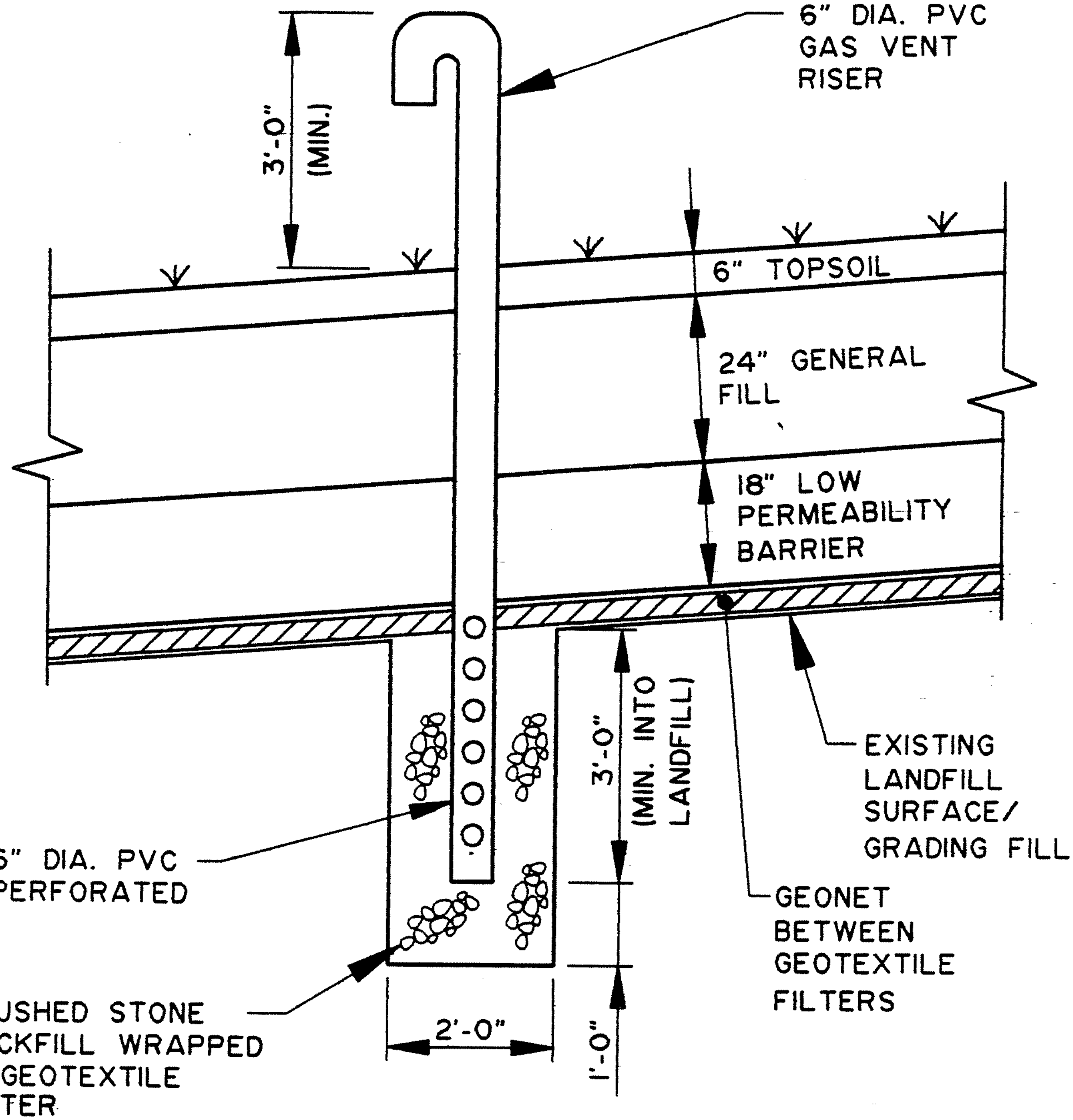
4.3 Cap Design

The cap will be designed with slopes varying from 4 to 20 percent to provide positive drainage and a stable final cover. The cap will be installed on areas where the slopes of the existing landfill surface are less than 33 percent. Areas that have slopes between 20 to 33 percent will be regraded to a slope of no more than 20 percent. The 20 percent maximum slope was selected based on the final landfill cap and a interface friction angle between the low permeability soil and geotextile fabric of 17 degrees. Regrading will consist of the optimization of cutting and filling of the landfill to minimize the amount of off-site fill material required. In the event that landfill waste is encountered during grading operations, such material will be handled in accordance with provisions contained in the Project Specifications.

The cap will be installed to the limits defined in the RI/FS excluding the steep embankment where slopes are higher than 33 percent and the area along the wetland located on the northwestern edge of the site where waste has been extended to the edge of the wetland. The remediation of these slopes and the area along the wetland are discussed in section 5 and 6 respectively.

The typical perimeter transition will be approximately 12 feet, based on a 3 horizontal: 1 vertical slope of the transition to tie into one of the four following condition:

- Existing Grades: The cap will be terminated at the limits of fill, as identified in the RI/FS reports, and transition to the existing grade.



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URS CONSULTANTS, INC.	TYPICAL PART 360 CAP CROSS SECTION	FIGURE 4-1

- Railroad: The cap will be terminated at the right-of-way where it will meet existing grade.

- Steep Slopes: The cap will be terminated at the limit of final cover and transition to the existing grade.

- Wetland: The cap will be terminated approximately 25 feet outside the limits of the wetlands and transition to the edge of the wetland. The waste outside the limit of the final cover will be removed and backfilled with grading fill.

5.0 STEEP SLOPES

5.1 Design Rationale

The western side of the landfill has steep slopes and is divided into two different areas as follows:

- "very steep" slopes at the south western edge of the site which are characterized by bedrock out-crops; and

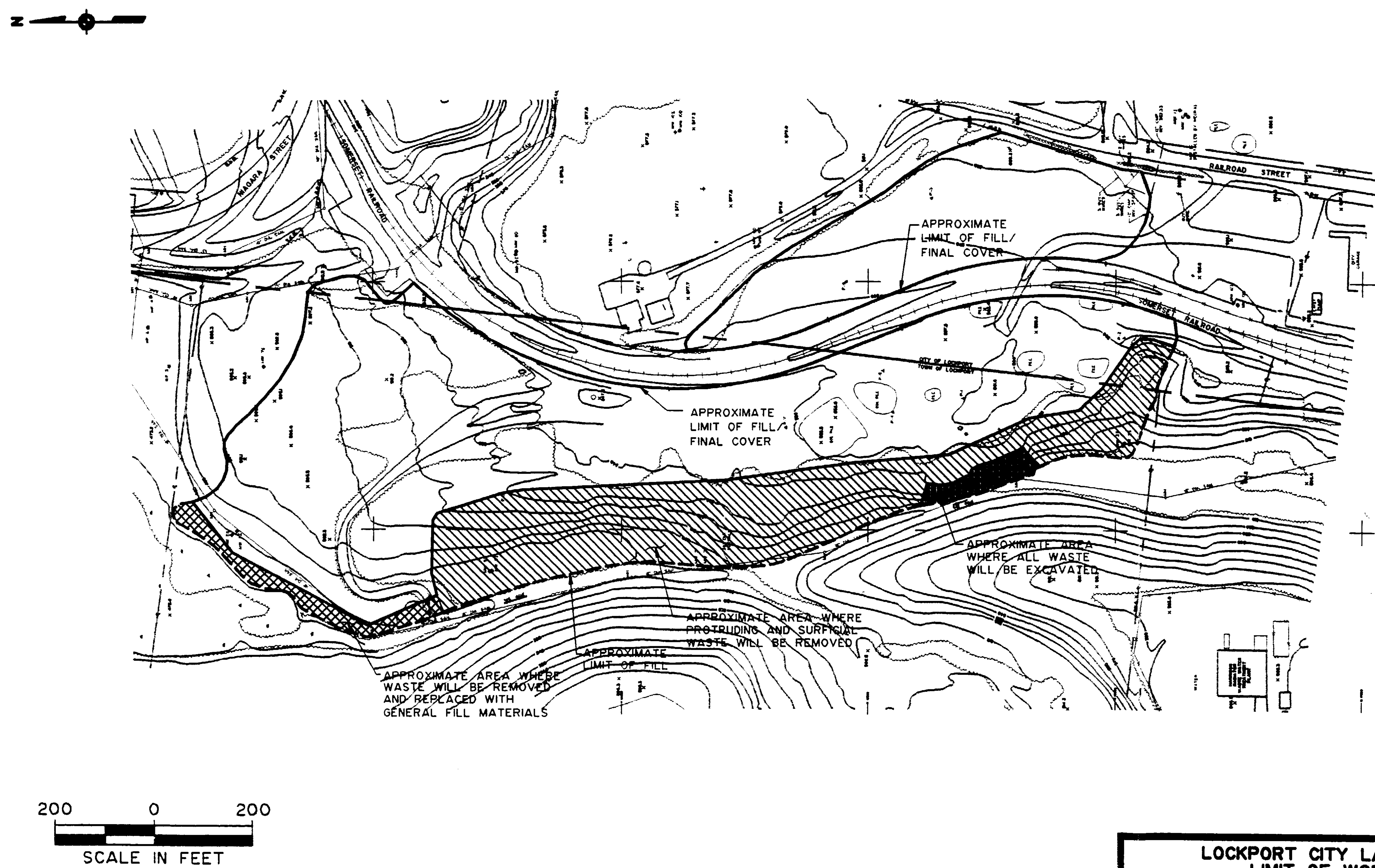
- "steep" slopes on the remaining steep slope area which has slopes that generally exceed 33 percent and have significant existing vegetative cover.

The limits of the "very steep" and "steep" areas of the landfill are shown on Figure 5-1.

Among the goals for the remedial program set forth in the ROD is the reduction of erosion of contaminated material from the site into The Gulf. To that end, as shown on Figure 5-1, the potentially unstable waste along the "very steep" slope at the southwestern edge of the site will be removed and placed under the cap. The remaining section of the steep slope area has existing dense vegetative cover which contributes significantly to the current stability of the slopes. In order to minimize the risk of contaminant migration through erosion during construction and the risk of actual slope failure as a result of vegetative cover removal, minimal disturbance is proposed. Waste removal in the "steep" slope area will consist of removing only protruding and surficial waste, with a minimal disturbance to the existing vegetative cover.

Figure 5.1 shows the relationship between the area to be capped and the steep slopes. The location of the limit of the cap is chosen for the following reasons:

- West of the selected cap location, the existing slopes encountered generally exceed the 33 percent slope maximum discussed in Section 4.3 - Cap Design. Although there are areas as shown on Figure 5-1 which appear to have relatively slight grades, there are steep slopes within those areas that do not appear as such



LOCKPORT CITY LANDFILL LIMIT OF WORK	
URS CONSULTANTS, INC.	FIGURE 5-1

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on the drawings because of the 10-foot contour line interval.

- During several site walk-through performed by URS throughout this project bedrock out-crops have been observed in the vicinity of the cap edge. As such it is anticipated that, in many instances, bedrock is only slightly below the existing surface. This condition would present major obstacles in regrading the steep slopes to a 20 percent maximum slope. As discussed in Section 4.3 -Cap Design, the 20 percent maximum slope has been selected based on a interface friction angle between the low permeability soil and geocomposite gas vent layer of 17 degrees.

- The location of the cap edge is essentially that of the eastern edge of the successional shrub savanna which extends to the western edge of the landfill. It would be best for the environment if that habitat were left undisturbed.

Deviations from the requirement of a final cover over the entire landfill surface are permitted under certain conditions under 6 NYCRR Part 375-1.10(c)(1)(i). Specifically the following sub-sections apply:

- (b) "Conformity to such standard or criterion will result in greater risk to the public health or to the environment than alternatives"

- (c) "Conformity to such standard or criterion is technically impracticable from an engineering perspective"

These sub-sections are applicable to this site because:

- As discussed in the RI, while there is virtually no present or future risk of harm to humans or the environment from the currently stable waste, there would be a substantial potential for short term damage if the vegetation is disturbed from:

  - Soil contaminant migration

- Sediment runoff to The Gulf
- Damage to existing habitat

- The presence of bedrock at or near the surface makes it technically impracticable to produce the grades required for cap design.

- Leaving the existing vegetation in-place will preserve the veneer stability of the steep slope area.

- The steep slope area is currently stable.

- The existing dense vegetation and steep slope promotes evapotranspiration and runoff which minimizes infiltration.

5.2 Infiltration/Runoff Analysis

The Hydrologic performance of the steep slope area was evaluated using the USEPA Hydrologic Evaluation of Landfill Performance (HELP) Model. In this evaluation, the following site parameters were used:

- 20 years of synthetic rainfall, temperature and solar radiation for Buffalo, NY
- 6 to 18 inches of clayey silt on top of waste
- 20 ft of waste
- site heavily vegetated

Based on the above assumptions, the following average annual distribution of total precipitation was obtained:

- Precipitation of ≈ 36 in/yr
- Run-off of 0.006 to 0.008 in/yr (≈0% of precipitation.)
- Evapotranspiration of ≃ 27 in/yr (≃ 75% of precipitation)
- Infiltration of ≈ 9 in/yr (≈ 25% of precipitation)

The analysis indicates that a significant portion of the average annual precipitation would result in evapotranspiration (75 percent) while only a small portion would result in run-off. However, the HELP model uses the Soil Conservation Service (SCS) method for calculating runoff. This procedure estimates the runoff based on the type of soil and land use. It was developed mostly for relatively large and flat areas and it does not account for the slope of the watershed. Therefore, we believe the HELP model significantly underestimates the runoff from steep slopes. HELP model analysis calculations are included in Appendix B.

Slopes on the steep slope area generally exceed 33 percent and will likely cause significant runoff. Runoff from the steep slope areaas during precipitation events can be estimated using the Rational Method. The following formula is used to estimate the runoff by this method:

$$Q = Cia$$

Where Q = runoff, cubic feet per second

C = runoff coefficient

i = rainfall intensity, inches per hour

a = drainage area, acres

The run-off coefficient represents a ratio of the runoff to rainfall. The Erie and Niagara Counties Regional Planning Board (ENCRPB) Storm Drainage manual was used to estimate the runoff coefficient for the steep slope area. Based on a hydrologic soil group D for open space and a slope of greater than 6 percent, the runoff coefficient is in the range of 0.31 to 0.41. Therefore, 30 to 40 percent of precipitation on the steep slope area would result in runoff during precipitation events. The Rational Method support data are included in Appendix B.

In summary, we believe the HELP model correctly demonstrates that a substantial portion of the total precipitation will result in evapotranspiration but that it underestimates the runoff from steep slopes. The Rational Method shows that a significant portion of the precipitation during rainfall events will result in runoff. While there is no direct correlation between the HELP model analysis and the Rational Method, we conclude that the percentage of infiltration will likely be in the range of 15 to 17.5 percent.

5.3 Slope Stability

Since the potentially unstable waste and the thin layer of the surficial soils along the "very steep" slope area will be excavated to bedrock, a slope stability analysis is not warranted for this area. As discussed earlier in Section 5.1, it is anticipated that bedrock is only slightly below the existing grade. This condition will be verified as part of the preconstruction activities.

URS conducted a visual inspection of the face of the "steep" slope area and found it to be densely vegetated and currently stable. In addition, preliminary calculations using the wedge method of analysis were completed. The wedge method of analysis was used for the following conditions:

a) Groundwater Level (GWL) at toe of slope - dry condition

b) GWL at Elevation +515 (seepage head of 15′)

c) GWL at Elevation +515 with final cap in place

Computed factors of safety for cases (a), (b) & (c) are 1.55, 1.38, and 1.38, respectively. By limiting the rise of the groundwater table after capping, adequate factors of safety for the overall landfill stability will be maintained.

In order to extend the cap over the steep face of the landfill, flattening of the steep slope areas would be necessary since existing slopes are as steep as 1.33 H on 1.0 V. Clearing the slope of dense vegetation would be necessary if slope flattening were to be undertaken. In the absence of slope flattening there are advantages to leaving the vegetation on the slopes as it exists. The cover and its root structure is effective in preserving veneer stability of the steep slope face. In addition, there is substantial potential for short term damage if the vegetation is disturbed from soil contaminant migration and sediment runoff to the Gulf. For the above reasons it is recommended that the existing vegetation be left in place over the steep landfill areas when the flat areas on the top are capped.

5.4 Soil Sampling/Analysis

As part of the Remedial Investigation, soil samples (SPS-2 and SPS-3) were collected in the vicinity of the steep slope area and analyzed for TCL list parameters. The surface soil results for SPS-2 and SPS-3 (copy included in Appendix B) were compared to NYSDEC TAGM for determination of soil cleanup objectives (November 1992). Six polynuclear aromatic hydrocarbons (PAHs), a pesticide, and a PCB exceeded recommended SCG values. This indicates that the surface soil is contaminated along the steep slope.

