FLORIDA DEPARTMENT OF ENVIRONMENTAL REGULATION SOLID WASTE MANAGEMENT FACILITY CONSTRUCTION PERMIT APPLICATION

FOR

SARASOTA COUNTY CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX

PREPARED FOR:

SARASOTA COUNTY 4058C 02034 W

PREPARED BY:

CAMP DRESSER & McKEE INC.
201 MONTGOMERY AVENUE
SARASOTA, FLORIDA 34243

JUNE, 1992

D. E. R.
JUN 15 1992
SOUTHWEST DISTRICT
TAMPA

SARASOTA COUNTY

SOLID WASTE MANAGEMENT FACILITY

CONSTRUCTION PERMIT APPLICATION

4058C02034 W

SARASOTA COUNTY GOVERNMENT
SARASOTA, FLORIDA

Solid Waste Operations Division

[redacted]
Manager
8350 Bee Ridge Road
Sarasota, Florida 34241
(813) 951-5096
FAX (813) 951-5161

D E R

June 11, 1992

JUN 15 1992

SOUTHWEST DISTRICT
TAMPA

Solid Waste Section
Department of Environmental Regulation
4520 Oak Fair Boulevard
Tampa, Florida 33610-7347

Gentlemen:

Subject: Central County Solid Waste Disposal Complex
Construction Permit Applications
Sarasota County Class I and III Landfill

Enclosed are four sets of permit applications for new Class I and Class III landfill facilities for Sarasota County. These facilities have been planned and designed to meet all FDER requirements in accordance with Chapter 17-701, F.A.C. The applications are accompanied with engineering documentation for your review and approval, as prepared by our consultant, Camp Dresser & McKee, Inc.

Sarasota County has invested considerable time and resources to complete these applications to the level of detail presented for FDER permitting evaluation. We believe the proposed facilities will provide all residents in Sarasota County a safe and environmentally sound solid waste disposal program for years to come. If you have any questions regarding this documentation, please call me for response. We appreciate your efforts in reviewing and permitting these important facilities.

Sincerely,

William K. Lederman, P.E.
Engineer IV

Enclosures
c: Robert L. LaSala, Sarasota County
Thomas G. Walker, P.E., Camp Dresser & McKee, Inc.

STATE OF FLORIDA

DEPARTMENT OF ENVIRONMENTAL REGULATION

TWIN TOWERS OFFICE BUILDING
2600 BLAIR STONE ROAD
TALLAHASSEE, FLORIDA 32301-8241

BOB GRAHAM
GOVERNOR

VICTORIA J. TSCHINKEL
SECRETARY

APPLICATION FOR PERMIT TO CONSTRUCT [X] OPERATE [ ]

A SOLID WASTE RESOURCE RECOVERY AND MANAGEMENT FACILITY

GENERAL REQUIREMENTS

Solid Waste Resource Recovery and Management Facilities shall be permitted pursuant to Section 403.707, Florida Statutes, and in accordance with Florida Administrative Code Rule 17-7. A minimum of six copies of the application shall be submitted to the Department District Office having jurisdiction over the facility. Complete appropriate sections for the type of facility for which application is made. Entries should be typed or printed in ink. All blanks should be filled in or marked not applicable. The application shall include all information, drawings, and reports necessary to evaluate the facility. Information required to support the application is listed on the attached pages of this form.

Facility Type: Existing ____ Proposed X

Sanitary Landfill:
- X Class I,
- __ Class II,
- X Class III: Trash/yard Trash
- __ Class III: Yard Trash Composting

Volume Reduction:
- __ Composting
- __ Shredder
- __ Incinerator/Trench Burner
- __ Resource Recovery: __ Energy __ Materials

Sludge Landspreading:
- __ Grade I
- __ Grade II
- __ Grade III
- __ Septage/Food Service

FACILITY NAME: Central County Solid Waste Disposal Complex / ____ (DER ID Number)

FACILITY LOCATION (main entrance): North end Knights Trail Road

S 1,2,3,4,9, 10,11,12, 13,14,15,16 (section), T 38S (township), R 19E (range) /Latitude 27 ° 12 ' 00 " Longitude 82 ° 23 ' 00 "

Applicant Name (operating authority): Sarasota County Solid Waste Department

Street Address & P. O. Box: 8350 Bee Ridge Road Sarasota, FL (City) Sarasota (County) 34241 (Zip)

Contact Person: William K. Lederman (Name) 813-951-5096 (Phone Number)

Authorized Agent/Consultant: Camp Dresser & McKee Inc. (Name) 813-351-7100 (Phone Number)

Contact Person: John Banks (Name) 201 Montgomery Avenue (Street) ____ (P. O. Box) 813-351-7100 (Phone Number)

Sarasota (City) Sarasota (County) FL (State) 34243 (Zip)

Landowner (if different than applicant): N/A

Address of Landowner: N/A (Street, P. O. Box) ____ (City) ____ (State) ____ (Zip)

Cities, Towns and Areas to be Served: All of Satasota County

Current and Projected Population to Served: 283,140 (1991) 475,000 (2025)

Acres within Waste Site Boundary: 310 Acres within Property Boundary: 6150 ±

Protecting Florida and Your Quality of Life

DER FORM 17-7.130(1) Effective 12/10/85

Volume of Solid Waste to be received: 1420 Ton/day ave. cu. yds/day tons/day gallons/day

Date Site Ready to Received Solid Waste: Jan. 15, 1995 Estimated Life of Facility 31 years

Estimated Cost of Construction, Total: $ 32,500,000 Estimated cost of Closing: $ 16,880,000

Anticipated Construction Starting and Completion Dates
From: June 1993 To: Dec. 30, 1994

REQUIRED ATTACHEMENTS FOR CONSTRUCTION/OPERATION PERMIT FOR A RESOURCE RECOVERY AND MANAGEMENT FACILITY

GENERAL
Permit application and supporting information shall include the following (17-7.030(2), F.A.C.):

	Completeness Check
1. A letter of transmittal to the Department; (17-7.030(3)(a) F.A.C.)	___
2. A table of contents listing the main sections of the application: (17-7.030(3)(b), F.A.C.)	___
3. The permit fee specified in Florida Aministrative Code Rule 17-4.05 in check or money order payable to the Department: (17-7.030(3)(c), F.A.C.)	___
4. Six copies, at minimum, of the completed application form, all supporting data, and reports; (17-7.030(2), F.A.C.)	___
5. Engineer seal; (17-7.030(2)(d), F.A.C.)	___
6. Engineer's letter of appointment if applicable; (17-7.030(3)(e), F.A.C)	___
7. Copy of any lease agreement, transfer of property agreement with right of entry for long-term care, or any other agreement between operator and property owner by which the closing and long-term care of the facility may be affected; (17-7.030(3)(h)	___
8. Proof of publication of notice of application for the proposed activity in a newspaper of general circulation; (17-7.03(4), F.A.C)	___

SPECIFICATION ATTACHMENT ITEMS

The following information items must be included in the application or an explanation given if they are not applicable.

Construction Permits:

A. Landfills - Submit items 1, 2, 3, 4, 5, 6, 7, 8, 10.
B. Volume Reduction - Submit items 1, 2, 3, 4, 5, 6, 7, 9, 10.
C. Sludge Landspreading - Submit items 2, 3, 4, 5, 6, 8, 10.

Operation Permits:

A. Landfills - All the items above.
B. Volume Reduction - All the items above.
C. Sludge Landspreading - All the items above.

NOTE: For facilities that have been satisfactorily constructed in accordance with their construction permit the information required for A, B, and C type facilities does not have to be resubmitted for an operation permit if the information has not changed during the construction period.

1. A foundation analysis (17-7.050(2)(b), F.A.C.)	___
2. Evidence that the facility is in conformance with local zoning (17-7.050(2)(c)4, F.A.C)	___

3. Facility Design (17-7.050(3), F.A.C.:

NOTE: All maps, plan sheets, drawings, isometrics, cross-sections, or aerial photographs shall be legible; be signed and sealed by the registered professional engineer responsible for their preparation; be of appripriate scale to show clearly all required details; be numbered, referenced to narrative, titled, have a legend of symbols used, contain horizontal and vertical scales (where applicable), and specify drafting or origination dates; and use uniform scales as much as possible, contain a north arrow, and use NGVD for all elevations.

Completeness Check

a. A map or aerial photograph of the area, no more than 1 year old, showing land use and zoning within 1 mile of the facility. (17-7.050(3)(a), F.A.C.) ___

b. Plot Plan (17-7.050(3)(b), F.A.C.) ___

NOTE: The plot plan on a scale not greater than 200 feet to the inch showing the following:

(1) Dimensions and Legal Description of the site ___

(2) Location and depth (NGVD) of soil borings ___

(3) Plan for trenching or disposal areas ___

(4) Fencing or other measures to restrict access ___

(5) Cross sections showing both original and propsed fill elevations ___

(6) Location, depth, and construction details of monitoring wells ___

c. Topographic Maps (17-7.050(3)(c), F.A.C.) ___

NOTE: The topographic maps, which may be combined with the plot plan (item 4b), on a scale not greater than 200 feet to the inch showing the following:

(1) Five foot contour intervals ___

(2) Proposed fill areas ___

(3) Borrow areas ___

(4) Access roads ___

(5) Grades required for proper drainage ___

(6) Typical cross sections of disposal site including lifts, borrow areas and drainage controls ___

(7) Special drainage devices ___

(8) Fencing ___

(9) Equipment facilities ___

(10) Other pertinent information based on intended use of facility ___

d. Report (17-7.050(3)(d), F.A.C.)

(1) Estimated population and area served by the proposed site with basis for the estimate ___

(2) Anticipated type, annual quantity, and source of solid waste ___

(3) Anticipated life of site ___

(4) Source and characteristics of cover material ___

e. Ground Water Monitoring Plan (17-7.050(3)(e), F.A.C.)

(1) Plan and hydrogeological survey, including foundation analysis, in accordance with 17-4.245(6), 17-7.030, and 17-7.050 F.A.C.; or ___

(2) A copy of a Department letter of approval of a previously submitted plan, if applicable. ___

Completeness Check

4. Landfill Performance and Design Standards (17-7.050(4), F.A.C.)

   a. Liner performance (17-7.050(4)(a)(b), F.A.C.) ___
      (1) Material type (soil, synthetic, other) ___
      (2) Adequate base support ___
      (3) Planned installation adequate to cover all surrounding earth ___
      (4) Equivalency to design standards ___

   b. Liner quality control plan (17-7.050(4)(c), F.A.C.) ___
      (1) Specifications ___
      (2) Construction/installation methods ___
      (3) Sampling and testing ___
      (4) Manufacturer's specifications and recommendations ___

   c. Leachate control and removal system performance (17-7.050(4)(e), F.A.C.) ___
      (1) Construction materials ___
      (2) Strength and thickness ___
      (3) Measures to prevent clogging ___
      (4) Central collection point for treatment and disposal ___
      (5) Leachate depth not to exceed one foot ___
      (6) Equivalency to design standards ___

   d. Surface water management system performance (17-7.050(4)(g), F.A.C) ___
      (1) Prevention of surface water flow onto waste-filled areas ___
      (2) Stormwater run-off controls; retention, detention ponds ___
      (3) Equivalency to design standards ___
      (4) Water management district approval ___

   e. Gas control system performance (17-7.050(4)(i), F.A.C.) ___
      (1) Prevention of methane migration ___
      (2) Prevention of damage to vegetation ___
      (3) Prevention of objectionable odors off site ___
      (4) Equivalency to design standards ___

5. Operations Plan (17-7.050(5)(b),(c)(d) & (e), F.A.C.) ___

   a. Designation of responsible person(s) ___
   b. Contingency operations ___
   c. Controlling the type of waste received at the site: ___

Completeness Check

d. Weighing or measuring incoming waste ___

e. Vehicle traffic control and unloading ___

f. Method and sequence of filling waste ___

g. Waste compaction and application of cover ___

h. Operations of gas, leachate, and storm water controls ___

i. Ground water monitoring ___

j. All weather access roads ___

k. Effective barrier ___

l. Signs indicating name of operating authority, traffic flow, hours of operation, and charges for disposal (if any) ___

m. Dust control methods ___

n. Litter control devices ___

o. Fire protection and fire fighting facilities ___

p. Attendant ___

q. Communication facilities ___

r. Adequate in-service and reserve equipment ___

s. Safety devices on equipment to shield and protect operators ___

.. Water Quality Standards (17-7.050(5)(g) & (h), F.A.C.) ___

Describe how surface runoff and leachate will be handled to meet water quality standards of Florida Administrative Code Rules 17-3 and 17-4. ___

7. Closure (17-7.070(2), F.A.C.)

a. Closure plan (17-7.073, F.A.C.) ___

(1) Design ___

(2) Final use ___

(3) Closure operations ___

(4) Post-closure (17-7.075, F.A.C.) ___

(5) Financial responsibility(17-7.071, F.A.C.) ___

b. Closure plan schedule (17-7.071, F.A.C) ___

8. Solid Waste Disposal Facility Data Form ___

9. Solid Waste-Volume Reduction and Resource Recovery Facility Data Form ___

10. Certification by Applicant and Engineer or Public Officer ___

SOLID WASTE DISPOSAL FACILITY DATA FORM

Date Form Completed: 6/15/92

Permit No.: ______ Issue Date: ______ Expires: ______

DER ACTION: X Add __ Delete __ Change __ Deactivate Site

1. DER IDENTIFICATION NUMBER
Unknown

2. SITE NAME
Central County Solid Waste Disposal Complex

3. COUNTY
Sarasota

4. FACILITY ADDRESS (Road, cross road, street)
North End of Knights Trail Road

4a. Facility Phone Number:

4b. Facility Site Supervisor

5a. 27 ° 12 ' 00 " Latitude 82 ° 23 ' 00 " Longitude

5b. 1-4 & 9-16 Township 38S Range 19E Section

6. Operating Authority Name
Solid Waste Dept. Sarasota County

7. Phone Number 813-951-5096

8. Operating Authority Address
8350 Bee Ridge Road
Sarasota, FL 34241

9. Owner of Site Property (if different from operator)
Same

1. Phone Number of Owner Same

11. Address of Owner
Same

12. Facility Type
X Class I, Sanitary Landfill
__ Class II, Sanitary Landfill
X Class III, Trash/Yard Trash
__ Class III Yard trash comp.

Sludge Landspreading:
__ Grade I
__ Grade II
__ Grade III
__ Septage

Type
__ Other Facility

13. Month Year Begun
January 1995

14. Disposal Area
310 Acres

15. Population Served
283,140 (current)

16. Expected Useful Lifetime
Approx. 30 Years

17. Weighing Scales
X Yes __ No

18. Security to Prevent Unauthorized Used
X Yes __ No

19. Depth of Water Table
16.5 - 20.0 Ft. (NGVD)

20. Quantity of Waste/Day
1420 tons or Yd^3

21. Charge
$ N/A yd/ton

22. Surrounding Land Use Zoning
X Residential __ None X Agricultural __ Commercial __ Industrial __ Other

23. Types of Waste Received
X Residential
X Commercial
__ Incinerator Residue
__ Pathological/Infectious
X Agricultural
__ Septic Tank
X Industrial
__ Water/Air Treat Sludge
X Yard Trash/Trash
__ Sewage Sludge
__ Industrial Sludge
__ Hospital
__ Other:

24. Number of Monitoring Wells 15

25. Number of Surface Monitoring Points 7

26. Gas Control / Recovery
X Yes __ No / __ Yes __ No

27. Salvaging Permitted
__ Yes X No

28. Attendant
X Yes __ No

29. Leachate Control Method - Liner Type: __ Natural X Emplaced Clay X Synthetic __ None __ Other______		
Collection Mehtod: __ Well Point __ Perimeter Ditch __ None X Under Site Drains __ Other______		
Treatment Method: __ Oxidation __ Recirculated __ Chemical X Advanced __ None X Other UF/RO		
30. Leachate Discharge __ Yes X No	Class of Receiving Water N/A	
31. Site Located in __ Floodplain X Wetlands X Other: Uplands Approx. 90%		
32. Surface Runoff Collected X Yes __ No	Type of Runoff Treatment Wet Detention	Class of Receiving Waters Class III
33. Property Recoreded as a Solid waste Disposal Site in County Land Records X Yes __ No		
34. Days of Operation Mon-Sat	Days of Cover Mon-Sat	Hours of Operation 8am-5pm
35. Name, Title and Phone Number of Person Completing Form John A. Banks, P.E. Environmental Engineer (813)351-7100		

NOTE: All blanks must be filled or marked as not applicable.

SOLID WASTE VOLUME REDUCTION AND RESOURCE RECOVERY FACILITY DATA FORM

Permit No.: ____________ Issue Date: ____________ Expires: ____________

Facility No. (DER Identification): ____________

DER ACTION: ☐ Add ☐ Delete ☐ Change ☐ Deactivate Site ☐ Other

1. County Sarasota	2. Site Name Central County Solid Waste Disposal Complex
3. Date Form Completed 6/15/92	4. Facility Address North End of Knights Trail Road
4a. Facility Phone No. None	4b. Facility Site Supervisor William K. Lederman
5a. 27° 12' 00" Latitude; 82° 23' 00" Longitude	5b. 38S Township; 19E Range; 1-4 & 9-16 Section
6. Operating Authority Name Solid Waste Dept. Sarasota County	8. Operating Authority Address 8350 Bee Ridge Road Sarasota, FL 34241
7. Phone Number 813-951-5096	
9. Owner of Site Property (if different from Operator) N/A	11. Address of Owner N/A
10. Phone Number of Owner N/A	

12. Facility Type (check one or more)
☐ Incinerator Only ☐ Biomass Gas Production ☐ Pyrolysis ☐ Other:
☐ Sludge Concentration ☐ Baler (compactor) ☐ Composting Plant
☒ Transfer Station ☐ Waterwall Incinerator ☐ Shredder (pulverizer)

13. Month/Year Begun January 1995	14. Disposal Area 310 Acres	15. Population Served 283,140
16. Expected Useful Lifetime 31 Years	17. Weighing Scales ☒ Yes ☐ No	18. Waste Processed Per Operational Day 1400 tons/~~day/wk~~
19. Charge/____ N/A	20. Days Operated S (M) (T) (W) (T) (F) (S)	21. Hours/Day Operated 8-5

22. Maximum Processing Rate None tons/day

23. Material Recovered, Tons/Week N/A
____ Paper ____ Glass Other:
____ Ferrous Metals ____ Non-Ferrous Metals
____ Aluminum ____ Plastics

24. Energy Recovery, in units shown N/A
____ High Pressure Steam-lb/hr ____ Chilled Water-gal/hr ____ Gas-ft³/hr
____ Low Pressure Steam-lb/hr ____ Oil-gal/hr ____ Gas-BTU/hr
____ Electricity-kw/hr ____ Oil-BTU/hr Other:

25. Process Water Recycled N/A ☐ Yes ☐ No | Treatment Method Used N/A

Discharged to: N/A ☐ Surface Waters ☐ Underground | Class Receiving Water N/A

26. Final Residue is N/A % of waste intake | Residue is disposed of at (Site Name) N/A

27. Supplementary Fuel Used N/A

Type N/A | Quantity Used/Hour N/A

28. Estimated Operating Costs Material — Energy Revenue $ N/A | Total Cost/Ton $ N/A | Net Cost/Ton $ N/A

29. Number of Staff N/A | 30. State Pollution Control Bond Financing Amount $ N/A | 31. Estimated Amount of Tax Exemptions that will be Requested $ N/A

32. Name and Title of Person Completing Form John A. Banks, P.E.

Note: All blanks must be filled or marked as not applicable.

CERTIFICATION BY APPLICANT AND ENGINEER OR PUBLIC OFFICER

A. Applicant

The undersigned applicant or authorized representative of Sarasota County is aware that statements made in this form and attached information are an application for a Solid Waste Management Facility Construction Permit from the Florida Department of Environmental Regulation and certifies that the information in this application is true, correct and complete to the best of his knowledge and belief. Further, the undersigned agrees to comply with the provisions of Chapter 403, Florida Statutes, and all rules and regulations of the Department. It is understood that the Permit is not transferable, and, the Department will be notified prior to the sale or legal transfer of the permitted facility.

Signature of Applicant or Agent

William K. Lederman, P.E.
Name and Title

Date: June 15, 1992

Attach letter of authorization if agent is not a governmental official, owner, or corporate officer.

B. Professional Engineer Registered in Florida or Public Officer as Required in Section 403.707 and 403.7075, Florida Statutes

This is to certify that the engineering features of this resource recovery and management facility have been designed/examined by me and found to conform to engineering principals applicable to such facilities. In my professional judgement, this facility, when properly maintained and operated, will comply with all applicable statutes of the State of Florida and rules of the Department. It is agreed that the undersigned will provide the applicant with a set of instructions of proper maintenance and operation of the facility.

Signature

John A. Banks, P.E.
Name and title (please type)

39387
Florida Registration Number
(please affix seal)

201 Montgomery Ave.
Mailing Address

Sarasota, FL 34243
City, State, Zip Code

(813) 351-7100
Telephone Number

Date: June 15, 1992

Construction Cost Estimate: $32,500,000 (see technical Appendix C for cost estimate breakdown)

Permit Number: ______________ Issue Date: ______________

Review Date: ______________ Expiration Date: ______________

SARASOTA COUNTY GOVERNMENT
SARASOTA, FLORIDA

Solid Waste Operations Division

[redacted]
Manager
8350 Bee Ridge Road
Sarasota, Florida 34241
(813) 951-5096
FAX (813) 951-5161

June 12, 1992

Thomas G. Walker, P.E.
Camp Dresser & McKee, Inc.
201 Montgomery Avenue
Sarasota, Florida 34243

Dear Mr. Walker:

Subject: Central County Solid Waste Disposal Complex
DER Construction Permit Applications
Sarasota County Class I and III Landfill

As per Section 17-701.030(5)(e), F.A.C., this is to confirm that Camp Dresser & McKee, Inc. has been appointed to act on behalf of the applicant with respect to the above-referenced permit application.

Sincerely,

William K. Lederman, P.E.
Engineer IV

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LIST OF APPENDICES

EXECUTIVE SUMMARY

In 1986, Sarasota County purchased a 6,150± acre site in Central Sarasota County using solid waste revenue bonds, for the purpose of developing new solid waste disposal facilities. In 1989, a Solid Waste Master Plan was prepared which provided a conceptual design for the facilities to be included at the proposed Central County Solid Waste Disposal Complex (CCSWDC).

In October of 1989, a Dredge and Fill Joint Permit Application was submitted to DER and the U.S. Army Corps of Engineers (ACOE). In March 1990, the DER granted Sarasota County a permit for the dredge and fill activities associated with the proposed impacts to the DER jurisdictional wetlands. The ACOE has requested additional information related to the proposed mitigation plan for impacts to ACOE jurisdictional wetlands.

In May 1992, Sarasota County submitted a Permit Application for the Management and Storage of Surface Waters to the Southwest Florida Water Management District (WMD). The WMD permit application includes the information related to the mitigation plan requested by the ACOE. This information will be provided to the ACOE to complete the information they have requested.

This permit application provides the information related to the design and operation of the Class I and Class III landfills proposed for the CCSWDC. Included in this application package are Technical Appendices which include a hydrogeotechnical report, a copy of the WMD permit design report, design calculations, specifications for the installation of the proposed liner, a design report for the leachate treatment system, and information related to financial responsibility requirements.

The hydrogeological evaluation of the site, performed by Ardaman & Associates, indicates that the soils on the site range from fine sands near the surface to calcareous clayey fine sand to silty fine sand at deeper elevations. Past and present geologic, hydrologic and geotechnical evidence indicates that the conditions favorable for the development of sinkholes do not exist at the site.

The Class I and Class III landfills will be constructed in accordance with Chapter 17-701 FAC. The maximum height of the Class I landfill will be 100-feet above existing grade as required by zoning stipulation. The Class I landfill will be lined with a composite liner system consisting of a clay barrier layer overlain by a 60 mil HDPE synthetic liner and a leachate collection system.

Because Sarasota County is implementing an aggressive recycling program which will include yard waste composting and a construction and demolition recycling facility, it is anticipated that Class III materials remaining for disposal will be well sorted and inspected. For this reason it is not necessary to provide protective liners to the Class III disposal area. The Class III landfill will consist of earthen berms and grading to segregate the disposal area from the stormwater collection area.

The site will also contain a yard waste composting facility and a wastewater sludge composting facility. These facilities will be located to the south of the Class I landfill. As with the landfills, the yard waste composting area will be constructed in phase based on the needs of the County, eventually reaching a maximum size of 120-acres.

Sarasota County Solid Waste Department will operate and maintain the landfill and associated facilities. The Central County Solid Waste Disposal Complex will be owned by Sarasota County and operated by the Sarasota County Solid Waste Department. William K. Lederman, P.E., Solid Waste Department Engineer will be the designated responsible person for the operation of the CCSWDC. The facility will be operated in compliance with all applicable regulations governing the operation of solid waste management facilities, wastewater management facilities and surface water management facilities. Assurance that these requirements will be met is based on the County's past record of excellent compliance.

This document presents the permit application for the construction and operation of Class I and Class III landfills and the leachate treatment and disposal facilities for the Class I landfill only. This application document provides the construction details for Phase I disposal areas, with the remaining phases to be constructed in a similar manner. Table E-1 provides an index for the permit drawings. Separate permit applications will be submitted for the associated yard waste compost facility and wastewater sludge composting facility.

TABLE E-1

PERMIT DRAWING INDEX

COVER	
G-1	Index and Project Location
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C-5	Phase I Section 6
C-6	Class III, Phase I Landfill Plan
C-7	Class III, Phase I Sections
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CD-2	Sump Details and Sections
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SECTION 1
FOUNDATION ANALYSIS

An onsite geotechnical and hydrogeologic investigation was performed by Ardaman & Associates, Inc. The report is included as Technical Appendix A. The geotechnical investigation involved construction of 79 Standard Penetration Test (SPT) borings onsite to determine the subsurface conditions and to recover soil samples for laboratory testing. Soil samples were collected during the drilling program including split spoon samples for visual classification of soils and laboratory testing including sieve analysis and wash gradation, Atterberg limit tests and determination of moisture content. Undisturbed Shelby tube samples also were collected from selected locations to evaluate soils compressibility, strength and permeability.

The predominant material underlying the sandy surficial soils is a silty to clayey fine sand with rock and shell fragments. The SPT "N-valves" for this material ranged from between 15 to greater than 50 blows. A stiff clay layer was found under a large portion of the landfill footprint from 18 to 40 feet. Below the 50 feet, high standard penetration test values were found to be greater than 50 blows per 12 inches.

Standard penetration testing boring TH-1 (Test Hole Number 1) through TH-26 were used to establish conditions beneath the landfill. Criteria for the settlement analysis assumed that the landfill will be 100 feet above grade and no refuse is to be placed below grade. Information collected indicates that only the shallow 18 feet of sandy soils and stiff clay layer will contribute to settlement. The calculated settlement in the sandy soils is approximately 2 inches while the stiff clay layer, assuming a maximum consolidated pressure of approximately 6,600 psf, will be 2 inches or less. Combining the settlements calculated for the two layers, the maximum expected foundation settlement under the weight of the landfill (4,500 lb/ft^2) is 4 inches. This degree of settlement will not adversely affect the performance of the liner.