In order to minimize exposure to this surface soil (via dust and soil erosion), we recommend not disturbing the dense vegetation already present in the steep slope area (all surface debris will be removed). Leaving the vegetation will help reduce erosion and also help keep the compounds which exceeded cleanup goals in the soil. It should also be noted that during the course of the Remedial Investigation, waste samples taken in the vicinity of the steep slopes were subjected to EPTOX testing with no results exceeding SCG values. (RI Report, April 1992, Table 6-9, page 6-15 copy included in Appendix B). Thus, it would appear that there is no health risk from that material. The EPTOX test results indicated that the resulting groundwater should not become contaminated

5.5 Waste Removal

Waste removal in the steep slope areas will be carried out as follows:

- Along the "very steep" slopes at the southwestern edge of the site, waste will be removed down to bedrock and hauled for placement under the cap.

- In the remaining steep slope area, only the protruding and surficial waste (drums, debris, etc.) will be removed and placed under the cap. Existing vegetation will remain undisturbed to the maximum extent feasible. The Contract Specifications will require that the Contractor prepare and submit a waste removal plan for review and approval of the Engineers. This plan will be submitted to the NYSDEC for review and comments prior to approval.

The boundary between these two areas will be verified in the field during construction by visual inspection of the following:

- Evidence of current and potential slope instability
- Integrity of vegetative cover
- Change in relative slope steepness.

All work will be performed in conformance with an Erosion and Sedimentation Control Plan which will be a specification requirement. This plan will be based upon:

- April 1991 Memorandum titled "Erosion and Sediment Control Guidelines for New Development" issued by NYSDEC Division of Water.

- New York State "Guidelines for Urban Erosion and Sediment Control."

The Remedial Action Contractor will be required to prepare an Erosion and Sedimentation Control Plan for review and approval by the Engineer. This Plan will be submitted to the NYSDEC for review and comments prior to approval. The Contractor will not be permitted to begin construction of the work until his erosion and sediment control plan is approved.

6.0 AREA ADJACENT TO THE WETLAND

As shown on Figure 5-1, waste materials extend laterally toward the edge of a wetland located on the northwest corner of the site. As discussed in Section 4.3 - Cap Design, the cap will be terminated approximately 25 feet outside the limits of the wetland and transition to the edge of it.

In order to minimize disruption in the wetland area, the contractor will use the following sequence for cap construction:

- The area between the limit of waste and the limit of the cap will be excavated to a depth which insures that the waste is removed.
- Excavated waste material will be placed on the surface of the landfill to be covered by the cap construction.
- The excavation will be filled with the same material used for the barrier protection layer, covered with topsoil, and seeded to matching existing grades.
- The barrier protection layer and topsoil layer will transition into the filled excavation in accordance with the design documents.

The contractor, as required by the Contract Specifications, will, prior to any excavation, prepare a description of erosion and sedimentation controls adequate to protect the wetlands. That description will be submitted to NYSDEC for approval prior to Engineer's approval.

7.0 SURFACE WATER MANAGEMENT PLAN

7.1 Design Rationale

In order to reduce erosion in the steep embankment area caused by the surface run-off from the landfill cap and to maintain the integrity of the cap from erosion caused by the surface run-on, the surface water must be managed. The surface water management plan will also serve to preserve the current drainage patterns to the maximum extent possible.

7.2 Surface Water Management Plan

References used in developing the surface water management plan are:

- Guidelines for Urban Erosion and Sediment Control, New York.
- Storm Drainage Design Manual, Erie and Niagara Counties Regional Planning Board.

Key components of the surface water management plan are described briefly as follows:

- A diversion berm (drainage swale) will be constructed at the western edge of the cap (along the edge of the steep embankment) to intercept surface run-off from areas of the landfill cap between the railroad tracks and steep slope areas. Where possible, the grass-lined diversion berm will have a ridgeline approximately 10 feet inside the western limit of the cap. It will convey surface run-off from south to north, draining to The Gulf through riprap-lined down chute and outlet protection apron into the wetland area.

- For the eastern portion of the landfill, a grass-lined perimeter drainage ditch will be constructed to collect surface run-on from areas north and east of the landfill. The perimeter drainage ditch will convey surface run-on to the existing ditch at the east side of the railroad tracks. The collected run-on will join the current drainage pathways in the northern portion of the landfill.

- In the extreme northwestern part of the landfill, the cap will be contoured to provide a final cover slope of between 4 and 20 percent, utilizing the current drainage pathway as much as possible. No diversion berm is to be constructed for this area.the surface run-off is expected to flow to the northwest, draining into the wetland.

- The existing drainage ditch east of the railroad will be retained, and a new drainage ditch will be constructed west of the railroad.

- The landfill cap would have a run-off drainage distance no greater than 600 feet. This is achieved by inclusion of a grass-lined drainage swale at the northern part of the landfill. This swale will intercept surface flow from the northern part of the landfill and convey the flow from east to west. It will join the riprap-lined down chute, draining to the wetland.

8.0 FENCING

8.1 Design Rationale

To protect the landfill cap against damage from unauthorized vehicles that may jeopardize the cap integrity, a chain-link fences will be installed as a part of the remedial action at the Lockport City Landfill.

8.2 Fence Installation

The site is bounded by the Gulf, running along the north and west boundary by Sutliff Rotary Park and Railroad Street on the east, and by the City Highway Garage on the south.

Fences will consist of six-foot high chain link fence including fabric, top and bottom, tension wire, bracing, and truss rods. Gates will also be installed as required.

APPENDIX A

DRAINAGE CALCULATIONS

A.1 - DITCH SIZING

A.2 - DOWNCHUTE SIZING

A.3 - CMP SIZING

APPENDIX A.1

DITCH SIZING

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PROJECT LOCKPORT
JOB NO.
SUBJECT DRAINAGE
MADE BY MW DATE 3/10/93
CHKD. BY MH DATE 7-6-93

REF. PAGE

SUMMARY

The following results were obtained from the drainage calculations for the Lockport City Landfill Cap

- See sheet 8 for the locations of the drainage structures
- Perimeter ditch S1
  - Flow $Q \approx 1.6\ ft^3/sec$
  - Depth of Flow $y \approx 0.21\ ft$
  - Flow Velocity $V \approx 2.9\ ft/sec$
  - Surface Grass
  - Trapezoidal channel, 2ft bottom, 1V:3H side slopes
- Perimeter ditch S2
  - Flow $Q = 1.6\ ft^3/sec$
  - Depth of Flow $y = 0.18$ min ; 0.30 ft max
  - Flow Velocity $V = 1.8$ min ; 3.5 ft/sec max
  - Surface Grass
  - Trapezoidal channel, 2ft bottom, 1V:3H side slopes
- Swale S3
  - Flow $Q = 27\ ft^3/sec$
  - Depth of Flow $y = 0.36\ ft$ min ; 0.57 ft max
  - Flow Velocity $V = 2.1\ ft/sec$ min ; 6.9 ft/s max
  - Surface Downchute part - rip-rap
  - Flat part - grass
  - Triangular channel, flat part 1:3 & 1:5, downchute part 1:3 & 1:3

Note: For a 5 yr time of concentration storm for Buffalo, NY

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PROJECT LOCKPORT
SUBJECT DRAINAGE

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1 PURPOSE

This calculation was performed in order to size the surface water diversion structures at the Lockport City Landfill site

2 METHODOLOGY

The flows were determined using a rational method with time of concentration. The diversion structures were sized using standard methods treating flows in open channels

- Flows

$$t_c = \left(\frac{L}{\alpha \, i_e^{m-1}} \right)^{1/m} \leftarrow$$

t_c - Time of conc [sec]
L - Length of overland flow [ft]
i_e - Rainfall excess [ft/sec]

$$i_e = C \cdot i$$

i - Rainfall intensity [ft/sec]
C - Runoff coefficient [-]
α - Factor

$$\alpha = \frac{K_m \sqrt{S}}{N}$$

S - Slope [-]
K_m - Conversion factor ($K_m = 1.49$ for ft & sec)
N - Effective roughness [-]
$m = 5/3$ for turbulent flow

See Ref 1, sheets 18, 19 of this calc.

An iterative technique is employed to find "t_c". Time of storm duration "t_r" is

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assumed. For the assumed "t_r", the rainfall intensity "i" and the excess rainfall "i_e" are determined from IDF curves. From that, "t_c" is calculated. The procedure is repeated until $t_r \approx t_c$. Once "t_c" is known, the appropriate intensity of the rainfall of the duration $t_r = t_c$ is found and the total flow in the channel is calculated as

$$Q = k_c C i A$$

Q - Flow [ft^3/s]
k_c - Conversion factor [$k_c \approx 1.0$ for units of ft^3/s, acres and in/hr]
i - Rainfall intensity [in/hr]
A - Area of drainage basin [acres]

See Ref 1, sheet 20 of this calc.

- Drainage structures
Manning's eq of uniform flow in open channels was used

$$Q = \frac{1.49}{N} A R_H \sqrt{S}$$

Ref 1, see sheet 21A of this calc

Q - Flow [ft^3/s]
N - Roughness coeff. [-]
A - Cross sectional flow area [ft^2]
R_H - Hydraulic Radius [ft]

$R_H = \frac{A}{P}$ P - wetted perimeter [ft]

s - Slope [-]

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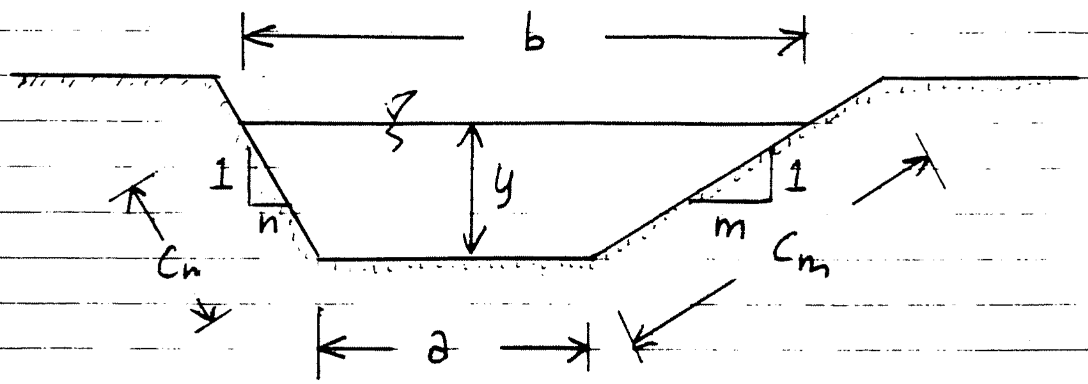
- For trapezoidal Channels



Area $A = \frac{1}{2}(a+b)y$

$$b = a + ny + my = a + y(m+n)$$

$$A = \frac{1}{2}\left[a + a + y(m+n)\right]y = \frac{1}{2}\left[2ay + y^2(m+n)\right]$$

Wetted perimeter $P = a + C_n + C_m$

$$P = a + y\sqrt{n^2+1} + y\sqrt{m^2+1}$$

$$P = a + y\left(\sqrt{n^2+1} + \sqrt{m^2+1}\right)$$

Hydraulic radius $R_H = A/P$

$$R_H = \frac{\frac{1}{2}\left[2ay + y^2(m+n)\right]}{a + y\left(\sqrt{n^2+1} + \sqrt{m^2+1}\right)}$$

Flow $Q = \frac{1.49}{N} A R_H^{2/3} \sqrt{S}$

$$Q = \frac{1.49}{N} * \frac{1}{2}\left[2ay + y^2(m+n)\right] * \left\{\frac{\frac{1}{2}\left[2ay + y^2(m+n)\right]}{a + y\left(\sqrt{n^2+1} + \sqrt{m^2+1}\right)}\right\}^{2/3} * \sqrt{S}$$

$$Q = \frac{1.49\sqrt{S}}{N} \frac{\left\{\frac{1}{2}\left[2ay + y^2(n+m)\right]\right\}^{5/3}}{\left[a + y\left(\sqrt{n^2+1} + \sqrt{m^2+1}\right)\right]^{2/3}} \leftarrow$$

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For this, "y" is determined by iterating

Velocity $V = Q/A$

$$V = \frac{Q}{\frac{1}{2}\left[2ay + y^2(n+m)\right]}$$

For a triangular channel, a = 0 and

$$Q = \frac{1.49\sqrt{S}}{N}\,\frac{\left[\frac{1}{2}(n+m)\right]^{5/3}}{\left[y\left(\sqrt{n^2+1}+\sqrt{m^2+1}\right)\right]^{2/3}}$$

$$y = \left\{\frac{QN\left(\sqrt{n^2+1}+\sqrt{m^2+1}\right)^{2/3}}{1.49\sqrt{S}\left[\frac{1}{2}(n+m)\right]^{5/3}}\right\}^{3/8}$$

$$V = Q/A = Q\Big/\tfrac{1}{2}y^2(n+m)$$

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3 PARAMETERS

a) Areas and drainage structures
The system was designed for the following distribution of flows
- Area A1, draining to the structure S1, along the northern portion of the perimeter of the eastern part of landfill
- Area A2, draining to structure S2, along the southern portion of the perimeter of the eastern part of landfill.
- Area A3, draining to structure S3, along the change in slope of the western portion of the landfill.

See sheet 8 of this calc for the designation of areas and drainage structures

b) Length of overland flow
- Area A1: $L \approx 950$ ft
- Area A2: $L \approx 1{,}000$ ft (Assumed, since there is no topo for the entire area)
- Area A3: $L \approx 350$ ft

c) Runoff coefficients
Assume heavy soil, overgrown with grass, on avg slopes (All areas).

$C \approx 0.2$ Ref 1, see sheet 21 of this calc

d) Slopes
Based on topo and proposed grading plan, assume
- Area A1: $S \approx \frac{620-570}{950} = 0.05$

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- Area A2: $S \approx 0.05$
- Area A3: $S \approx \frac{570-540}{350} \approx 0.09$

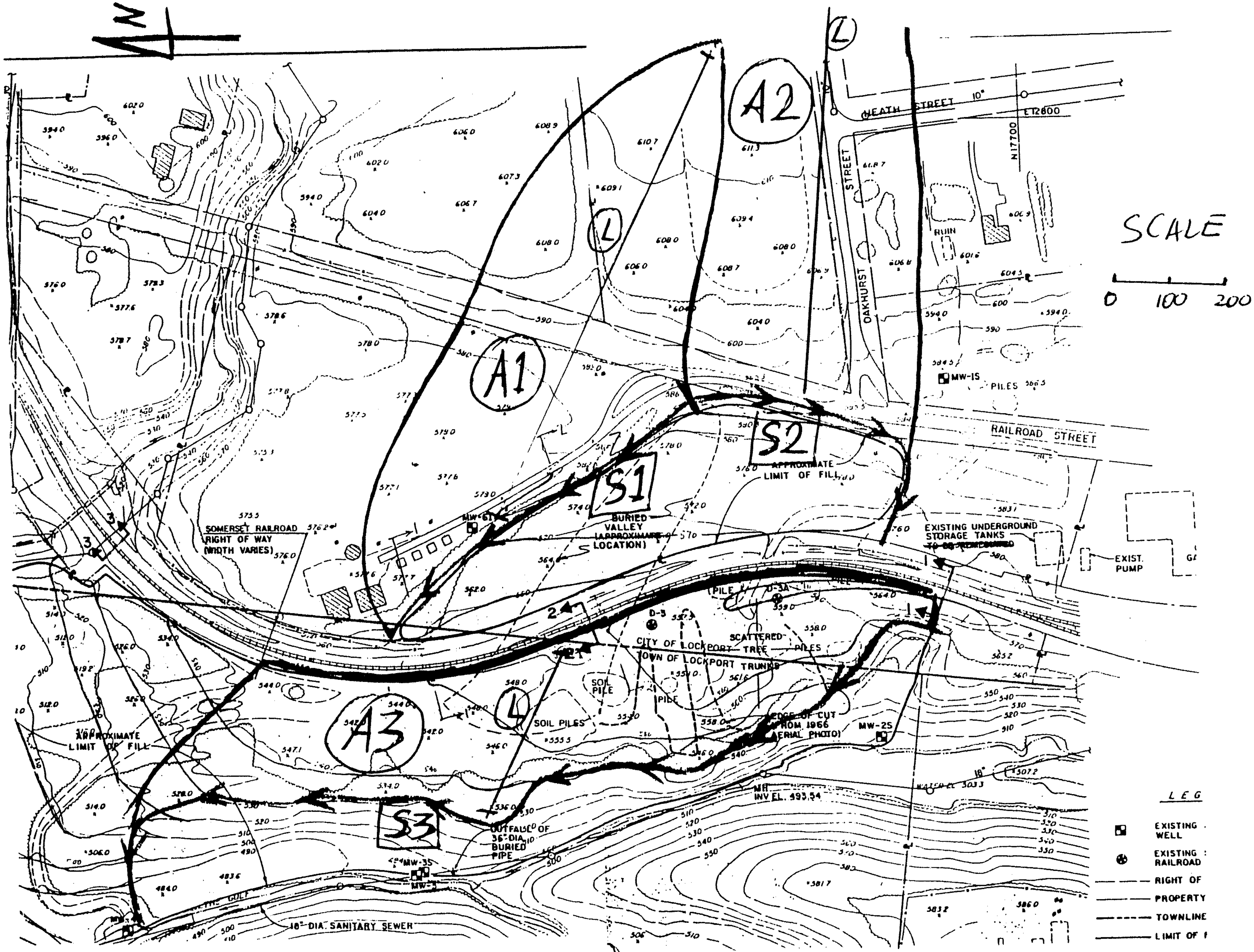
e) Effective roughness for overland flow
Assume
$N = 0.25$ Ref 1, see sheet 17

for grass

f) Areas
- Area A1: $A \approx 10$ acres
- Area A2: $A \approx 10$ acres
- Area A3: $A \approx 8$ acres

g) Slopes of channels
- Channel S1: $S = \frac{10}{250} = 0.04$
- Channel S2: $S = \frac{5}{400} = 0.01$
- Channel S3: $S = 0.01$ assumed

h) Manning's coefficient
Assume, for grass lined channels
$n = 0.03$ Ref 2, see sheet 23



SCALE
0 100 200
N
A1
A2
A3
S1
S2
S3
HEATH STREET
OAKHURST STREET
RAILROAD STREET
E12800
N17700
RUIN
MW-1S
PILES
MW-6
MW-2S
MW-3S
APPROXIMATE LIMIT OF FILL
BURIED VALLEY (APPROXIMATE LOCATION)
SOMERSET RAILROAD RIGHT OF WAY (WIDTH VARIES)
EXISTING UNDERGROUND STORAGE TANKS
EXIST. PUMP
D-3A
D-3
SCATTERED TREE TRUNKS
PILES
CITY OF LOCKPORT
TOWN OF LOCKPORT
SOIL PILE
SOIL PILES
PILE
EDGE OF CUT FROM 1966 AERIAL PHOTO
MH INV.EL. 493.54
OUTFALL OF 36"-DIA. BURIED PIPE
18"-DIA. SANITARY SEWER
APPROXIMATE LIMIT OF FILL
LEG
EXISTING WELL
EXISTING RAILROAD
RIGHT OF
PROPERTY
TOWNLINE
LIMIT OF

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Summary of data

	A1	A2	A3
L [ft]	850	1,000	350
C [-]	0.2	0.2	0.2
S [-]	0.05	0.05	0.09
A [acres]	10	10	8
N [-]	0.25	0.25	0.25

	S1	S2	S3
S [-]	0.04	0.01	0.01
N [-]	0.03	0.03	0.03

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4. CALCULATIONS
- Flows

$$t_c = \left(\frac{L}{\alpha\, i_e^{m-1}} \right)^{1/m}$$

see sheet 2

$$t_c = \left(\frac{L}{\frac{k_m \sqrt{S}}{n} (C i)^{m-1}} \right)^{1/m}$$

$$t_c = \left[\frac{L}{\frac{1.49 \sqrt{S}}{n} (C i)^{m-1}} \right]^{1/m}$$

This is in units "ft" and "sec". To convert rainfall intensity "i" from in/hr to ft/sec

$$i_{ft/sec} = i_{in/hr} * (2.3E-5)$$

Also, $m = 5/3$, $C = 0.2$, $n = 0.25$

$$t_c = \left[\frac{L}{\frac{1.49 \sqrt{S}}{0.25} (0.2 * 2.3E-5 * i)^{2/3}} \right]^{3/5}$$

$$t_c = 46.7 \left(\frac{L}{\sqrt{S}\, i^{2/3}} \right)^{3/5}$$

"t_c" in sec
"i" in in/hr

in minutes

$$t_c = 46.7 \left(\frac{L}{\sqrt{S}\, i^{2/3}} \right)^{3/5} \frac{1}{60} = 0.78 \left(\frac{L}{\sqrt{S}\, i^{2/3}} \right)^{3/5}$$

"t_c" in min
"i" in in/hr

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Using IDF curves for Buffalo, NY, 5yr frequency

- Area 1

$$t_c = 0.78 \cdot \left(\frac{350}{\sqrt{.05}\, i^{2/3}}\right)^{3/5} = 117.2\, i^{-2/5}$$

$$t_c \approx 120 \text{ min} \rightarrow i \approx 0.8 \text{ in/hr}$$

$$t_c = 117.2 \cdot (0.8)^{-2/5} \approx 128 \text{ min} \approx OK$$

- Area 2

$$t_c = 0.78 \times \left(\frac{1{,}000}{\sqrt{.05}\, i^{2/3}}\right)^{3/5} = 120.9\, i^{-2/5}$$

$$t_c \approx 120 \text{ min} \rightarrow i \approx 0.8 \text{ in/hr}$$

$$t_c = 120.9 \cdot [0.8]^{-2/5} = 132 \text{ min} \approx OK$$

- Area 3

$$t_c = 0.78 \times \left(\frac{350}{\sqrt{.05}\, i^{2/3}}\right)^{3/5} = 54\, i^{-2/5}$$

$$t_c \approx 40 \text{ min} \rightarrow i \approx 1.7 \text{ in/hr}$$

$$t_c = 54\, i^{-2/5} = 54\,(1.7)^{-2/5} = 44 \text{ min} \approx OK$$

So, flows are

	C [-]	i [in/hr]	A [acres]	Q = CiA [ft³/s]
Area 1	0.2	0.8	10	1.6
Area 2	0.2	0.8	10	1.6
Area 3	0.2	1.7	8	2.7

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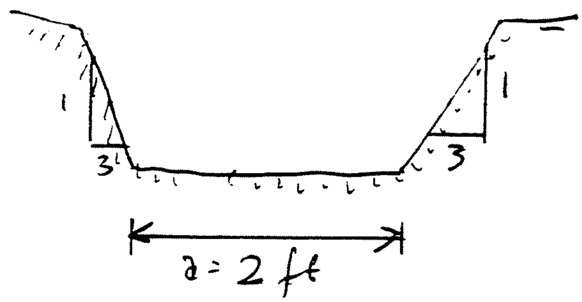
– Drainage structures
• Perimeter ditch S1
Assume a trapezoidal channel, grass lined



$n = m = 3$)

$S = 0.04$, $n = 0.03$, $Q = 1.6\ ft^3/sec$

$$Q = \frac{1.49\sqrt{S}}{n}\,\frac{\left\{\frac{1}{2}\left[2ay + y^2(m+n)\right]\right\}^{5/3}}{\left[a + y\left(\sqrt{n^2+1} + \sqrt{m^2+1}\right)\right]^{2/3}}$$

see sheet 4

$$Q = \frac{1.49\sqrt{.04}}{0.03}\,\frac{\left\{\frac{1}{2}\left[2\cdot 2\cdot y + y^2(3+3)\right]\right\}^{5/3}}{\left[2 + y\left(\sqrt{3^2+1} + \sqrt{3^2+1}\right)\right]^{2/3}}$$

$$Q = 9.9\cdot\frac{1}{2}^{5/3}\,\frac{\left[4y + 6y^2\right]^{5/3}}{\left[2 + 6.3y\right]^{2/3}} = 3.1 \times \frac{\left[4y + 6y^2\right]^{5/3}}{\left[2 + 6.3y\right]^{2/3}}$$

for 1.6cfs, this holds for $y \cong 0.21\ ft$

$$Q = 3.1 \times \frac{\left[4\cdot 0.21 + 6\cdot 0.21^2\right]^{5/3}}{\left[2 + 6.3\cdot 0.21\right]^{2/3}} = 1.64 \simeq 1.6$$

ok

Velocity

$$V = \frac{Q}{\frac{1}{2}\left[2ay + y^2(n+m)\right]} = \frac{1.6}{\frac{1}{2}\left[2\cdot 2\cdot 0.21 + 0.21^2(3+3)\right]} = 2.9\ ft/sec$$

Use a grass lined channel

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• Perimeter ditch S2

Use same dimensions as for S1

$$S = 0.01 \quad , Q = 1.6 \ ft^3/s$$

$$Q = \frac{1.49 \sqrt{.01}}{0.03} \frac{1}{2}^{5/3} \frac{[4y + 6y^2]^{5/3}}{[2 + 6.3y]^{2/3}}$$

$$Q = 1.6 \frac{[4y + 6y^2]^{5/3}}{[2 + 6.3y]^{2/3}}$$

For $Q = 1.6 \ ft^3/s$, this holds for $y \approx 0.3$

$$Q = 1.6 \frac{[4 \cdot 0.3 + 6 \cdot 0.3^2]^{5/3}}{[2 + 6.3 \cdot 0.3]^{2/3}} = 1.63 \approx 1.6 \ \text{ok}$$

$$V = \frac{1.6}{\frac{1}{2}[2 \cdot 2 \cdot 0.3 + 0.3^2(3+3)]} = 18 \ ft/s$$

Use grass lined channel.

The ~~drainchute~~ part of this ditch has a slope of

$$s \approx \frac{10}{150} = 0.07$$

$$Q = \frac{1.49 \sqrt{.07}}{0.03} \frac{1}{2}^{5/3} \frac{[4y + 6y^2]^{5/3}}{[2 + 6.3y]^{2/3}}$$

$$Q = 4.1 \frac{[4y + 6y^2]^{5/3}}{[2 + 6.3y]^{2/3}}$$

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For $Q = 1.6\ ft^3/sec$, it holds for $y \simeq 0.18\ ft$

$$Q = 4.1 \cdot \frac{[4 \cdot 0.18 + 6 \cdot 0.18^2]^{5/3}}{[2 + 6.3 \cdot 0.18]^{2/3}} \simeq 1.65 \approx 1.6$$

ok

Velocity

$$V = \frac{1.6}{\frac{1}{2}\left[2 \cdot 2 \cdot 0.18 + 0.18^2 (3+3)\right]} \simeq 3.5\ ft/sec$$

Use also a grass lined channel

- Swale S3

Assume dimensions

$n = 3,\ m = 5$

$S = 0.01$, $n = 0.03$, $Q = 2.7\ ft^3/s$

For a triangular channel

$$y = \left\{ \frac{Q n \left(\sqrt{n^2+1} + \sqrt{m^2+1}\right)^{2/3}}{1.49 \sqrt{S} \left[\frac{1}{2}(n+m)\right]^{5/3}} \right\}^{3/8}$$

$$y = \left\{ \frac{2.7 \times 0.03 \left(\sqrt{3^2+1} + \sqrt{5^2+1}\right)^{2/3}}{1.49 \sqrt{0.01} \left[\frac{1}{2}(3+5)\right]^{5/3}} \right\}^{3/8}$$

$y \simeq 0.57\ ft$

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Velocity $$V = \frac{Q}{\frac{1}{2} y^2 (n+m)} = \frac{2.7}{\frac{1}{2} 0.57^2 (3+5)} = 2.1 \text{ ft/sec}$$

Use a grass lined channel.

The downchute part of the channel
(use $n = m = 3$)
$S = 0.2$

$$y = \left\{ \frac{2.7 \cdot 0.03 \left(\sqrt{3^2+1} + \sqrt{3^2+1} \right)^{2/3}}{1.49 \sqrt{.2} \left[\frac{1}{2} (3+3) \right]^{5/3}} \right\}^{3/8} = 0.36 \text{ ft}$$

$$V = \frac{2.7}{\frac{1}{2} \cdot 0.36^2 (3+3)} = 6.9 \text{ ft/sec}$$

Use rip-rap lined downchute

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Hydrology and Floodplain Analysis

Philip B. Bedient
RICE UNIVERSITY

Wayne C. Huber
UNIVERSITY OF FLORIDA

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TABLE 4.2

Effective Roughness Parameters for Overland Flow

SURFACE	N
Dense Growth*	0.4–0.5
Pasture*	0.3–0.4
Lawns*	0.2–0.3
Bluegrass sod**	0.2–0.5
Short-grass prairie**	0.1–0.2
Sparse vegetation**	0.05–0.13
Bare clay-loam soil (eroded)**	0.01–0.03
Concrete/asphalt	
Very Shallow Depths* (depths less than 1/4 inch)	0.10–0.15
Small Depths* (depths on the order of 1/4 inch to several inches)	0.05–0.10

AVG = 0.25

* From Crawford and Linsley (1966)

** From Woolhiser (1975)

Rewriting equations in terms of flow per unit width for overland flow, we have

$$q_0 = \alpha_0 y_0^{m_0}, \tag{4.44}$$

where

$\alpha_0 = \dfrac{1.49}{N}\sqrt{S_0}$ = conveyance factor,

m_0 = 5/3 from Manning's equation,

S_0 = average overland flow slope,

y_0 = mean depth of overland flow.

The continuity equation is

$$\frac{\partial y_0}{\partial t} + \frac{\partial q_0}{\partial x} = i - f, \tag{4.45}$$

where

$i - f$ = rate of excess rainfall (ft/s),

q_0 = flow rate per unit width (cfs/ft),

y_0 = mean depth of overland flow (ft).

1. The time of concentration is the travel time of a *wave* to move from the hydraulically most distant point in the catchment to the outlet.
2. The time of concentration is the time to equilibrium of the catchment under a steady rainfall excess (i.e., when the outflow from the catchment equals the rainfall excess onto the catchment).

Note especially that t_c is *not* the travel time taken by a parcel of water to move down the catchment, as is so often cited in texts. The catchment is in equilibrium when the time t_c is reached because the outlet then "feels" the inflow from every portion of the catchment. Since a wave moves faster than a parcel of water does, the time of concentration (and equilibrium) occurs sooner than if based on overland flow (or channel) water velocities (Overton and Meadows, 1976). For overland flow, the wave speed (celerity) is usually given by the kinematic wave equation

$$c = mV = \alpha m y^{m-1}, \tag{6.2}$$

where

c = wave speed,
V = average velocity of water,
y = water depth,
α, m = kinematic wave parameters in the uniform flow momentum equation, and

$$q = yV = \alpha y^m \tag{6.3}$$

where q = flow per unit width of the catchment.

Using Manning's equation, the parameter m is 5/3 for turbulent flow and 3 for laminar flow. Thus the wave can travel at a rate of from 1.7 to 3 times as fast as does a parcel of water. Corresponding values of the parameter α for turbulent and laminar flow, respectively, are

$$\alpha = \frac{k_m \sqrt{S}}{n} \qquad (m = 5/3) \qquad \text{turbulent flow,} \tag{6.4}$$

$$\alpha = gS/3\nu \qquad (m = 3) \qquad \text{laminar flow,} \tag{6.5}$$

where

S = slope (small, such that $\sin S \approx \tan S \approx S$),
n = Manning's roughness,
g = acceleration of gravity,
ν = kinematic viscosity, and
k_m = 1.49 for units of ft and s in Eq. (6.4),
= 1.0 for units of m and s.

The time of concentration for overland flow by kinematic wave theory is

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$$t_c = \left(\frac{L}{\alpha i_e^{m-1}}\right)^{1/m}, \tag{6.6}$$

where

L = length of overland flow plane,

i_e = rainfall excess.

Care should be taken to use consistent units in Eq. (6.6). It can be seen that t_c depends inversely on the rainfall excess; thus higher-intensity rainfall excess will reduce t_c. Alternative equations for t_c based on water parcel travel times seldom include this inherent physical property of the catchment (McCuen et al., 1984; Huber, 1987).

EXAMPLE 6.2

KINEMATIC WAVE FLOW COMPUTATION

Water flows down an asphalt parking lot that has a 1% slope. What are the values of α and m (Eq. 6.3) for the following two cases:

a) laminar flow ($T = 20°C$),

b) turbulent flow ($n = 0.013$)?

What is the flow per unit width q (cfs/ft) for each of the two cases, for a water depth of 0.3 in.?

SOLUTION

Laminar flow has an exponent $m = 3$ and a coefficient α, given by Eq. (6.5). At a temperature of 20°C, $\nu = 1.003 \times 10^{-2}$ cm²/s $= 1.086 \times 10^{-5}$ ft²/s (Table C.1). Thus

$$\alpha = gS/3\nu = \frac{32.2 \times 0.01}{3 \times 1.080 \times 10^{-5}} = 9938 \text{ ft}^{-1}\text{s}^{-1}.$$

For turbulent flow, the exponent $m = 5/3$, and from Eq. (6.4),

$$\alpha = \frac{1.49 S^{0.5}}{n} = \frac{1.49 \times 0.01^{0.5}}{0.013} = 11.46 \text{ ft}^{1/3}/\text{s}.$$

A depth of 0.3 in. = 0.025 ft. The flow per unit width is given by Eq. (6.3):

Laminar: $q = 9938 \times 0.025^3 = 0.155$ cfs/ft,

Turbulent: $q = 11.46 \times 0.025^{5/3} = 0.024$ cfs/ft.

For the same depth, the laminar flow rate is higher, but in most actual cases turbulent flow is observed.

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no need to make an arbitrary assumption about the shape of the hyetograph. The use of historical storms also has an advantage when dealing with the public, because a design can be presented to prevent the flooding that occurred from a specific real event in the public's memory. The main disadvantage is simply the extra effort involved in a continuous simulation or a frequency analysis based on storm events rather than on conditional frequencies readily available from published IDF data. As a result, synthetic design storms are most often used in practice.

6.4 METHODS FOR QUANTITY ANALYSIS

Peak Flow, Volume, or Hydrograph?