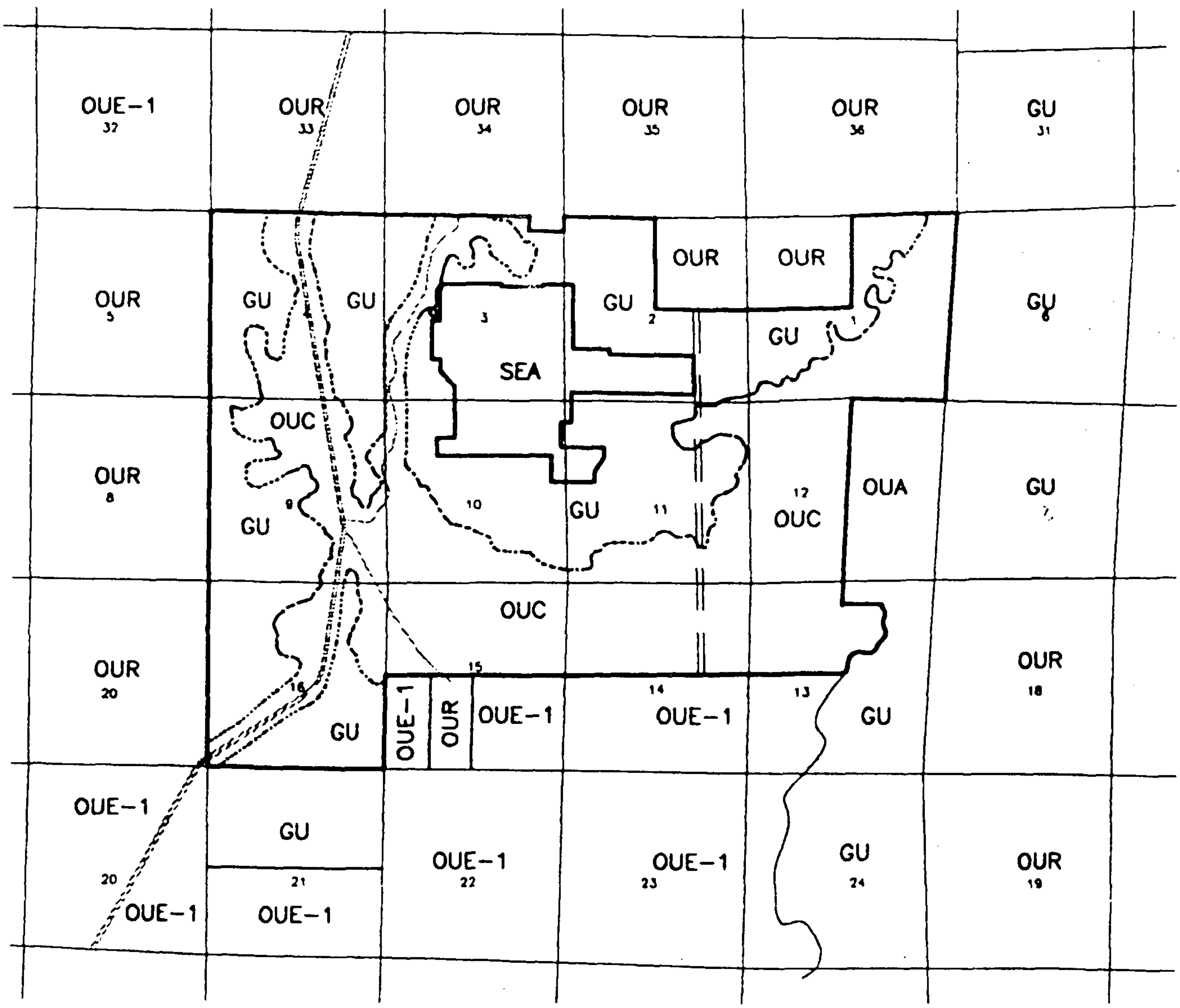
SECTION 2

EVIDENCE THAT THE FACILITY IS IN CONFORMANCE WITH LOCAL ZONING

The site of the proposed facilities has two zoning classifications and a special exception area designating various uses for different areas of the site. The majority of the site, 3,180-acres, is zoned Government Use (GU), of which 550-acres are designated as a Special Exception Area (SEA) for solid waste facilities. All solid waste activities are required to be within the SEA. The remainder of the site, 2,970-acres, has been designated as Open Use Conservation (OUC), to provide a large area of land to be preserved for its habitat and passive recreational value. The land use surrounding the site generally seems compatible with the proposed facilities. These include Open Use Rural, Open Use Estates, and Government Use. On the north and west sides of the site, are active groves and/or ranches. The Myakka River State Park is located along the northeast property boundary, with the Knight's Trail Park (gun range and other facilities) and some residential areas located along the south property boundary (see Figure 2-1).

LEGEND

GU	GOVERNMENT USE
OUA	OPEN USE AGRICULTURE
OUR	OPEN USE RURAL
OUC	OPEN USE CONSERVATION
SEA	SPECIAL EXCEPTION AREA
OUE-1	OPEN USE ESTATE - 5 ACRE MIN. LOT SIZE



SARASOTA COUNTY

CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
ZONING MAP

CDM
environmental engineers, scientists, planners, & management consultants

Figure No. 2

SECTION 3
FACILITY DESIGN

3A Map or aerial photograph of the area not more than one year old showing land use and zoning within one mile of the facility.

Aerials of the site are provided on Sheet G-4 of the Permit Drawings. The aerials were flown in February 1991 and have a 1-inch equal 1500 foot scale. Current land use zoning designations within one mile of the proposed landfill site are delineated on Figure 2-1.

3B Plot Plan.

3B(1) Dimensions and Legal Description of the Site.

The dimensions and legal description of the site are shown on Sheet G-5 of the Permit Drawings. The legal description indicates the entire site acreage to be 6,150 acres more or less.

The actual landfill facilities are to be located within the designated 550 acre special exception area which is located at the approximate center of the site. The locations of the special exception area and the landfill facilities are shown on Sheet G-6 of the Permit Drawings.

3B(2) Location and Depth of Soil Borings.

The boring locations were determined in the field from points of reference with the use of a mosaic aerial photograph. Borings were constructed on 1,000 foot centers within the proposed landfill area for foundation analyses and around the proposed landfill area where potential borrow areas are designated. The boring locations and grid systems are shown on Figure 2 of Technical Appendix A.

3B(3) Plan for Disposal Areas.

The primary object of the facility layout is to maximize available disposal volume for the area utilized. This approach will reduce the cost per cubic yard of fill capacity and extend the service life of the facility and site.

The ratio of volume to base area (footprint) is maximized by using a symmetrical rectangular shape. The maximum width of the landfill base is limited by the ability to clean and maintain the leachate collection laterals. Collection laterals must be at least one-half as long as the greatest width of the lined landfill. Equipment is capable of jetting the collection laterals as designed. Sheets C-1 through C-5 show the plans for the Class I disposal areas.

3B(4) Fencing or Other Measures to Restrict Access.

Access will be controlled at all times at two separate locations. The special exception area will be enclosed by an eight foot high chain-link fence. Vehicles entering the landfill will be required to stop at the scale house for weighing or approval before proceeding. Access or contaminant barriers will include a locking access gate at the entrance to the special exception area. In addition, the existing fence along the property boundary lines restricts access to the landfill site, except through a main entrance gate to be located along the main access roadway at the south property line.

3B(5) Cross Sections Showing Original and Proposed Fill Elevations.

Original and proposed final fill elevations are identified on the landfill cross sections shown on Sheet C-11 of the Permit Drawings.

3B(6) Location, Depth, and Construction Details of Monitoring Wells.

The hydrogeologic investigation performed by Ardaman & Associates included the development of a groundwater monitoring plan which includes the installation of 14 observation (or monitor) wells. One additional monitor well is proposed in the area of the leachate holding pond. In addition, 17 existing piezometers and 7 existing surface water elevation stations will be maintained for monitoring water elevations. The locations of these wells and stations are depicted on Figure 2 of Attachment A. The monitor wells are to be used for collecting groundwater samples and measuring groundwater levels. Fourteen surficial monitor wells are to be installed at 1,000 feet spacing along the down gradient side of the Class I and Class III landfills. The background monitor well will be located about 500 feet northeast of the Class I landfill in the area of highest groundwater elevation on the site. The location of the proposed monitor wells are also shown on Sheet G-6 of the Permit Drawings. The typical monitor well construction detail is shown on Figure 18 of Technical Appendix A.

The piezometers which are used to measure groundwater levels were set at depth ranging from 15 to 95 feet. The depths of each piezometer, and construction information are shown on Table 3 of Technical Appendix A. The surface water elevation stations are surveyed posts that are placed at strategic locations throughout the site to supplement the groundwater elevation data from the shallow wells and piezometers.

3C Topographic Maps.

3C(1) Five-Foot Contour Intervals.

One-foot contour intervals are shown at a scale of 1-inch equals 100 feet on Sheets C-1, C-6 and G-6 of the Permit Drawings.

3C(2) Proposed Fill Areas.

Sheets C-1, C-6 and G-6 of the Permit Drawings are topographic maps with one-foot contour intervals showing the proposed location of fill areas.

3C(3) Borrow Areas.

Borrow will be provided from the excavation of stormwater management ponds and from three on-site borrow areas, totalling 347 acres and the products produced by the composting operations. Existing and proposed final elevations for the three borrow lakes are shown on the grading and drainage plan included in Sheets C-8 and C-9 of the Permit Drawings.

3C(4) Access Roads.

Ingress and egress to and from the CCSWDC is proposed via an all-weather access roadway beginning at the existing Knight's Trail Road and terminating at the proposed landfill entrance. This roadway will be designed and constructed to meet Florida Department of Transportation (FDOT) standards. The access road is located on Sheets G-6 and G-7 of the on permit drawings.

There are two intersections proposed for this project. One is at the origin of the roadway at the point of tie-in to the existing Knight's Trail Road. The second intersection is proposed to allow access to the new roadway from the unpaved portion of Knight's Trail Road, north of Rustic Road.

The proposed pavement elevations will be designed a minimum of one (1) foot above the 100-year flood elevation of the Cow Pen Slough. Along the slough, the road will be comprised of a divided section with incoming and outgoing lanes separated by a grass median. The pavement will be sloped at two (2) percent toward a swale within the grass median. All other portions of the roadway will be a standard rural roadway section with a crown sloping at two (2) percent toward the outside shoulders.

The divided traveled lanes along Cow Pen Slough will be designed so that they slope toward the grass median where runoff will be collected and treated in a linear retention area and ultimately drained to wetlands adjacent to the roadway. Other roadway sections will also be designed with roadside swales for treatment/retention. Design information for stormwater treatment is provided in Technical Appendix B, WMD Permit Application.

The design features of the roadway include, but are not limited to:

- large radius curves that are adequately super elevated,
- 4 to 1 sideslopes to increase and promote stable slopes,
- adequate sight distances,
- slopes that promote drainage and prevent ponding of water,
- properly designed transitions between different roadway cross sections,
- friction course to provide adequate skid resistance properties,
- special additional safety design features will include 12-foot travel lanes and 10-foot paved shoulders.

a) Pavement

The pavement structure will be designed to support traffic loads and distribute them to the underlying soils. This will be done by utilizing the proper design techniques that employ the correct design vehicles, the Average Annual Daily Traffic (AADT), and the appropriate 18-Kip Equivalent Single Axle Load (ESAL). The FDOT Flexible Pavement Design Manual for New

Construction and Pavement Rehabilitation, January 1990 edition, will be used as a guide for this design.

b) Roadway Geometry

The roadway geometry will be designed based on function and classification utilizing two FDOT design manuals - the Manual of Uniform Minimum Standards for Design, Construction and Maintenance for Street and Highways (Green Brook) - 1989 edition, and Roadway and Traffic Design Standards - January, 1990 edition. To promote consistency with the exiting roadway, a similar design speed (50 miles per hour) will be used. This will allow for a posted speed limit of 40-45 mph. Horizontal curves, super elevations, vertical curves, shoulders and transitions will all be designed to the standards required by the FDOT.

3C(5) Grades Required for Proper Drainage

Grading on fill areas will be developed to divert surface runoff away from active cell areas to reduce the amount of potential leachate. See Sheet C-12 of the Permit Drawings. Clean surface water run-off accumulated in cells not containing solid wastes will be pumped to perimeter stormwater swales. The closeout surface water drainage plan is illustrated on Sheet C-10 of the Permit Drawings. The surface water management ponds will be constructed to adequately handle surface water drainage in accordance with FDER requirements in Chapters 17-3 and 17-4 FAC, as well as the requirements of the Southwest Florida Water Management District. A detailed evaluation of existing and proposed drainage and surface management features is included in the stormwater management facility design report, Attachment B.

3C(6) Typical Cross Sections of Disposal Site, Including Lifts, Borrow Areas and Drainage Controls.

Sheet C-11 of the Permit Drawings shows cross sections through the proposed landfill area. Proposed bench terraces will be located along the 5:1 side slopes during closure of fill areas to minimize erosion. The perimeter swale/ditch between the landfill toe of slope and the perimeter road directs the surface water run-off to the surface water management ponds. Sheet C-12 of the Permit Drawings show a typical cross section of a cell including interior drainage, swales, berms, and lifts.

3C(7) Special Drainage Devices.

No special drainage devices will be necessary for stormwater management. All flow patterns are accomplished by gravity.

3C(8) Fencing.

See response 3B(4).

3C(9) Equipment Facilities.

The maintenance yard located on Sheet G-6 of the Permit Drawings will contain an operations and maintenance building for servicing and storing equipment as well as providing office and locker areas.

3C(10) Other Pertinent Information Based on Intended Use of Facility.

The Class I and Class III landfills will be accepting mixed municipal waste and construction/demolition debris respectively. An area near the entrance to the facility will be designated for storage of recyclable items including white goods as shown on Sheet G-6 of the Permit Drawings. A used tire storage area will also be provided near the entrance to the facilities.

The site will also contain a Yard Waste Composting Facility. This facility will be located to the south of the Class I landfill. As with the landfills, the composting area will be constructed in phases based on the needs of the County, eventually reaching a maximum size of 120-acres. The stormwater which falls on this area will be collected and treated in the stormwater management system. Wastewater sludge will also be composted within the SEA area. These areas will be covered to prevent stormwater from contacting the sludge compost.

A sludge dewatering and sidestream treatment facility may be constructed as part of the sludge composting system. This facility will operate separately from the leachate treatment system. The liquid sidestream wastes from this facility may be treated and used for irrigation at the CCSWDC site.

3D Report

3D(1) Service Area and Population.

The landfill will serve both the unincorporated areas and municipalities of Sarasota County. Cites include: Sarasota, Venice, North Port, Englewood and the town of Longboat Key. Solid waste disposal at the landfill will include Class I materials (i.e. mixed municipal solid waste);

Class III materials primarily construction and demolition wastes and yard wastes unsuitable for composting.

Population projections for Sarasota County are shown in Table 3-1. Population data for Sarasota County was based on information provided by the Sarasota County Planning Department.

3D(2) Solid Waste Generation.

Solid waste collected from Sarasota County include residential, commercial, industrial and special wastes. Special wastes include tires, oversize bulky wastes, Class III wastes and C&D debris. No hazardous wastes will be accepted for disposal at the landfill.

A Solid Waste Master Plan (CDM, 1989) contained projections of solid waste generation for Sarasota County based on weight records at the existing landfill and data available on waste recycled at other locations (construction and demolition debris and land clearing waste). These projections will be used as the base waste stream flow which, after subtracting projected recycling quantities, are used to calculate the expected quantity of material requiring disposal in a landfill. The disposal quantity was used to estimate the capacity of the landfill configuration developed during design.

The Sarasota County Charter Article IV requires the County to reduce solid waste quantities by 50 percent by volume. It has been reported by Article IV authors that the reduction goal is to represent the volume of materials compacted in a landfill. Due to the various densities of recyclables, it is estimated that a volume reduction of 50% (compacted in a landfill) will reduce the weight of the disposed waste by approximately 38% (see Technical Appendix C Design Calculations).

TABLE 3-1

FUNCTIONAL POPULATION PROJECTIONS*

YEAR	POPULATION
1992	354,311
1995	380,319
2000	418,608
2005	454,327
2010	487,859

* Sarasota County Comprehensive Plan.

Table 3-2 outlines the volume of material requiring disposal using the original medium waste stream projections contained in the Solid Waste Master Plan. The resulting disposal volume requirements will be the volume used to project the required disposal capacities, in terms of time, for the various elements of the landfill.

The Solid Waste Master plan projections based on available population projection are limited to the year 2010. Additional projections were determined for extended years based on available population data and extrapolation. Table 3-3 provides an extended projection of disposal needs.

3D(3) Anticipated Service Life of Landfill

The landfill facilities are designed to maximize the life of the site within certain engineering and environmental constraints. The resulting landfill footprint and height limitations will provide a service life of approximately thirty (30) years based on the estimated solid waste quantities reflecting a 50 percent reduction in the total solid waste stream from recycling activities as required by Sarasota County Charter Article IV. The Phase I area of the Class I landfill comprises approximately 60 acres of disposal area and will provide disposal capacity for 5 years based on the projections from Table 3-2.

3D(4) Source and Characteristics of Cover Material

There will be two sources of on-site material for use as daily, intermediate and final cover on the landfill. These include the designated borrow lakes and compost to be generated on-site. The borrow lakes will provide material for the initial construction of facilities while the remainder of the material will be stockpiled to be readily available for use on the active landfill.

TABLE 3-2
PROJECTED DISPOSAL VOLUME REQUIREMENTS

YEAR	MASTER PLAN MEDIUM PROJECTION TONS/YEAR	SOLID WASTE WEIGHT REDUCTION (38%) TONS/YEAR	MINIMUM REQUIRED DISPOSAL VOLUME @ 1200 LB/CY CY/YEAR
1995	537,000	332,940	554,900
2000	587,000	363,940	606,600
2005	637,000	394,940	658,233
2010	678,000	420,360	700,600

TABLE 3-3
LONG TERM WASTE STREAM PROJECTIONS

YEAR	PROJECTED (1) FUNCTIONAL POPULATION	TOTAL SOLID WASTE (2) (T/Y)	SW 38% WT REDUCTION (T/Y)	(3) VOLUME IN-PLACE (CY)	CUMULATIVE TOTALS (CY)
2020	551,207	766,000	474,920	791,533	17,622,467
2021	558,481	776,000	481,120	801,867	18,424,333
2022	565,754	786,000	487,320	812,200	19,236,533
2023	573,028	797,000	494,140	823,567	20,060,100
2024	580,301	807,000	500,340	833,900	20,894,000
2025	588,050	817,000	506,540	844,233	21,738,233
2026	594,240	826,000	512,120	853,533	22,591,767
2027	600,430	835,000	517,700	862,833	23,454,600
2028	606,620	843,000	522,660	871,100	24,325,700
2029	612,810	852,000	528,240	880,400	25,206,100
2030	619,500	861,000	533,820	889,700	26,095,800
2031	625,695	870,000	539,400	899,000	26,994,800
2032	631,890	878,000	544,360	907,267	27,902,067
2033	638,085	887,000	549,940	916,567	28,818,633
2034	644,280	896,000	555,520	925,867	29,744,500
2035	651,000	905,000	561,100	935,167	30,679,667

(1) Extrapolated from Sarasota County Planning Department Projections
(2) Based on Solid Waste Master Plan (CDM 1989) Generation Rates
(3) Based on compacted densityof 1200 lbs/cubic yard

The main borrow lake, because of its large size, will be developed in at least two stages. The Phase I stage will be excavated for a relatively short period of time and the excess material stockpiled in future phase areas (as shown on Sheet G-6) , out of the way of the next phase, but convenient to the landfill operations. Preliminary estimates indicate that the Phase I borrow lake area will provide five years of cover material for the landfill in addition to the initial construction requirements. This will provide the County a continuous supply of cover material accessible by standard landfill earthmoving equipment.

The earthen cover material from the borrow lake stockpile will be supplemented by use of compost generated by the adjacent yard waste and sludge facilities. Compost can be used as a replacement for soil for all cover types and should be utilized for this purpose to the greatest extent possible. Use of compost as cover material will extend the life of the five year stockpile described above. Since the compost product will be generated on a daily basis, consideration must be given to utilizing it on a continuous basis, whereas additional borrow material need not be excavated until it is needed. In this way it is anticipated that excavation of the Phase II borrow area could be postponed. For the life of the facility the proposed borrow lakes will supply some of the cover needs while compost will provide the remainder. Table 3-4 illustrates the cover requirements and sources over the life of the facility.

3E Groundwater Monitoring Plan

3E(1) Plan and Hydrogeological Survey Including Foundation Analysis in Accordance with 17-4.245(6) and 17-7.050 FAC

The geotechnical/hydrogeologic investigation report, Technical Appendix A, evaluates the structural compressibility, consolidation, and differential settlement of soils that will form the base of the proposed landfill. The report contains information about soils borings, monitoring

TABLE 3-4

SUMMARY OF COVER NEEDS AND SUPPLIES

Total cover material required for 30-years of landfill operation	6,750,000 c.y.
Available cover material from borrow lakes (Borrow lake volume minus structural fill requirements)	3,500,000 c.y.
Cover material needs to be met by compost	3,250,000 c.y.
Average daily cover material required from compost sources	350 c.y.
Estimated average daily yard waste compost available	750 c.y.

well and piezometer installation, groundwater and surface water measurements, groundwater quality, and vertical and horizontal aquifer hydraulic conductivity.

Beginning of Page 22 of the report, a description of the monitoring plan providing methods and procedures used to monitor groundwater in the vicinity of the proposed facilities is included. The proposed monitoring well network locations are illustrated on Sheet G-6 of the Permit Drawings. The proposed monitor well construction detail is included as Figure 18 of Technical Appendix A and is described in the report.

SECTION 4

LANDFILL PERFORMANCE AND DESIGN STANDARDS

4A LINER PERFORMANCE

The proposed liner system for the Class I disposal area will consist of a composite liner with a primary leachate collection system.

The composite liner system profile consist of (from bottom to top):

1. Prepared subbase compacted to 95 percent modified proctor density ASTM D 1557.
2. 18-inch barrier soil layer with a saturated hydraulic conductivity of not greater than $1x10^{-7}$ cm/sec hydraulic conductivity, compacted to 95 percent modified proctor density.
3. 60 mil HDPE synthetic liner overlaying the barrier soil layer.
4. Two HDPE geonet lateral drainage layers of .2200 $\pm$ .002 inches thick each.
5. 80 mil thick (6 oz.) geotextile fabric installed on top of the geonet to prevent fines from clogging the geonet.
6. 12-inch uncompacted silica sand drainage layer with hydraulic conductivity not less than $1x10^{-3}$ cm/sec.
7. 12-inch local sand protective layer with a hydraulic conductivity of $1x10^{-3}$ cm/sec to $1x10^{-4}$cm/sec.

4A(1) Material Types (Soil, Synthetic, Other)

The proposed 18 inches of barrier soil liner of not greater than $1x10^{-7}$ cm/sec hydraulic conductivity, in conjunction with the 60 mil synthetic liner, acts as a composite bottom liner. The material for the clay barrier layer will be obtained from preapproved sources demonstrating an ability to provide material meeting the required specifications. Technical Appendix D contains specifications for the installation of the barrier soil liner.

The synthetic liner will be a 60 mil thick high density polyethylene material that meets, or exceeds, the minimum requirements of the National Sanitation Foundation Standard Number 54, Flexible Membrane Liners (revised November 1985). High density polyethylene (HDPE) was selected as the liner material for several reasons. HDPE contains no additives or fillers which may leak out and cause deterioration over time. Most importantly, HDPE is resistant to a wide range of chemicals including acids, bases, oils, and heavy metals which may be present in landfill leachate. Technical Appendix D contains detailed specifications for the materials and installation of the synthetic liner.

4A(2) Adequate Base Support

Technical Appendix A provides a detailed foundation analysis which indicates a total maximum settlement of 4-inches. The geologic conditions present beneath the site include a zone of very stiff, consolidated clays and other clay-sand soils. These conditions represent an excellent foundation for a landfill.

4A(3) Planned Installation Adequate to Cover all Surrounding Earth.

The installation of the liner will be performed in phases. Typical seam welding will be accomplished by using an automatic hot wedge fusion welder. A hand held extrusion welder will be used for areas difficult to access with the automatic equipment. The liner installation plan is designed to contain all solid waste materials and soils to be in contact with waste or leachate. Sheet C-1 of the Permit Drawings outlines the limits of liner for Phase I of the Class I landfill.

The waste material will be contained in each cell by interior and exterior berms. The liner in each cell will completely cover the base of the cell and exterior berms. The liner will be anchored permanently at the top of the exterior berms around the perimeter of the landfill. Temporary anchorage will be provided at the exterior berms located within the limits of the Class I landfill for future tie-in of the expansion phases. The liner details are illustrated on Sheet C-5 and CD-1 of the Permit Drawings. The leachate collection system has been designed to eliminate the need for any penetrations of the liner system.

4A(4) Equivalent to Design Standards.

The synthetic liner material will meet the specifications for permeability and strength as required in FDER 17-7.050(4)(a) (refer to Technical Appendix D). The proposed liner material (HDPE) is highly resistant to chemical degradation.

Certification of performance in a leachate environment will be provided by the liner manufacturer/supplier. Additional materials test data will be provided by the manufacturer including tensile strength, burst strength, impact puncture strength, friction pullout and permeability.

4B LINER QUALITY CONTROL

4B(1) Specifications

Liner specifications including a Liner Quality Control Plan are provided in Technical Appendix D.

4B(2) Construction/Installation Methods.

The method of installation of the liner and recommended construction practices are included within the Liner Quality Control Plan in Technical Appendix D. Liner installation will occur by phased construction (see Sheet C-13 of Permit Drawings). Additional liner will be fused to the in-place liner as more cell areas are developed for landfilling.

The liner will be protected by a minimum 24-inch layer of uncompacted sand placed above the liner. This layer will attenuate the fractive and compressive forces of construction and operations equipment as well as acting as a vertical drainage layer. Construction methods and techniques will be in accordance with EPA publication EPA 1600/2-88/052, September 1988, "Lining of Waste Contaminant and Other Impoundment Facilities."

4B(3) Sampling and Testing.

As part of the liner Quality Control Plan presented in Technical Appendix D, all completed field seams will be subject to non-destructive vacuum testing. Samples from completed seams will be tested for shear and peel strength as specified in the Technical Appendix D.

4B(4) Manufacturers Specifications and Recommendations.

Manufacturer specifications and recommendations will be used if they are more rigorous/stringent than those in Technical Appendix D. Manufacturer recommendations will be followed, upon FDER concurrence, if improved methods of installation/construction for specific materials have been developed since EPA publication EPA/600/2-88/052 was issued.

4C LEACHATE CONTROL AND REMOVAL SYSTEM PERFORMANCE

4C(1) Construction Materials.

The proposed leachate collection and removal system includes a HDPE geonet lateral drainage layer, 8 inch perforated HDPE collector pipes, and a leachate collection sump with a submerged leachate pump located inside a 24-inch riser pipe.

The lateral drainage layer for the primary liner in the Class I area (from top to bottom) consists of 12 inches of uncompacted local sand material (in the range of $1x10^{-3}$ to $1x10^{-4}$ cm/sec), 12 inches of imported silica sand with a hydraulic conductivity of not less than $1x10^{-3}$ cd/sec, followed by a 6 oz. geotextile fabric, two layers of HDPE geonet placed below the geotextile followed by a 60 mil liner and 18-inches of barrier clay with a saturated hydraulic conductivity of not more than $1x10^{-7}$ cm/sec.

Entrenched in the 24 inch sand layer and placed in a shallow sump at the low point of the cell slope, perforated leachate collector pipes will be surrounded with inert stone or gravel (one or two inches in diameter) and, wrapped with geotextile fabric. The stone promotes leachate flow into the collector pipes and the geotextile is used to prevent fine particles from entering the pipe.

The perforated pipe and stone are placed at the bottom of the two opposing two percent slopes in each cell. A shallow "sump" is created so that the perforations in the pipe will be positioned approximately 1 inch below the adjacent liner elevation. Each cell will be equipped with a leachate extraction pumping system that will transport leachate to a double walled 6 inch HDPE header pipe located at the top of the landfill cell containment berm. This header pipe will transport the leachate to an on-site treatment facility or will bypass to a leachate storage pond.

4C(2) Strength and Thickness.

Wall thickness for leachate collection pipes will be greater than or equal to SDR 17. Leachate force main pipes will have a wall thickness of SDR 17. See calculations in Technical Appendix C. The lateral drainage layer of geonet will be composed of two layers of .2200 $\pm$ .002 inches thick. The geotextile fabric will be 6 oz. material of approximately 80 mil. thickness.

4C(3) Measure to Prevent Clogging.

The leachate collection system pipes will be surrounded by stone (gravel) to improve leachate flow into the pipes. Geotextile filter fabric will be wrapped around the stone to prevent the protective sand layer from clogging the stones. Leachate collection pipes have a minimum diameter of 8" and are spaced at a 400' interval (center to center). Collection pipes are provided with cleanouts and are limited to 1,400 ft. in length. Investigations were conducted to assure the availability of cleanout equipment up to 1,500 ft. (See technical Appendix C). Collector and transport pipes are designed to maintain sufficient velocities to prevent sediment accumulations in the system. (See technical Appendix C).

4C(4) Central Collection Point for Treatment Disposal.

The leachate will be conveyed by gravity to leachate extraction pumps at the low point of each cell. The leachate will then be pumped to a central onsite treatment facility. The quality and quantity of leachate generated will be monitored on a daily basis as part of the operational procedures for the treatment system. Technical Appendix E provides detailed information for the leachate treatment system.

4C(5) Leachate Depth and Generation Modeling.

The quantity of leachate generated and the anticipated depth over the synthetic liner was computed for four scenarios using the Hydrologic Evaluation of Landfill Performance Model (HELP). The liner was assumed to be impermeable. This assumption maximizes the calculated leachate head over the liner produced from the model runs.

For the purpose of this analysis the composite liner was input as a barrier soil liner at the base with hydraulic conductivity (HC) of not more than 5×10^{-7} cm/sec and a thickness of not less than 18-inches. The soil layer is covered by a synthetic membrane with a thickness of not less than 60 mils. For the model analysis, this layer is overlaid with one synthetic geonet to improve lateral drainage along the membrane. The geonet is then overlaid by a geotextile filter fabric to prevent fine particles from clogging the geonet openings.

Above the base liner is the leachate collection system (LCS) consisting of a sloped 8-inch perforated pipe, drainage aggregate, and geotextile filter fabric. This system is designed in such a manner to eliminate accumulation of leachate on the base liner. The LCS will be embedded in a protective layer of 24-inches of uncompacted local sand.

Four separate HELP model runs were performed to estimate leachate depths and generation rates during common landfill operating scenarios. The purpose of model Run1 is to estimate the maximum possible flow through the lateral drainage layer to properly design the leachate collection and removal system. The purpose of model Run2 is to simulate the conditions at a relatively young landfill. Model Run3 simulates the conditions in a landfill that has been operating for several years, and model Run4 is the final capped condition which simulates the leachate generation for the closed landfill.

The average daily volumes of leachate generation were estimated using climatological data from the Bradenton, Florida Weather Station No. 8-9045, 5-ESE over the thirty-year period from 1951 to 1980, and was provided from NOAA document No. 181. Hence, the one-year model run represents a thirty-year average of climatic conditions. The rainfall data for the single peak day has been adjusted to simulate a 25-year, 24-hour storm event (8-inch, which is representative of this area).

For years 1951-1980 the average monthly precipitation results are as follows:

Month	Rainfall (inches)
January	2.77
February	3.03
March	2.92
April	2.03
May	3.24
June	12.46
July	8.83
August	9.59
September	8.45
October	3.10
November	1.97
December	2.37
Total	60.76

The following are descriptions of the model runs:

1. **RUN1, 5 FEET OF SOLID WASTE, NO COVER**

   Layer 1: 60-inches solid waste, $1x10^{-4}$ cm/s HC

   Layer 2: 24-inches local sand (uncompacted), 2×10^{-4} cm/s HC

   Layer 3: Geotextile fabric, 0.4 cm/s HC

   Layer 4: Geonet, 9 cm/s HC

   Layer 5: 18-inches barrier soil, $1x10^{-7}$ cm/s HC with a 60-mil synthetic membrane overlaying it

2. **RUN2, 20 FEET OF SOLID WASTE, 12 INCH COVER**

   Layer 1: 12-inches local sand (compacted), $1x10^{-5}$ cm/s HC

   Layer 2: 240-inches solid waste, $2x10^{-4}$ cm/s HC

   Layers 3, 4, 5 and 6 same as RUN1 Layers 2, 3, 4, and 5

3. **RUN3, 50 FEET OF SOLID WASTE, 12 INCH COVER**

   Layer 1: 12-inches local sand (compacted), $1x10^{-5}$ cm/s HC

   Layer 2: 600-inches solid waste, $2x10^{-4}$ cm/s HC

   Layers 3, 4, 5 and 6 same as RUN1 Layers 2, 3, 4, and 5

4. **RUN2, GOOD GRASS, CLOSED CELL, 100 FEET OF SOLID WASTE**

   Layer 1: 18-inches local sand (Uncompacted), $8x10^{-4}$ cm/s HC

   Layer 2: Geotextile fabric barrier liner protector, 0.4 cm/s HC

   Layer 3: 12-inches barrier soil , $1x10^{-7}$ cm/s HC

   Layer 4: 12-inches local sand (compacted), $1x10^{-5}$ cm/s HC

   Layer 5: 1200-inches solid waste, $2x10^{-4}$ cm/s HC

   Layer 6, 7, 8 and 9 same as RUN1 Layers 2, 3, 4, and 5

Several leachate collection system designs were analyzed using the previously described model runs. The final design chosen is a 400 foot spacing between collection pipes with a 2 percent slope on the liner beginning one half way between the collection pipes (high point) and sloping to the collection pipes for a distance of 200 feet. Table 4-1 provides the results of the HELP model analysis of this design.

Table 4-1 shows that the static head buildup over the primary liner for all runs are less than 0.1 inches, demonstrating an adequate system design.

The output of each of the HELP model runs may be found in Technical Appendix C. Table 4-1 also summarizes leachate volumes for the peak day average day for modeled conditions, as well as maximum and average calculated leachate hydrostatic head.

4D SURFACE WATER MANAGEMENT SYSTEM PERFORMANCE

4D(1) Prevention of Surface Water Flow into Waste Filled Cells.

Prevention of surface water flow into waste filled areas will be accomplished by the design and implementation of several different common engineering practices. First, the proposed access road encompassing the landfill area will be elevated above existing ground elevations by approximately 5 feet and thus the road will prevent any surface water from entering the waste filled cells. Secondly, large swales will be located at the base slope of the landfill on the interior side of the access road. The swales have been designed to receive runoff from the predeveloped (i.e., unused) and closed-out portions of the landfill areas and transport it to the detention basins. Additionally, the proposed berm to be constructed interior to the stormwater ditch will be constructed with a composite liner to an elevation 4 feet above the design high-water elevation of the perimeter ditch. Sheet CD-3 of the Permit Drawings provides a detail which illustrates this point. In the event of any standing water within the ditch, the liner will

TABLE 4-1

ESTIMATED LEACHATE GENERATION VOLUMES
HELP MODEL SUMMARY RESULTS

Run Conditions	Peak Day Volume/Acre-Day					Average Day Volume/Acre-Day				
	Surface Run-Off		Leachate Flow			Surface Run-Off		Leachate Flow		Ave. Head In.
	CF	GAL	CF	GAL	Head In.	CF	GAL	CF	GAL	
1. New Cell Condition	2,814.1	21,054	965.4	7,221	0.1	20.6	154	185.8	1,390	<0.1
2. Early Operating Condition	5,491.3	41,075	23.8	178	<0.1	159	1,190	22.5	168	<0.1
3. Intermediate Operating Condition	5,491.3	41,075	23.4	175	<0.1	159	1,190	22.8	171	<0.1
4. Final Capped Condition	13,238.6	99,025	17.4	130	<0.1	46.5	348	14.7	110	<0.1

prevent any lateral flow into the landfill area. Finally, a clay cap will be installed on top of the waste layers upon closure to inhibit vertical infiltration of rainfall.

4D(2) Stormwater Runoff Controls, Retention/Detention Ponds.

The stormwater management system for this project will consist of a series of swales, pipes, and detention ponds. The system has been designated to comply with all of the requirements of both Chapters 17-25 FAC and 40-D4 FAC. Sheet CD-3 of the Permit Drawings provides several details related to stormwater runoff controls to be installed on the landfill. Appendix B also provides information related to the design of the retention facilities

4D(3) Equivalency to Design Standards.

In accordance with Chapter 17-701 FAC, the surface water collection system for the landfill cells will be designed to route surface water away from solid waste fill areas. The drainage ditches/swales and surface water management ponds are designed to satisfy both the Chapter 17-25, FAC, criteria of DER, and the 40-D4 criteria of SWFWMD. See Technical Appendix B of this application for detailed information related to the stormwater management system.

4D(4) Water Management District Approval.

All pertinent information for the Management and Storage of Surface Water (MSSW) permit application (Technical Appendix B) has been submitted to the SWFWMD for approval.

4E GAS CONTROL SYSTEM PERFORMANCE

4E(1) Prevention of Methane Migration

The Class I landfill will require a means of releasing landfill gases that migrate to the top of the landfill after the final cap is in place. Therefore, a gas venting system will be installed during installation of the final cap. A portion of the Phase I area will receive a final cap once filling in the Phase II area commences. Sheet C-15 of the Permit Drawings illustrates the location of the final cap and the location of proposed passive gas vents. A methane gas vent detail is also provided on Sheet C-15 of the Permit Drawings. The impermeable liner installed at the bottom and side slopes will prevent lateral migration of methane.

It is proposed that an active landfill gas extraction system be installed in the future once adequate volumes of gas are generated. For this purpose several passive wells in the Phase I area will be constructed with special features which will allow the wells to be converted to an active system in the future. These well locations correspond with the layout of an active gas collection system for the entire Class I landfill area shown on Sheet C-15. Details for the conversion of the passive wells to active wells are also provided on Sheet C-15.

When working in and around the leachate pond manholes, stormwater drains, or when installing methane gas vents, appropriate "No Smoking" rules will be observed. Signs will be posted warning of danger in these areas.

The effectiveness of the landfill gas management system will be monitored through inspection of the final cover. Evidence of ineffective collection of landfill gas could include stress to the final cover vegetation.

4E(2) Prevention of Damage to Vegetation.

The landfill is designed as an aboveground facility; therefore, no lateral migration of landfill gas will be possible, thus preventing any opportunity for damage to vegetation.

4E(3) Prevention of Objectionable Odors Offsite.

The landfill is located near the center of a 6,000 acre site. The closest distance from the proposed waste limits to the site property line is approximately 1,900 feet. The closest residence is approximately 1.5 miles from the Class I landfill area. The buffers and good operational practices will be sufficient to prevent objectionable odors offsite. Strict operational procedures will include a limited working face and an adequate supply of available cover material.

4E(4) Equivalency to Design Standards.

The gas control vents will be designed and located to adequately reduce gas pressure to prevent explosion or fires. A gas vent detail is shown on Sheet C-15 of the Permit Drawings.

SECTION 5
OPERATIONS PLAN

5A DESIGNATION OF RESPONSIBLE PERSONS

The Central County Solid Waste Disposal Complex will be owned by Sarasota County and operated by the Sarasota County Solid Waste Department. William K. Lederman, P.E., Solid Waste Department Engineer will be the designated responsible person for the operation of the CCSWDC. The facility will be operated in compliance with all applicable regulations governing the operation of solid waste management facilities, wastewater management facilities and surface water management facilities. Assurance that these requirements will be met is based on the County's past record of landfill operation.

5B(1) Emergency Provisions

Emergency conditions at the landfill site may occur as a result of a natural disaster (hurricane, tornado, flooding, etc). or fire.

Refuse is not normally delivered to the site during emergency conditions; however, should a major storm occur, the following procedures shall be taken.

- Daily cover shall be applied to all exposed refuse before a major storm arrives, if possible.
- All landfill equipment shall be parked near any natural wind screens such as earthen mounds and berms.
- All lightweight signs and equipment shall be secured.
- When operation is resumed work shall commence in dry areas only (upstream of active face). Refuse shall not be deposited in standing water.

- Contract agreements with local contractors, equipment suppliers, or cooperative lending agreements with other County departments will be pursued for backup equipment, if necessary.

Small fires on the working face will be controlled by a bulldozer or landfill compactor and ample cover material to extinguish the fire. On-site stockpiles of soil cover material will always be available for suppressing fires. In the event an uncontrollable fire does occur at the landfill site, the Nokomis Fire Department will be contacted. The Nokomis Fire Department presently maintains a fire station at 111 Pavonia Road in Nokomis, approximately 7.5 miles from the proposed facility. This station has equipment capable of drafting water from surface sources. Therefore, the large stormwater retention basins adjacent to the landfill will serve as the water source for fire fighting purposes. The proposed source of potable water will also be available as a back-up source of water for firefighting purposes.

These basic emergency procedures should protect the landfill and equipment, and allow re-activation of the operation in an orderly and timely manner.

5B(2) Wet Weather Operations

Steps to be taken for accommodating wet weather solid waste disposal include: 1) set-aside elevated tipping areas with limestone or shell approaches as needed to allow uninhibited vehicular movement, 2) set-aside elevated sandy cover material, and 3) drainage and treatment facility inspection and maintenance.

In the event of excessive accumulation of standing water in the area of the working face a small area of daily cover will be removed by grading to allow direct percolation to the underlying refuse and leachate collection system.

5C CONTROLLING THE TYPE OF WASTE RECEIVED AT THE SITE

The CCSWDC will only accept wastes which are permitted for Class I sanitary landfills and Class III disposal facilities, as provided in Chapter 17-701, FAC. Hazardous or biohazardous waste, as defined by the U.S. EPA and FDER, will not be accepted at the site for disposal.

All materials entering the facility must pass through a scale facility. At this point the nature of the material must be disclosed for proper charging and direction to the correct receiving facility. At least three random loads of solid waste delivered to the landfill each week will be examined. A detailed inspection of the discharged material will be made for any unauthorized waste.

Special wastes such as white goods, recyclable materials, batteries, tires and waste oil will be accepted and stored in designated areas. These materials will be periodically collected by recycling companies or removed for alternate disposal. Wastes to be removed for alternate disposal include automotive batteries, pesticide formulations, and solvents.

All landfill personnel will be trained and instructed to watch for barrels and other suspicious containers to control materials that may be classified as hazardous; halt their unloading; and report the incident to the foreman.

5D WEIGHING OR MEASURING INCOMING WASTES

All waste entering the landfill site will be weighed. A minimum of three (3) electronic 50 ton scales will be installed at the entrance facility. An Information Management System (IMS) will be linked to the scales to facilitate accurate data collection and measurement of incoming materials.

5E VEHICLE TRAFFIC CONTROL AND UNLOADING

Directional signs will be placed to safely direct vehicles to the current waste unloading area. These signs will have large legible letters and will be cleaned when necessary. Signs will be strategically placed so that the route is clear to the drivers. Speed limit, safety, and prohibitive practice signs will be placed as necessary to encourage a safe, clean operating area.

Unloading will be permitted only at designated working faces of currently operating cells. On the fill areas, temporary signs, barricades and flagged stakes will be used to direct vehicles to the proper tipping areas. Haulers will be responsible for unloading their own vehicles. Wastes requiring special handling will be coordinated with and unloaded under the direct supervision of landfill personnel.

5F METHOD AND SEQUENCE OF FILLING WASTE

The overall phasing plan for the facilities is depicted on Sheet C-13 of the Permit Drawings. The layout for the cells comprising Phase I of the Class I landfill is shown on Sheet C-1. The maximum lift height in each cell is 15 feet, as shown on Sheet C-12 of the Permit Drawings. The liner and leachate collection systems for all of Phase I of the Class I landfill will be initially constructed prior to commencing filling activities while the temporary roads and swales for access and surface water drainage will be phased in as the Phase I area is filled.

The method of waste disposal in an individual cell is described as follows. All incoming solid waste will be directed to the working face and placed against the side slope of the previous day's refuse. The first row in a new lift will act as a berm to provide a guide for the placement of refuse for the remaining rows. A slope of 3 to 1 will be maintained on an approximate 200 foot wide working face. This will provide maneuvering area for large private and commercial vehicles.

Solid waste will be placed at the bottom of the working face, spread up toward the top in 2-foot layers. The solid waste will be compacted with a minimum of three to five passes of a compactor. The spreading of refuse will be continuous operation. The construction and demolition material to be deposited in the Class III landfill will be uniformly spread to reduce unused voids.

Of critical importance will be maintaining the stormwater management system during the filling sequence. As each cell is constructed, two sets of temporary diversion berms will be constructed. One set will isolate the working face from the remaining covered areas. Stormwater which accumulates in the area of the working face will be retained and allowed to percolate into the landfill where it will eventually be collected in the leachate collection system. The second set of berms will serve as erosion and sediment traps on the newly covered landfilled areas. Sediments which reach the perimeter ditch will collect behind the ditch blocks (shown on Sheet C-1 of the Permit Drawings) and will require periodic removal.

As filling progresses beyond the first terrace, the first set of let-down structures will be constructed and the side slopes of the first fill layer will receive intermediate and vegetative cover. This operating procedure will minimize the amount of erosion and sediment accumulation that must periodically be removed from the perimeter ditches by stabilizing the sideslopes at the earliest available time.

The filling of the lined cell areas within the Phase I area will follow the sequence outlined below: (Refer to Sheet C-12)

- The entire cell area will be initially filled with a 5 to 6 ft. lift to bring daily cover grade 1-2 feet above the containment berms to promote stormwater runoff.
- Filling of lift No. 1 shall progress from the low end of the cell (north end on Phase I) to the high end while providing a 0.5 percent slope on the cover.

- Subsequent lifts shall progress from the new upslope end (to minimize runoff into the working face area) moving in the opposite direction as the previous lift.
- A new cell shall be opened once insufficient room exist for the next lift. A minimum of 200 ft. should be provided for a lift width.
- The surface runoff from unused portions of cells shall be directed away from solid waste by using temporary diversion berms.
- Cover slopes shall maximize surface runoff away from the working face and to the stormwater drainage areas to minimize leachate generation.

Efficient use of these techniques will reduce the need for intermediate over, and decrease leachate volumes.

Final cover will be applied over fill areas within 180 days after the design elevations have been achieved in the area depicted for Phase I closure shown on Sheet C-15 of the Permit Drawings. Final cover will consist of 18 inches of clayey material covered with 18 inches of native soils. The top four inches of top soil will be uncompacted and vegetated with native grasses or other vegetation to promote evapotranspiration and minimize erosion. See Sheet CD-3 of the Permit Drawings for final cap details.

5G WASTE COMPOSITION AND APPLICATION OF COVER

Cover material for daily operations of the landfills will be developed from designated borrow lakes and compost generated from yard waste recycling. As shown on Sheet C-13 of the Permit Drawings, the borrow areas will be divided into five phases, corresponding to the five phases of landfill development. The first phase will be the largest area of excavation in order to provide material necessary for construction of the initial facilities including the access roadway, compost area and perimeter stormwater ditches and Phase I of the landfill base. In addition, Phase I will provide cover material for the first five years of landfill cover needs. This material will be deposited in the location shown on Sheet G-6. The designated stockpile area will result in a stockpile no higher than 10-feet with 5:1 side slopes in order to minimize erosion. A silt fence will be installed at the toe of the stockpile area and side slopes grassed to further reduce and control erosion.

Application of initial, intermediate, and final cover is to be performed as required per Chapter 17-701 FAC. For the Class I landfill, six inches of initial cover will be applied to the top of the lift when solid waste will be placed on the working face within 18 hours. Intermediate cover consisting of 1 foot of compacted native sandy soils or compost will be applied within 7 days of cell completion if final cover or an additional lift is not to be applied within 180 days of cell completion. Any intermediate areas that will not be landfilled or covered with final cover within 6 months will be seeded and covered with wood chip, straw or other appropriate cover material to avoid slope erosion. The materials deposited in the Class III landfill will receive six inches of initial cover monthly.

Final cover will be applied to the Class I landfill once the final design grades are reached. The initial final slope on top of the landfill areas will not exceed 4 percent. The perimeter sides of all completed cells will have a slope of 5:1 to minimize erosion. Areas with final cover will be seeded or planted with grass or suitable cover vegetation.

5H OPERATIONS OF GAS, LEACHATE, AND STORMWATER CONTROLS

The Central County Solid Waste Disposal Complex will be located near the center of a 6,000 acre site. The minimum distance from the Class I landfill to the nearest property line will be 1,900 feet. The nearest exiting residence is over 8,000 feet form the proposed Class I landfill. These distances represent a substantial buffer to allow for dispersion of odors normally associated with MSW landfill operations.

The landfill gas management system design will involve a series of vertical gas wells which can be utilized as passive vents during the early development phase and later connected to an active system. This method will ensure an effective venting system during the early phases of landfill operation.

The Class I landfill will have a leachate collection system consisting of a geonet drainage layer and perforated collection pipe above the liner system to collect and convey leachate. The leachate will be conveyed to sumps and pumped to a treatment/disposal facility located onsite. The leachate will be collected via a piping system generally consisting of 8-inch perforated polyethylene pipe. This pipe will be sloped in such as manner that leachate flowing through the solid waste of the landfill is collected and transported by gravity to a sump in which a leachate pump will be located. The discharge line from the sump pump will be connected to a double walled HDPE header line via a valve vault. Provisions for sampling the leachate as well as monitoring flows and pressure will be provided in the valve vault (See Sheet CD-1) provisions will also be provided to allow discharge of accumulated stormwater from a cell prior to receiving solid wastes.

The stormwater management system for this project will consist of a series of swales, culverts and detention ponds. The system will be designed to comply with all of the requirements of both Chapters 17-25 FAC and 40 D-4 FAC.

All stormwater runoff will be conveyed to retention facilities located to the east of the old Cow Pen Slough via a perimeter drainage ditch. Ditch blocks will be located in the perimeter ditch at strategic locations. These ditch blocks will act as sediment traps and will require periodic maintenance.

The ultimate discharge of the retention facilities will be the old slough or isolated wetlands through fixed control weirs and spreader swales. The drainage and retention facilities are designed to meet or exceed all applicable requirements for treatment and water quality. In addition, the retention facilities, in combination with the proposed wetland mitigation area, are designed to meet the special stipulation that post development runoff volumes shall not exceed pre-developed runoff volumes for the mean annual storm event. (See Technical Appendix B)

The active landfill cell areas will be provided sediment treatment facilities to minimize siltation of the main retention areas. In addition, the active fill area(s) will be isolated by berming to prevent any runoff which comes in contact with the waste from being mixed with other stormwater. This runoff will be allowed to percolate into the landfill for collection by the leachate collection system.

5I GROUNDWATER MONITORING PLAN

The groundwater monitoring network and the results of the background water sampling are discussed in the geotechnical/hydrogeological investigation report, Technical Appendix A. The proposed long term monitoring network for the site is presented in the Monitoring Plan Section of (Technical Appendix A). This plan complies with Chapter 2.424(6) FAC. Monitoring well locations are shown on Sheet G-6 of the Permit Drawings.

5J ALL WEATHER ACCESS ROADS

A paved entrance will be constructed from Knights Trail Road and terminate at the landfill perimeter roadway. All weather access roads will be constructed to route traffic to the active working face. In addition, all weather perimeter roads will be constructed around the landfill areas as shown on Sheet G-6 of the Permit Drawings.

5K EFFECTIVE BARRIER

Access controls or other barriers will include a locking access gate at the property line of the CCSWDC site. In addition, the special exception area will include a perimeter fence with a locking gate at the scale house entrance.

5L SIGNS INDICATING NAME OF OPERATING AUTHORITY, TRAFFIC FLOW, HOURS OF OPERATION, AND CHARGES FOR DISPOSAL

There will be a permanent sign at the south property line along the access road to the facility identifying the Sarasota County Central County Solid Waste Disposal Facility and indicating hours of operation and charges for different types of loads. The sign will also indicate materials that are not accepted for disposal in the landfill.

Signs indicating approach and exit routes and one-way roads will be strategically placed so traffic at the landfill will move smoothly and efficiently to and from the working face area.

5M DUST CONTROL METHODS

Dust will be controlled on unpaved roads and around construction areas by a water spray truck which will wet areas immediately adjacent to the working face. Dust masks will also be available to personnel working in excessively dusty areas.

5N LITTER CONTROL DEVICES

Litter will be controlled by requiring covered loads, efficient unloading and cover operations, perimeter fencing, and by routine clean-up.

5O FIRE PROTECTION AND FIRE FIGHTING FACILITIES

Small fires on the working fence will be controlled by use of a bulldozer to move earth cover material over hot areas. In the event that an uncontrollable fire does occur at the landfill site, the Nokomis Fire Department will be contacted immediately. The Nokomis Fire Department is equipped with pumper trucks capable of drafting water from surface sources.

A hot load area will be provided in a location away from the working face to allow vehicles arriving at the landfill with a fire in their load to dump quickly in an area where the material can be spread out and quickly covered with soil. The location of the hot load area will change from time to time with the changing working face locations. Hot loads will not be dumped on the working face until sufficiently cool to avoid combustion.

5P ATTENDANT

There will be an attendant at the scale house and a landfill spotter at the working face during all operating hours. The spotter will be responsible for guiding vehicles and promoting an efficient operation during normal operating hours.

5Q COMMUNICATION FACILITIES

A telephone will be installed at the scale house and the maintenance/administration building. Radios and other communication devices will be installed in select landfill equipment to provide safe conditions for landfill personnel.

5R ADEQUATE IN-SERVICE AND RESERVE EQUIPMENT

Equipment proposed for the Sarasota County Landfill will include the equipment listed in Table 5-1. Cooperative lending agreements between other County departments and standing agreements with local equipment suppliers will provide a means for procuring additional back-up equipment.

5S SAFETY DEVICES ON EQUIPMENT TO SHIELD AND PROTECT OPERATORS

All safety devices will be maintained in accordance with recommended safe operating procedures. All landfill equipment are provided fire extinguishers.

TABLE 5-1

PROPOSED EQUIPMENT AT THE SARASOTA COUNTY LANDFILL

NUMBER	EQUIPMENT
2	Compactors
2	Bulldozers
1	Pan-Scraper
2	Roll-off Containers
1	Water Truck W/Spray Boom
1	Front-end Loader

SECTION 6

WATER QUALITY STANDARDS

The proposed landfill facility is designed to meet water quality standards of Chapters 17-3 and 17-4 of the Florida Administrative Code (FAC) by minimizing leachate formation; installing a positive barrier between deposited refuse and groundwater; collecting and treating any leachate that is generated, diverting offsite stormwater away from disposal areas; and treating stormwater runoff from active site areas.

6A(1) Surface Water Management

Prevention of surface water flow into waste filled areas will be accomplished by the design and implementation of several common engineering practices. First, the proposed perimeter road encompassing the landfill area will be elevated above existing ground elevations and thus will prevent any surface water from entering the waste filled area. Secondly, a large ditch will be located at the base slope of the landfill on the interior side of the access road. The ditch will be designed to receive runoff from the predeveloped (i.e., unused) and closed-out portions of the landfill area and convey it to the stormwater detention treatment ponds. The perimeter ditch will contain ditch blocks at strategic locations to act as sediment traps. Additionally, the proposed berm to be constructed interior to the swale will be lined to an elevation above the design high-water elevation of the swale, the liner will prevent any lateral flow into the landfill area. Finally, an impervious cap will be installed on top of the waste layers upon closure to inhibit vertical infiltration/percolation of rainfall. Technical Appendix B provides details related to the surface water management plan. Sheet CD-3 of the Permit Drawings provides details related to drainage features for the landfill.