Recall that there are several possible parameters to be determined in an urban hydrologic analysis, but most often they include peak flow, runoff volume, or the complete runoff hydrograph. Related parameters might be the hydraulic grade line or flooding depths. If hydrographs are predicted, then peaks and volumes are an implicit part of the analysis, but some of the simpler methods will not necessarily provide all parameters of possible interest.

Peak Flows by the Rational Method

The rational method dates from the 1850s in Ireland (Dooge, 1973) and is called the Lloyd-Davies method in Great Britain. It is one of the simplest and best-known methods routinely applied in urban hydrology, although it contains subtleties that are not always appreciated. Peak flows are predicted by the simple product

$$Q_p = k_c CiA, \tag{6.8}$$

where

Q_p = peak flow (cfs or m^3/s),
C = runoff coefficient,
i = rainfall intensity (in./hr or mm/hr),
A = catchment area (ac or ha),
k_c = conversion factor.

When U.S. customary units are used, the conversion factor $k_c = 1.008$ to convert ac-in./hr to cfs and is routinely ignored; this conversion is the basis for the term "rational" in the rational method. (The approximate equivalence of ac-in./hr to cfs is worth remembering for convenient rough calculations.) For the alternative metric units given with Eq. (6.8), the conversion factor $k_c = 0.00278$ to convert ha-mm/hr to m^3/s.

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TABLE 6.8

Typical C Coefficients for 5- to 10-yr Frequency Design*

DESCRIPTION OF AREA	RUNOFF COEFFICIENTS
Business	
Downtown areas	0.70–0.95
Neighborhood areas	0.50–0.70
Residential	
Single-family areas	0.30–0.50
Multiunits, detached	0.40–0.60
Multiunits, attached	0.60–0.75
Residential (suburban)	0.25–0.40
Apartment dwelling areas	0.50–0.70
Industrial	
Light areas	0.50–0.80
Heavy areas	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yard areas	0.20–0.40
Unimproved areas	0.10–0.30
Streets	
Asphalt	0.70–0.95
Concrete	0.80–0.95
Brick	0.70–0.85
Drives and walks	0.75–0.85
Roofs	0.75–0.95
Lawns, Sandy Soil	
Flat, 2%	0.05–0.10
Average, 2–7%	0.10–0.15
Steep, 7%	0.15–0.20
Lawns, Heavy Soil	
Flat, 2%	0.13–0.17
Average, 2–7%	0.18–0.22 AVG = 0.2
Steep, 7%	0.25–0.35

* Viessman et al., 1977.

To avoid the complications of an iterative process, constant overland flow inlet times are often used to approximate the time of concentration. These vary from 5 to 30 min with 5 to 15 min most commonly used (American Society of Civil Engineers and Water Pollution Control Feder-

21A

Then

$$\boxed{b = 2.08 \text{ m.}}$$

EXAMPLE 7.2

UNIFORM FLOW IN A TRAPEZOIDAL CHANNEL

A trapezoidal channel with side slopes of 2 to 1 is designed to carry a normal flow of 200 cfs. The channel is grass-lined with a Manning's n of 0.025 and has a bottom slope of 0.0006 ft/ft. Determine the normal depth, bottom width, and top width (see Fig. E7.2) assuming normal flow and the bottom width (BW) as 1.5 times the normal depth.

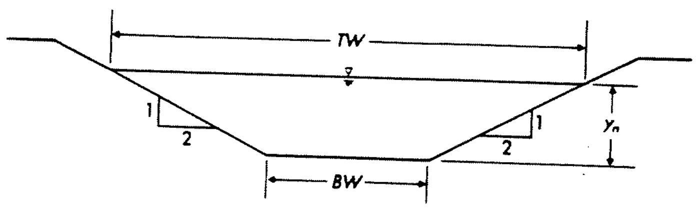

FIGURE E7.2

SOLUTION

Given

$$Q = 200 \text{ cfs},$$
$$n = 0.025,$$
$$S_0 = 0.0006 \text{ ft/ft},$$
$$BW = 1.5\, y_n.$$

Equation (7.4) gives

$$V = \frac{1.49}{n} R^{2/3}\sqrt{S_0}$$

for U.S. customary units. Since $Q = VA$,

$$Q = \frac{1.49}{n} AR^{2/3}\sqrt{S_0}.$$

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22

OPEN-CHANNEL HYDRAULICS

VEN TE CHOW, Ph.D.

Professor of Hydraulic Engineering
University of Illinois

REF 2

McGRAW-HILL BOOK COMPANY

New York Toronto London

1959

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT *n* (*continued*)

Type of channel and description	Minimum	Normal	Maximum
C. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
D. NATURAL STREAMS			
D-1. Minor streams (top width at flood stage <100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT *n* (*continued*)

Type of channel and description	Minimum	Normal	Maximum
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
D-2. Flood plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
D-3. Major streams (top width at flood stage >100 ft). The *n* value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	.....	0.060
b. Irregular and rough section	0.035	.....	0.100

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APPENDIX A.2

DOWNCHUTE SIZING

URS CONSULTANTS, INC.

PROJECT: LOCKPORT

SUBJECT: DRAINAGE

PAGE ...... OF ......

SHEET NO. 1 OF

JOB NO.

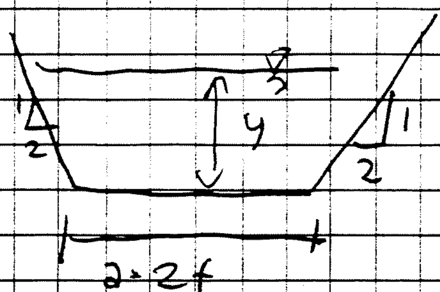
MADE BY: [illegible] DATE: 3/11/9-

CHKD. BY: MH DATE: 7-6-93

REF. PAGE

Design the outlet protection apron for the downchute

Downchute previously designed as triangular
Change to trapezoidal



$n = m = 2$

$Q = 2.7\ ft^3/sec$ - say, $Q = 3\ ft^3/s$

$S = 0.2$

$N_{riprap} \approx 0.033$

$$Q = \frac{1.49\sqrt{S}}{N} \frac{(ay + my^2)^{5/3}}{(a + 2y\sqrt{m^2+1})^{2/3}}$$

$$Q = \frac{1.49\sqrt{.2}}{0.033} \frac{(2y + 2y^2)^{5/3}}{(2 + 2y\sqrt{5})^{2/3}}$$

$$Q = 20.2 \times \frac{2^{5/3}}{2^{2/3}} \frac{(y + y^2)^{5/3}}{(1 + 2.24y)^{2/3}}$$

$$Q = 40.4 \frac{(y + y^2)^{5/3}}{(1 + 2.24y)^{2/3}}$$

For $Q = 3\ ft^3/s$ it holds for $y \approx 0.2\ ft$

$$Q = 40.4 \frac{(0.2 + 0.2^2)^{5/3}}{(1 + 2.24 \cdot 0.2)^{2/3}} = 2.83 \approx 3 \quad ok$$

URS CONSULTANTS, INC.

PROJECT: LOCKPORT
SUBJECT: DRAINAGE

PAGE ... OF ...
SHEET NO. 2 OF ...
JOB NO.
MADE BY Hro DATE 3/11/93
CHKD. BY MH DATE 7-6-93

REF. PAGE

$$V = \frac{Q}{A} = \frac{Q}{2y + 4y^2} = \frac{3.0}{2 \cdot 0.22 + 2 \cdot 0.22^2} \approx 6.25 \ ft/s$$

For a D = 0.2 ft pipe flowing full at V = 6.25 ft/sec., the discharge is

$$Q = 6.25 * \frac{\pi 0.2^2}{4} = 0.2 \ ft^3/sec$$

This, according to the Fig 4.47 of the NY Guidelines for Urban Erosion and Sediment Control, March 1988, does not require outlet protection.
However, the velocity of 6.25 ft/s seems to high for grass. So, design the outlet as if

$$D = 2 \ ft \ (\text{channel bottom width})$$

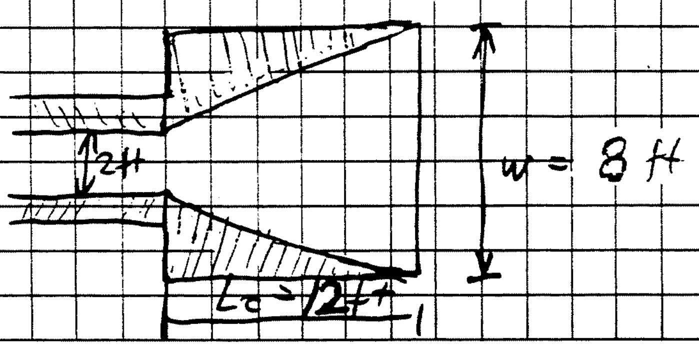
$$Q = 6.25 * \frac{\pi 2^2}{4} = 20 \ ft^3/sec$$

From Fig 4.47 for d = 2 ft = 24" and Q = 20 ft³/sec (See sheet 3)

$$d_{50} = 0.5 \ ft \quad \text{rip-rap}$$

$$L_a = 12 \ ft \quad \text{length of apron}$$

Use apron



3

Figure 4.47
Outlet Protection Design - Minimum Tailwater Condition

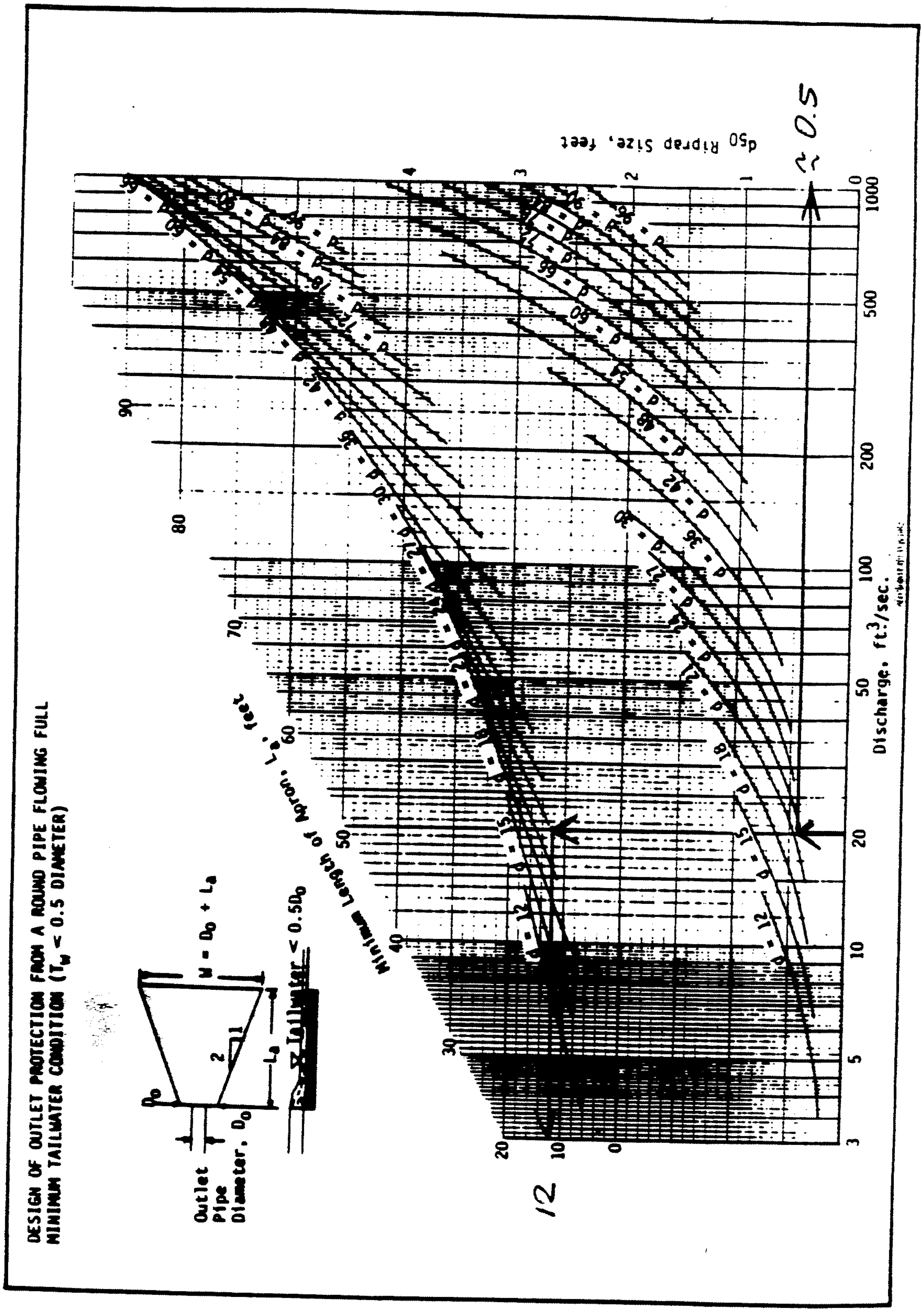


EXHIBIT 5.5-2

URS Consultants, Inc.
CALCULATION COVER SHEET

Client: CITY OF LOCKPORT Project: LOCKPORT CITY LANDFILL CLOSURE

Project/Calculation Number: 35180.02

Title: LOCKPORT CITY LANDFILL: FINAL COVER CONSTRUCTION

Total number of pages (including cover sheet): 18

Total number of computer runs: ________

Originator: JOHN MACDOWELL Date: 8/10/93

Checker: [illegible] Date: ________

Description and Purpose: SIZE CMP CULVERTS FOR DITCH A AND C.

Design bases/references/assumptions:

REFER TO PAGE 1 FOR REFERENCES

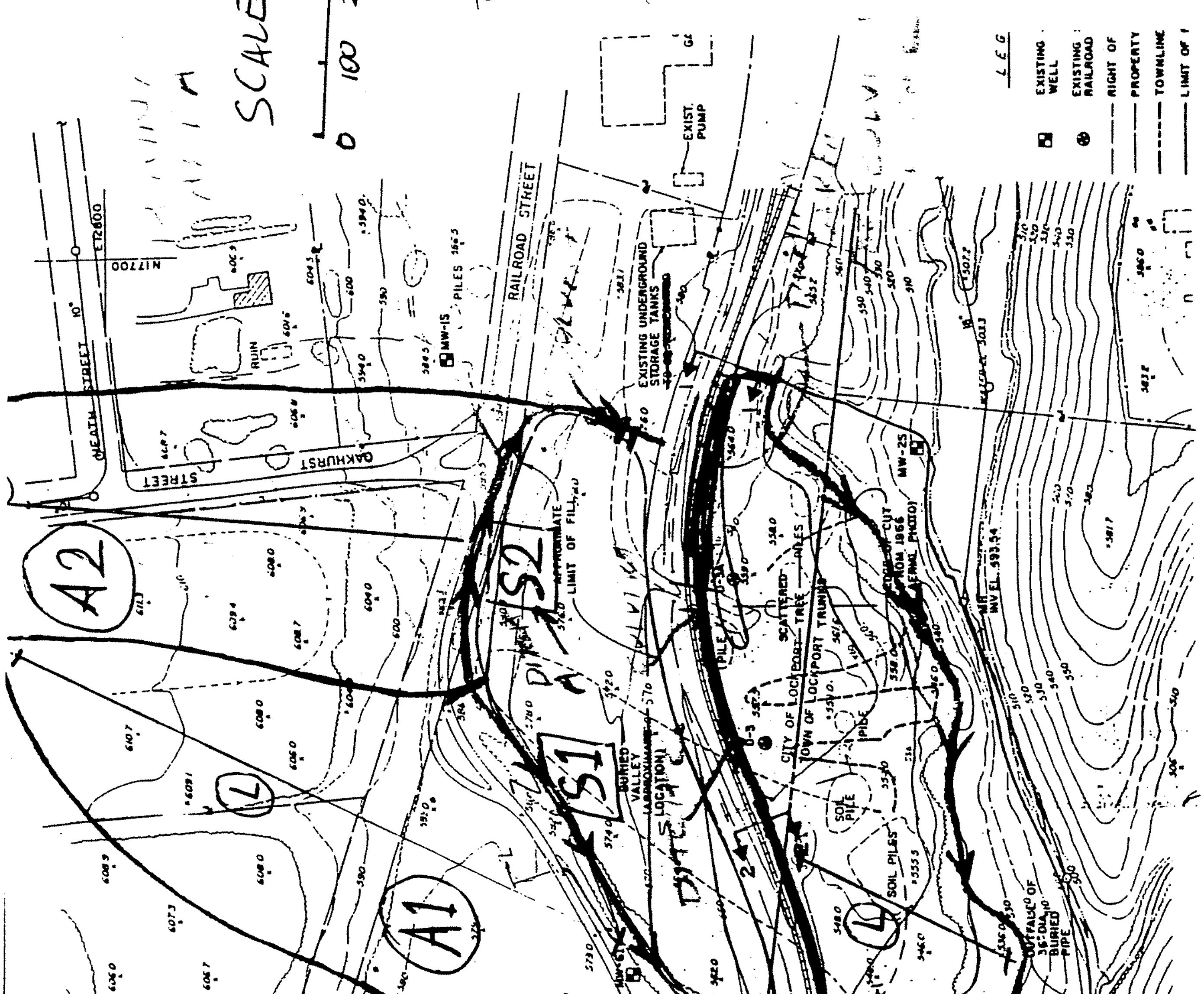
Remarks/conclusions:

Revision No.	Description of Revision	Approved by/date

Project Manager

Revision 1 - 11/01/91

REF. E (SITE MAP)



APPENDIX A.3

CMP SIZING

PROJECT LOCKPORT CITY LANDFILL
SUBJECT SIZING OF CMP CULVERTS

JOB NO. 35180.02.00.300
MADE BY JFM DATE 3/10/93
CHKD. BY ... DATE ...

REF. PAGE

REFERENCES:

A. HYDROLOGY AND FLOOD PLAIN ANALYSIS, BEDIENT AND HUBER, ADDISON-WESLEY PUB. COMP., 1975.
B. STORM DRAINAGE DESIGN MAN, ERIE AND NIAGARA COUNTIES REGIONAL PLANNING BOARD, 1972, AMMENDED 1981.
C. OPEN CHANNEL HYDRAULICS, CHOW, MCGRAW-HILL, 1988
D. ENGINEERING FLUID MECH, BERTIN, PRENTICE HALL, 1987
E. DRAINAGE CALCULATION BY MO FOR DITCH SIZING, 3/93

LOCATION #1 - PERIMETER DITCH A (SEE MAP)

P376 REF D

MANNING EQUATION

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

WHERE,

$Q = 1.6$ CFS

REF E PREVIOUS CALCS BY MO ON 3/10/93

$n =$ ROUGHNESS COEFF. $= .024$

p.110 REF C

$AR^{2/3} =$ AREA (HYDRAULIC RADIUS)$^{2/3}$

$S^{1/2} =$ (SLOPE OF CMP)$^{1/2}$ $= (.04)^{1/2}$

$$AR^{2/3} = \frac{Qn}{1.49 S^{1/2}}$$

$$AR^{2/3} = \frac{1.6(.024)}{1.49(.04^{1/2})}$$

$$AR^{2/3} = .129$$

DETERMINE CMP DIAMETER

ASSUMPTION: NORMAL DEPTH SHOULD NOT EXCEED 75% OF PIPE DIAMETER.

$$\therefore \quad Y/D_o = .75$$

FROM FIG 6-1 ATTACHED WITH $Y/D_o = .75$

$$AR^{2/3}/d_o^{8/3} = .3$$

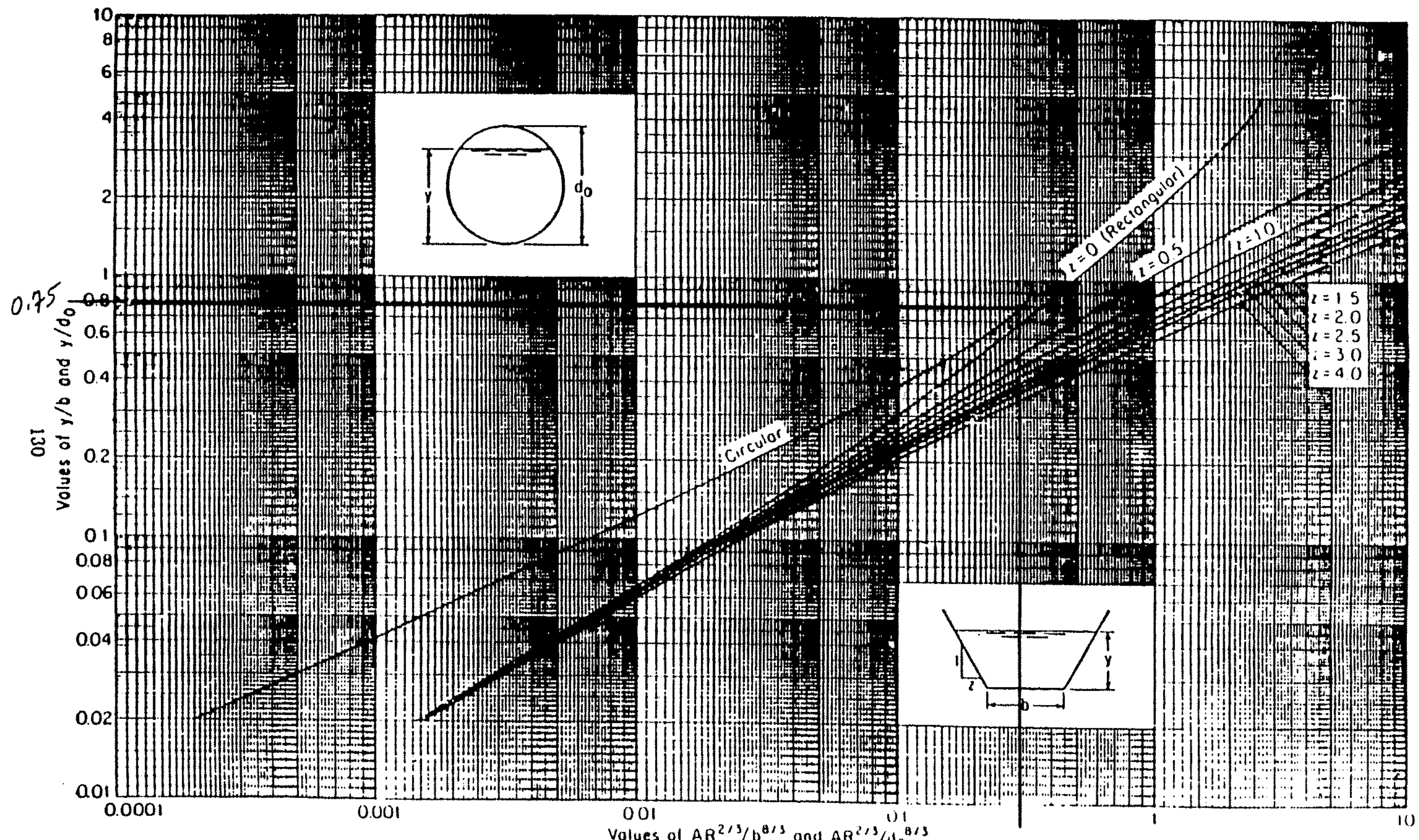


FIG. 6-1. Curves for determining the normal depth.

2/5

SINCE $AR^{2/3} / d_o^{8/3} = .3$ AND $AR^{2/3} = .129$ ✓

$$\frac{.258}{d_o^{8/3}} = .3$$

$$.3\, d_o^{8/3} = .129$$

$$d_o^{8/3} = .43$$

$$d_o = .73 \text{ ft} = 8.7 \text{ in.}$$

USE 12" CMP ✓

- LOCATION #2 - PERIMETER DITCH C (SEE MAP)

DETERMINE FLOW RATE:

$Q = C\,i\,A$

WHERE, C = .2 ⇒ SEE MO. CALC. 3/10/93.
A = .34 Ac. ⇒ SEE MAP

DETERMINE i:

$$T_c = \frac{1.8(1.1 - C)\, D^{1/2}}{S^{1/3}}$$

$$T_c = \frac{1.8(1.1 - .2)(300)^{.5}}{(.037)^{1/3}}$$

$T_c = 84.2$ MIN

C = .2 ✓

S = AVG. SLOPE = $\frac{4.89 + 2.5}{2}$ = 3.7%.

D = MAX FLOW DIST.

D = 300 ft ✓

PROJECT ......
SUBJECT ......

REF. PAGE

FROM EXIBIT III-6, ATTACHED,

$T_L = 84.2$ min @ 5 yr storm

$i = 1.22$ in/hr.

DETERMINE FLOW

$\therefore \quad Q = (.2)(1.22)(.34) = .082$ CFS.

MANNING EQUATION

$$AR^{2/3} = \frac{Qn}{1.49\, S^{1/2}}$$

$$AR^{2/3} = \frac{(.082)(.024)}{1.49\,(.007)^{1/2}} = .016$$

DETERMINE CMP DIAMETER

FROM PREVIOUS CALC.

$$AR^{2/3} / d_0^{8/3} = .3$$

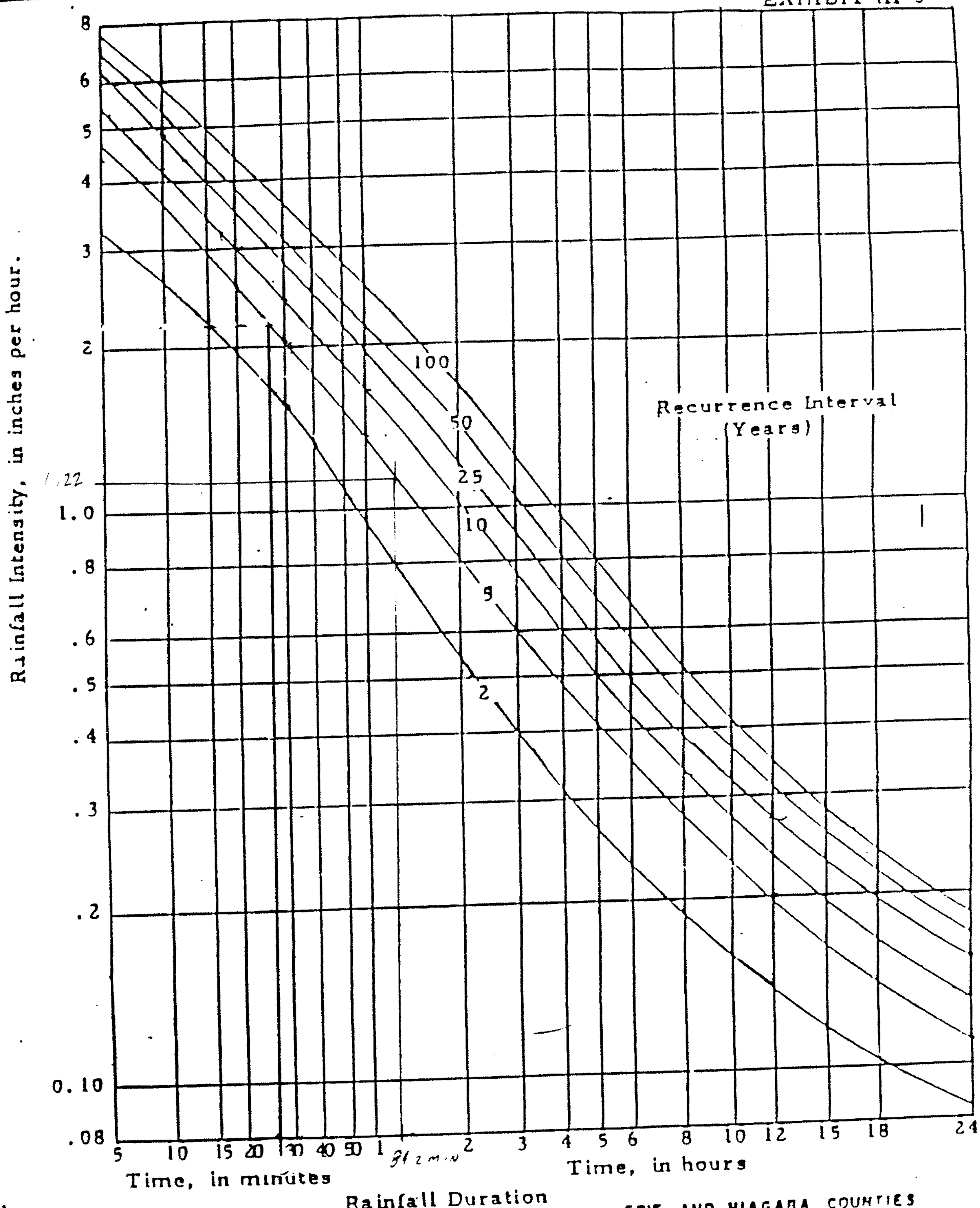
$$\frac{.016}{d_0^{8/3}} = .3$$

$d_0 = .33$ ft. $= 4''$

USE 12" CMP

5/6

EXHIBIT III-6



ERIE AND NIAGARA COUNTIES
REGIONAL PLANNING BOARD

REGIONAL STORM-SURFACE WATER
DRAINAGE MANAGEMENT STUDY

RAINFALL INTENSITY-DURATION-FREQUENCY, BUFFALO, NEW YORK

The printing of this report was financially aided through a federal grant from the Department of Housing and Urban Development under the Comprehensive Planning Assistance Program authorized by Section 701 of the Federal Housing Act of 1954, as amended. This report was printed under the Comprehensive Planning Assistance Program for the New York State Department of State. It was financed in part by the State of New York and Erie and Niagara Counties.

Source: U.S. Weather Bureau

REF A

Hydrology and Floodplain Analysis

Philip B. Bedient
RICE UNIVERSITY
Wayne C. Huber
UNIVERSITY OF FLORIDA

ADDISON-WESLEY PUBLISHING COMPANY
Reading, Massachusetts ▪ Menlo Park, California ▪ New York
Don Mills, Ontario ▪ Wokingham, England ▪ Amsterdam ▪ Bonn
Sydney ▪ Singapore ▪ Tokyo ▪ Madrid ▪ Bogotá ▪ Santiago ▪ San Juan

no need to make an arbitrary assumption about the shape of the hyetograph. The use of historical storms also has an advantage when dealing with the public, because a design can be presented to prevent the flooding that occurred from a specific real event in the public's memory. The main disadvantage is simply the extra effort involved in a continuous simulation or a frequency analysis based on storm events rather than on conditional frequencies readily available from published IDF data. As a result, synthetic design storms are most often used in practice.

6.4 METHODS FOR QUANTITY ANALYSIS

Peak Flow, Volume, or Hydrograph?

Recall that there are several possible parameters to be determined in an urban hydrologic analysis, but most often they include peak flow, runoff volume, or the complete runoff hydrograph. Related parameters might be the hydraulic grade line or flooding depths. If hydrographs are predicted, then peaks and volumes are an implicit part of the analysis, but some of the simpler methods will not necessarily provide all parameters of possible interest.

Peak Flows by the Rational Method

The rational method dates from the 1850s in Ireland (Dooge, 1973) and is called the Lloyd-Davies method in Great Britain. It is one of the simplest and best-known methods routinely applied in urban hydrology, although it contains subtleties that are not always appreciated. Peak flows are predicted by the simple product

$$Q_p = k_c CiA, \tag{6.8}$$

where

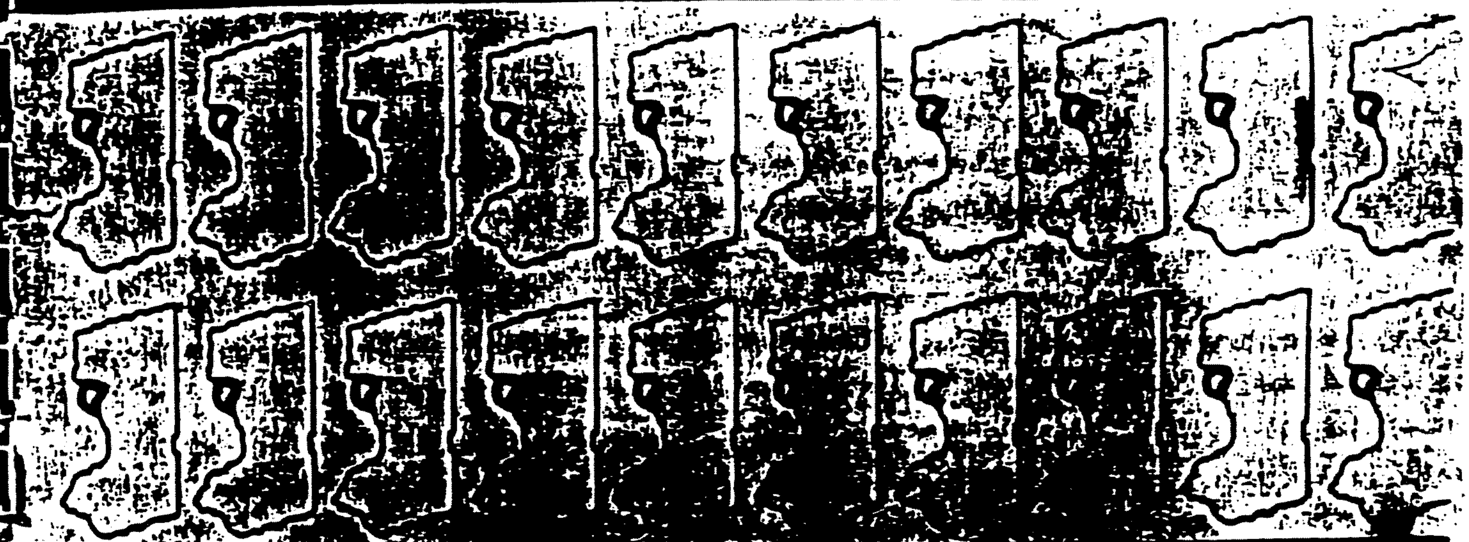
Q_p = peak flow (cfs or m³/s),
C = runoff coefficient,
i = rainfall intensity (in./hr or mm/hr),
A = catchment area (ac or ha),
k_c = conversion factor.

When U.S. customary units are used, the conversion factor k_c = 1.008 to convert ac-in./hr to cfs and is routinely ignored; this conversion is the basis for the term "rational" in the rational method. (The approximate equivalence of ac-in./hr to cfs is worth remembering for convenient rough calculations.) For the alternative metric units given with Eq. (6.8), the conversion factor k_c = 0.00278 to convert ha-mm/hr to m^3/s.