6A(2) Leachate Management

Available options for treatment and disposal of leachate and wastewaters have been developed using the estimates of qualities and quantities of the waste streams. Based on these estimates, consideration was given to several treatment and disposal scenarios. The available options for treatment and disposal were evaluated based on capital and operating costs along with other factors including environmental protection, permeability, and operational concerns. Technical Appendix E provides the design for leachate treatment system to be used. Included in this report is a determination of leachate quantities using the HELP model and landfill phasing plan, and of leachate quality based on analysis of existing landfills of similar design and serving similar populations.

After reviewing several treatment options it was determined that an on-site treatment facility using ultrafiltration and reverse osmosis (UF/RO) will provide the most reliable and effective method for leachate treatment.

The UF/RO leachate treatment plant design is based on direct feed to the plant from the leachate sump pumps. As determined in the water balance, discussed in Technical Appendix E, a peak flow of 100,000 gpd is required for the plant. Therefore the plan is sized to treat 100,000 gpd of raw leachate flow producing 70,000 gpd of treated effluent. A 700,000 gallon leachate storage pond is provided to temporarily store peak flows. An effluent irrigation area of 70 acres is provided based on a one quarter inch per week irrigation rate. Finally an effluent storage pond providing 70 days of effluent storage is also provided. These facilities are located on Sheet G-6 of the Permit Drawings.

Calculations for the sizing of the above facilities are shown in Technical Appendix C. Leachate prevention, control, and removal system performance is addressed in Section 4.

SECTION 7

CLOSURE

7A CLOSURE PLAN

Sections 1 through 8, of Chapter 17-701.073, F.A.C, addressing general and specific site information required for closure, have been presented in Sections 1 through 6 of this document.

7A(1) Design.

Closure of the landfill site will be carried out in accordance with the phasing plan shown on sheets, C-13 of the Permit Drawings.

Final cover will consist of a minimum of 36 inches of soil, in addition to intermediate and daily cover, placed on all fill areas. The final cover consists of 18 inches of local clay with a maximum hydraulic conductivity of 1×10^{-5} cm/sec and 18 inches of local common soil material capable of supporting vegetative cover. A typical final cover profile is shown on Sheet CD-3 of the Permit Drawings.

The final cover design slope for closed portions of the landfill is 4% on the top of the cell, and 20% on the side slopes. To help prevent erosion, the large area on top of the landfill has been designed to limit the sheet flow distances from the sloped areas. This was accomplished by providing a ridge and furrow type arrangement as shown on the final closure plan Sheet C-10 of the Permit Drawings. Adequate drainage will be provided by letdown structures as indicated on Sheets C-10 and CD-3. Spacing of letdown structures was limited to 400 feet maximum in order to limit the contributing areas to each structure.

Final vegetative cover will be provided by applying hydroseed mixtures such as winter rye or browntop millet with Pensacola Bahia or Bermuda grass on flatter slopes, as appropriate and depending on the season. Pensacola Bahia sod will be used on sideslopes and in critical drainage areas.

Whenever permanent vegetation is established at a given location, liming and fertilization may also be needed. Liming will not be required if the soil pH is above 6.0.

7A(2) Final Use.

The final use of the landfill is expected to be for recreational purposes. Subject to DER approval post development land use would commence after all onsite landfilling activities are completed and sufficient time for stabilization has passed (5 to 7 years).

7(3) Closure Operations.

The Closure Plan Consist of Three Phases:

I. Activities and information developed during site operation.
II. Activities accomplished at site closure (within one year of last active disposal), and
III. Activities and information developed during 20 years after closure. This item will be addressed in this section in Paragraph 7A(4).

I. Stage I - Site Pre-Closure Operations

Normal site operations will include activities necessary for final closure decisions.

Siltation and erosion control procedures are described in Section 4, Paragraph 4D(2). Vegetative cover procedures are described in this Section, in Paragraph 7A(1). Surface water and groundwater quality monitoring is described in Technical Appendix A. Leachate monitoring is described in Section 4, Paragraph 4C(4). Methane monitoring is described in Section 4, Paragraph 4E(1).

A log of the site activities and monitoring results will be kept at the site as well as a daily log of site rainfall. Annual surveys will be made to establish quantity and in place density of solid wastes until one year after each cell closure.

II. <u>Stage II - Site Closure Operations</u>

1. Continue closure related activities from Phase I.
2. Four months prior to closing, place signs at the entrance gate stating: the site is closed; the penalty for dumping; the location and hours of operation of the alternative site; and the name of the operating agency.
3. Within one year of closure, establish all final slopes, vegetative cover and other site improvements for erosion and stormwater runoff control.
4. Final closure inspection by FDER.
5. Record closure in the official records of the Sarasota County property recording office.

7A(4) Post Closure.

a. The designated responsible party or their designee will inspect the site semi-annually for 5 years following total site closure. The inspection shall direct attention to: 1) possible settling of landfill cover that could create ponding, 2) stormwater ponds operations, 3) perimeter ditches and berms, general site vegetative cover, drainage, and erosion control, and 4) evidence of any unauthorized use of the site.

Repairs to restore the site to the intended condition will be made immediately. A record of the inspections and findings shall be kept in the supervisor's office.

b. Similar inspections shall be made annually for 20 years.

c. Groundwater and methane gas monitoring will be continued annually.

7A(5) Financial Responsibility.

Sarasota County has accepted financial responsibility for the closure of the landfill as shown in Technical Appendix F.

7B CLOSURE PLAN SCHEDULE

The landfill will be closed in portions upon reaching the final grades in each of the five phases. The closure of a phase will proceed by grading, covering, seeding, one or two sides of the phase first, prior to complete closure. One year prior to the projected date when wastes will no longer be accepted in the phase to be closed, a written notice for closure will be submitted to FDER. A Closure Permit Application will follow at least 90 days before the closing date.

Signs will be posed at the site, two months before the closing date, to advise haulers and other users of the new area in the next phase to be used after closure of the preceding area. Landfill personnel will direct the users to unload ashes in the new designated area.

For closure of the remaining phases: 1) a notice will be submitted to FDER six months before the expected closing date of the phase and 2) signs will be posted to advise users of the new fill area.

For Phase 5 closure (Site Final Closure), the following procedures will be taken: 1) one year prior to the projected closing date of the landfill, FDER will be notified of the final closure; 2) four months prior to the closing date, a sign will placed at the entrance of the facility as described in Section 7A(3) for stage II operations; and 3) ten days prior to the closing date, a notice of the landfill closure will be published in the legal section of a local newspaper. A proof of publication will be submitted by FDER.

TOC

TECHNICAL APPENDIX A

GEOTECHNICAL REPORT
AND GROUNDWATER MONITORING PLAN

See Separate Document Labeled Appendix A

TECHNICAL APPENDIX B

WMD PERMIT APPLICATION
DESIGN REPORT

PERMIT APPLICATION

SWFWMD USE ONLY

SOUTHWEST FLORIDA
WATER MANAGEMENT DISTRICT
2379 BROAD STREET • BROOKSVILLE, FL 34609-6899
(904) 796-7211 or FLORIDA WATS 1 (800) 423-1476

SURFACE WATER MANAGEMENT — CONSTRUCTION PERMIT

PERMITS ARE REQUIRED PURSUANT TO SECTION 373.413, FLORIDA STATUTES (F.S.) AND RULES 40D-4.041, 40D-40.041 AND 40D-40.042, FLORIDA ADMINISTRATIVE CODE (F.A.C.). THIS APPLICATION WILL BE PROCESSED IN ACCORDANCE WITH THE PROCEDURE DESCRIBED IN RULE 40D-1.603, F.A.C. WITHIN 30 DAYS AFTER RECEIPT OF THIS APPLICATION, THE DISTRICT WILL NOTIFY YOU IF THE APPLICATION IS COMPLETE OR MAY REQUEST ADDITIONAL INFORMATION NECESSARY TO COMPLETE THE APPLICATION. IF YOU HAVE ANY QUESTIONS REGARDING THE APPLICATION, YOU ARE ENCOURAGED TO SCHEDULE A PRE-APPLICATION CONFERENCE WITH THE DISTRICT STAFF BY CALLING THE APPROPRIATE DISTRICT PERMITTING DIVISION WHICH HANDLES PERMITS FOR PROJECTS WITHIN YOUR COUNTY.

- **PROVIDE ALL APPLICABLE INFORMATION BELOW. ATTACH ADDITIONAL INFORMATION IF NECESSARY.**
- **PLEASE SUBMIT FOUR COPIES OF APPLICATION, DRAWINGS, CALCULATIONS, ETC.**
- **A PROCESSING FEE IS REQUIRED WITH THIS APPLICATION IN ACCORDANCE WITH RULE 40D-0.201.**

I. GENERAL INFORMATION

PLEASE CHECK APPROPRIATE BOX: [X] CONSTRUCTION/OPERATION PERMIT [ ] MODIFICATION OF EXISTING PERMIT, NO.:

OWNER(S) OF LAND:	AGENT OF OWNER, AUTHORIZED TO SECURE PERMIT *(IF APPLICABLE)*:
NAME: Sarasota County	NAME: William K. Lederman, P.E.
ADDRESS: Solid Waste Dept., 8350 Bee Ridge Road	ADDRESS: Solid Waste Dept., 8350 Bee Ridge Road
CITY, STATE, ZIP: Sarasota, Fl 34241	CITY, STATE, ZIP: Sarasota, Fl 34241
TELEPHONE: (813)951-5096	TELEPHONE: (813)951-5096

PERSON WHO PREPARED PLANS AND SPECIFICATIONS:	PERSON WHO WILL CONSTRUCT PROPOSED WORK *(IF KNOWN)*:
NAME: Michael G. Taylor, P.E.	NAME: Unknown
ADDRESS: One Tampa City Center, Suite 1750	ADDRESS:
CITY, STATE, ZIP: Tampa, Fl 33602	CITY, STATE, ZIP:
COMPANY: Camp Dresser & McKee Inc. — TELEPHONE: (813)221-2833	TELEPHONE: ( )

DESCRIPTION OF LAND:

COUNTY:	SECTION	TOWNSHIP	RANGE
Sarasota	1,2,3,4,9, 10,11,12,13, 14,15,16	38S	19E

PROJECT NAME *(INCLUDING PHASE)*: Central Solid Waste Disposal Complex

TOTAL LAND AREA *(ACRES)*: 6151

PROJECT SIZE *(ACRES)* IF DIFFERENT THAN TOTAL LAND AREA: 869

BRIEF DESCRIPTION OF PROJECT: *(TYPE, LAND USE, ETC.)*

See Attachment "A", Project Description, Section 1.0

HAVE YOU HAD A PRE-APPLICATION CONFERENCE WITH DISTRICT STAFF? [X] YES [ ] NO

DATE OF CONFERENCE: / /

WITH WHOM? See Attachment A, Page 4

II. CONTENT OF APPLICATION - 40D-4.101

INSTRUCTIONS: THE FOLLOWING INFORMATION IS NECESSARY TO PROPERLY EVALUATE YOUR PERMIT APPLICATION. THE APPLICANT SHOULD SUBMIT AS MUCH OF THE INFORMATION LISTED AS THE COMPLEXITY OF THE PROJECT AND SENSITIVITY OF THE AREA NECESSITATES.

- Respond to each item leaving no blanks.
- If you feel that an item **DOES NOT** apply to your project, write **"NOT APPLICABLE"** or **"N/A"** in that space.
- Your response to each item must indicate where the requested information can be found in the accompanying plans, maps, reports, etc.
- Please be specific when identifying the plan sheet number(s), map(s), report(s), page(s), etc. that contain all supporting information.

A. SITE INFORMATION 40D-4.101(2)(a)

1 **40D-4.101(2)(a) 1** Provide a location sketch of sufficient detail to allow someone to locate the project for a site visit.

See Attachment "A", Figure 1

2 **40D-4.101(2)(a) 7** Provide recent aerial photo map(s), legible for photo interpretation with a scale no smaller than 1" = 800'; with total land area, project area, and any on-site wetlands delineated.

See Attachment "F"

3 **40D-4.101(2)(a) 3** Provide a hydrologic features map of the overall area showing existing runoff patterns and size, location, topography, and land use of off-site areas which drain through, onto, and from the project.

See Attachment "B", Sheets 1, 2 and 3

4 **40D-4.101(2)(a) 2** Provide a detailed topographic map (with contours) of the site and adjacent hydrologically related areas, which shall include the location and description of bench marks (minimum of one per major water control structure).

See Attachment "B"

5 **40D-4.101(2)(a) 6** Provide a description of vegetative cover. Location of wetland areas in and adjacent to the project area are to be identified. Quantify the acreage of existing on-site wetlands. If activities are proposed in wetlands, provide a description of wetland type and functions, and an analysis of impact alternatives that were considered.

See Attachment "D", Attachment A, Section 1.0 provides background as alternatives.

6 **40D-4.101(2)(a) 5** If the project is in the known floodplain of a stream or other watercourse, the floodplain should be identified and approximate flooding elevations determined. The 100 year flood plain elevation and limits are to be identified. If no 100 year flood plain exists on site, please state below.

See Attachment "A", Appendix C

7 **40D-4.101(2)(a) 4** Provide identification of wet season high water table elevations, including normal pool and seasonal high elevations in wetlands

Attachment C, Figure F for wet season water table.

See Attachment "D" for wetland water elevations.

8 **40D-4.101(2)(a) 9** Provide soils information representative of actual conditions; including percolation tests, if effluent filtration, percolation or exfiltration systems are proposed.

See Attachment "C"

9 **40D-4.101(2)(a) 8** Provide construction drawings signed and sealed by the design engineer for the paving, grading, and drainage plans with special attention to perimeter site grading.

See Attachment "H"

10 **40D-4.101(2)(a) 10** Provide a complete description of measures to be implemented during the construction period to mitigate adverse quantity and quality impacts off-site, including erosion or sediment control measures to be employed.

See Attachment "A", Sections 5.0 and 8.1

B. MASTER DRAINAGE PLAN 40D-4.101(2)

1 **40D-4.101(2)(b) 1** Provide the location of all water bodies with details of size, side slopes, elevations and depths used for water quantity, quality and environmental functions, including compensation plan details for wetland impacts and floodplain enroachment.

See Attachment "B" and Attachment "E" and Attachment "G"

2	40D-4.101(2)(b) 2	Provide the location and site details of all major water control structures, including design elevations of the discharge structures along with any seasonal water level fluctuation or drawdown schedules.

ee Attachment "B" and Attachment "G"

3	40D-4.101(2)(b) 4	Provide the locations of roads and buildings along with their proposed elevations.

See Attachment "H"

4	40D-4.101(2)(b) 6	Provide the location and size of internal minor water management facilities.

See Attachment "B" and Attachment "H"

5	40D-4.101(2)(b) 5	Provide the right-of-way and easement locations for the drainage system including all areas to be reserved for water management purposes, with the legal method to be utilized.

See Attachment "A", Appendix i

6	40D-4.101(2)(b) 3	Provide drainage basin boundaries showing direction and path of flow, taking into account off-site runoff being routed through or around the project.

See Attachment "B", Sheets 1, 2 and 3

7	40D-4.101(2)(b) 7	Provide the location and description of any nearby existing off-site water resource facilities which might be affected by the proposed construction or development. The names and addresses of the owners of such facilities should also be submitted.

See Attachment "C", Table 11 and Attachment "A", Section 9.0

C. DRAINAGE CALCULATIONS 40D-4.101(2)(c)

1	40D-4.101(2)(c) 1	Provide the design storms used including depth, duration and distribution.

See Attachment "B", Section 2.2

2	40D-4.101(2)(c) 2	Provide the details of any off-site inflows.

ee Attachment "B", Section 2.1 and Sheet 3

3	40D-4.101(2)(c) 4	Provide the acreages and percentages of property proposed as: a. Impervious surfaces (excluding waterbodies); b. Pervious surfaces (green areas); c. Lakes, canals, retention areas, etc.; d. Wetlands; and, e. Total acreage of project.

See Attachment "B", Section 3.3

4	40D-4.101(2)(c) 3	Provide the stage-storage computations for the project and stage-discharge computations for the outfall structure(s).

See Attachment "B", Section 3.4

5	40D-4.101(2)(c) 5	Provide the runoff calculations signed and sealed by the design engineer showing pre- and post-development discharges, elevations, and volumes retained and/or detained during applicable storm events; including the hydrologic parameters and modeling input and output data. Mathematical computations may be required to demonstrate that the proposed development will not significantly alter net storage from the project area for events up to the required design storm nor cause adverse affects due to floodplain encroachment up to the 100 year event.

See Attachment "B", Section 3

6	40D-4.101(2)(c) 6	Provide the calculations required for determination of minimum building flood and road elevations.

See Attachment "B", Table 3-3

D. LEGAL AND INSTITUTIONAL INFORMATION 40D-4.101(2)(d)

1	40D-4.101(2)(d) 4	Provide the identification of agencies and organizations contacted in connection with the project. Include meeting summaries and/or responses. Give the status of local approvals indicating if site plan and/or subdivision approval has been granted, final plats recorded and building or construction permits issued.

See Attachment "A", Section 1.0 and Section 2.0

2	40D-4.101(2)(d) 6	Provide a copy of the boundary survey and/or a legal description and acreage of the total land area of contiguous property owned or controlled by the applicant adjacent to and including the project. Also provide a legal description and acreage of the project area required to construct, operate and maintain the system; if different from the total land area. Provide a copy of the deed or other evidence of ownership.
	See Attachment "A", Appendix d, e, f and i	
3	40D-4.101(2)(d) 5	Provide the present zoning classification and status of any DRI application or proposed zoning reclassification.
	See Attachment "A", Section 2.0, Figure No. 2	
4	40D-4.101(2)(d) 3	Provide indication of how water and wastewater service will be supplied. Letters of intended commitment from off-site suppliers must be included.
	See Attachment "A", Section 10.0	
5	40D-4.101(2)(d) 7	Provide documentation of legal and physical availability of the receiving water system to receive project discharge, if such is not evident.
	N/A	
6	40D-4.101(2)(d) 1	Identify the entity to be responsible for operation and maintenance of the Surface Water Management System upon completion of construction. Include all operation and maintenance guidelines and schedules.
	See Attachment "A", Section 8.0	
7	40D-4.101(2)(d) 2	Provide a letter or other evidence of potential acceptance by the operation and maintenance entity, if the entity is to be a public body such as a city or drainage district. If the entity is a homeowners or other association, documents verifying either the present or imminent existence of such an organization and its ability to accept operation and maintenance responsibility are required.
	See Attachment "A", Appendix h	

E. OTHER INFORMATION

1	17-21.10(4)	If there are any existing wells located within the project site, please indicate how they will be utilized or properly abandoned by a licensed water well contractor in accordance with Rules 40D-3 and 17-21.10(4), F.A.C.
	See Attachment "A", Section 9.0	
2		Are there existing Consumptive Use Permits from Southwest Florida Water Management District within the total land area? If so, provide the permit number(s).
	No	
3		Provide letter or other certification from the owner clearly identifying and authorizing the agent who is applying for the permit to sign for and bind the owner, if applicable.
	See Attachment "A", Appendix g	
4	40D-4.101(3)	The application must be signed by the owner or his authorized agent, and submitted to the District with four copies of all information required in subsections 40D-4.101 (1) and (2).

COMMENTS:

III. APPLICATION CERTIFICATIONS

• STATEMENTS BY APPLICANT •

A. I hereby acknowledge that:

1. Construction of any portion of this project prior to receiving a District Construction Permit is a violation of Section 373.413, Florida Statutes (F.S.), and Rules 40D-4.041, 40D-40.041 and 40D-40.042, Florida Administrative Code.

2. The District has the authority, pursuant to Chapter 373, F.S., to enter and inspect the property described in this application for the purpose of determining compliance with district rules.

B. By executing this application the applicant states that the property legally described in the survey drawing and/or legal description of the total land area is owned/controlled by the undersigned applicant and encompasses the project area referenced in the permit application.

C. Within 30 days after completion of the permitted Surface Water Management System, the owner shall submit to the District a Statement of Completion. The Statement of Completion shall contain As-Built drawings and certification by a Professional Engineer that the project has been constructed in accordance with the permitted design and a request to transfer the permit to an approved operation and maintenance entity. The owner hereby agrees to retain the below-named Engineer to be in responsible charge of observing on-site construction of this project and submitting the As-Built drawings and certification.

ENGINEER'S NAME	FL REGISTRATION NO.	COMPANY NAME
Thomas G. Walker	35948	Camp Dresser & McKee Inc.

ADDRESS	
201 Montgomery Ave.	
CITY, STATE, ZIP	**PHONE**
Sarasota, Fl 34243	(813) 351-7100

D. By executing this application the owner hereby agrees to accept responsibility for operation and maintenance of the Surface Water Management System authorized by the permit. If the owner is required to or otherwise intends to request transfer of the permit upon completion of the system to another entity who shall be responsible for operation and maintenance, the owner shall provide the information requested below. (See Permit Information Manual, Part B, Section 3.1.6).

OPERATION & MAINTENANCE ENTITY	NAME OF CONTACT PERSON *(PRINT)*
Sarasota County	William K. Lederman

ADDRESS	
Engineer IV, Solid Waste Dept., 8350 Bee Ridge Road	
CITY, STATE, ZIP	**PHONE**
Sarasota, Fl 34241	(813) 951-5096

E. I hereby certify that the information contained herein is true and accurate and that I have legal authority to execute this application to perform construction activities on the property identified herein.

OWNER'S NAME *(PRINT)*		NAME OF PERSON SIGNING APPLICATION *(PRINT)*
Sarasota County		William K Lederman
SIGNATURE OF OWNER OR AUTHORIZED AGENT:	**DATE**	**TITLE *(IF APPLICABLE)***
X William K Lederman	5, 14, 92	**COMPANY NAME *(IF APPLICABLE)***

NOTE: INFORMATION REQUESTED BELOW, OTHER THAN THE SIGNATURE, DATE AND SEAL, NEED ONLY BE PROVIDED IF DIFFERENT FROM THAT PROVIDED IN SECTION "C", PAGE 5 OF 6.

IV. APPLICATION CERTIFICATIONS

• STATEMENT BY PROFESSIONAL ENGINEER REGISTERED IN FLORIDA •

This is to certify that the engineering features of this Surface Water Management System, designed by me or under my responsible charge, are designed in my professional opinion in conformity with sound engineering principles and in accordance with all applicable rules and specifications of Chapters 40D-4 and 40D-40, Florida Administrative Code (F.A.C.), including the "Basis of Review for Surface Water Management Permit Applications" referenced in Rule 40D-4.091, F.A.C. I further agree that I or my engineering firm will furnish the owner with a set of guidelines and schedules for maintenance and operation of the Surface Water Management System.

ENGINEER'S NAME *(TYPE OR PRINT)* Michael G. Taylor	FL REGISTRATION NO. 36664	• AFFIX SEAL •
COMPANY NAME Camp Dresser & McKee Inc.		
ADDRESS One Tampa City Center, Suite 1750		
CITY, STATE, ZIP Tampa, Fl 33602	PHONE (813) 221-2833	
ENGINEER'S SIGNATURE X	DATE 5/14/92	

ATTACHMENT B

STORMWATER MANAGEMENT FACILITY DESIGN REPORT

SARASOTA COUNTY

CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX SITE

PREPARED FOR:

SARASOTA COUNTY

Prepared by:

CAMP DRESSER & McKEE INC.
201 Montgomery Avenue
Sarasota, Florida 34243

May 1992

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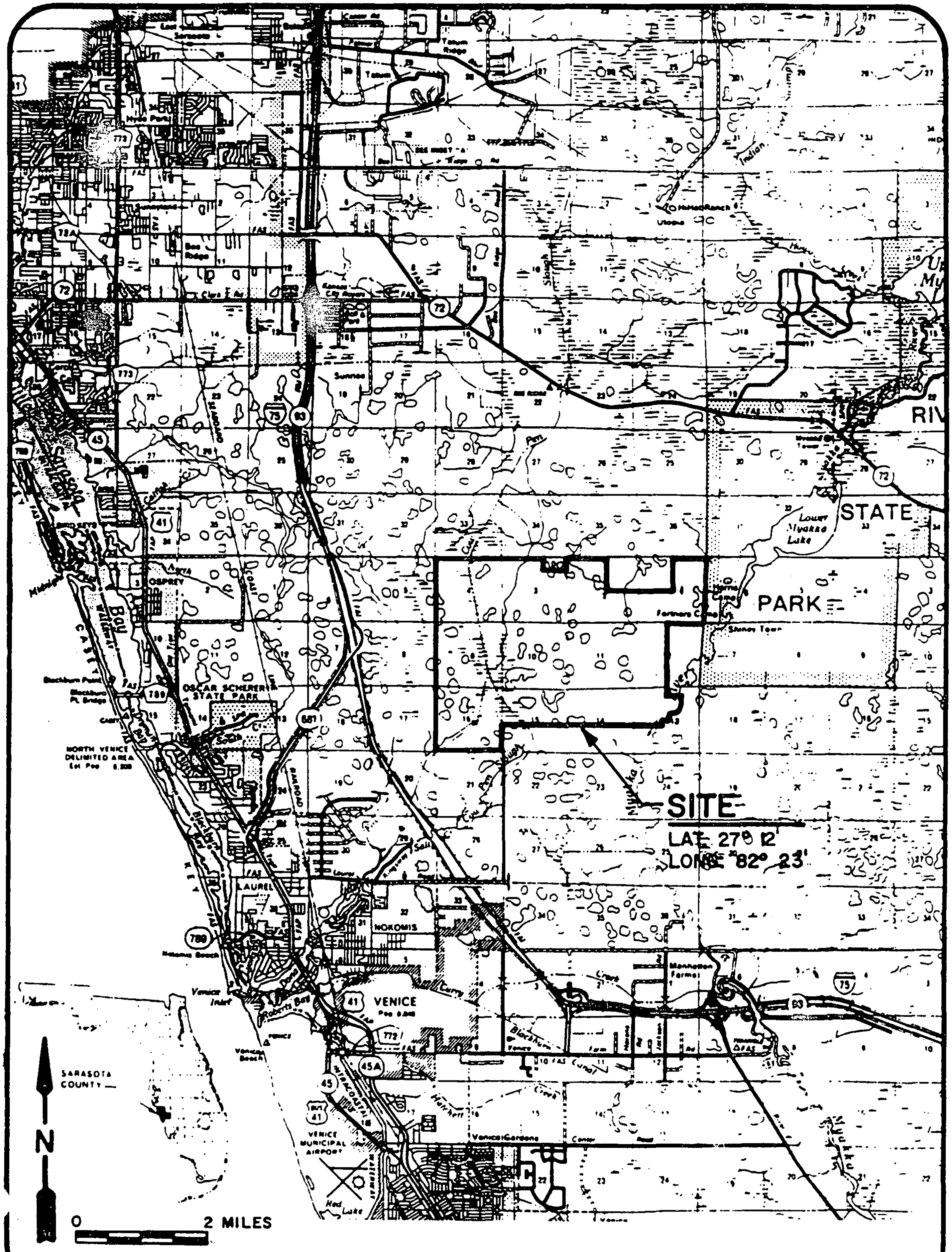
1.0 INTRODUCTION

1.1 PROJECT LOCATION

The Sarasota County Central Solid Waste Disposal Complex Site is located on a 6,151-acre parcel, called the Walton Tract, located approximately 1-1/2 miles east of Interstate 75 and two miles north of Laurel Road in central Sarasota County (see Figure 1-1).