81

REF. B

STORM DRAINAGE DESIGN MANUAL



B2

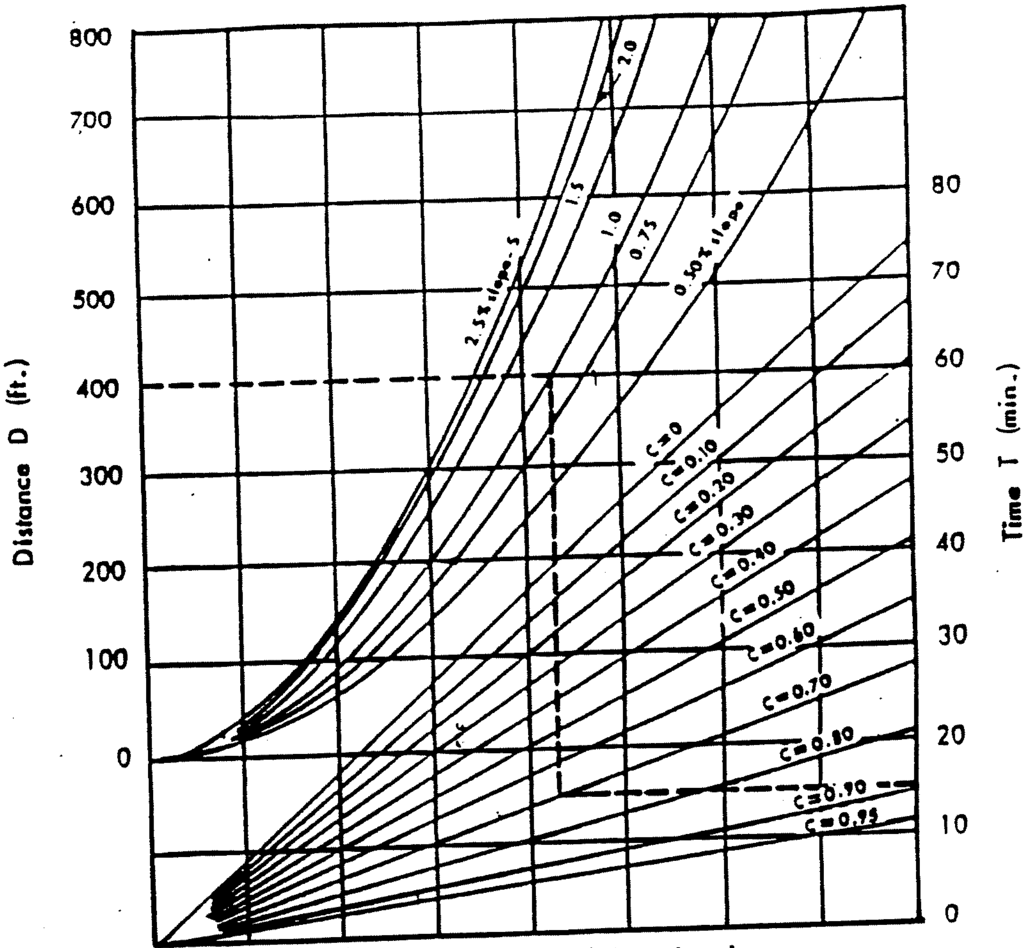
RUNOFF COEFFICIENTS
FOR USE IN THE RATIONAL FORMULA

Hydrologic Soil Group	A			B			C			D		
Slope Range	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+
LAND USE												
Industrial	0.67 1/ 0.85 2/	0.68 0.85	0.68 0.86	0.68 0.85	0.68 0.86	0.69 0.86	0.68 0.86	0.69 0.86	0.69 0.87	0.69 0.86	0.69 0.86	0.70 0.88
Commercial	0.71 0.88	0.71 0.89	0.72 0.89	0.71 0.89	0.72 0.89	0.72 0.89	0.72 0.89	0.72 0.89	0.72 0.90	0.72 0.89	0.72 0.89	0.72 0.90
High Density 3/ Residential	0.47 0.58	0.49 0.60	0.50 0.61	0.48 0.59	0.50 0.61	0.52 0.64	0.49 0.60	0.51 0.62	0.54 0.66	0.51 0.62	0.53 0.64	0.56 0.69
Medium Density 4/ Residential	0.25 0.33	0.28 0.37	0.31 0.40	0.27 0.35	0.30 0.39	0.35 0.44	0.30 0.38	0.33 0.42	0.38 0.49	0.33 0.41	0.36 0.45	0.42 0.54
Low Density 5/ Residential	0.14 0.22	0.19 0.26	0.22 0.29	0.17 0.24	0.21 0.28	0.26 0.34	0.20 0.28	0.25 0.32	0.31 0.40	0.24 0.31	0.28 0.35	0.35 0.46
Agricultural	0.08 0.14	0.13 0.18	0.16 0.22	0.11 0.16	0.15 0.21	0.21 0.28	0.14 0.20	0.19 0.25	0.26 0.34	0.18 0.24	0.23 0.29	0.31 0.41
Open Space	0.05 0.11	0.10 0.16	0.14 0.20	0.08 0.14	0.13 0.19	0.19 0.26	0.12 0.18	0.17 0.23	0.24 0.32	0.16 0.22	0.21 0.27	0.28 0.39
Freeways and Expressways	0.57 0.70	0.59 0.71	0.60 0.72	0.58 0.71	0.60 0.72	0.61 0.74	0.59 0.72	0.61 0.73	0.63 0.76	0.60 0.73	0.62 0.75	0.64 0.78

1/ Lower runoff coefficients for use with storm recurrence intervals less than 25 years.
2/ Higher runoff coefficients for use with storm recurrence intervals of 25-years or more.
3/ High Density Residential - greater than 15 dwelling units per acre
4/ Medium Density Residential - 4 to 15 dwelling units per acre
5/ Low Density Residential - 1 to 4 dwelling units per acre

B3

OVERLAND TIME OF FLOW



SOURCE: Airport Drainage Federal Aviation Agency
Department of Transportation Circular-AC 150-5320-5B
Washington, D.C., 1970

Where: T = time - minutes
D = distance - feet
S = slope - percentage
C = runoff coefficient

$$T = \frac{1.8\,(1.1 - C)\,D^{1/2}}{S^{1/3}}$$

The printing of this report was financially aided through a federal grant from the Department of Housing and Urban Development under the Comprehensive Planning Assistance Program authorized by Section 701 of the Federal Housing Act of 1954, as amended. This report was printed under the Comprehensive Planning Assistance Program for the New York State Department of State. It was financed in part by the State of New York and Erie and Niagara Counties.

ERIE AND NIAGARA COUNTIES REGIONAL PLANNING BOARD

REF. C

McGRAW HILL CIVIL ENGINEERING SERIES

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT n
(Boldface figures are values generally recommended in design)

Type of channel and description	Minimum	Normal	Maximum
A. CLOSED CONDUITS FLOWING PARTLY FULL			
A-1. Metal			
a. Brass, smooth	0.009	0.010	0.013
b. Steel			
1. Lockbar and welded	0.010	0.012	0.014
2. Riveted and spiral	0.013	0.016	0.017
c. Cast iron			
1. Coated	0.010	0.013	0.014
2. Uncoated	0.011	0.014	0.016
d. Wrought iron			
1. Black	0.012	0.014	0.015
2. Galvanized	0.013	0.016	0.017
e. Corrugated metal			
1. Subdrain	0.017	0.019	0.021
2. Storm drain	0.021	0.024	0.030
A-2. Nonmetal			
a. Lucite	0.008	0.009	0.010
b. Glass	0.009	0.010	0.013
c. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
d. Concrete			
1. Culvert, straight and free of debris	0.010	0.011	0.013
2. Culvert with bends, connections, and some debris	0.011	0.013	0.014
3. Finished	0.011	0.012	0.014
4. Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
5. Unfinished, steel form	0.012	0.013	0.014
6. Unfinished, smooth wood form	0.012	0.014	0.016
7. Unfinished, rough wood form	0.015	0.017	0.020
e. Wood			
1. Stave	0.010	0.012	0.014
2. Laminated, treated	0.015	0.017	0.020
f. Clay			
1. Common drainage tile	0.011	0.013	0.017
2. Vitrified sewer	0.011	0.014	0.017
3. Vitrified sewer with manholes, inlet, etc.	0.013	0.015	0.017
4. Vitrified subdrain with open joint	0.014	0.016	0.018
g. Brickwork			
1. Glazed	0.011	0.013	0.015
2. Lined with cement mortar	0.012	0.015	0.017
h. Sanitary sewers coated with sewage slimes, with bends and connections	0.012	0.013	0.016
i. Paved invert, sewer, smooth bottom	0.016	0.019	0.020
j. Rubble masonry, cemented	0.018	0.025	0.030

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT n (continued)

Type of channel and description	Minimum	Normal	Maxi...
B. LINED OR BUILT-UP CHANNELS			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0 0[illegible]
2. Painted	0.012	0.013	0 0[illegible]
b. Corrugated	0.021	0.025	0 0[illegible]
B-2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0 0[illegible]
2. Mortar	0.011	0.013	0 0[illegible]
b. Wood			
1. Planed, untreated	0.010	0.012	0 0[illegible]
2. Planed, creosoted	0.011	0.012	0 0[illegible]
3. Unplaned	0.011	0.013	0 0[illegible]
4. Plank with battens	0.012	0.015	0 0[illegible]
5. Lined with roofing paper	0.010	0.014	0 0[illegible]
c. Concrete			
1. Trowel finish	0.011	0.013	0 0[illegible]
2. Float finish	0.013	0.015	0 0[illegible]
3. Finished, with gravel on bottom	0.015	0.017	0 0[illegible]
4. Unfinished	0.014	0.017	0 0[illegible]
5. Gunite, good section	0.016	0.019	0 0[illegible]
6. Gunite, wavy section	0.018	0.022	0 0[illegible]
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0 0[illegible]
2. Random stone in mortar	0.017	0.020	0 0[illegible]
3. Cement rubble masonry, plastered	0.016	0.020	0 0[illegible]
4. Cement rubble masonry	0.020	0.025	0 0[illegible]
5. Dry rubble or riprap	0.020	0.030	0 0[illegible]
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0 0[illegible]
2. Random stone in mortar	0.020	0.023	0 0[illegible]
3. Dry rubble or riprap	0.023	0.033	0 0[illegible]
f. Brick			
1. Glazed	0.011	0.013	0 0[illegible]
2. In cement mortar	0.012	0.015	0 0[illegible]
g. Masonry			
1. Cemented rubble	0.017	0.025	0 0[illegible]
2. Dry rubble	0.023	0.032	0 0[illegible]
h. Dressed ashlar	0.013	0.015	0 0[illegible]
i. Asphalt			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
j. Vegetal lining	0.030	.....	0 5[illegible]

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT *n* (*continued*)

Type of channel and description	Minimum	Normal	Maximum
C. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
D. NATURAL STREAMS			
D-1. Minor streams (top width at flood stage <100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150

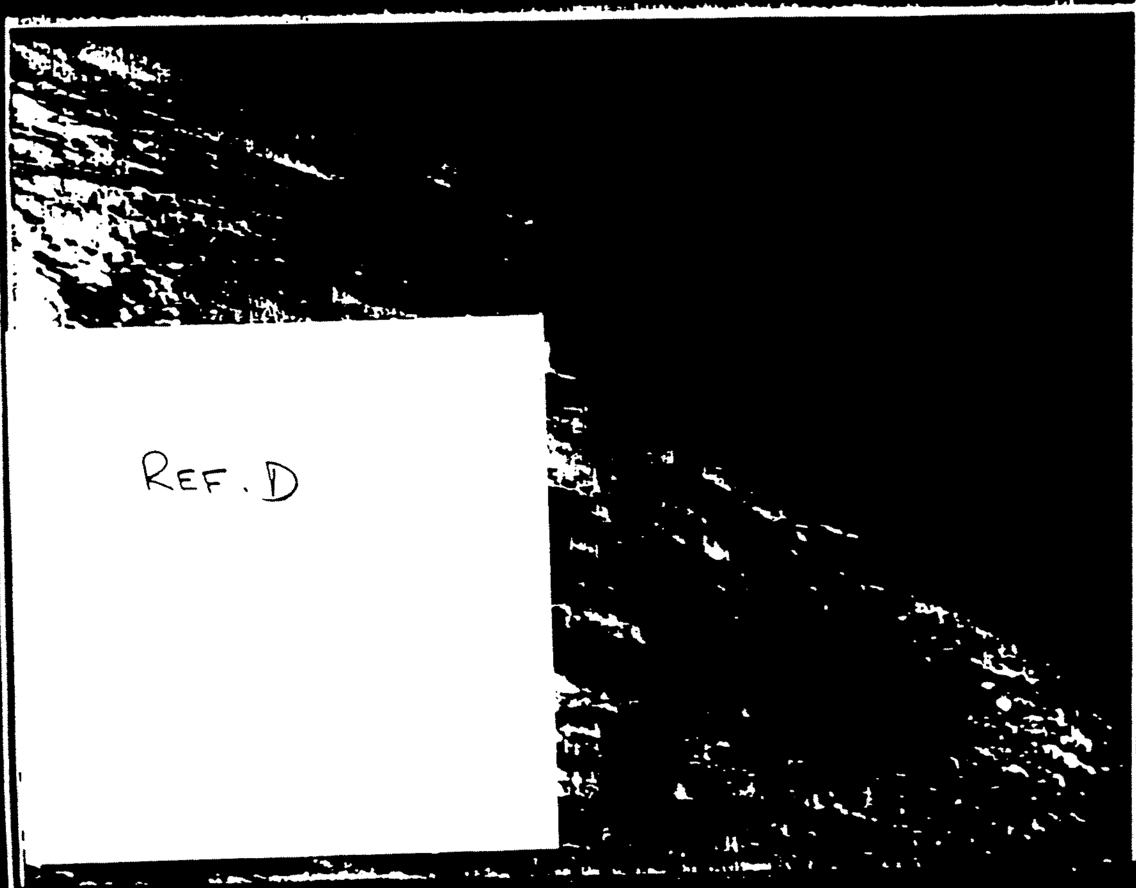
TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT *n* (*continued*)

Type of channel and description	Minimum	Normal	Ma[illegible]
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	[illegible]
2. Bottom: cobbles with large boulders	0.040	0.050	[illegible]
D-2. Flood plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	[illegible]
2. High grass	0.030	0.035	[illegible]
b. Cultivated areas			
1. No crop	0.020	0.030	[illegible]
2. Mature row crops	0.025	0.035	[illegible]
3. Mature field crops	0.030	0.040	[illegible]
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	[illegible]
2. Light brush and trees, in winter	0.035	0.050	[illegible]
3. Light brush and trees, in summer	0.040	0.060	[illegible]
4. Medium to dense brush, in winter	0.045	0.070	[illegible]
5. Medium to dense brush, in summer	0.070	0.100	[illegible]
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	[illegible]
2. Cleared land with tree stumps, no sprouts	0.030	0.040	[illegible]
3. Same as above, but with heavy growth of sprouts	0.050	0.060	[illegible]
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	[illegible]
5. Same as above, but with flood stage reaching branches	0.100	0.120	[illegible]
D-3. Major streams (top width at flood stage >100 ft). The *n* value is less than that for minor streams of similar description, because banks offer less effective resistance			
a. Regular section with no boulders or brush	0.025		[illegible]
b. Irregular and rough section	0.035		[illegible]

SECOND EDITION

ENGINEERING FLUID MECHANICS

John J. Bertin



REF. D

TABLE 9.2 VALUES OF n FOR VARIOUS STAGES OF THE NISHNABOTNA RIVER FOR THE AVERAGE GROWING SEASON

		Floodplain cover				
Depth of water, ft	Channel section	Corn	Pasture	Meadow	Small grains	Brush and waste
Under 1	0.03	0.06	0.05	0.10	0.10	0.12
1 to 2	0.03	0.06	0.05	0.08	0.09	0.11
2 to 3	0.03	0.07	0.04	0.07	0.08	0.10
3 to 4	0.03	0.07	0.04	0.06	0.07	0.09
Over 4	0.03	0.06	0.04	0.05	0.06	0.08

Source: From *Open Channel Hydraulics* by Ven Te Chow. Copyright © 1959 by McGraw-Hill Book Company. Used with the permission of McGraw-Hill Book Company.

If the discharge is so high that the stream overflows its banks, a portion of [the] flow will be along the floodplain. The n value of the floodplain is generally greater [than] that of the channel proper, the exact value depending on surface conditions and v[ege]tation, as illustrated in Table 9.2.

The corresponding formulas for the average velocity and the volumetric flow are

$$V_{av} = \frac{1.49}{n}[R_h\ (\mathrm{ft})]^{2/3}S_0^{0.5} = \frac{1.0}{n}[R_h\ (\mathrm{m})]^{2/3}S_0^{0.5} \qquad (9.$$

and

$$\boxed{Q = \frac{1.49}{n}A[R_h\ (\mathrm{ft})]^{2/3}S_0^{0.5}} = \frac{1.0}{n}A[R_h\ (\mathrm{m})]^{2/3}S_0^{0.5} \qquad (9.$$

respectively.