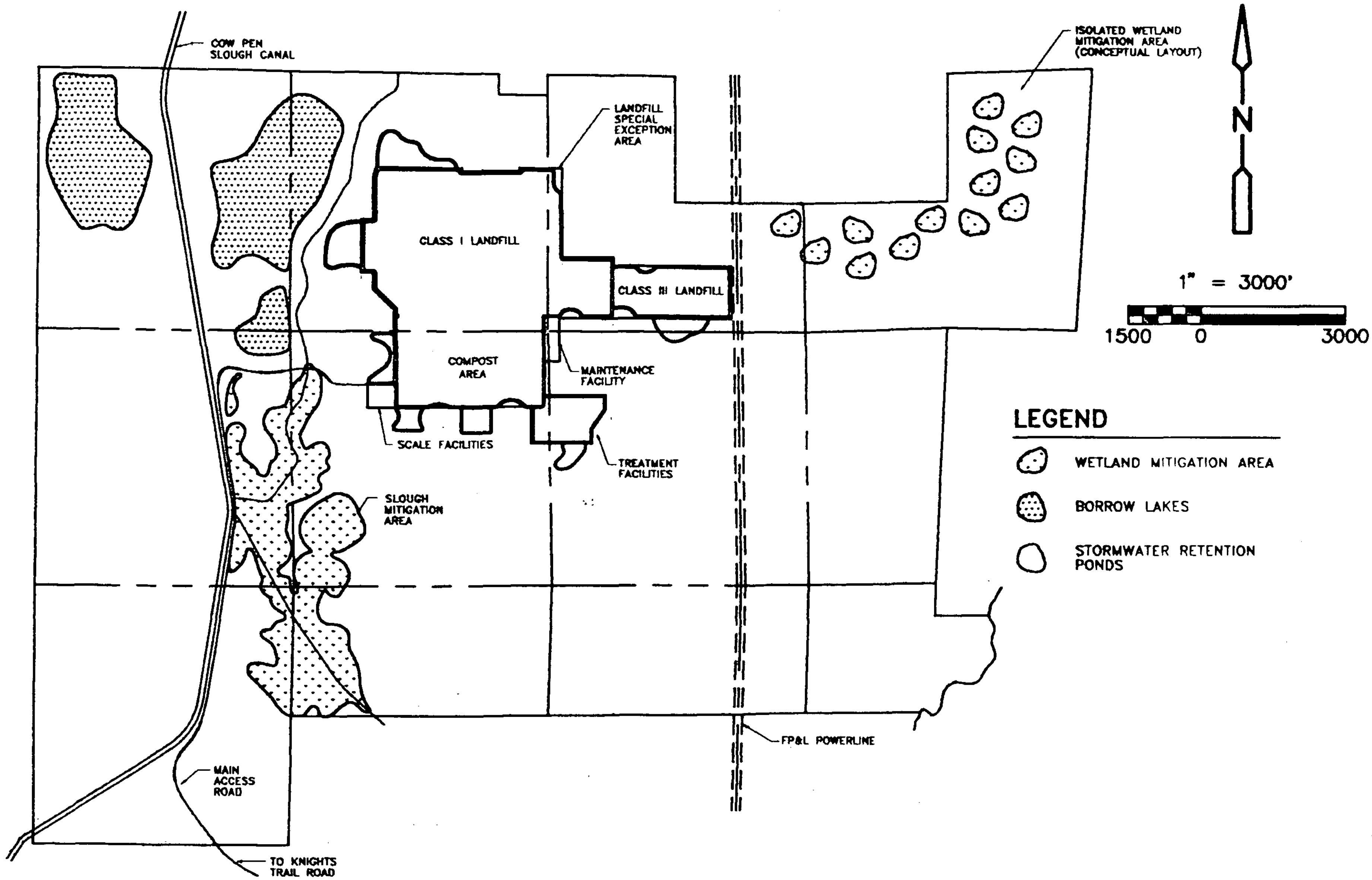
The proposed active solid waste disposal facilities include a 294-acre Class I landfill, a 55-acre Class III landfill, a 133-acre compost area, a 30-acre leachate treatment and sludge dewatering facility, a 7-acre maintenance yard, a 6-acre entrance/scale facilities area and approximately 15-acres for internal roads and ditches. These facilities are to be located within a designated 550-acre special exception area for solid waste facilities which is located at the approximate center of the parcel. Locations of the proposed facilities are shown on Figure 1-2. In addition, a 3.2-mile long access road, approximately 100-feet in width, will be constructed from the intersection of Knights Trail Road and Rustic Road (one mile north of Laurel Road) to the site entrance. Also, three borrow pits totalling 278-acres will be excavated within the parcel to provide cover and fill material for construction of the solid waste disposal facilities.

The solid waste disposal facilities will be constructed in phases over an approximate 40-year period and will necessitate the filling of approximately 83 acres of SWFWMD jurisdictional wetlands for the landfill. As mitigation for the filling of these and other wetlands along the access road route, a number of isolated wetlands similar to those being impacted will be constructed in the northeast portion of the County's parcel. Also, creation of a large area of wetlands and restoration of severely drained and highly impacted wetlands along the Cow Pen Slough and Cow Pen Slough Canal will be accomplished. Rehydration of this area will occur by constructing weir structures within the drained slough and replacing control structures along the canal.



environmental engineers, scientists, planners, & management consultants **CDM**

FIGURE 1-1



SARASOTA COUNTY
CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX

FACILITIES SITE PLAN

CDM
environmental engineers, scientists, planners, & management consultants

Figure No. 1-2

1.2 BASIN DESCRIPTION

The site of the new Sarasota County Central County Solid Waste Disposal Complex is an area of minor topographical relief characterized by a number of wetlands. There are three major drainage basins of which the 6,151-acre site is a part:

- The Myakka River drains a portion of the site on the eastern boundary.

- A major man-made conveyance system, the Cow Pen Slough Canal, cuts through the 6,151-acre site on the western portion, flowing south into Shackett Creek and subsequently into Dona Bay. Portions of the site drain directly into the canal via culverts placed through the canal spoil banks. During extreme flood events, direct drainage from the site into the canal can occur via low segments purposely left in the canal spoil banks.

  The Cow Pen Slough Canal was constructed during the late 1960's and was designed to provide flood control for a 10-year flood event. This system severed most of the upstream drainage area to the Old Cow Pen Slough and presently handles the flow from most of the Cow Pen Slough Drainage Basin. The southern boundary of the Central Solid Waste Disposal Complex site lies at river mile 2.27 of the canal. From this point, the upstream contributing area is approximately 65 square miles. This area represents approximately 85 percent of the entire Cow Pen Slough Canal watershed.

  The canal system will receive a minimal impact as a result of this project. This impact will consist primarily of replacing the existing connection between the Cow Pen Slough Canal and the north leg of the Old Cow Pen Slough and some spoil bank modifications where the access road will traverse the east bank. Due to these minimal impacts, a detailed study was not conducted on the Cow Pen Slough canal system.

- The third major drainage system of which the complex is a part is the Old Cow Pen Slough drainage basin. This basin originates north of the site boundary and flows south, traversing the site and discharging into the canal under normal flow conditions. This northern off-site drainage area is approximately 1430 acres. The land use in this area is agricultural, predominantly improved pasture lands for cattle grazing. This area is served by a ridge and furrow type irrigation/drainage system.

  There is also another portion of the Old Cow Pen Slough, called the southern leg, which is no longer connected to the northern leg, but can still serve to convey excess flood water that cannot be discharged into the canal. The south leg of the Old Cow Pen Slough was channelized in the 1950's to control flooding in the farm areas to the north. Both upstream and downstream of the site, control structures are installed to manage the flows in the system. These structures consist primarily of flashboard risers and do not appear to be operated regularly.

1.3 STORMWATER MANAGEMENT FACILITY DESIGN CRITERIA

The stormwater management system for the proposed solid waste disposal complex site will consist of a series of swales, channels, culverts, wet detention ponds, and water control structures which will be combined into an overall plan to provide both treatment of stormwater runoff from the site and integrate with a wetland mitigation plan to compensate for wetland impacts on the site. The system is designed to comply with requirements of both Chapter 17-25 F.A.C and 40D-4 and 40D-40 F.A.C.

2.0 STORMWATER MODEL DEVELOPMENT - EXISTING CONDITIONS

For the purpose of this design, the Sarasota County Central County Solid Waste Disposal Complex site drainage basin was modeled using the Advanced Interconnected Pond Routing Model (ADICPR) Version 1.4 marketed by Streamline Technologies, Inc. Advanced ICPR is used to model hydrologic conditions to generate a hydrograph from a drainage subbasin and to hydrodynamically route stormwater flow through the stormwater conveyance system. As such, it combines the hydrologic and hydraulic modeling functions into one model.

ADICPR is a sophisticated and powerful stormwater management analysis and design tool. There are routines to input and edit data, compute runoff hydrographs and route flood hydrographs through complex pond and conveyance systems. Time-variable tailwater conditions, flow reversals and looped hydraulic networks are included in its solution algorithm. ADICPR is able to analyze an extensive array of hydraulic structures, including culverts under all flow regimes. Results can be reviewed and analyzed with an extensive data base retrieval system. A variety of tabular and graphical reports can be easily generated for review by the user or for submittal to reviewing agencies.

2.1 DRAINAGE FACILITY INVENTORY MAPPING AND SUBBASIN DELINEATION

The first step in the development of any stormwater management model is the inventory and definition of existing drainage facilities and drainage patterns. This required much field reconnaissance of the site and the survey of cross-sections, dimensions and inverts of major drainage features such as culverts and defined channels. The basis for mapping of drainage patterns was 1"=200' aerial topographic mapping provided by the County for the entire 6151-acre site. In addition, SWFWMD aerial topographic maps of the same scale were used to supplement the more recent mapping of the site and to provide the basis of defining the northern and southern offsite drainage basins.

Overlays to these aerial topographic maps were prepared which show major drainage facilities and subbasin delineations. CDM personnel conducted a field reconnaissance of the basin while recording drainage patterns, mapping drainage facilities, measuring facility dimensions, and inspecting the condition of the drainage facilities in the field. Subbasin boundaries were defined for each of the individual isolated wetlands on the site. For modeling purposes, only those portions of the site which were determined to drain to Cow Pen Slough or to the Cow Pen Slough Canal were considered in the drainage system definition. Sheets 1 and 2 show the existing on-site drainage system and subbasin delineations which were defined for modeling purposes. A total of 106 separate subbasins are shown on these sheets, each defining a separate wetland system and given a corresponding wetland number designation. The average area of each subbasin is 35 acres.

In addition, the northern offsite contributing drainage area of the north leg of Old Cow Pen Slough was divided into a more coarse set of 23 subbasins averaging 62 acres in area. This northern offsite drainage basin is defined on Sheet 3.

On the southern boundary of the site, a southern offsite drainage area was defined to the point of the Rustic Road crossing of the south leg of Cow Pen Slough and what was the original unchannelized Cow Pen Slough. This southern offsite area, defined by 5 subbasins averaging 77 acres in drainage area, is shown on Sheet 2. The model was extended to this point for the purpose of sizing the new solid waste disposal complex access road culverts where the road will cross the slough and old slough channels.

There is a 168-acre drainage area of the site (see Sheet 1) on the eastern boundary (Subbasins 88, 89, 90, 97, 100, 103, and 105) which was determined to presently drain eastward into the Myakka River. The historical drainage pattern of these areas is difficult to determine due to disruptions of flow caused by the FP&L power line easement and off-road vehicle trails within this area. These drainage subbasins were incorporated into the stormwater management model since the Class III landfill will directly impact these basins. The model boundary point was defined at the FP&L power line easement where Subbasin 105 discharges through a 36" CMP culvert.

2.2 HYDROLOGIC DATA

Hydrologic modeling with the ADICPR model was performed utilizing the SCS Unit Hydrograph method as a means of computing stormwater runoff rates into the conveyance system. The SCS Unit Hydrograph method requires the following information:

1. A non-dimensional unit hydrograph with associated peaking factor.
2. Drainage area.
3. Percentage of directly connected impervious areas.
4. Runoff curve numbers for all areas which are not considered to be directly connected impervious areas.
5. Times-of-concentration.

Whenever the SCS Unit Hydrograph method is selected, a non-dimensional unit hydrograph must be specified. The unit hydrograph and peaking factor directly affect the peak runoff rate. The proper selection of a unit hydrograph depends on the geographical area and local conditions. For example, flatter coastal areas with above-average depressional storage would be expected to have lower peaking factors than hilly areas. ADICPR has three standard curvilinear unit hydrograph files included. These correspond to peaking factors of 256, 323 and 484. ADICPR also allows the user to vary unit hydrographs and peaking factors with each drainage basin. The user is not limited to a single unit hydrograph for all basins. In the existing conditions model a unit hydrograph peaking factor of 256 was used for all subbasins.

As previously discussed, to develop the model of the solid waste disposal complex site, the drainage basin was divided into the 134 discrete subbasins shown on Sheets 1, 2, and 3. These subbasins were delineated from aerial topographic mapping and by field reconnaissance of the study area. Subbasin designations were selected to correspond to individual isolated wetlands as much as possible. Subbasin areas were measured from the 1" = 200' scale aerial photography. Table 2-1 lists subbasin areas and other hydrologic data for each of the model subbasins.

Directly connected impervious area (DCIA) is defined as impervious areas that are connected to the basin outlet point (i.e. node) without flowing over any pervious surfaces. In this model application, only water surface areas associated with pond surfaces were included as DCIA. Wetland areas determined to be the extent of water during the normal wet season were input as DCIA since the wetland itself is considered the storage node in model formulation. This area generally coincides with what is considered to be a seasonal high water elevation in the wetland unless it is drained artificially by a man-made structure such as a channel and/or culvert.

Hydrologic modeling with the ADICPR model was performed using the SCS Runoff Curve Number Method to compute a runoff volume from rainfall. The runoff curve number (CN) is a function of a number of factors including hydrologic soil group, cover type, treatment, hydrologic condition, water table elevation and antecedent moisture condition.

The natural soil types and hydrologic soil groups within the site are mostly poorly drained to very poorly drained soils, primarily Pineda and EauGallie fine sands, with hydrologic soil group classifications of D and B/D. All of these soil types have depths to seasonal high water table documented as less than 10 inches for one or more months during the year. The areas east of the Cow Pen Slough (north and south leg) have not been altered enough to lower the groundwater table and allow a soil group classification of B to be used. The existing land use of the subject area is mostly palmetto prairies and pine flatwoods broken up by freshwater marsh wetland systems. Based on the ground cover, hydrologic soil group, and poorly drained soil

TABLE 2-1

ADICPR WETLAND NODE DATA SUMMARY
EXISTING CONDITIONS

Wetland Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
4	20.1	79	12.8	29	16.5	17.1
6	33.8	79	10.6	30	15.5	17.1
11	14.7	79	28.1	21	16.5	17.0
12	14.6	79	8.2	28	16.0	17.0
14	36.6	79	7.0	34	15.4	16.3
15	36.2	79	0.3	32	15.5	15.9
17	28.1	79	14.3	25	15.0	15.8
18	50.9	79	23.1	36	15.0	15.8
20	160.6	79	0.5	31	8.0*	15.8
21	37.5	79	22.5	32	15.0	15.7
29	54.7	79	19.2	30	14.8	15.8
31	74.8	79	4.8	55	14.0	15.6
33	29.8	79	1.0	25	12.4*	15.6
34	27.6	79	3.2	27	12.8*	15.3
35	64.8	79	13.4	35	17.1	18.0
36	28.6	79	6.4	30	18.2	18.6
37	17.3	79	10.6	21	18.0	18.7
38	32.5	79	1.1	34	16.0	17.6
38A	35.7	79	3.1	37	16.3	17.8
39	50.0	79	1.1	48	16.0	17.4

*** Wetland Drained By Channel or Culvert**

TABLE 2-1 (continued)

ADICPR WETLAND NODE DATA SUMMARY
EXISTING CONDITIONS

Wetland Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
40	15.4	79	12.8	17	16.4	17.2
41	29.7	79	8.8	27	15.5	16.8
42	33.0	79	0.6	32	11.1*	16.0
43	10.4	79	2.1	28	13.4*	16.0
44	7.9	79	2.3	25	13.1*	15.8
45	62.8	79	0.6	30	12.0*	16.2
45A	64.0	79	0.6	35	12.5*	16.7
47	26.4	79	0.4	40	13.3*	15.8
48	43.9	79	0.7	27	11.1*	15.9
49	17.8	79	1.1	27	11.1*	15.8
49A	59.9	79	0.8	30	10.0*	15.8
50	205.2	79	0.7	48	8.9*	15.7
51	21.6	79	28.9	17	15.8	16.3
52	42.7	79	42.8	25	16.5	17.0
54	49.2	79	5.4	29	15.0	15.9
56	17.4	79	15.8	29	17.9	18.2
57	51.7	79	1.6	35	13.7*	17.9
58	34.2	79	37.9	25	19.2	20.2
59	9.5	79	26.7	17	19.4	20.2
60	18.2	79	35.4	17	19.4	20.2

* Wetland Drained By Channel or Culvert

TABLE 2-1 (continued)

ADICPR WETLAND NODE DATA SUMMARY
EXISTING CONDITIONS

Wetland Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
61	40.7	79	0.0	27	17.5*	20.5
62	12.1	79	32.8	17	20.8	21.4
63	11.2	79	13.8	30	19.0	19.7
64	8.8	79	12.9	17	20.0	20.5
65	8.2	79	19.6	17	19.5	20.5
66	21.9	79	16.4	29	19.4	20.4
67	15.8	79	30.2	17	20.1	20.7
70	103.9	79	15.6	37	19.5	21.2
71	52.8	79	1.0	35	13.5*	17.9
71A	72.6	79	1.3	33	13.0*	17.5
72	18.0	79	15.3	25	18.6	19.0
73	21.0	79	19.0	27	19.0	19.5
74	29.0	79	13.3	27	20.0	20.8
75	28.1	79	24.1	17	19.5	19.9
76	27.4	79	2.9	35	20.0	20.8
77	7.2	79	20.4	17	20.5	21.2
78	32.6	79	24.3	25	20.5	21.2
79	31.1	79	23.6	25	20.5	21.3
80	18.2	79	35.4	17	20.2	21.2
81	33.7	79	19.9	27	20.0	21.2

* Wetland Drained By Channel or Culvert

TABLE 2-1 (continued)

ADICPR WETLAND NODE DATA SUMMARY
EXISTING CONDITIONS

Wetland Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
82	39.3	79	19.2	31	18.0	18.4
83	15.7	79	29.8	17	18.0	18.4
84	37.6	79	5.0	38	18.5	19.6
85	22.3	79	33.3	25	19.7	20.2
86	15.0	79	4.7	33	20.1	20.7
87	13.4	79	31.9	17	19.5	20.4
88	30.6	79	14.3	29	19.8	21.0
89	7.9	79	27.8	13	20.6	21.1
90	8.3	79	12.4	17	19.9	20.2
92	19.7	79	31.1	17	17.5	18.3
92A	39.0	79	0.0	48	17.2	18.6
93	19.1	79	18.2	25	18.5	18.9
94	23.7	79	19.0	27	18.7	19.3
95	12.1	79	28.4	17	19.1	20.4
96	6.7	79	13.2	17	19.4	20.4
97	30.7	79	3.0	40	20.0	21.0
98	23.0	79	14.2	37	18.6	19.2
99	15.7	79	16.1	17	19.2	19.8
100	15.4	79	17.6	27	19.7	20.8
103	9.7	79	15.8	25	19.0	20.0

* Wetland Drained By Channel or Culvert

TABLE 2-1 (continued)

ADICPR WETLAND NODE DATA SUMMARY
EXISTING CONDITIONS

Wetland Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
105	65.8	79	6.7	35	17.5	19.3
106	15.6	79	9.0	25	18.0	18.7
107	14.9	79	4.8	33	17.9	18.7
108	24.0	79	30.7	27	18.1	18.7
109	48.4	79	24.4	30	17.8	18.4
110	16.6	79	15.5	28	17.8	18.5
111	14.4	79	5.9	28	17.8	18.6
112	4.6	79	12.4	17	18.5	18.6
113	58.4	79	22.6	28	15.8	16.2
114	27.1	79	24.1	27	16.6	17.5
115	21.0	79	32.4	25	17.2	17.8
116	28.8	79	29.9	27	16.9	17.4
118	9.3	79	18.5	25	17.7	18.3
120	16.1	79	21.0	25	16.0	16.5
121	14.1	79	27.8	25	16.0	16.7
122	68.9	79	18.7	40	16.8	17.4
123	64.5	79	15.1	32	16.5	17.6
127	86.2	79	19.7	40	14.1*	15.8
127A	36.5	79	22.8	27	14.0*	15.7
128	12.7	79	19.7	17	13.5*	15.7

* Wetland Drained By Channel or Culvert

TABLE 2-1 (continued)

ADICPR WETLAND NODE DATA SUMMARY
EXISTING CONDITIONS

Wetland Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
129	26.8	79	19.7	27	15.5	16.2
130	104.8	79	13.2	46	15.5	16.3
131	24.9	79	15.6	17	16.5	17.2
133	171.6	79	15.1	35	15.7	16.4
142	17.3	79	23.0	25	16.0	17.0
143	29.9	79	7.4	27	16.0	17.1
ON1	43.2	79	37.3	17	25.0	25.8
ON2	205.5	79	12.5	41	23.5	25.5
ON3	51.3	79	0.4	17	22.2	25.3
ON4	111.4	79	0.0	81	24.0	26.4
ON5	41.2	79	15.4	25	24.0	25.2
ON6	41.8	79	18.2	26	21.0	23.1
ON7	292.3	79	25.1	29	19.8	21.6
ON8	25.0	79	3.7	17	19.1	21.6
ON9	49.5	79	4.9	28	20.3	21.4
ON10	33.6	79	2.9	32	20.5	21.4
ON11	103.4	79	27.4	25	20.3	21.4
ON12	28.4	79	19.4	17	19.8	20.5
ON13	34.0	79	0.8	21	19.0	21.4
ON14	19.9	79	17.6	25	19.0	21.6

*** Wetland Drained By Channel or Culvert**

TABLE 2-1 (continued)

ADICPR WETLAND NODE DATA SUMMARY
EXISTING CONDITIONS

Wetland Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
ON15	43.8	79	13.4	25	19.5	21.6
ON16	29.4	79	21.7	17	20.5	21.4
ON17	48.8	79	8.8	25	19.0	21.1
ON18	53.2	79	8.6	25	19.0	20.5
ON19	53.9	79	8.7	27	19.0	21.2
ON20	41.5	79	9.7	27	19.5	21.2
ON21	21.4	79	0.0	20	20.5	21.6
ON22	14.9	79	14.8	25	19.5	21.4
ON23	42.3	79	25.9	25	20.5	21.2
OS1	93.8	79	0.3	32	11.8*	14.4
OS2	61.6	79	0.0	37	11.8*	15.2
OS3	95.9	79	1.7	46	12.8*	15.2
OS4	84.0	79	7.9	25	9.0*	15.0
OS5	48.3	79	0.7	32	6.4*	14.1

* **Wetland Drained By Channel or Culvert**

condition, a CN value of 79 was used for the unimproved upland areas, and wetlands were treated as directly connected impervious area. Table 2-1 lists the CN values used in the ADICPR model.

The subbasin time-of-concentration (TC) represents the amount of time it takes for a particle of water to travel from the hydrologically most distant point in the drainage basin to its outlet. Several methods can be used to calculate this parameter, but the most common and universally accepted method is that detailed in the SCS TR-55 Manual, Urban Hydrology for Small Watersheds. In this method, TC is computed by summing all the travel times for consecutive components of the drainage conveyance system. TR-55 provides graphs for computing flow times for sheet flow, shallow concentrated flow, and open channel flow as a function of the lengths, slope and type of flow path. Table 2-1 lists TC's used in the ADICPR model of the existing drainage system.

In accordance with flood control criteria which have been adopted by the Southwest Florida Water Management District, the primary basis for the design of the disposal complex stormwater management facilities was the 25-year, 24-hour storm event. The total rainfall for this event is 8.0 inches, as determined from the SWFWMD MSSW Permit Information Manual. In accordance with SWFWMD design criteria, the SCS Type II Florida-Modified distribution curve was used to develop the design storm hyetographs for the modeling runs.

2.3 HYDRAULIC DATA

Advanced ICPR hydrodynamically simulates unsteady flow in a drainage system that consists of open channels, ponds, culverts, and a variety of control structures. The model takes subbasin runoff hydrographs and routes them through a series of reaches. The model can also accommodate time-varying boundary conditions.

The ADICPR program requires input from five basic data groups to describe a stormwater system. The data groups are: control and initialization data, stage/storage data, conveyance reach data, boundary conditions, and inflow hydrographs.

2.3.1 CONTROL AND INITIALIZATION DATA

As previously discussed, the project area was divided into 134 drainage subbasins for input into the ADICPR model. Each of these subbasins represents an area contributing runoff to a wetland which acts as a natural reservoir for the existing drainage system. These wetlands serve to attenuate the peak runoff rates from the contributing areas by detention and reduce the volume of runoff through retention. The wetlands are know as nodes in the nomenclature used for the ADICPR model. The starting storage elevation in each of the nodes was determined by interpreting seasonal high water table and normal pool elevations selectively flagged and surveyed in the field. The nodes are interconnected by reaches represented as weirs or channels to form systems. The reach invert elevations were determined by interpreting aerial topographic mapping, surveyed cross-sections, and seasonal high water table elevations.

To initiate the model simulation of the 25-year storm event in the basin, each wetland node must have a starting water surface elevation. It was assumed that this design storm event would occur during the wet season and the wetlands would be at or near their normal seasonal high water (not to be confused with an extreme seasonal high water). The initial starting water surface elevations for model simulation are listed with the subbasin data in Table 2-1. Note that those wetlands along the canal and both legs of the slough, which are directly drained and severely impacted by these man-made conveyances, were started at their respective drained control elevations, generally coinciding with channel bottom and/or control weir inverts. These nodes are asterisked in Table 2-1.

2.3.2 STAGE/STORAGE DATA

A stage/storage relationship was derived for each of the 134 wetland nodes using topographic mapping of the project area. The stage/storage information was obtained by planimetering successive 1 foot contours around each wetland from base topography and survey information provided by the County. The stage/storage curve provides a range of data within which the ADICPR program adds inflow volumes and subtracts outflow volumes to calculate the resultant stage within each wetland at each designated time step of the storm simulation. The ADICPR input data summary in Appendix A lists the stage/storage data for each node.

2.3.3 CONVEYANCE REACH DATA

The ADICPR program models the hydraulic connections between nodes, the channel reaches, by simulating either open channel flow, pipe flow, weir flow, or orifice or gate flow. Most of the nodes within the project area were modeled with two reaches, one entering and one exiting. However, many of the nodes had multiple reaches entering and exiting. Figure 2-1 shows a model schematic of the existing drainage system as represented by the model. Nodes, designated as circles with identification numbers, are connected by conveyance reaches, shown as connecting lines with corresponding identifications. The existing drainage system is represented by 158 nodes interconnected by 182 reaches. Not all the nodes correspond to a wetland and may not have storage associated with them. These nodes simply provide a point for designating a change in a conveyance reach, such as a culvert.

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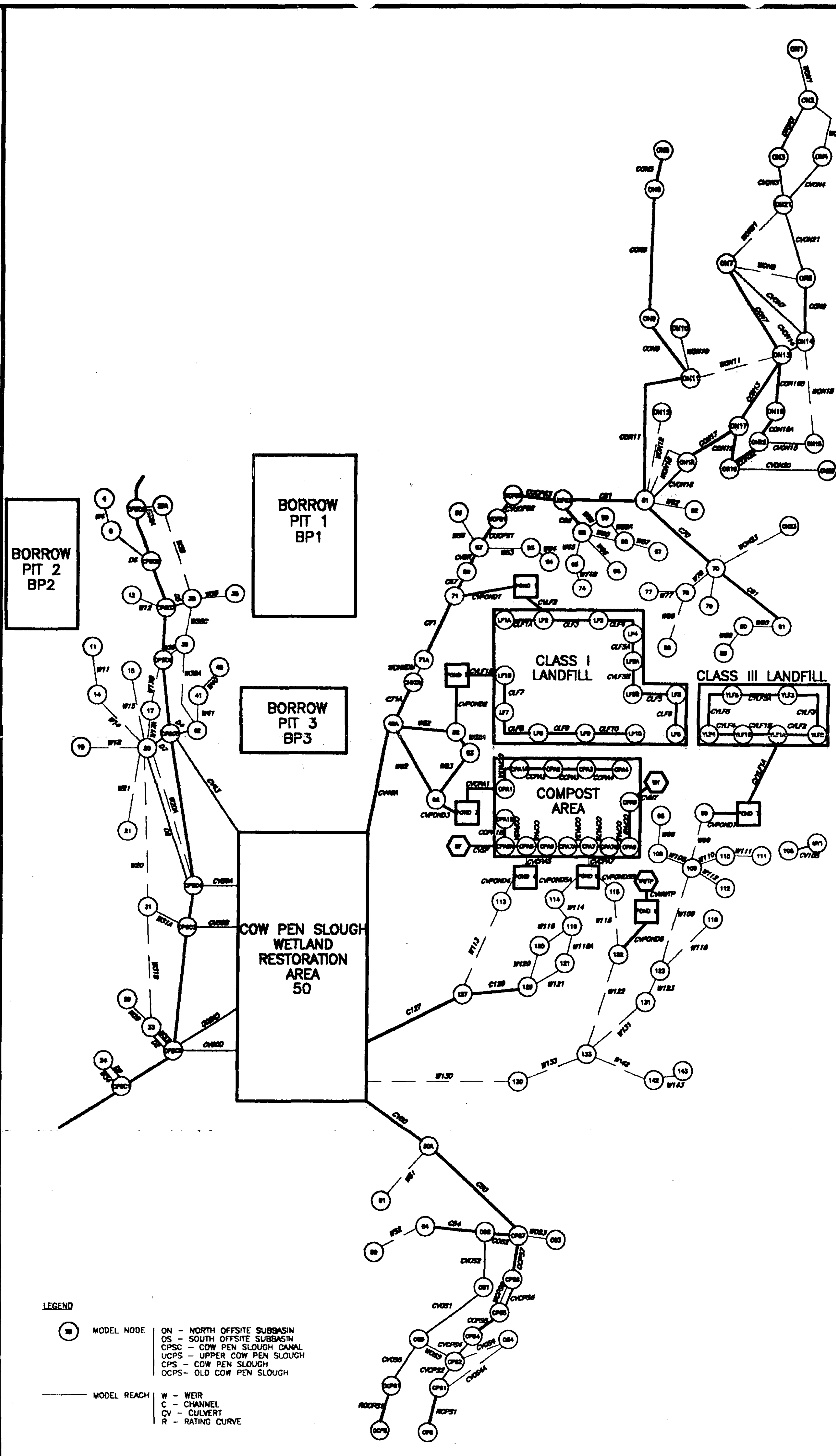


CENTRAL SOLID WASTE DISPOSAL COMPLEX
ADICPR MODEL SCHEMATIC
PROJECT CONDITIONS

Figure No. 3-1

The majority of the reaches which interconnect the natural undisturbed wetlands were defined as broad-crested weirs, representing the sheet flow of water as it breaches the ill-defined basin divides between adjacent wetlands. In this manner, excess stormwater runoff "cascades" down the system at a very shallow depth, following the general land topography. There are 112 weirs, designated with a "W" prefix shown on the model schematic in Figure 2-1. Table 2-2 lists the pertinent input data for the weir reaches.

The remaining reaches consist of 41 channels ("C" prefix), 27 culverts ("CV" prefix), and 2 rating curves ("R" prefix). Where a defined channel exists, the channel invert and cross section shape of each channel reach was determined from field survey information provided by the County. In some instances, cross section dimensions were estimated from field reconnaissance and/or aerial topographic mapping. Culvert dimensions, inverts, and top of road elevations were similarly provided by County surveyors and supplemented by topographic maps. There are 10 culverts which are equipped with an upstream flashboard weir and were represented as such in the ADICPR model according to County survey information. Tables 2-3, 2-4 and 2-5 list input data for the channel and culvert reaches of the existing drainage system as represented in the ADICPR model.

2.3.4 BOUNDARY CONDITIONS

Within the ADICPR model configuration, the drainage system must eventually outfall to a boundary node or nodes which has known or assumed characteristics such as a discharge rating curve (stage/discharge relationship) or fixed stage/time increments. For the model representation of the project area, several boundary nodes were input. The two southern most points of discharge, the south leg of Cow Pen Slough (CPS), and the Old Cow Pen Slough (OCPS), were modeled as rating curves.

TABLE 2-2

ADICPR MODEL WEIR DATA SUMMARY
EXISTING CONDITIONS

Reach No.	From Node	To Node	Weir Crest Elev., ft.	Weir Length, Ft.	Type of Weir
W4	4	6	16.6	100	Broad Crested Rect.
W6	6	6A	15.5	100	Broad Crested Rect.
W11	11	14	16.8	100	Broad Crested Rect.
W12	12	CPSC7	16.5	100	Broad Crested Rect.
W14	14	20	15.8	100	Broad Crested Rect.
W15	15	20	15.5	100	Broad Crested Rect.
W17A	17	20	15.5	100	Broad Crested Rect.
W17B	17	CPSC6	16.7	100	Broad Crested Rect.
W18	18	20	15.5	100	Broad Crested Rect.
W20	20	31	15.0	100	Broad Crested Rect.
W20A	20	CPSC4	14.5	100	Broad Crested Rect.
W21	21	20	15.5	100	Broad Crested Rect.
W29	29	33	15.5	200	Broad Crested Rect.
W31A	31	CPSC3	14.5	100	Broad Crested Rect.
W31B	31	33	15.0	100	Broad Crested Rect.
W33	33	CPSC2	15.2	100	Broad Crested Rect.
W34	34	CPSC1	15.5	100	Broad Crested Rect.
W35	35	38B	17.6	100	Broad Crested Rect.
W36	36	71	18.3	100	Broad Crested Rect.
W37	37	38	18.5	100	Broad Crested Rect.
W38	38	38A	17.5	200	Broad Crested Rect.
W38A	38A	CPSC9	17.0	100	Broad Crested Rect.
W38B	38	38B	16.5	100	Broad Crested Rect.
W38C	38	39	16.8	100	Broad Crested Rect.
W39	39	CPSC6	17.0	100	Broad Crested Rect.

TABLE 2-2 (continued)

ADICPR MODEL WEIR DATA SUMMARY
EXISTING CONDITIONS

Reach No.	From Node	To Node	Weir Crest Elev., ft.	Weir Length, Ft.	Type of Weir
W39A	39	42	17.0	100	Broad Crested Rect..
W40	40	41	17.0	100	Broad Crested Rect.
W41	41	42	16.5	100	Broad Crested Rect.
W43	43	CPSC5	16.0	100	Broad Crested Rect.
W43A	43	42	14.6	100	Broad Crested Rect.
W44	44	43	15.5	100	Broad Crested Rect.
W44A	44	47	14.5	100	Broad Crested Rect.
W47	47	CPSC4	16.2	100	Broad Crested Rect.
W47A	47	49	13.8	100	Broad Crested Rect.
W49	49	CPSC4	17.2	100	Broad Crested Rect.
W49A1	49	49A	13.8	400	Broad Crested Rect.
W49A2	49A	CPSC3	16.1	100	Broad Crested Rect.
W50	50	CPSC2	16.0	100	Broad Crested Rect.
W51	51	50	16.1	100	Broad Crested Rect.
W52	52	54	16.8	100	Broad Crested Rect.
W54	54	0S2	15.5	100	Broad Crested Rect.
W56	56	57	18.0	100	Broad Crested Rect.
W59	59	58	19.9	100	Broad Crested Rect.
W59A	59	60	19.9	100	Broad Crested Rect.
W60	60	58	19.9	100	Broad Crested Rect.
W62	62	61	21.2	100	Broad Crested Rect.
W63	63	57	19.4	100	Broad Crested Rect.
W64	64	63	20.3	100	Broad Crested Rect.
W65	65	58	20.5	100	Broad Crested Rect.
W66	66	58	20.2	100	Broad Crested Rect.
W67	67	60	20.5	100	Broad Crested Rect.
W72	72	71	18.8	200	Broad Crested Rect.

TABLE 2-2 (continued)

ADICPR MODEL WEIR DATA SUMMARY
EXISTING CONDITIONS

Reach No.	From Node	To Node	Weir Crest Elev., ft.	Weir Length, Ft.	Type of Weir
W73	73	72	19.2	100	Broad Crested Rect.
W74	74	73	20.6	100	Broad Crested Rect.
W74A	74	75	20.6	100	Broad Crested Rect.
W75	75	73	20.0	100	Broad Crested Rect.
W75A	75	71A	19.6	100	Broad Crested Rect.
W76	76	74	20.5	100	Broad Crested Rect.
W77	77	78	21.1	100	Broad Crested Rect.
W78	78	70	20.9	100	Broad Crested Rect.
W79	79	70	21.1	100	Broad Crested Rect.
W80	80	81	21.1	100	Broad Crested Rect.
W82	82	45A	18.2	200	Broad Crested Rect.
W82A	82	83	17.8	100	Broad Crested Rect.
W83	83	92	18.2	100	Broad Crested Rect.
W85	85	84	20.0	100	Broad Crested Rect.
W86	86	87	20.5	100	Broad Crested Rect.
W87	87	95	19.7	100	Broad Crested Rect.
W88	88	97	20.3	100	Broad Crested Rect.
W89	89	100	20.9	100	Broad Crested Rect.
W90	90	103	20.0	100	Broad Crested Rect.
W92	92	45	18.0	100	Broad Crested Rect.
W93	93	92A	18.7	100	Broad Crested Rect.
W94	94	106	19.0	100	Broad Crested Rect.
W98	98	108	19.0	100	Broad Crested Rect.
W99	99	109	19.5	100	Broad Crested Rect.
W103	103	105	19.8	100	Broad Crested Rect.
W106	106	92A	18.4	100	Broad Crested Rect.
W108	108	115	18.4	100	Broad Crested Rect.

TABLE 2-2 (continued)

ADICPR MODEL WEIR DATA SUMMARY
EXISTING CONDITIONS

Reach No.	From Node	To Node	Weir Crest Elev., ft.	Weir Length, Ft.	Type of Weir
W109	109	123	17.9	100	Broad Crested Rect.
W110	110	109	18.2	100	Broad Crested Rect.
W111	111	110	18.4	100	Broad Crested Rect.
W112	112	109	18.5	100	Broad Crested Rect.
W113	113	127	15.9	200	Broad Crested Rect.
W114	114	116	17.4	100	Broad Crested Rect.
W115	115	122	17.4	100	Broad Crested Rect.
W116	116	120	17.2	100	Broad Crested Rect.
W116A	116	121	17.5	100	Broad Crested Rect.
W118	118	123	18.2	100	Broad Crested Rect.
W120	120	129	16.2	100	Broad Crested Rect.
W121	121	129	16.5	100	Broad Crested Rect.
W122	122	133	17.0	200	Broad Crested Rect.
W123	123	131	16.8	100	Broad Crested Rect.
W130	130	128	15.5	200	Broad Crested Rect.
W131	131	133	16.5	100	Broad Crested Rect.
W133	133	130	15.7	400	Broad Crested Rect.
W142	142	133	16.8	100	Broad Crested Rect.
W143	143	142	16.9	100	Broad Crested Rect.
WOS3	OS3	CPS7	12.8	100	Broad Crested Rect.
WOS4	OS4	CPS	15.2	100	Broad Crested Rect.
WOS5	CPS2	OS5	15.0	100	Broad Crested Rect.
WON1	ON1	ON2	25.5	100	Broad Crested Rect.
WON4	ON4	ON2	26.3	400	Broad Crested Rect.
WON8	ON8	ON7	21.1	200	Broad Crested Rect.
WON10	ON10	ON11	20.5	100	Broad Crested Rect.
WON11	ON11	ON13	20.5	10	Trap SS=20:1

TABLE 2-2 (continued)

ADICPR MODEL WEIR DATA SUMMARY
EXISTING CONDITIONS

Reach No.	From Node	To Node	Weir Crest Elev., ft.	Weir Length, Ft.	Type of Weir
WON12	ON12	61	20.0	100	Broad Crested Rect.
WON15	ON15	ON14	20.5	100	Broad Crested Rect.
WON18	ON18	61	19.5	100	Broad Crested Rect.
WON21	ON21	ON7	21.2	200	Broad Crested Rect.
WON23	ON23	70	20.5	100	Broad Crested Rect.
WCPS6	CPS6	CPS5	16.0	200	Broad Crested Rect.

TABLE 2-3

**ADICPR MODEL CHANNEL REACH DATA SUMMARY
EXISTING CONDITIONS**

Reach No.	From Node	To Node	Upstream I.E., ft	Downstream I.E., ft	Maximum Depth, ft.	Length, ft.	Bottom Width, ft.	Left Sideslope, ft./ft.	Right Sideslope, ft./ft.	Manning's n
C48	48	49	11.0	8.4	6′	1000′	**	**	**	.060
C45	45	48	12.0	11.0	6′	1400′	**	**	**	.060
C45A	45A	45	12.5	12.0	6′	1600′	**	**	**	.060
C71A	71A	45A	13.0	12.5	6′	1700′	**	**	**	.060
C71	71	71A	13.5	13.0	6′	1750′	**	**	**	.060
C57	57	71	13.7	13.5	6′	1150′	**	**	**	.060
CUCPS1	UCPS1	57	15.3	13.7	5′	1800′	**	**	**	.060
CUCPS3	UCPS3	UCPS2	16.0	15.4	5′	850′	**	**	**	.060
C61	61	UCPS3	17.5	16.0	5′	1750′	**	**	**	.060
CON17	ON17	ON18	17.7	17.2	4′	1800′	**	**	**	.060
CON13	ON13	ON17	18.4	17.7	4′	1200′	**	**	**	.060
CON8	ON8	ON14	19.1	18.5	3′	1100′	8′	3:1	3:1	.060
CON5	ON5	ON6	24.0	21.0	2′	1400′	5′	10:1	10:1	.060
CON6	ON6	ON9	21.5	20.3	2′	1400′	5′	10:1	10:1	.060
CON9	ON9	ON11	20.3	20.0	2′	2000′	5′	10:1	200:1	.060

** **Irregular Section**

TABLE 2-3 (continued)

ADICPR MODEL CHANNEL REACH DATA SUMMARY
EXISTING CONDITIONS

Reach No.	From Node	To Node	Upstream I.E., ft	Downstream I.E., ft	Maximum Depth, ft.	Length, ft.	Bottom Width, ft.	Left Sideslope, ft./ft.	Right Sideslope, ft./ft.	Manning's n
CON11	ON11	61	20.5	18.0	2′	2400′	5′	10:1	10:1	.060
CON7	ON7	ON13	19.8	19.0	2′	900′	5′	4:1	4:1	.060
CON16A	ON16	ON22	20.5	20.0	2′	550′	5′	2:1	2:1	.060
CON16B	ON16	ON13	20.5	20.0	2′	900′	5′	2:1	2:1	.060
CON19	ON19	ON17	19.0	19.0	2′	400′	5′	2:1	2:1	.060
CON22	ON22	ON19	19.5	19.0	2′	450′	5′	2:1	2:1	.060
C70	70	61	19.5	18.1	3′	2300′	**	**	**	.060
C81	81	70	20.0	19.5	3′	400′	**	**	**	.060
C58	58	UCPS3	19.2	17.7	2′	250′	5′	4:1	4:1	.060
C49A	49A	50	11.0	10.0	6′	1500′	20′	6:1	6:1	.080
C50	50	CPS7	8.5	8.9	8′	4700′	20′	4:1	4:1	.080
CCPS7	CPS7	CPS6	8.9	8.0	8′	1100′	20′	2:1	2:1	.080
CCPS5	CPS5	CPS4	8.0	7.0	8′	3300′	20′	2:1	2:1	.080
C54	54	0S2	15.0	14.0	2′	1200′	5′	10:1	10:1	.060
COS2	OS2	CPS7	12.2	12.0	4′	600′	5′	8:1	8:1	.060
C84	84	82	18.6	17.5	1′	1400′	5′	4:1	100:1	.060
C95	95	84	19.5	19.0	1′	1850′	5′	4:1	50:1	.060

** Irregular Section

TABLE 2-3 (continued)

ADICPR MODEL CHANNEL REACH DATA SUMMARY
EXISTING CONDITIONS

Reach No.	From Node	To Node	Upstream I.E., ft	Downstream I.E., ft	Maximum Depth, ft.	Length, ft.	Bottom Width, ft.	Left Sideslope, ft./ft.	Right Sideslope, ft./ft.	Manning's n
C96	96	95	19.6	19.5	1′	600′	5′	4:1	50:1	.060
C92A	92A	45	17.2	17.0	1′	1800′	5′	4:1	200:1	.060
C107	107	92A	17.9	17.2	1′	2000′	5′	4:1	50:1	.060
C97	97	100	20.0	19.7	1′	1300′	5′	4:1	100:1	.060
C100	100	103	19.7	19.5	1′	1300′	5′	4:1	100:1	.060
C127A	127A	50	14.0	10.9	2′	500′	5′	100:1	100:1	.060
C127	127	127A	14.5	14.0	2′	200′	5′	100:1	100:1	.060
C129	129	127	15.5	14.5	2′	500′	5′	200:1	200:1	.060
C128	128	50	13.5	12.5	3′	300′	5′	100:1	100:1	.060

** Irregular Section

TABLE 2-4

ADICPR MODEL CULVERT DATA SUMMARY
EXISTING CONDITIONS

Reach No.	From Node	To Node	Upstream I.E., ft	Downstream I.E., ft	Type	Entrance Loss Coef., k_e	Manning's n	Length, ft.	Road Overtop
CV105	105	MY1	16.95	16.9	36" CMP	0.7	.024	30′	L = 50′ @ 19.2
CVUCPS2	UCPS2	UCPS1	15.4	15.3	42" CMP	0.7	.024	20′	L = 200′ @ 18.0
CVOS1	OS1	OS5	11.8	11.8	24" x 38" RCP	0.5	.013	30′	L = 50′ @ 13.9; SS = 100:1
CVOS2	OS2	OS1	12.6	12.9	2 - 18" CMP	0.7	.024	30′	L = 50′ @ 15.0; SS = 100:1
CVOS4	OS4	CPS2	10.8	10.4	24" CMP	0.7	.025	50′	
CVOS5	OS5	OCPS1	6.4	6.2	24" RCP	0.5	.013	50′	L = 20′ @ 11.5; SS = 50:1
CVCPS2	CPS2	CPS1	6.4	6.2	48" CMP	0.7	.024	50′	L = 200′ @ 15.5
CVCPS4	CPS4	CPS2	5.6	4.7	72" CMP	0.7	.024	50′	L = 100′ @ 15.5
CVON2	ON2	ON3	23.5	23.5	3 - 36" CMP	0.7	.024	50′	L = 200′ @ 25.4
CVON3	ON3	ON21	22.2	22.2	2 - 42" CMP	0.7	.024	50′	L = 200′ @ 26.3
CVON4	ON4	ON21	24.0	24.0	24" CMP	0.7.	.024	40′	L = 200′ @ 26.3
CVON7	ON7	0N14	19.8	19.8	24" CMP	0.7	.024	50′	L = 200′ @ 21.2
CVON14	ON14	ON13	18.5	18.5	48" CMP	0.7	.024	50′	L = 200′ @ 21.5
CVON15	ON15	ON22	19.5	19.5	24" CMP	0.7	.024	40′	L = 200′ @ 21.5
CVON 20	ON20	ON19	19.5	19.5	24" CMP	0.7	.024	40′	L = 200′ @ 21.5
CVON21	ON21	ON8	20.5	20.5	2 - 36" CMP	0.7	.024	100′	L = 200′ @ 21.4
CVOS4A	OS4	CPS1	9.0	8.2	19" x 30" RCP	0.5	.013	50′	L = 100′ @ 15.2

TABLE 2-4

ADICPR MODEL CULVERT DATA SUMMARY
EXISTING CONDITIONS

Reach No.	From Node	To Node	Upstream I.E., ft	Downstream I.E., ft	Type	Entrance Loss Coef., k_e	Manning's n	Length, ft.	Road Overtop
CV105	105	MY1	16.95	16.9	36" CMP	0.7	.024	30'	L = 50' @ 19.2
CVUCPS2	UCPS2	UCPS1	15.4	15.3	42" CMP	0.7	.024	20'	L = 200' @ 18.0
CVOS1	OS1	OS5	11.8	11.8	24" x 38" RCP	0.5	.013	30'	L = 50' @ 13.9; SS = 100:1
CVOS2	OS2	OS1	12.6	12.9	2 - 18" CMP	0.7	.024	30'	L = 50' @ 15.0; SS = 100:1
CVOS4	OS4	CPS2	10.8	10.4	24" CMP	0.7	.025	50'	
CVOS5	OS5	OCPS1	6.4	6.2	24" RCP	0.5	.013	50'	L = 20' @ 11.5; SS = 50:1
CVCPS2	CPS2	CPS1	6.4	6.2	48" CMP	0.7	.024	50'	L = 200' @ 15.5
CVCPS4	CPS4	CPS2	5.6	4.7	72" CMP	0.7	.024	50'	L = 100' @ 15.5
CVON2	ON2	ON3	23.5	23.5	3 - 36" CMP	0.7	.024	50'	L = 200' @ 25.4
CVON3	ON3	ON21	22.2	22.2	2 - 42" CMP	0.7	.024	50'	L = 200' @ 26.3
CVON4	ON4	ON21	24.0	24.0	24" CMP	0.7.	.024	40'	L = 200' @ 26.3
CVON7	ON7	0N14	19.8	19.8	24" CMP	0.7	.024	50'	L = 200' @ 21.2
CVON14	ON14	ON13	18.5	18.5	48" CMP	0.7	.024	50'	L = 200' @ 21.5
CVON15	ON15	ON22	19.5	19.5	24" CMP	0.7	.024	40'	L = 200' @ 21.5
CVON 20	ON20	ON19	19.5	19.5	24" CMP	0.7	.024	40'	L = 200' @ 21.5
CVON21	ON21	ON8	20.5	20.5	2 - 36" CMP	0.7	.024	100'	L = 200' @ 21.4
CVOS4A	OS4	CPS1	9.0	8.2	19" x 30" RCP	0.5	.013	50'	L = 100' @ 15.2

TABLE 2-5

ADICPR MODEL CULVERT-WITH-RISER DATA SUMMARY
EXISTING CONDITIONS

Reach No.	From Node	To Node	Upstream I.E., ft	Downstream I.E., ft	Type	Entrance Loss Coef., k_e	Manning's n	Length, ft.	Riser Weir Lengths
D2	33	CPSC2	8.7	7.7	24" CMP	0.7	.024	80′	L = 3′ @ 12.4 L = 5′ @ 14.8
D3	49	CPSC4	7.6	6.6	54" CMP	0.7	.024	80′	L = 7′ @ 11.1 L = 11′ @ 14.1
D4	42	CPSC5	9.0	8.0	24" CMP	0.7	.024	80′	L = 3′ @ 11.1 L = 5′ @ 15.0
D5	38B	CPSC7	9.1	8.1	24" CMP	0.7	.024	80′	L = 3′ @ 11.1 L = 5′ @ 15.2
D6	6A	CPSC8	10.3	9.3	24" CMP	0.7	.024	80′	L = 3′ @ 13.1 L = 5′ @ 16.4
D7	20	CPSC5	5.1	4.1	24" CMP	0.7	.024	80′	L = 3′ @ 8.0 L = 5′ @ 11.2
D8	20	CPSC4	8.5	7.5	24" CMP	0.7	.024	80′	L = 3′ @ 10.2 L = 5′ @ 14.6
D9	34	CPSC1	8.8	7.8	24" CMP	0.7	.024	80′	L = 3′ @ 12.8 L = 5′ @ 14.9
CVCPS6	CPS6	CPS5	6.3	5.6	72" CMP	0.7	.024	30′	L = 4′ @ 8.9 L = 4′ @ 9.6
CVON18	ON18	61	16.2	16.0	48" CMP	0.7	.024	40′	L = 6′ @ 19.0

Rating curves were developed to simulate a stage versus discharge relationship at these downstream boundary discharge points at Rustic Road. The rating curves were developed from a hydraulic analysis of the downstream channels using the U.S. Army Corps of Engineers HEC-2 backwater computer program. The HEC-2 program is designed to calculate water surface profiles for steady gradually varied flow in natural or man-made channels. The computational procedure used in the program, generally known as the Standard Step Method, is based on the energy equation with losses due to friction evaluated with the Manning's equation. The energy losses of channel expansion and contraction, and the effects of flow obstructions such as bridges, culverts, weirs, embankments, dams, channel improvements, and other structures in the flood plain can be considered in the computations.

The input data required by the HEC-2 model to perform the backwater computations include: flow regime (flow at each cross section), starting elevation, loss coefficients for structures, cross section geometry for channels and structures, Manning's "n", and reach lengths. More detailed data are required at crossings (bridge or culverts) to describe the section at which low flow, pressure flow (e.g., through culverts) or pressure and weir flow (e.g., flow through culverts and over top the roadway) could occur under various flow conditions.

The basic input data required to develop the hydraulic model of the project area outfalls included the cross section data for the downstream channels. These data were obtained from survey information and from Southwest Florida Water Management District 1"=200' scale aerial topographic maps. The basic channel cross section data (stations and elevations) provide the HEC-2 model with the geometry required to characterize the flow-carrying capabilities of the channels and adjacent floodplain areas.

The HEC-2 model was used to generate a series of backwater profiles for each of the outfalls. The results of the backwater analysis provided water surface profiles in each of the outfall channels for each of a set of selected discharge rates. The resultant rating curves for the outfalls at the project area boundary were input into the ADICPR program to provide the tailwater condition for the drainage system as it discharges into the outfall channel. Table 2-6 lists the rating curve used for each of the outfall channels.

In addition to the two southernmost discharge points, the model representation includes a number of outfalls to the Cow Pen Slough Canal (CPSC) as shown on the model schematic (Figure 2-1). These outfalls consist of the 8 culverts with risers listed in Table 2-5 plus several weir connections representing the low breaks in the canal spoil banks on both sides of the canal. These direct connections to the canal allow floodwater to pass either into or out of the canal, depending on relative water surface elevations on either side of the spoil banks.

The only source of available information by which the canal boundary condition could be defined is the "Flood Plain Management Study, Cow Pen Slough Watershed, Sarasota County, Florida" (SCS, 1985). This study presents a set of flood profiles along the canal for the 10-, 50-, and 100-year frequency flood events. Although this study is generally considered to grossly overestimate the 100-year flood elevation due to modeling misrepresentations (channel cross sections were not extended out into the floodplain), it can be judged to fairly show the 10-year flood profile since it is contained within the bank. The 25-year flood profile was estimated using the 10- and 50-year flood profiles as points of reference.