Example 9.1

Use both the Manning formula and the friction factor analysis to calculate the disch[arge] ... water flowing in a trapezoidal channel. Using the nomenclature of Fig. 9.8, y_n = ... $b = 4\ m$, and $\phi = 50°$. The floor of the channel slopes at 0.5°. The lining of the c[hannel] is finished concrete.

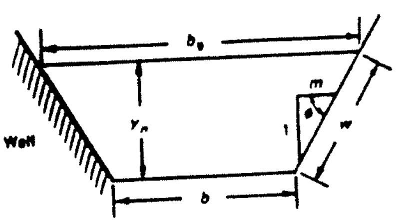


Figure 9.8 Nomenclature for a trapezoidal section.

REF E.

URS CONSULTANTS, INC.

PROJECT LOCKPORT

SUBJECT DRAINAGE

PAGE ... OF ...
SHEET NO. 1 OF ...
JOB NO.
MADE BY MW DATE 3/10/9
CHKD. BY MH DATE 7-6-9

REF. PAGE

SUMMARY

The following results were obtained from the drainage calculations for the Lockport City Landfill Cap

- See sheet 8 for the locations of the drainage structures
- Perimeter ditch S1
  - Flow $Q \approx 1.6\ ft^3/sec$
  - Depth of Flow $y \approx 0.21\ ft$
  - Flow Velocity $V \approx 2.9\ ft/sec$
  - Surface Grass

  Trapezoidal channel, 2ft bottom, 1V:3H side slopes
- Perimeter ditch S2
  - Flow $Q = 1.6\ ft^3/sec$
  - Depth of Flow $y = 0.18$ min ; $0.30\ ft$ max
  - Flow Velocity $V = 1.8$ min ; $3.5\ ft/sec$ max
  - Surface Grass

  Trapezoidal channel, 2ft bottom, 1V:3H side slopes
- Swale S3
  - Flow $Q = 27\ ft^3/sec$
  - Depth of Flow $y = 0.36 ft$ min ; $0.57 ft$ max
  - Flow Velocity $V = 2.1\ ft/sec$ min ; $6.9\ ft/s$ max
  - Surface: Downchute part - rip-rap; Flat part - grass

  Triangular channel, flat part 1:3 & 1:5, downchute part 1:3 & 1:3

Note: For a 5yr time of concentration storm for Buffalo, NY

REF E.

, 21

TABLE 6.8

Typical C Coefficients for 5- to 10-yr Frequency Design*

DESCRIPTION OF AREA	RUNOFF COEFFICIENTS
Business	
Downtown areas	0.70–0.95
Neighborhood areas	0.50–0.70
Residential	
Single-family areas	0.30–0.50
Multiunits, detached	0.40–0.60
Multiunits, attached	0.60–0.75
Residential (suburban)	0.25–0.40
Apartment dwelling areas	0.50–0.70
Industrial	
Light areas	0.50–0.80
Heavy areas	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yard areas	0.20–0.40
Unimproved areas	0.10–0.30
Streets	
Asphalt	0.70–0.95
Concrete	0.80–0.95
Brick	0.70–0.85
Drives and walks	0.75–0.85
Roofs	0.75–0.95
Lawns, Sandy Soil	
Flat, 2%	0.05–0.10
Average, 2–7%	0.10–0.15
Steep, 7%	0.15–0.20
Lawns, Heavy Soil	
Flat, 2%	0.13–0.17
Average, 2–7%	0.18–0.22 AVG = 0.2
Steep, 7%	0.25–0.35

* Viessman et al., 1977.

To avoid the complications of an iterative process, constant overland flow inlet times are often used to approximate the time of concentration. These vary from 5 to 30 min with 5 to 15 min most commonly used (American Society of Civil Engineers and Water Pollution Control Feder-

APPENDIX B

STEEP SLOPE AREA SUPPORT DATA

B.1 - EPTOX RESULTS

B.2 - ANALYTICAL SOIL RESULTS

B.3 - RUNOFF/INFILTRATION ANALYSIS

B.4 - SLOPE STABILITY

APPENDIX B.1

EPTOX RESULTS

TABLE 6-9
LOCKPORT CITY LANDFILL
SOIL BORINGS AND WASTE SAMPLES EPTOX RESULTS AND SCGs

SAMPLE-ID		SB-1	SB-5	SB-9	SB-13	SB14	SB-18	SB-19	WS-1	WS-2	WS-3	SCG	SOURCE
ROUND		1st	1st	1st	1st	1st	1st	1st	1st	1st	1st	VALUE	
PARAMETER	TYPE												
CORROSIVITY	MISC	NON	NON	NON	NON	NON	NON	NON	NON	NON	NON		
IGNITABILITY	MISC	NON	NON	NON	NON	NON	NON	NON	NON	NON	NON		
REACTIVITY	MISC	NON	NON	NON	NON	NON	NON	NON	NON	NON	NON		
pH, S.U.	MISC	8.36	8.88	8.08	7.30	7.65	7.82	7.83	7.90	7.54	7.14		
SULFUR (%)	MISC	0.13	0.11	0.15	0.16	0.14	0.16	0.56	0.019	0.010	0.010		
CHLORINE (PPM)	MISC		0.030						137		109		
ASH WEIGHT(%)	MISC	61.30	52.91	54.25	56.03	54.47	63.37	53.51	61.68	70.52	32.41		
HEAT OF COMBUSTION (BTU/lb	MISC									443	3,046		
EP Tox - ARSENIC	MCP					18.5	26.4				22	5000	EPTox
EP Tox - BARIUM	MCP	456	610	625	656	530	985	704	1070	1000	293	100000	EPTox
EP Tox - CADMIUM	MCP	32.9	4.4 B	1.7 B	75.1	27.8	26.1	12.1	38.3	6.9		1000	EPTox
EP Tox - CHROMIUM	MCP		4.3 B	3 B		8.5 B	10.7	3.3 B			14.6	5000	EPTox
EP Tox - LEAD	MCP	86.4		28.6	1990	130	279	231	496	14.1	1190	5000	EPTox
EP Tox - MERCURY	MCP											200	EPTox
EP Tox - SELENIUM	MCP											1000	EPTox
EP Tox - SILVER	MCP											5000	EPTox
EP Tox - Lindane	PEST											400	EPTox
EP Tox - Endrin	PEST											20	EPTox
EP Tox - Methoxychlor	PEST											10000	EPTox
EP Tox - Toxaphene	PEST											500	EPTox
EP Tox - 2,4-D	HERB											10000	EPTox
EP Tox - Silvex	HERB											1000	EPTox
WATER REACTIVITY	WASTE			NEG	NEG	NEG	NEG	NEG	NEG	NEG	NEG		
HEXANE SOLUBILITY	WASTE			NEG	NEG	NEG	NEG	NEG	NEG	NEG	NEG		
WATER SOLUBILITY	WASTE			NEG	NEG	NEG	NEG	NEG	NEG	NEG	NEG		
pH	WASTE			8.08	7.30	7.65	7.82	7.83	7.90	7.54	7.14		
FLAMMABILITY	WASTE			NEG	NEG	NEG	NEG	NEG	NEG	NEG	NEG		
RADIOACTIVITY(mR/h)	WASTE												
S/CN RELEASE	WASTE			NEG	NEG	NEG	NEG	NEG	NEG	NEG	NEG		
BEILESTEIN TEST	WASTE			NEG	NEG	NEG	NEG	NEG	NEG	NEG	NEG		
PEROXIDE TEST(mg/Kg)	WASTE												
REDOX POTENTIAL(mV)	WASTE			-48.4	-24.9	-58.1	-30.7	-33.2	-88.3	-104	90.1		

All results reported in μg/L (ppb) unless otherwise specified.
Only detected results reported.

NA - Not Analyzed
NEG - No reaction
NON - Non-corrosive, non-reactive, non-ignitable
B - Less than quantitation limit but greater than or equal to instrument detection limit.

APPENDIX B.2

ANALYTICAL SOIL RESULTS

LOCKPORT CITY LANDFILL: SURFICIAL SOILS

Analytical Results

SAMPLE-ID		SPS-1	SPS-2	SPS-2-DL	SPS-3	SPS-3RE	SPS-4	SPS-5	NYSDEC Recommended Soil Cleanup Objective (ppb)
COLLECTION DATE		12/8/89	12/8/89	12/8/89	12/8/89	12/8/89	12/11/89	12/12/89	
PARAMETER	TYPE								
4-NITROPHENOL	SEMI								
DIBENZOFURAN	SEMI		240 J	230 JD	180 J	170 J			6,200
2,4-DINITROTOLUENE	SEMI								
DIETHYLPHTHALATE	SEMI		33 J						7,100
4-CHLOROPHENYL-PHENYL ETHER	SEMI								
FLUORENE	SEMI	43 J	430 J	510 JD	280 J	300 J			50,000
4-NITROANILINE	SEMI								
4,6-DINITRO-2-METHYLPHENOL	SEMI								
N-NITROSODIPHENYLAMINE	SEMI								
4-BROMOPHENYL-PHENYL ETHER	SEMI								
HEXACHLOROBENZENE	SEMI								
PENTACHLOROPHENOL	SEMI						320 J		1,000
PHENANTHRENE	SEMI	350 J	4,800	6,000 D	3,100	3,000	370 J	160 J	50,000
ANTHRACENE	SEMI	97 J	760	890 JD	680 J	770	77 J	36 J	50,000
DI-N-BUTYLPHTHALATE	SEMI	R	3,700 B	6,000 BD	R	R	R	R	8,100
FLUORANTHENE	SEMI	800	3,900	8,700 D	5,000	4,300	850	400 J	50,000
PYRENE	SEMI	420	10,000 E	7,200 D	5,800	4,200	730	350 J	50,000
BUTYLBENZYLPHTHALATE	SEMI				130 J				50,000
3,3'-DICHLOROBENZIDINE	SEMI								
BENZO(A)ANTHRACENE	SEMI	310 J	4,200	5,000 D	2,900	3,000	300 J	180 J	220
CHRYSENE	SEMI	310 J	4,100	4,800 D	2,800	2,700	320 J	190 J	400
BIS(2-ETHYLHEXYL)PHTHALATE	SEMI	680 B	3,900 B	4,000 BD	1,900 B	1,700 B	R	1,400 B	50,000
DI-N-OCTYL PHTHALATE	SEMI	58 J	260 J					88 J	50,000
BENZO(B)FLUORANTHENE	SEMI	320 J	4,600	5,900 D	3,500	3,800	390 J	260 J	1,100
BENZO(K)FLUORANTHENE	SEMI	300 J		790 JD	3,200	2,200		220 J	1,100
BENZO(A)PYRENE	SEMI	280 J	3,700	4,100 D	2,700	2,400	280 J	180 J	61
INDENO(1,2,3-CD)PYRENE	SEMI		1,700	2,100 D	1,600	1,200			3,200
DIBENZ(A,H)ANTHRACENE	SEMI		1,200			200 J			14
BENZO(G,H,I)PERYLENE	SEMI		3,300	1,800 D	830	1,100			50,000

All results reported in μg/kg (ppb).
Only detected results are reported.

J – Indicates the result is less than the sample quantitation limit but greater than zero.
R – Analyte rejected due to blank contamination (See Appendix O).
D – Indicates compounds were analyzed at a secondary dilution factor.
B – Analyte detected in associated method blank.
E – Compound concentration exceeded the calibration range for that specific analysis.

LOCKPORT CITY LANDFILL: SURFICIAL SOILS

Analytical Results

SAMPLE-ID		SPS-1	SPS-2	SPS-2-DL	SPS-3	SPS-3RE	SPS-4	SPS-5	NYSDEC Recommended Soil Cleanup Objective (ppb)
COLLECTION DATE		12/8/89	12/8/89	12/8/89	12/8/89	12/8/89	12/11/89	12/12/89	
PARAMETER	TYPE								
PHENOL	SEMI								
BIS(2-CHLOROETHYL)ETHER	SEMI								
2-CHLOROPHENOL	SEMI								
1,3-DICHLOROBENZENE	SEMI								
1,4-DICHLOROBENZENE	SEMI								
BENZYL ALCOHOL	SEMI								
1,2-DICHLOROBENZENE	SEMI								
2-METHYLPHENOL	SEMI								
BIS(2-CHLOROISOPROPYL)ETHER	SEMI								
4-METHYLPHENOL	SEMI								
N-NITROSO-DI-N-PROPYLAMINE	SEMI								
HEXACHLOROETHANE	SEMI								
NITROBENZENE	SEMI								
ISOPHORONE	SEMI								
2-NITROPHENOL	SEMI								
2,4-DIMETHYLPHENOL	SEMI								
BENZOIC ACID	SEMI		420 J	510 JD	220 J	280 J	77 J	70 J	— NA
BIS(2-CHLOROETHOXY)METHANE	SEMI								
2,4-DICHLOROPHENOL	SEMI								
1,2,4-TRICHLOROBENZENE	SEMI								
NAPHTHALENE	SEMI		150 J	160 JD	160 J	160 J			13,000
4-CHLOROANILINE	SEMI								
HEXACHLOROBUTADIENE	SEMI								
4-CHLORO-3-METHYLPHENOL	SEMI								
2-METHYLNAPHTHALENE	SEMI		180 J	180 JD	150 J	150 J			36,400
HEXACHLOROCYCLOPENTADIENE	SEMI								
2,4,6-TRICHLOROPHENOL	SEMI								
2,4,5-TRICHLOROPHENOL	SEMI								
2-CHLORONAPHTHALENE	SEMI								
2-NITROANILINE	SEMI								
DIMETHYLPHTHALATE	SEMI		1,300	1,400 D					2,000
ACENAPHTHYLENE	SEMI	120 J	210 J	180 JD	430 J	280 J			41,000
2,6-DINITROTOLUENE	SEMI								
3-NITROANILINE	SEMI								
ACENAPHTHENE	SEMI		410 J	420 JD	180 J	200 J			50,000
2,4-DINITROPHENOL	SEMI								

All results reported in μg/kg (ppb).
Only detected results are reported.

J – Indicates the result is less than the sample quantitation limit but greater than zero.
D – Indicates compounds were analyzed at a secondary dilution factor.

NA – No NYSDEC guidance value available

LOCKPORT CITY LANDFILL: SURFICIAL SOILS

Analytical Results

SAMPLE-ID		SPS-1	SPS-2	SPS-2-DL	SPS-3	SPS-3RE	SPS-4	SPS-5	NYSDEC Recommended Soil Cleanup Objective (ppb)
COLLECTION DATE		12/8/89	12/8/89	12/8/89	12/8/89	12/8/89	12/11/89	12/12/89	
PARAMETER	TYPE								
ALPHA-BHC	PST			NA		NA			
BETA-BHC	PST			NA		NA			
DELTA-BHC	PST			NA		NA			
GAMMA-BHC (LINDANE)	PST			NA		NA			
HEPTACHLOR	PST			NA		NA			
ALDRIN	PST			NA	67	NA	20		41
HEPTACHLOR EPOXIDE	PST			NA		NA			
ENDOSULFAN I	PST			NA		NA			
DIELDRIN	PST			NA		NA			
4,4'-DDE	PST			NA		NA			
ENDRIN	PST			NA		NA			
ENDOSULFAN II	PST			NA		NA			
4,4'-DDD	PST			NA		NA			
ENDOSULFAN SULFATE	PST			NA		NA			
4,4'-DDT	PST			NA		NA			
METHOXYCHLOR	PST			NA		NA			
ENDRIN KETONE	PST			NA		NA			
ALPHA-CHLORDANE	PST			NA		NA			
GAMMA-CHLORDANE	PST	7.9 J		NA		NA			540
TOXAPHENE	PST			NA		NA			
AROCLOR-1016	PCB			NA		NA			
AROCLOR-1221	PCB			NA		NA			
AROCLOR-1232	PCB			NA		NA			
AROCLOR-1242	PCB			NA		NA			
AROCLOR-1248	PCB		9,300	NA		NA			1,000 *
AROCLOR-1254	PCB	36 J		NA	1,900	NA	170 J	160 J	1,000 *
AROCLOR-1260	PCB			NA		NA			

All results reported in μg/kg (ppb).
Only detected results are reported.