Table 2-7 lists a set of boundary condition stage/time series which were developed at nine points along the Cow Pen Slough Canal as it passes through the project site. The initial elevation of 7.5 feet at the start of the design storm is representative of the water level maintained by a downstream canal control structure during the wet season. A stage hydrograph was developed

TABLE 2-6

BOUNDARY CONDITION RATING CURVES
DOWNSTREAM OF RUSTIC ROAD

	Flow, cfs	Tailwater Elevation, ft
Cow Pen Slough		
	0	7.0
	20	7.9
	50	9.0
	100	10.3
	200	12.2
	400	14.7
	600	16.6
Old Cow Pen Slough		
	0	6.4
	20	8.8
	50	10.5
	100	12.1
	200	13.9
	400	15.3
	600	16.2

TABLE 2-7

COW PEN SLOUGH CANAL BOUNDARY CONDITIONS
25-YEAR FLOOD ELEVATIONS

NODE	TIME FROM START OF STORM, HRS						
	0	6	12	15	18	24	36
CPSC1	7.5	8.0	9.0	11.0	13.0	15.0	14.0
CPSC2	7.5	8.0	9.0	11.1	13.2	15.2	14.2
CPSC3	7.5	8.0	9.0	11.2	13.6	15.7	14.6
CPSC4	7.5	8.0	9.0	11.3	14.0	16.0	15.0
CPSC5	7.5	8.0	9.0	11.5	14.5	16.5	15.5
CPSC6	7.5	8.0	9.0	11.6	14.7	16.7	15.7
CPSC7	7.5	8.0	9.0	11.7	15.0	17.0	16.0
CPSC8	7.5	8.0	9.0	11.9	15.5	17.5	16.5
CPSC9	7.5	8.0	9.0	12.0	16.5	18.5	17.5

starting at this initial point and peaking at Hour 24 of simulation. This reflects an estimated 12-hour lag time in the canal from the time of peak rainfall intensity (Hour 12). This lag time was estimated by examination of historical streamflow gaging records of rivers of similar watershed drainage areas as the canal (65 square miles) for the May 1979 storm event in Hillsborough and Pinellas Counties. Model simulations were carried out to Hour 36 to account for canal impacts as its flood wave passes through the site.

2.4 EXISTING CONDITION MODEL RESULTS

The results of the ADICPR model simulation of the 25-year design storm event for existing conditions on the project site are listed in Table 2-1 for each of the wetland nodes. The ADICPR model creates a nodal peak condition summary at the completion of each analysis. Appendix A contains this summary table. The peak flood stage and corresponding storage are given for each node. In addition, the total peak outflow is given for each node. Appendix A also contains a peak condition summary of model reaches which lists the peak flow, time of peak, and the corresponding upstream and downstream node elevations.

The 25-year peak discharge from the northern offsite drainage area (north leg of Cow Pen Slough) was simulated to be 70 cfs at a flood elevation of 20.53 feet (Node 61). This seems to be a small flow for such a large storm event, but the large amount of wetland storage in this upper basin, especially in ON7, buffers the flood surge. Also, the north leg of the slough is full to capacity at this flow amount and restricts any further discharge from the northern offsite area.

Discharge to the canal from the north leg of the slough at Node 49 peaks at 101 cfs with a corresponding peak elevation of 15.82 feet. At the south boundary of the site (Node 50), discharge to the south leg of the slough peaks at 149 cfs at a peak elevation of 15.65 feet. At the model southern boundary points at Rustic Road, existing condition peak 25-year flood flows are 104 cfs to the channelized Cow Pen Slough, and 230 cfs to the Old Cow Pen Slough. The latter flow is so high since the upstream drainage areas to the Old Slough, bermed off from the channelized slough by its western spoil bank, have virtually no wetland storage to attenuate peak discharges. This results in overtop of Rustic Road at a peak elevation of 14.1 feet while the main slough channel peaks at elevation 13.7 feet.

Upon examination of Table 2-1, a comparison of the calculated peak stage from a 25-year, 24-hour storm event and the initial starting elevation, representing a normal seasonal high water table elevation at each wetland, shows that the 25-year, 24-hour rainfall event generally increases the stage in a wetland between 0.5 and 1 foot. This figure is representative of the natural undrained wetlands in the eastern portion of the site. This rise is reasonable because of the flatness of the land near the wetlands. A large rainfall event will cause the stormwater collected in a wetland to spread out into the palmetto and pine flatwoods where a large volume of storage is available with only a small rise in stage. This outer floodplain area, while providing a large volume of storage, is mostly comprised of type D soils which do not allow much stormwater infiltration. When stormwater spreads out into the surrounding palmetto and pine flatwoods, it does not infiltrate into the ground, but either spills over into the next adjacent wetland or is instead "stored" until the wetland water level recedes and can accept the excess stormwater.

3.0 STORMWATER MANAGEMENT FACILITIES DESIGN

3.1 DESIGN OBJECTIVES

The design objectives for the stormwater management facilities design for the Sarasota County Central County Solid Waste Disposal Complex site are:

- Provide facilities for the capture, conveyance, storage and treatment of all runoff from the complex and appurtenant facilities. All stormwater facilities are sized for the 25-year, 24-hour duration design storm event. Treatment of stormwater runoff from the complex is designed to meet or exceed the requirements of Chapters 17-25 and 40D-4 F.A.C.

- Discharges from the County's property are controlled such that downstream peak discharge to the Cow Pen Slough and Cow Pen Slough Canal does not exceed that of existing conditions for the 25-year design storm event. Also, peak flood elevations in upstream off-site areas will not exceed pre-development flood elevations for the 25-year design event.

- No discharges from the complex facilities will enter the Myakka River drainage basin.

- Direct connections of the County's property to the Cow Pen Slough Canal are maintained to retain the same floodplain storage for the canal as exists today.

- Storage provided by the site stormwater management facilities, borrow pits, and wetland restoration areas will assure that the volume of discharge from the mean annual storm event will not exceed the predevelopment volume.

- Buffer of 50 feet or more from adjacent jurisdictional wetland lines is provided for all stormwater management facilities.

- Amenities to preserve, enhance, and/or restore existing wetlands on the County's property are incorporated in the stormwater management facilities design where possible. The stormwater management system is designed with the basic goals of preserving existing drainage patterns, preserving existing wetlands and upland buffers, mimicking the existing wetland hydroperiods, and providing compensation for wetland impacts and flood storage.

3.2 GENERAL DESCRIPTION

The stormwater management facilities design for the Sarasota County Central County Solid Waste Disposal Complex site is shown on Sheets 4 and 5. Detailed construction plans for the stormwater facilities are included as a separate attachment to this report.

In general, all stormwater runoff from the solid waste disposal complex (within the special exception area boundary) will be collected within perimeter ditches which encircle the entire complex and conveyed to seven stormwater treatment ponds. The perimeter ditches will be elevated above the natural grade with ditch bottoms at existing grade elevations. A perimeter berm will form the outer ditch bank and will have a minimum top width of 20 feet to provide access for maintenance of facilities. The perimeter ditches have been designed around several of the wetlands which abut or cross into the special exception area in order to minimize wetland impacts and/or provide buffer zones. In addition, ditch blocks will be constructed within the perimeter ditches at key locations to provide both pretreatment of stormwater runoff and storage of runoff during dry weather periods to be available for percolation and hydration of adjacent wetlands.

The seven stormwater treatment ponds are designed as wet detention treatment systems which will treat more than one inch of runoff from their respective contributing drainage areas. As such, each pond has been designed according to the criteria of Part B Basis of Review for Surface Water Management Permit Applications Within the Southwest Florida Water Management District. These ponds are designed to provide the required littoral zones, permanent pool volumes and depths, and bleed down orifices. The pond outfalls are designed to attenuate peak flows and convey discharges along existing drainage paths to preserve downstream wetlands. Spreader swales at 30 foot setbacks from jurisdictional lines will be used to minimize impacts to downstream wetlands at points of discharge.

Three (3) borrow pits totalling 278 acres will be excavated, as shown on Sheet 4, within the County's parcel to provide cover and fill material for construction of the solid waste disposal facilities. These borrow pits will be surrounded by a perimeter berm that will be elevated 2 feet above existing grade and serve to isolate the borrow pits entirely from the drainage basin (i.e. no furface water will enter or exit the borrow pits).

A 3.2-mile long access road, approximately 100 feet in total width, will be constructed from the intersection of Knights Trail Road and Rustic Road to the solid waste disposal complex site entrance. This access road will be elevated above the natural grade and will drain to either a median swale system or two side swales. The swales will be constructed with bottoms above the seasonal high water elevation. Elevated inlets and ditch blocks will store 1/2-inch of runoff from the roadway approximately 3-5 inches deep in the swale system to provide the required treatment volume for a dry retention treatment system. Percolation of the treatment volume into the elevated swale bottom will provide the mode of treatment.

The alignment of the access road will carry it across several water bodies, as shown in Sheets 4 and 5, and cross culverts will be constructed to maintain existing drainage paths at these locations. In addition, the roadway will fill in the breaks in the Cow Pen Slough Canal east spoil bank at four locations. At each of these points, cross culverts will be constructed to maintain the floodplain connection between the site and the canal.

In an effort to restore the drastically altered wetlands along the Cow Pen Slough and Cow Pen Slough Canal, the stormwater management facilities design includes the construction or modification of structures which will rehydrate these wetlands and restore their natural hydroperiods. Two structures will be constructed on the north leg of the slough and one on the south leg at the south property line. These structures will dam the man-made channels with weirs set to impound water upstream at the natural wetland control elevation. The control structure on the south leg of the slough will include a containment berm that will impound excess floodwater during the 25-year design storm event and reduce downstream peak discharges to below predevelopment conditions. In addition, peak discharge of the north leg of the slough to the Cow Pen Slough Canal will be reduced for the design storm event. A second upstream check dam on the north leg of the slough will serve to rehydrate the upper slough system and minimize any potential dewatering affects of the adjacent borrow pits. Section 3.5 contains a more detailed description of the slough wetland restoration area.

The series of culverts which directly drain adjacent wetlands into the Cow Pen Slough Canal through the canal spoil banks will be modified by replacement of the riser inlet structures with standard FDOT inlets. The grate elevations of the new inlets will be set at the historical seasonal high water elevations of the drained wetlands.

3.3 STORMWATER MODEL - PROJECT CONDITIONS

The design of the stormwater management facilities for the Sarasota County Central County Solid Waste Disposal Complex site was accomplished through modification of the ADICPR model of the existing site drainage system (see Section 2.0) to reflect the developed site conditions.

The initial step of this process was to overlay the solid waste disposal complex site plan on the existing drainage basin map and calculate the changes in contributing drainage basin areas. The complex site and borrow pits will eliminate 21 isolated wetlands located within their perimeters. In addition, the drainage basins of 39 other wetlands will be reduced in size by the construction of the complex, borrow pits, ponds, and access road. Table 3-1 provides a summary of changes in acreages from the existing site drainage conditions model. It can be seen from this table that the total amount of directly contributing drainage area which will become a part of the disposal complex facilities is 591 acres, in addition to the 278 acres of borrow pit.

Table 3-2 provides a listing of the wetland node hydrologic data as modified to reflect the project conditions. In addition to changes in contributing drainage areas of wetland nodes, the disposal complex site facilities also result in changes to directly connected impervious area percentages, times of concentration, stage-storage relationships, and initial elevations, as input to the project conditions ADICPR model. Note in this table that the Cow Pen Slough Wetland Restoration Area is represented as Node 50 and incorporates all of the previously designated Nodes 43, 44, 45, 47, 48, 49, 49A, 127A and 128 into one area. This is not to be interpreted that these wetlands are to be eliminated.

The stormwater management facilities for the solid waste disposal complex (see Sheets 4 and 5) are added into the ADICPR model as shown schematically in Figure 3-1. The perimeter ditches of the complex site are represented as a series of nodes connected by open channels. Each node has a contributing drainage subbasin, as listed in Table 3-3. Runoff hydrographs from the subbasins are input to the respective perimeter ditch node (no associated storage) and routed to the seven stormwater treatment and attenuation ponds, as shown in Figure 3-1. Note in Table 3-3 that nodes and subbasins with the 'LF' prefix represent the Class I landfill, 'YLF' denotes the Class III landfill, and 'CPA' represents the compost area. In addition, the 'MY' subbasin is the maintenance yard, 'SF' is the entrance and scale facilities, and 'WWTP' is the leachate pretreatment and sludge dewatering facilities.

TABLE 3-1

SUMMARY OF CONTRIBUTING DRAINAGE AREAS
EXISTING VERSUS PROJECT CONDITIONS

	Contributing Drainage Areas, ac	
	Existing Condition	**Project Condition**
Northern Offsite (ON1-ON23)	1429.7	1429.7
Southern Offsite (OS1-OS5)	383.6	383.6
West of Cow Pen Slough Canal	620.0	597.5
On-Site East of Cow Pen Slough Canal	3072.2	2312.3
Class I Landfill	----	293.5
Class III Landfill	----	54.2
Leachate Treatment & Sludge Dewatering Facilities	----	30.1
Compost Area, Scale Facilities, & Maintenance Yard	----	144.9
Wet Detention Ponds (Ponds 1-7)	----	67.9
Subtotal	----	590.6
Borrow Pit 1	----	144.3
Borrow Pit 2	----	109.2
Borrow Pit 3	----	25.0
Subtotal	----	278.5
	5505.5	**5592.2***

* **NOTE:** Total contributing drainage area increases under project condition because Borrow Pit 2 includes 86.7 acres on western property edge which does not presently drain to the Cow Pen Slough Canal.

TABLE 3-2

ADICPR WETLAND NODE DATA SUMMARY
PROJECT CONDITIONS

Wetland Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
4	17.9	79	14.3	29	16.5	17.3
6	17.3	79	20.7	30	16.5*	17.3
11	13.2	79	31.1	21	16.5	17.0
12	12.3	79	9.7	28	16.0	17.0
14	36.6	79	7.0	34	15.4	16.3
15	36.2	79	0.3	32	15.5	16.0
17	28.1	79	14.3	25	15.0	16.0
18	50.9	79	23.1	36	15.0	16.0
20	160.6	79	48.4	31	15.0*	16.0
21	37.5	79	22.5	32	15.0	16.0
29	54.7	79	19.2	30	15.0	16.0
31	74.8	79	4.8	55	15.0	16.0
33	29.8	79	22.4	25	15.0*	16.0
34	27.6	79	9.3	27	15.0*	15.8
35	55.6	79	15.7	35	17.1	18.0
38	22.8	79	15.7	28	16.5	17.5
38A	35.7	79	3.1	37	16.3	17.8
39	17.1	79	3.3	24	16.0	17.4
40	14.4	79	13.8	17	16.4	17.2
41	28.9	79	9.0	27	15.5	17.2
42	16.1	79	31.1	22	15.0*	17.2
45A	49.4	79	5.4	32	15.0*	17.0
50	440.4	79	60.7	48	15.0*	16.3
50A	43.4	79	3.2	28	8.9	15.0
51	21.6	79	28.9	17	15.8	16.3
52	42.7	79	42.8	25	16.5	17.0

* Wetland Enhanced By Control Structure

TABLE 3-2 (continued)

ADICPR WETLAND NODE DATA SUMMARY
PROJECT CONDITIONS

Wetland Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
54	49.2	79	5.4	29	15.0	15.9
56	16.2	79	17.0	29	17.9	18.9
57	46.3	79	2.7	32	17.5*	18.9
58	34.2	79	37.9	25	19.2	20.2
59	9.5	79	26.7	17	19.4	20.2
60	18.2	79	35.4	17	19.4	20.2
61	40.7	79	0.0	27	17.5	20.6
62	12.1	79	32.8	17	20.8	21.4
63	11.2	79	13.8	30	19.0	19.6
64	7.9	79	14.4	17	20.0	20.5
65	8.2	79	19.6	17	19.5	20.5
66	20.4	79	17.7	29	19.4	20.3
67	15.8	79	30.2	17	20.1	20.7
70	103.9	79	15.6	37	19.5	21.2
71	30.2	79	13.2	32	17.5*	18.7
71A	31.5	79	16.6	22	17.5*	18.7
74	3.0	79	26.7	27	20.0	20.8
77	4.8	79	30.6	17	20.5	21.2
78	30.9	79	25.6	25	20.5	21.2
79	31.1	79	23.6	25	20.5	21.3
80	18.2	79	35.4	17	20.2	21.2
81	33.7	79	19.9	27	20.0	21.2
82	22.1	79	34.1	22	18.0	18.4
83	10.0	79	46.8	15	18.0	18.4
88	13.9	79	22.7	29	20.5	21.2
89	10.4	79	21.1	13	20.6	21.2
92	16.1	79	38.1	15	17.5	18.4
98	13.2	79	24.7	37	18.6	19.1
99	11.7	79	21.6	22	19.2	19.8

* Wetland Enhanced By Control Structure

TABLE 3-2 (continued)

ADICPR WETLAND NODE DATA SUMMARY
PROJECT CONDITIONS

Wetland Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
105	47.6	79	9.2	35	17.5	18.9
108	22.9	79	32.2	27	18.1	18.9
109	44.8	79	26.4	30	17.8	18.5
110	15.5	79	16.6	25	17.8	18.5
111	12.7	79	6.7	28	17.8	18.6
112	4.6	79	12.4	17	18.5	18.6
113	43.9	79	30.1	22	15.8	16.3
114	14.5	79	45.0	15	16.6	17.6
115	13.5	79	50.4	18	17.2	17.6
116	24.4	79	35.3	25	16.9	17.6
118	9.3	79	18.5	25	17.7	18.3
120	16.1	79	21.0	25	16.0	16.4
121	14.1	79	27.8	25	16.0	16.7
122	46.9	79	27.5	28	16.8	17.5
123	57.3	79	17.0	32	16.5	17.6
127	86.2	79	27.8	40	15.0*	16.3
129	26.8	79	19.7	27	15.5	16.3
130	104.8	79	13.2	46	15.5	16.3
131	24.9	79	15.6	17	16.5	17.2
133	171.6	79	15.1	35	15.7	16.3
142	17.3	79	23.0	25	16.0	17.0
143	29.9	79	7.4	27	16.0	17.1
ON1	43.2	79	37.3	17	25.0	25.8
ON2	205.5	79	12.5	41	23.5	25.6
ON3	51.3	79	0.4	17	22.2	25.3
ON4	111.4	79	0.0	81	24.0	26.4
ON5	41.2	79	15.4	25	24.0	25.2
ON6	41.8	79	18.2	26	21.0	23.1
ON7	292.3	79	25.1	29	19.8	21.6

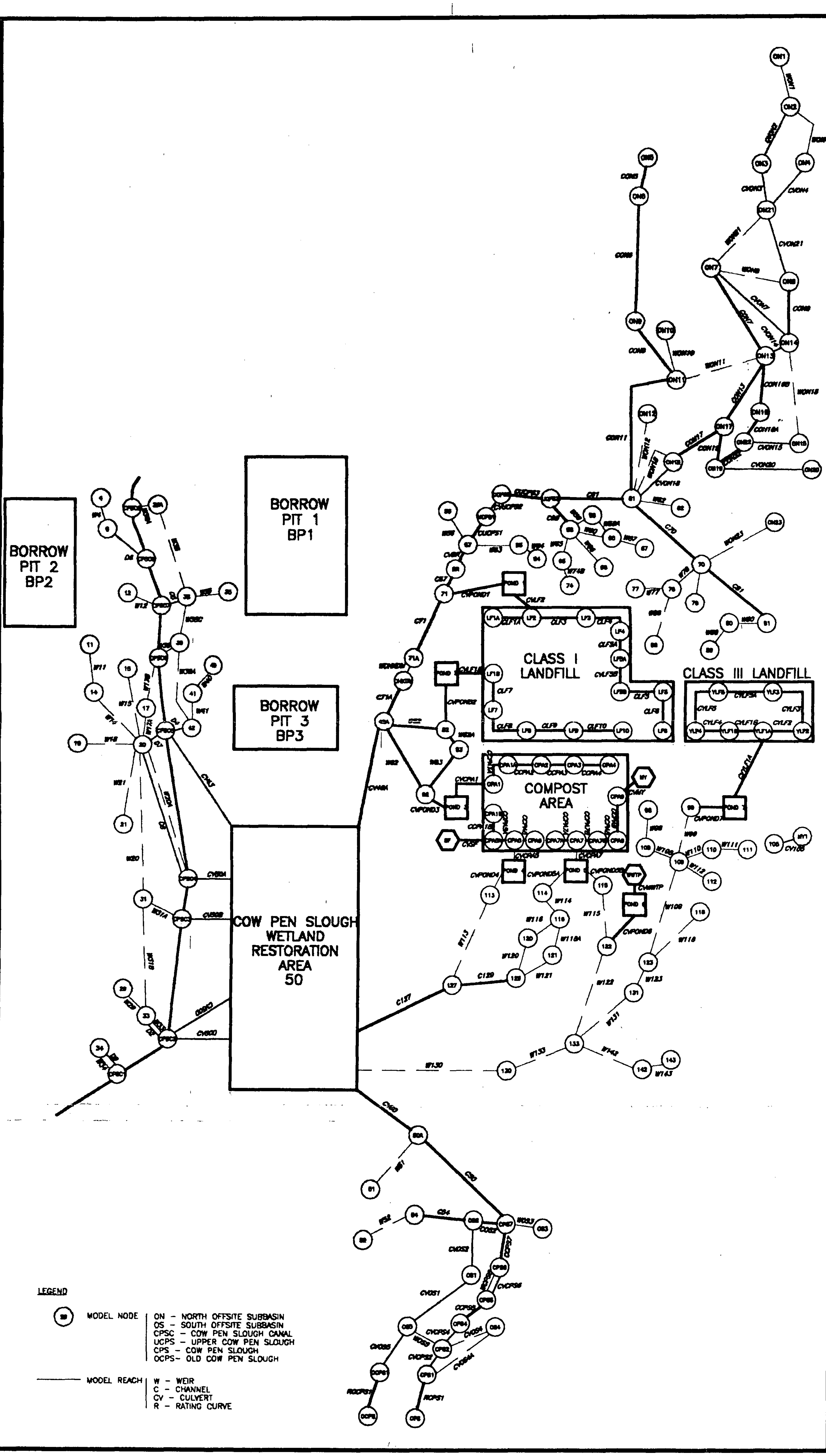
* Wetland Enhanced By Control Structure

TABLE 3-2 (continued)

ADICPR WETLAND NODE DATA SUMMARY
PROJECT CONDITIONS

Wetland Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
ON8	25.0	79	3.7	17	19.1	21.6
ON9	49.5	79	4.9	28	20.3	21.4
ON10	33.6	79	2.9	32	20.5	21.4
ON11	103.4	79	27.4	25	20.3	21.4
ON12	28.4	79	19.4	17	19.8	20.6
ON13	34.0	79	0.8	21	19.0	21.4
ON14	19.9	79	17.6	25	19.0	21.6
ON15	43.8	79	13.4	25	19.5	21.6
ON16	29.4	79	21.7	17	20.5	21.4
ON17	48.8	79	8.8	25	19.0	21.1
ON18	53.2	79	8.6	25	19.0	20.6
ON19	53.9	79	8.7	27	19.0	21.2
ON20	41.5	79	9.7	27	19.5	21.2
ON21	21.4	79	0.0	20	20.5	21.6
ON22	14.9	79	14.8	25	19.5	21.4
ON23	42.3	79	25.9	25	20.5	21.2
OS1	93.8	79	0.3	32	10.5	14.2
OS2	61.6	79	0.0	37	10.8	15.1
OS3	95.9	79	1.7	46	12.8	14.8
OS4	84.0	79	7.9	25	9.0*	15.0
OS5	48.3	79	0.7	32	6.4*	14.0

* Wetland Enhanced By Control Structure



CENTRAL SOLID WASTE DISPOSAL COMPLEX

ADICPR MODEL SCHEMATIC
PROJECT CONDITIONS

Figure No. 3-1

CDM
environmental engineers, scientists, planners, & management consultants

TABLE 3-3

ADICPR SOLID WASTE DISPOSAL COMPLEX NODE DATA SUMMARY
PROJECT CONDITIONS

Site Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
LF1A	34.0	92	0	18	21.4	24.5
LF1B	13.9	92	0	18	19.6	23.6
LF2	45.0	92	0	18	21.4	24.5
LF3	23.2	92	0	18	21.4	24.5
LF4	31.1	92	0	18	21.4	24.7
LF5	13.7	92	0	18	21.4	25.2
LF6	7.0	92	0	18	21.4	25.2
LF7	27.3	92	0	18	19.6	23.7
LF8	59.4	92	0	18	20.0	25.0
LF9	30.7	92	0	18	20.5	25.1
LF10	8.2	92	0	18	21.0	25.1
YLF1A	7.3	92	0	7	21.0	22.8
YLF1B	5.4	92	0	7	21.0	22.8
YLF2	11.9	92	0	7	21.0	22.8
YLF3	9.6	92	0	7	21.0	23.0
YLF4	8.2	92	0	7	21.0	22.9
YLF5	11.8	92	0	7	21.2	23.0
MY	6.6	79	15.2	10	18.7	20.8
CPA1A	13.5	73	0	24	19.3	21.0
CPA1B	8.5	73	0	24	18.1	20.1
CPA2	17.7	73	0	24	19.3	21.3
CPA3	17.7	73	0	24	19.3	21.6
CPA4	14.1	73	14.2	24	19.3	21.8
CPA5	11.5	73	0	24	18.1	20.1
CPA5A	---	---	---	---	18.1	20.1
CPA6	9.6	73	0	24	18.1	20.1

* Wetland Enhanced By Control Structure

TABLE 3-3 (continued)

ADICPR SOLID WASTE DISPOSAL COMPLEX NODE DATA SUMMARY
PROJECT CONDITIONS

Site Node No.	Drainage Area, ac	Runoff Curve Number, CN	Directly Connected Impervious Area, %	Time of Concentration, min.	Initial Elevation, ft.	25-Year Flood Elevation, ft.
CPA7	12.3	73	0	24	18.7	20.5
CPA7A	8.6	73	0	24	18.7	20.5
CPA7B	9.3	73	0	24	18.7	20.5
CPA8	10.0	73	0	24	18.7	20.5
CPA9	---	---	---	---	18.7	20.5
SF	5.5	73	18.2	10	18.1	20.1
WWTP	30.1	79	6.6	23	18.4	20.1
POND1	16.9	79	85.9	5	21.4	24.5
POND2	15.2	79	83.8	5	19.6	23.6
POND3	7.7	79	78.8	5	19.3	20.9
POND4	5.8	79	79.1	5	18.1	20.1
POND5	7.9	79	85.3	5	18.7	20.5
POND6	5.2	79	80.0	5	18.4	19.9
POND7	9.2	79	85.8	5	21.0	22.8
BP1	144.3	79	92.9	5	18.0	18.7
BP2	109.2	79	91.9	5	17.0	17.7
BP3	25.0	79	85.2	5	16.5	17.2

* Wetland Enhanced By Control Structure

3.4 SOLID WASTE DISPOSAL COMPLEX STORMWATER MANAGEMENT FACILITIES

3.4.1 CLASS I LANDFILL

The Class I landfill portion of the solid waste disposal complex site (294 acres) will split drainage areas between two stormwater treatment ponds, Pond 1 (16.9 acres) and Pond 2 (15.2 acres), as shown on Sheet 4. Stormwater runoff will be collected around the top perimeter of the 100-foot high landfill and conveyed by storm sewers down the steep sides of the landfill. Terraces on the sides of the landfill will be formed every 20 feet of vertical drop to collect runoff from the sideslopes and inlet to the storm sewer down pipes. These pipes will outfall at the base to the perimeter ditch system (see construction plans) which will then convey storm runoff to the treatment ponds.

Tables 3-4 and 3-5 list the pertinent design data for Ponds 1 and 2, respectively. As previously discussed, the outer bank of the perimeter ditch for the Class I landfill will be an elevated berm with a top of bank elevation of 26.0 feet. This berm will also act as a road for landfill construction vehicles and, along with the ponds, will be fully constructed prior to landfill activities. The interior berm for the ditch will be formed from the base of the landfill. The top elevation of this berm will be set approximately 2 feet above the outer berm. Thus, the primary stormwater management facilities will all be in place for the entire duration of landfill life. Refer to Table 3-3 for pertinent model input data for the Class I landfill.