J – Indicates the realut is less than the sample quantitation limit but greater than zero.
NA – Not Analyzed

* guidance value for total PCBs

APPENDIX B.3

RUN-OFF/INFILTRATION ANALYSIS

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 LOCKPORT CITY LANDFILL INFILTRATION ANALYSIS
 BY M.O., AUG 17, 1993
 RUN #1: 6" SOIL #8(VPL), 240" SOIL #18(VPL)

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                              EXCELLENT GRASS

                                 LAYER  1
                                 --------

                        VERTICAL PERCOLATION LAYER
   THICKNESS                              =       6.00 INCHES
   POROSITY                               =       0.4630 VOL/VOL
   FIELD CAPACITY                         =       0.2320 VOL/VOL
   WILTING POINT                          =       0.1157 VOL/VOL
   INITIAL SOIL WATER CONTENT             =       0.2614 VOL/VOL
   SATURATED HYDRAULIC CONDUCTIVITY       =       0.001850000001 CM/SEC


                                 LAYER  2
                                 --------

                        VERTICAL PERCOLATION LAYER
   THICKNESS                              =     240.00 INCHES
   POROSITY                               =       0.5200 VOL/VOL
   FIELD CAPACITY                         =       0.2942 VOL/VOL
   WILTING POINT                          =       0.1400 VOL/VOL
   INITIAL SOIL WATER CONTENT             =       0.3300 VOL/VOL
   SATURATED HYDRAULIC CONDUCTIVITY       =       0.000199999995 CM/SEC


                          GENERAL SIMULATION DATA
                          -----------------------

   SCS RUNOFF CURVE NUMBER                =       69.27
   TOTAL AREA OF COVER                    =    43560. SQ FT
   EVAPORATIVE ZONE DEPTH                 =       36.00 INCHES
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UPPER LIMIT VEG. STORAGE = 18.3780 INCHES
INITIAL VEG. STORAGE = 12.3318 INCHES
SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

SYNTHETIC RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND SOLAR RADIATION FOR BUFFALO NEW YORK

MAXIMUM LEAF AREA INDEX = 5.00
START OF GROWING SEASON (JULIAN DATE) = 138
END OF GROWING SEASON (JULIAN DATE) = 279

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
23.50	24.50	33.00	45.40	56.10	66.00
70.70	68.90	62.10	51.50	40.30	28.80

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.06	2.24	2.73	2.91	2.87	2.00
	2.91	3.92	3.01	3.10	4.02	3.17
STD. DEVIATIONS	0.91	0.97	0.86	0.87	0.88	0.75
	1.75	2.03	1.31	1.31	1.20	0.89
RUNOFF						
TOTALS	0.000	0.000	0.004	0.000	0.000	0.000
	0.000	0.002	0.000	0.002	0.000	0.000
STD. DEVIATIONS	0.001	0.000	0.014	0.000	0.000	0.000
	0.000	0.007	0.000	0.011	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.402	0.635	2.068	3.233	3.136	5.450
	2.782	3.516	2.781	1.813	0.824	0.455
STD. DEVIATIONS	0.113	0.136	0.239	0.600	0.971	0.748
	1.236	1.350	0.791	0.380	0.084	0.100

PERCOLATION FROM LAYER 2

TOTALS	0.6631	0.7448	1.0365	1.0733	1.0191	0.8660
	0.7712	0.6707	0.5751	0.5335	0.4730	0.5246
STD. DEVIATIONS	0.2606	0.2927	0.4079	0.4441	0.3718	0.2721
	0.2116	0.1635	0.1267	0.1074	0.0890	0.1664

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	35.95	(4.196)	130488.	100.00
RUNOFF	0.008	(0.019)	31.	0.02
EVAPOTRANSPIRATION	27.096	(3.033)	98357.	75.38
PERCOLATION FROM LAYER 2	8.9509	(2.6721)	32492.	24.90
CHANGE IN WATER STORAGE	-0.108	(2.711)	-392.	-0.30

PEAK DAILY VALUES FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)
PRECIPITATION	2.81	10200.3
RUNOFF	0.063	229.3
PERCOLATION FROM LAYER 2	0.0783	284.3
SNOW WATER	3.42	12422.8

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4036
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1359

FINAL WATER STORAGE AT END OF YEAR 20

LAYER	(INCHES)	(VOL/VOL)
1	1.59	0.2652
2	77.06	0.3211
SNOW WATER	0.00	

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 LOCKPORT CITY LANDFILL INFILTRATION ANALYSIS
 BY M.O., AUG 17, 1993
 RUN #2: 12" SOIL #8(VPL), 240" SOIL #18(VPL)

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EXCELLENT GRASS

LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4630 VOL/VOL
FIELD CAPACITY	=	0.2320 VOL/VOL
WILTING POINT	=	0.1157 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2631 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.001850000001 CM/SEC

LAYER 2

VERTICAL PERCOLATION LAYER

THICKNESS	=	240.00 INCHES
POROSITY	=	0.5200 VOL/VOL
FIELD CAPACITY	=	0.2942 VOL/VOL
WILTING POINT	=	0.1400 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3302 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000199999995 CM/SEC

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	69.27
TOTAL AREA OF COVER	=	43560. SQ FT
EVAPORATIVE ZONE DEPTH	=	36.00 INCHES

UPPER LIMIT VEG. STORAGE = 18.0360 INCHES
INITIAL VEG. STORAGE = 11.7939 INCHES
SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

SYNTHETIC RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND SOLAR RADIATION FOR BUFFALO NEW YORK

MAXIMUM LEAF AREA INDEX = 5.00
START OF GROWING SEASON (JULIAN DATE) = 138
END OF GROWING SEASON (JULIAN DATE) = 279

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
23.50	24.50	33.00	45.40	56.10	66.00
70.70	68.90	62.10	51.50	40.30	28.80

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.06	2.24	2.73	2.91	2.87	2.00
	2.91	3.92	3.01	3.10	4.02	3.17
STD. DEVIATIONS	0.91	0.97	0.86	0.87	0.88	0.75
	1.75	2.03	1.31	1.31	1.20	0.89
RUNOFF						
TOTALS	0.000	0.000	0.003	0.000	0.000	0.000
	0.000	0.002	0.000	0.002	0.000	0.000
STD. DEVIATIONS	0.001	0.000	0.010	0.000	0.000	0.000
	0.000	0.007	0.000	0.009	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.403	0.637	2.076	3.236	3.154	5.221
	2.698	3.556	2.787	1.809	0.828	0.457
STD. DEVIATIONS	0.113	0.138	0.241	0.608	0.966	0.722
	1.238	1.394	0.815	0.388	0.085	0.102

PERCOLATION FROM LAYER 2

TOTALS	0.6968	0.7756	1.0689	1.0948	1.0354	0.8788
	0.7836	0.6830	0.5867	0.5448	0.4854	0.5483
STD. DEVIATIONS	0.2648	0.2948	0.4076	0.4383	0.3652	0.2672
	0.2082	0.1613	0.1252	0.1063	0.0891	0.1742

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	35.95	(4.196)	130488.	100.00
RUNOFF	0.006	(0.015)	23.	0.02
EVAPOTRANSPIRATION	26.863	(3.075)	97512.	74.73
PERCOLATION FROM LAYER 2	9.1821	(2.6536)	33331.	25.54
CHANGE IN WATER STORAGE	-0.104	(2.693)	-379.	-0.29

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)
PRECIPITATION	2.81	10200.3
RUNOFF	0.044	161.4
PERCOLATION FROM LAYER 2	0.0787	285.9
SNOW WATER	3.42	12419.5

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.3872

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1318

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FINAL WATER STORAGE AT END OF YEAR 20

LAYER	(INCHES)	(VOL/VOL)
1	3.34	0.2780
2	77.03	0.3210
SNOW WATER	0.00	

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LOCKPORT CITY LANDFILL INFILTRATION ANALYSIS
BY M.O., AUG 17, 1993
RUN #3: 18" SOIL #8(VPL), 240" SOIL #18(VPL)

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EXCELLENT GRASS

LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	18.00 INCHES
POROSITY	=	0.4630 VOL/VOL
FIELD CAPACITY	=	0.2320 VOL/VOL
WILTING POINT	=	0.1157 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2658 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.001850000001 CM/SEC

LAYER 2

VERTICAL PERCOLATION LAYER

THICKNESS	=	240.00 INCHES
POROSITY	=	0.5200 VOL/VOL
FIELD CAPACITY	=	0.2942 VOL/VOL
WILTING POINT	=	0.1400 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3302 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000199999995 CM/SEC

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	69.27
TOTAL AREA OF COVER	=	43560. SQ FT
EVAPORATIVE ZONE DEPTH	=	36.00 INCHES

UPPER LIMIT VEG. STORAGE = 17.6940 INCHES
INITIAL VEG. STORAGE = 11.2627 INCHES
SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

SYNTHETIC RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND SOLAR RADIATION FOR BUFFALO NEW YORK

MAXIMUM LEAF AREA INDEX = 5.00
START OF GROWING SEASON (JULIAN DATE) = 138
END OF GROWING SEASON (JULIAN DATE) = 279

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
23.50	24.50	33.00	45.40	56.10	66.00
70.70	68.90	62.10	51.50	40.30	28.80

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.06	2.24	2.73	2.91	2.87	2.00
	2.91	3.92	3.01	3.10	4.02	3.17
STD. DEVIATIONS	0.91	0.97	0.86	0.87	0.88	0.75
	1.75	2.03	1.31	1.31	1.20	0.89
RUNOFF						
TOTALS	0.000	0.000	0.002	0.000	0.000	0.000
	0.000	0.002	0.000	0.002	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.008	0.000	0.000	0.000
	0.000	0.007	0.000	0.009	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.404	0.639	2.088	3.234	3.159	4.972
	2.646	3.590	2.793	1.821	0.835	0.460
STD. DEVIATIONS	0.114	0.141	0.242	0.623	0.953	0.678
	1.209	1.440	0.825	0.396	0.086	0.103

PERCOLATION FROM LAYER 2

TOTALS	0.7303	0.8059	1.0986	1.1128	1.0479	0.8890
	0.7942	0.6939	0.5970	0.5552	0.4978	0.5743
STD. DEVIATIONS	0.2696	0.2971	0.4066	0.4321	0.3579	0.2617
	0.2045	0.1589	0.1236	0.1051	0.0898	0.1827

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	35.95	(4.196)	130488.	100.00
RUNOFF	0.006	(0.013)	20.	0.02
EVAPOTRANSPIRATION	26.642	(3.123)	96709.	74.11
PERCOLATION FROM LAYER 2	9.3969	(2.6348)	34111.	26.14
CHANGE IN WATER STORAGE	-0.097	(2.711)	-353.	-0.27

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)
PRECIPITATION	2.81	10200.3
RUNOFF	0.038	138.9
PERCOLATION FROM LAYER 2	0.0793	287.9
SNOW WATER	3.42	12414.8

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3723
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1278

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FINAL WATER STORAGE AT END OF YEAR 20

LAYER	(INCHES)	(VOL/VOL)
1	5.06	0.2808
2	77.09	0.3212
SNOW WATER	0.00	

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APPENDIX B.4

SLOPE STABILITY ANALYSIS

URS CONSULTANTS, INC.

PROJECT Lockport City Landfill
SUBJECT Slope Stability Analysis
Preliminary Results

SHEET NO. OF
JOB NO. 35180.02
MADE BY F.M. DATE 8/26/93
CHKD. BY MSA DATE 8/30/93

REF. PAGE

Wedge Method (NAFAC DM-7.1)

GWL at El + 490 (dry slope)

Wedge 1: $\phi = 34°$ $c = 0$ $\gamma_T = 0.08$ kcf $\alpha_1 = 62°$

$$\omega_1 = \frac{24}{2} \times 24 \tan(28) \times 0.08 = 12.3^{k/l}$$

$$P_{\alpha_1} = 12.3 \times \frac{1.88 - 0.68/F_s}{1 + 1.28/F_s} = \frac{23.1F_s - 8.4}{F_s + 1.28}$$

Wedges 2 & 3: $\phi = 30°$ $c = 0$ $\gamma = 0.08$ kcf $\alpha_2 = 14°$

$$\left.\begin{aligned} \omega_2 &= \frac{24+16}{2} \times 35 \times 0.08 = 56^{k/l} \\ \omega_3 &= \frac{16}{2} \times 35 \times 0.08 = 22^{k/l} \end{aligned}\right\} 78^{k/l}$$

$$P_{\alpha_2} = 78 \tan(14) - 78 \cos(14) \frac{\tan(30)/F_s}{\cos(14)}$$

$$P_{\alpha_2} = 19.5 - 45/F_s$$

Factor of Safety, F_s (for $\Sigma P_\alpha = 0$)

$$\frac{23.1F_s - 8.4}{F_s + 1.28} + 19.5 - 45/F_s = 0$$

F_s	ΣP_α
1.0	-19
1.6	-1

$\therefore F_s = 1.55$

PROJECT Lockport City Landfill
SUBJECT Slope Stability Analysis
Preliminary Results

REF. PAGE

GWL at El. +515 (seepage parallel to slope)

Wedge 1: $\phi = 34$ $c = 0$ $\gamma_T = 0.08$ kcf $\alpha_1 = 62°$

$$W_1 = \frac{24}{2} \times 24 \tan(28) \times 0.08 = 12.3 \ k/l$$

$$P_{w_1} = \frac{7}{2} \times 0.062 \times \frac{7}{\sin(62)} = 1.7 \ k/l$$

$$P_{\alpha_1} = (12.3 - 0.8) \ \frac{1.88 - 0.68/F_s}{1 + 1.28/F_s} + 1.7 \sin(62)$$

$$P_{\alpha_1} = \frac{21.6 F_s - 7.8}{F_s + 1.28} + 1.5$$

Wedge 2 $\phi = 30$ $c = 0$ $\gamma_T = 0.08$ kcf $\gamma_{SAT} = 0.095$ kcf
$\alpha_2 = 14°$

$$W_2 = \frac{24+16}{2} \times 35 \times 0.08 = 56 \ k/l$$

$$W_{2_{sub}} = 16 \times 35 \times 0.08 + \frac{10}{2} \times 35 \times 0.032 = 45 + 6 = 51 \ k/l$$

$$P_{w_2} = \frac{8}{2} \times 0.062 \times 8 = 2.0 \ k/l$$

$$P_{\alpha_2} = 56 \tan(14) + 2.0 - 51 \cos(14) \frac{\tan(30)}{\cos(14)} / F_s$$

$$P_{\alpha_2} = 16 - 29.5/F_s$$

Wedge 3 $\phi = 30$ $c = 0$ $\gamma_T = 0.08$ kcf $\alpha_3 = 14°$

$$W_3 = \frac{1}{2} \times 16 \times 35 \times 0.08 = 22.4 \ k/l$$

$$P_{\alpha_3} = 22.4 \tan(14) - 22.4 \cos(14) \frac{\tan(30)}{\cos(14)} / F_s$$

$$P_{\alpha_3} = 5.6 - 12.9/F_s$$

Factor of Safety (for $\Sigma P_\alpha = 0$)

$$\frac{21.6 F_s - 7.8}{F_s + 1.28} + 23.1 - 42.4/F_s = 0$$

F_s	ΣP_α
1.0	-13.2
1.5	+3.6
1.4	+1.2

$\therefore$ $F_s = 1.38$

GWL at El. +515 (with Final Cap)

Wedge 1: $\phi = 34°$ $c = 0$ $\gamma_T = 0.08$ kcf $\alpha_1 = 62°$

$W_1 = \frac{1}{2} \times 18 \times 4 \times 0.125 + \frac{1}{2} \times 24 \times 24 \tan(28) \times 0.08$

$W_1 = 4.5 + 12.3 = 16.8$ k/l

$P_{W_1} = \frac{1}{2} \times 7 \times 0.062 \frac{7}{\sin(62)} = 1.7$ k/l

$P_{K_1} = (16.8 - 0.8) \frac{1.88 - 0.68/F_s}{1 + 1.28/F_s} + 1.7 \sin(62)$

$P_{K_1} = \frac{30.0 F_s - 10.9}{F_s + 1.28} + 1.5$

Wedge 2: $\phi = 30$ $c = 0$ $\gamma_T = 0.08$ kcf $\gamma_{SAT} = 0.095$ kcf $\alpha_2 = 14°$

$W_2 = \frac{24+16}{2} \times 45 \times 0.08 = 72$ k/l

$W_{2\,SUB} = 16 \times 25 \times 0.08 + 18 \times 20 \times 0.08 + \frac{10}{2} \times 45 \times 0.032 = 32 + 29 + 7 = 68$ k/l

$P_{W_2} = \frac{8}{2} \times 0.062 \times 8 = 2.0$ k/l

$P_{K_2} = 72 \tan(14) + 2.0 - 68 \cos(14) \frac{\tan(30)}{\cos(14)} / F_s$

$P_{K_2} = 20 - 39.3/F_s$

Wedge 3: $\phi = 30$ $c = 0$ $\gamma_T = 0.08$ kcf $\alpha_3 = 14°$

$W_3 = \frac{1}{2} \times 16 \times 35 \times 0.08 = 22.4$ k/l

$P_{K_3} = 22.4 \tan(14) - 22.4 \cos(14) \frac{\tan(30)}{\cos(14)} / F_s$

$P_{K_3} = 5.6 - 12.9/F_s$

Factor of Safety (for $\Sigma P_K = 0$)

$$\frac{30.0 F_s - 10.9}{F_s + 1.29} + 27.1 - 52.2/F_s = 0$$

F_s	ΣP_K
1.0	-16.7
1.5	+4.5
1.4	+1.4

$\therefore F_s = 1.38$

URS CONSULTANTS, INC.

PROJECT Lockport City Landfill
SUBJECT Slope Stability Analysis

PAGE 3A OF 3
SHEET NO. OF
JOB NO. 35180.02
MADE BY F.M DATE 8/26/93
CHKD. BY MSA DATE 8/30/93

REF. PAGE