TABLE 3-4

POND 1 DESIGN DATA SUMMARY

FACILITY: Pond 1

CONTRIBUTING AREA: Class I Landfill ACREAGE: 170.9ac (incl. pond)

ELEVATION, ft.	AREA, ac	VOLUME, ac-ft.	NOTES
12.5	7.96	0	Bottom
18.2	9.64	50.2	Begin Littoral Zone
20.1	12.18	70.9	Seasonal High Water (Wetland 64)
21.4	14.51	88.2	Treatment Elev.
26.0	16.57	159.7	Top of Bank (Inner)
26.0	16.9	159.7	Top of Bank (Outer)

LITTORAL ZONE: 18.2 to 21.4 ACREAGE: 4.87ac (40%)

INITIAL ELEVATION: 21.4

25-YEAR PEAK INFLOW, cfs: 520

25-YEAR PEAK OUTFLOW, cfs: 55

25-YEAR PEAK ELEVATION, ft.: 24.5

TREATMENT VOLUME: 17.3 ac. ft., 1.2 inches

DISCHARGE CULVERT INTO FACILITY: 2 - 5'X 8' BC INVERT ELEV: 17.5

OUTFALL STRUCTURE: FDOT Type "D" Inlet

BLEED DOWN: 9" diameter orifice INVERT ELEV: 20.1 50% BLEED DOWN TIME: 65.1 hrs

OVERFLOW:	L=3'	CREST ELEV:	21.4 (notch)
	L=11'	CREST ELEV:	24.5 (grate)
DISCHARGE PIPE:	36" RCP	INVERT ELEV:	19.0
SPREADER SWALE:	L=150'	CREST ELEV:	19.0

TABLE 3-4 (Continued)

POND 1 BLEED DOWN RATE CALCULATION:

Elevation	Area, ac	Discharge Rate, cfs	Average Q, cfs	Incremental Volume, ac-ft	Incremental Time, hrs	Cumulative Time, hrs
21.4	14.51	2.04				0
			1.98	1.44	8.8	
21.3	14.33	1.93				8.8
			1.87	1.42	9.2	
21.2	14.15	1.81				18.0
			1.74	1.41	9.8	
21.1	13.97	1.68				27.8
			1.61	1.39	10.4	
21.0	13.79	1.54				38.2
			1.46	1.37	11.4	
20.9	13.61	1.39				49.6
			1.30	1.35	12.6	
20.8	13.43	1.21				62.2
			1.11	1.33	14.5	
20.7	13.25	1.01				76.7

50% Bleed Down Time: 65.1 hrs

TABLE 3-5

POND 2 DESIGN DATA SUMMARY

FACILITY: Pond 2

CONTRIBUTING AREA: Class I Landfill ACREAGE: 154.7ac (incl. pond)

ELEVATION, ft.	AREA, ac	VOLUME, ac-ft.	NOTES
10.5	6.73	0	Bottom
16.5	7.84	43.7	Begin Littoral Zone
18.3	11.13	60.8	Seasonal High Water (Wetland 82)
19.6	12.74	76.3	Treatment Elev.
26.0	14.76	164.3	Top of Bank (Inner)
26.0	15.2	164.3	Top of Bank (Outer)

LITTORAL ZONE: 16.5 to 19.6 ACREAGE: 4.90ac (44%)

INITIAL ELEVATION: 19.6

25-YEAR PEAK INFLOW, cfs: 473

25-YEAR PEAK OUTFLOW, cfs: 17

25-YEAR PEAK ELEVATION, ft.: 23.6

TREATMENT VOLUME: 15.5 ac. ft., 1.2 inches

DISCHARGE CULVERT INTO FACILITY: 2 - 5'X 6' BC INVERT ELEV: 17.5

OUTFALL STRUCTURE: FDOT Type "D" Inlet

BLEED DOWN: 8" diameter orifice INVERT ELEV: 18.3 50% BLEED DOWN TIME: 71.7 hrs

OVERFLOW: L=0.5' CREST ELEV: 19.6 (notch)
L=13.5' CREST ELEV: 24.0 (grate)

DISCHARGE PIPE: 30" RCP INVERT ELEV: 17.5

SPREADER SWALE: L=50' CREST ELEV: 18.1

TABLE 3-5 (Continued)

POND 2 BLEED DOWN RATE CALCULATION:

Elevation	Area, ac	Discharge Rate, cfs	Average Q, cfs	Incremental Volume, ac-ft	Incremental Time, hrs	Cumulative Time, hrs
19.6	12.74	1.65				0
			1.60	1.27	9.6	
19.5	12.62	1.56				9.6
			1.51	1.26	10.1	
19.4	12.49	1.47				19.7
			1.42	1.24	10.6	
19.3	12.37	1.37				30.3
			1.31	1.23	11.4	
19.2	12.24	1.26				41.7
			1.20	1.22	12.3	
19.1	12.12	1.15				54.0
			1.08	1.21	13.6	
19.0	12.00	1.02				67.6
			0.94	1.19	15.3	
18.9	11.88	0.87				82.9

50% Bleed Down Time: 71.7 hrs

As listed in Tables 3-4 and 3-5, the stormwater treatment ponds will also have a top of bank elevation of 26.0 feet and will provide wet detention treatment for 1.2 inches of runoff from their contributing drainage areas. The bleed down orifice invert elevation of Pond 1 is set at the seasonal high water (SHW) elevation of Wetland 64 of 20.1 feet and overflow at the control structure is at elevation 21.4 feet. Peak discharge to the Cow Pen Slough during the 25-year design storm event is 55 cfs at a pond elevation of 24.5 feet. Discharge will be evenly distributed over the 150-foot length of a spreader swale. The bleed down time for the 9-inch diameter orifice is calculated to be 65 hours for the initial 50 percent of the treatment volume.

The control elevation for Pond 2 is set at the Wetland 82 SHW elevation of 18.3 feet and overflow at the control structure is at elevation 19.6 feet. Peak discharge to Wetland 82 for the 25-year design storm event is 17 cfs at a pond elevation of 23.6 feet. Again, a 50-foot long spreader swale will be used to evenly distribute flow to the downstream wetland. The bleed down time for the 8-inch diameter orifice is calculated to be 72 hours for the first 50 percent of the treatment volume.

In addition to the treatment provided by the wet detention pond systems, there will be four ditch blocks constructed in the Class I landfill perimeter ditch to capture and percolate stormwater runoff up to 1 foot deep in the ditch (see construction plans for location of ditch blocks). This feature of the design will provide both pretreatment of runoff and serve as an additional source of recharge to the adjacent wetlands (Wetlands 56, 74, 77, 78, 82, 83, 88, 98, and 99) during dry weather periods.

3.4.2 COMPOST AREA, MAINTENANCE YARD, AND ENTRANCE/SCALE FACILITIES

The compost area (133 acres), maintenance yard (6.6 acres), and entrance scale facilities (5.5 acres) portions of the solid waste disposal complex site will split drainage areas between three stormwater treatment ponds, Ponds 3, 4 and 5. The reason for splitting drainage into so many ponds is to provide outfalls to all of the adjacent wetlands and maintain existing drainage paths. All drainage from the disposal complex facilities will be collected via perimeter ditches constructed with an outside berm with a top elevation of 24.0 feet. At present, there is no site layout for maintenance yard buildings or the entrance/scale facilities. The drainage design for these facilities will provide for collection and discharge to the compost area perimeter ditch. Ditch blocks at elevation 24.0 feet will divide the contributing drainage basin flows to the appropriate ponds, as shown on the construction plans.

Table 3-6, 3-7, and 3-8 list the pertinent design data for Ponds 3, 4 and 5, respectively. The top of bank elevation for all three ponds will be 24.0 feet. All stormwater management facilities will be constructed prior to commencement of solid waste disposal activities and thus the treatment ponds will be in place and functional for the life of the complex. Refer to Table 3-3 for model input data for the compost area.

As listed in Tables 3-6, 3-7 and 3-8, the treatment ponds are designed to provide wet detention treatment for 1.1, 1.3, and 1.4 inches of runoff, respectively, from their contributing drainage areas. The bleed down orifices (5-, 4-, and 4-inch diameter) will provide bleed down of the first 50 percent of the treatment volumes in 75 hours, 82 hours, and 62 hours, respectively, for Ponds 3, 4, and 5. Note that Pond 5 has two separate outfalls, one each to Wetland 114 and Wetland 115.

The bleed down orifice invert elevation of Pond 3 is set at the SHW elevation of 18.1 feet for Wetland 83 with overflow of the control structure at elevation 19.3 feet. The Pond 3 outfall discharges a 25-year peak flow of 48 cfs at a pond elevation of 20.9 feet to Wetland 92. A 150' long spreader swale will be used to evenly distribute flow to the wetland.

TABLE 3-6

POND 3 DESIGN DATA SUMMARY

FACILITY: Pond 3

CONTRIBUTING AREA: Compost Area ACREAGE: 70.7ac (incl. pond)

ELEVATION, ft.	AREA, ac	VOLUME, ac-ft.	NOTES
10.5	2.50	0	Bottom
16.2	3.33	16.6	Begin Littoral Zone
18.1	5.18	24.7	Seasonal High Water (Wetland 83)
19.3	6.07	31.4	Treatment Elev.
22.0	6.94	49.0	Top of Bank (Inner)
22.0	7.7	49.0	Top of Bank (Outer)

LITTORAL ZONE: 16.2 to 19.3 ACREAGE: 2.74ac (53%)

INITIAL ELEVATION: 19.3

25-YEAR PEAK INFLOW, cfs: 163

25-YEAR PEAK OUTFLOW, cfs: 48

25-YEAR PEAK ELEVATION, ft.: 20.9

TREATMENT VOLUME: 6.7 ac. ft., 1.1 inches

DISCHARGE CULVERT INTO FACILITY: 4'X 8' BC INVERT ELEV: 15.5

OUTFALL STRUCTURE: FDOT Type "D" Inlet

BLEED DOWN: 5" diameter orifice INVERT ELEV: 18.1 50% BLEED DOWN TIME: 75.3 hrs

OVERFLOW:	L=8'	CREST ELEV:	19.3 (notch)
	L=6'	CREST ELEV:	21.0 (grate)
DISCHARGE PIPE:	36" RCP	INVERT ELEV:	17.0
SPREADER SWALE:	L=150'	CREST ELEV:	17.8

TABLE 3-6 (Continued)

POND 3 BLEED DOWN RATE CALCULATION:

Elevation	Area, ac	Discharge Rate, cfs	Average Q, cfs	Incremental Volume, ac-ft	Incremental Time, hrs	Cumulative Time, hrs
19.3	6.07	0.65				0
			0.64	0.60	11.3	
19.2	6.00	0.62				11.3
			0.60	0.60	12.1	
19.1	5.92	0.58				23.4
			0.56	0.59	12.7	
19.0	5.85	0.55				36.1
			0.52	0.58	13.5	
18.9	5.77	0.50				49.6
			0.48	0.57	14.4	
18.8	5.70	0.46				64.0
			0.44	0.57	15.7	
18.7	5.62	0.41				79.7

50% Bleed Down Time: 75.3 hrs

TABLE 3-7

POND 4 DESIGN DATA SUMMARY

FACILITY: Pond 4

CONTRIBUTING AREA: Compost Area & Entrance/Scale Facilities   ACREAGE: 40.9ac (incl. pond)

ELEVATION, ft.	AREA, ac	VOLUME, ac-ft.	NOTES
9.5	2.24	0	Bottom
15.2	2.83	14.4	Begin Littoral Zone
17.1	4.06	21.0	Seasonal High Water (Wetland 114)
18.1	4.59	25.3	Treatment Elev.
22.0	5.35	44.7	Top of Bank (Inner)
22.0	5.8	44.7	Top of Bank (Outer)

LITTORAL ZONE: 15.2 to 18.1   ACREAGE: 1.76ac (43%)

INITIAL ELEVATION: 18.1

25-YEAR PEAK INFLOW, cfs: 92

25-YEAR PEAK OUTFLOW, cfs: 5

25-YEAR PEAK ELEVATION, ft.: 20.1

TREATMENT VOLUME: 4.3 ac. ft., 1.3 inches

DISCHARGE CULVERT INTO FACILITY: 54" RCP   INVERT ELEV: 15.5

OUTFALL STRUCTURE: FDOT Type "D" Inlet

BLEED DOWN: 4" diameter orifice   INVERT ELEV: 17.1   50% BLEED DOWN TIME: 81.6 hrs

OVERFLOW:	L=0.5'	CREST ELEV:	18.1 (notch)
	L=13.5'	CREST ELEV:	20.5 (grate)
DISCHARGE PIPE:	24" RCP	INVERT ELEV:	16.0
SPREADER SWALE:	L=15'	CREST ELEV:	16.5

TABLE 3-7 (Continued)

POND 4 BLEED DOWN RATE CALCULATION:

Elevation	Area, ac	Discharge Rate, cfs	Average Q, cfs	Incremental Volume, ac-ft	Incremental Time, hrs	Cumulative Time, hrs
18.1	4.59	0.38				0
			0.37	0.46	15.0	
18.0	4.53	0.36				15.0
			0.35	0.45	15.6	
17.9	4.48	0.33				30.6
			0.32	0.45	17.0	
17.8	4.42	0.31				47.6
			0.30	0.44	17.7	
17.7	4.37	0.28				65.3
			0.26	0.43	20.0	
17.6	4.32	0.24				85.3

50% Bleed Down Time: 81.6 hrs

TABLE 3-8

POND 5 DESIGN DATA SUMMARY

FACILITY: Pond 5

CONTRIBUTING AREA: Compost Area ACREAGE: 54.7ac (incl. pond)

ELEVATION, ft.	AREA, ac	VOLUME, ac-ft.	NOTES
10.0	3.96	0	Bottom
15.5	4.66	23.7	Begin Littoral Zone
17.7	5.97	35.4	Seasonal High Water (Wetland 115)
18.7	6.74	41.8	Treatment Elev.
22.0	7.51	65.3	Top of Bank (Inner)
22.0	7.9	65.3	Top of Bank (Outer)

LITTORAL ZONE: 15.5 to 18.7 ACREAGE: 2.08ac (35%)

INITIAL ELEVATION: 18.7

25-YEAR PEAK INFLOW, cfs: 131

25-YEAR PEAK OUTFLOW, cfs: 9

25-YEAR PEAK ELEVATION, ft.: 20.5

TREATMENT VOLUME: 6.4 ac. ft., 1.4 inches

DISCHARGE CULVERT INTO FACILITY: 4' X 6' BC INVERT ELEV: 15.5

OUTFALL STRUCTURE: FDOT Type "D" Inlet (2)

BLEED DOWN: 4" diameter orifice (2) INVERT ELEV: 17.7 50% BLEED DOWN TIME: 61.5 Hrs

OVERFLOW:	L=0.5' (2)	CREST ELEV:	18.7 (notch)
	L=13.5' (2)	CREST ELEV:	20.5 (grate)
DISCHARGE PIPE:	24" RCP (2)	INVERT ELEV:	16.5
SPREADER SWALE:	L=15' (2)	CREST ELEV:	17.4

TABLE 3-8 (Continued)

POND 5 BLEED DOWN RATE CALCULATION:

Elevation	Area, ac	Discharge Rate, cfs	Average Q, cfs	Incremental Volume, ac-ft	Incremental Time, hrs	Cumulative Time, hrs
18.7	6.74	0.76				0
			0.74	0.67	11.0	
18.6	6.66	0.72				11.0
			0.69	0.66	11.6	
18.5	6.58	0.66				22.6
			0.64	0.65	12.3	
18.4	6.50	0.62				34.9
			0.59	0.65	13.3	
18.3	6.42	0.56				48.2
			0.52	0.64	14.9	
18.2	6.35	0.48				63.1

50% Bleed Down Time: 61.5 hrs

The Pond 4 outfall has its 4-inch diameter bleed down orifice set at elevation 17.1 feet, as determined from the SHW of Wetland 114, and an overflow at elevation 18.1 feet. The 25-year peak discharge, via a 15-foot long spreader swale to Wetland 113, is 5 cfs at a pond elevation of 20.1 feet.

The invert elevations of the two 4-inch diameter Pond 5 bleed down orifices are 17.7 feet, as determined by the SHW elevation of Wetland 115. Overflow at the outfall structures will occur at elevation 18.7 feet and discharge will be equally split to Wetlands 114 and 115. The 25-year peak discharge at each outfall, via 15-foot long spreader swales, is 4.5 cfs at a pond elevation of 20.5 feet.

In addition to the treatment provided by the wet detention pond systems, there will be four ditch blocks constructed in the perimeter ditch to capture and percolate stormwater runoff up to 1 foot deep in the ditch. This feature of the design will provide both pretreatment of runoff and serve as an additional source of recharge to the adjacent wetlands (Wetlands 83, 108, 114, and 115) during dry weather periods.

3.4.3 LEACHATE TREATMENT AND SLUDGE DEWATERING FACILITIES

Pond 6 will provide treatment for 1.3 inches of runoff from the 30-acre portion of the solid waste disposal complex site designated for leachate treatment and sludge dewatering facilities. The site layout and design for these facilities is not presently completed. The site drainage design for the facilities will provide for collection of runoff in a perimeter ditch with a top of bank elevation of 21.0 feet and discharge into Pond 6.

Table 3-9 lists the pertinent design data for Pond 6. The site stormwater collection system and pond will be constructed prior to the initiation of solid waste disposal activities. The top of bank elevation for Pond 6 will be 21.0 feet and the wet detention treatment volume will be provided

TABLE 3-9

POND 6 DESIGN DATA SUMMARY

FACILITY: Pond 6

CONTRIBUTING AREA: Leachate treatment Plant & Sludge Dewatering ACREAGE: 35.3ac (incl. pond)

ELEVATION, ft.	AREA, ac	VOLUME, ac-ft.	NOTES
9.5	1.84	0	Bottom
15.5	2.53	13.1	Begin Littoral Zone
17.4	3.83	19.2	Seasonal High Water (Wetland 115)
18.4	4.16	23.1	Treatment Elev.
21.0	4.94	35.0	Top of Bank (Inner)
21.0	5.2	35.0	Top of Bank (Outer)

LITTORAL ZONE: 15.5 to 18.4 ACREAGE: 1.63ac (43%)

INITIAL ELEVATION: 18.4

25-YEAR PEAK INFLOW, cfs: 107

25-YEAR PEAK OUTFLOW, cfs: 40

25-YEAR PEAK ELEVATION, ft.: 19.9

TREATMENT VOLUME: 3.9 ac. ft., 1.3 inches

DISCHARGE CULVERT INTO FACILITY: N/A INVERT ELEV: N/A

OUTFALL STRUCTURE: FDOT Type "D" Inlet

BLEED DOWN: 4" diameter orifice INVERT ELEV: 17.4 50% BLEED DOWN TIME: 74.1 hrs

OVERFLOW: L=8' CREST ELEV: 18.4 (notch)
L=6' CREST ELEV: 20.0 (grate)

DISCHARGE PIPE: 36" RCP INVERT ELEV: 16.5

SPREADER SWALE: L=120' CREST ELEV: 17.2

TABLE 3-9 (Continued)

POND 6 BLEED DOWN RATE CALCULATION:

Elevation	Area, ac	Discharge Rate, cfs	Average Q, cfs	Incremental Volume, ac-ft	Incremental Time, hrs	Cumulative Time, hrs
18.4	4.16	0.38				0
			0.37	0.41	13.4	
18.3	4.13	0.36				13.4
			0.35	0.41	14.2	
18.2	4.09	0.33				27.6
			0.32	0.41	15.5	
18.1	4.06	0.31				43.1
			0.30	0.40	16.1	
18.0	4.02	0.28				59.2
			0.26	0.40	18.6	
17.9	3.99	0.24				77.8

50% Bleed Down Time: 74.1 hrs

above the bleed down orifice invert elevation of 17.4 feet, as set from the SHW elevation of Wetland 115. Overflow of the outfall structure is at elevation 18.4 feet and discharge will be distributed by a 120-foot long spreader swale to Wetland 122. The 25-year design storm peak discharge will be 40 cfs at the pond design high water elevation of 19.9 feet. The 4-inch diameter orifice will provide a bleed down time of 74 hours for the first 50 percent of the treatment volume.

3.4.4 CLASS III LANDFILL

The Class III landfill portion of the solid waste disposal complex site (54 acres) will discharge, similar to the Class I landfill, to a perimeter ditch created by construction of an exterior road elevated above existing grade with a top of berm elevation of 24.0 feet. Stormwater runoff will be collected around the top perimeter of the landfill and conveyed by storm sewers down the steep sideslopes. Terraces on the sideslopes will catch runoff from the landfill sides and transport flow to inlets to the storm sewers for discharge into the perimeter ditch at the base (see construction plans).

Discharges from the Class III landfill will be routed through the perimeter ditch to Pond 7. The perimeter ditch and pond will be constructed prior to commencement of solid waste disposal complex operation and thus will be in place over the life of the facility.

Table 3-10 lists the pertinent design data for Pond 7. The top of bank elevation for this pond is set at 24.0 feet. The outfall structure will have a 6-inch diameter bleed down orifice with an invert elevation of 20.0 feet, as determined by the SHW elevation of Wetland 99. With the outfall overflow weir crest set at elevation 21.0 feet, Pond 7 will provide wet detention treatment of 1.4 inches of runoff from its contributing drainage area. The calculated bleed down time for the first 50 percent of this treatment volume is 69 hours.

TABLE 3-10

POND 7 DESIGN DATA SUMMARY

FACILITY: Pond 7

CONTRIBUTING AREA: Class III Landfill ACREAGE: 63.4ac (incl. pond)

ELEVATION, ft.	AREA, ac	VOLUME, ac-ft.	NOTES
12.0	3.90	0	Bottom
17.8	4.87	25.4	Begin Littoral Zone
20.0	6.84	38.3	Seasonal High Water (Wetland 99)
21.0	7.89	45.7	Treatment Elev.
24.0	8.64	70.5	Top of Bank (Inner)
24.0	9.2	70.5	Top of Bank (Outer)

LITTORAL ZONE: 17.8 to 21.0 ACREAGE: 3.02ac (44%)

INITIAL ELEVATION: 21.0

25-YEAR PEAK INFLOW, cfs: 201

25-YEAR PEAK OUTFLOW, cfs: 37

25-YEAR PEAK ELEVATION, ft.: 22.8

TREATMENT VOLUME: 7.4 ac. ft., 1.4 inches

DISCHARGE CULVERT INTO FACILITY: 4'X 10' BC INVERT ELEV: 17.5

OUTFALL STRUCTURE: FDOT Type "D" Inlet

BLEED DOWN: 6" diameter orifice INVERT ELEV: 20.0 50% BLEED DOWN TIME: 69.2 hrs

OVERFLOW:	L=5'	CREST ELEV:	21.0 (notch)
	L=9'	CREST ELEV:	22.8 (grate)
DISCHARGE PIPE:	36" RCP	INVERT ELEV:	18.5
SPREADER SWALE:	L=110'	CREST ELEV:	19.8

TABLE 3-10 (Continued)

POND 7 BLEED DOWN RATE CALCULATION:

Elevation	Area, ac	Discharge Rate, cfs	Average Q, cfs	Incremental Volume, ac-ft	Incremental Time, hrs	Cumulative Time, hrs
21.0	7.89	0.82				0
			0.79	0.78	11.9	
20.9	7.78	0.76				11.9
			0.73	0.77	12.8	
20.8	7.68	0.70				24.7
			0.66	0.76	13.9	
20.7	7.57	0.63				38.6
			0.59	0.75	15.4	
20.6	7.47	0.56				54.0
			0.51	0.74	17.6	
20.5	7.36	0.47				71.6

50% Bleed Down Time: 69.2 hrs

Pond 7 will have discharges distributed evenly over the length of a 110' long spreader swale to Wetland 99. The 25-year design storm peak discharge is 37 cfs at a pond design high water elevation of 22.8 feet. Similar to the other perimeter ditches, the Class III landfill perimeter ditch will have ditch blocks located at three points over its length to impound stormwater runoff up to 1 foot deep within the ditch. These will provide pretreatment of stormwater and a source of recharge to adjacent wetlands by percolation during dry weather periods. Wetlands 80, 89, and 99 will directly benefit from this additional source of recharge.

3.4.5 BORROW PITS

The three borrow pits that are shown on Sheet 4 and in the construction plans will provide the primary source of construction fill and cover material for the solid waste disposal complex over the duration of its active life. As such, these borrow pits will be phased in construction, beginning with Borrow Pit 1. Stormwater runoff will be prevented from entering or exiting the borrow pits by 2-foot high berms, serving as haul roads, around their perimeters. These haul roads will be constructed at elevations 20.0 feet, 19.0 feet, and 20.0 feet for Borrow Pits 1, 2, and 3, respectively. Littoral shelves will be constructed within the borrow pits totalling 15% of the surface area of each, as per Sarasota County regulations, and the maximum depth of excavation will be 20 feet below existing grade (see construction plans).

3.4.6 ACCESS ROAD

The access road construction plans show the details for the stormwater management facilities for this 3.2-mile long stretch of road. The access road is to be elevated above natural grade to meet two criteria:

- The road top is to be a minimum of 1 foot above the 100-year flood elevation.

- The road will drain to either a median swale or two side swales which will have a bottom elevation above the seasonal high groundwater elevation.

Thus, as can be seen in the construction plans, the road is generally a minimum of 3.5 to 4 feet above existing grade. The road begins at the intersection of Knights Trail Road and Rustic Road and crosses two major channels; the south leg of Cow Pen Slough and an older, natural channel which was once the original slough prior to channelization. Table 3-11 lists the dimensions of the cross culverts that will be constructed at these locations. These culverts are designed to pass the 25-year peak flow with no increase in the upstream predevelopment peak stage.

Upon reaching the County's south property line, the access road alignment carries it along the east bank of the Cow Pen Slough Canal where it will be constructed across four breaks in the spoil bank. These low spots in the spoil bank were left to provide 'communication' between the canal and its floodplain. To retain this connection, the access road design includes four sets of double 48-inch diameter RCP cross culverts at these breaks in the spoil bank (see Table 3-11). These pairs of culverts will have a weir box constructed on the eastern headwall with a crest elevation of 16.0 feet and a weir length of 20 feet. This crest elevation is the approximate elevation of the existing spoil bank breaks and will provide the ability for flow on either side to discharge to the other depending on the relative head difference.

A similar structure will be constructed at the location where the north leg of the Cow Pen Slough currently discharges through a 54-inch CMP to the canal. This will be replaced with a single 72-inch RCP with an upstream 20-foot long weir box with a crest elevation of 15.0 feet (see Table 3-11). This structure is designed to act in conjunction with a control structure on the south leg to restore the Cow Pen Slough wetland to its original undrained state (see Section 3.5).

TABLE 3-11

STORMWATER MANAGEMENT FACILITIES
ROAD CROSS CULVERTS

Location	Road Station	Culvert(s)	Invert Elev., ft.	Road Profile Elev., ft.	25-Year Storm Peak Flow, cfs	25-Year Storm Peak Elev.	Weir Length, ft.	Weir Crest Elev.
Upper Cow Pen Slough (CV45A)	218 + 80	72" RCP	12.8	22.7	98	17.0	—	—
Canal Bank (CV43)	194 + 48	2 - 48" RCP's	11.0	21.0	8	16.2	20'	16.0
Upper Cow Pen Slough Outfall to Canal (CV50A)	176 + 07	72" RCP	10.0	21.4	82	16.2	20'	15.0
Canal Bank (CV50B)	156 + 96	2 - 48" RCP's	11.0	20.6	9	16.3	20'	16.0
Canal Bank (CV50C)	139 + 50	2 - 48" RCP's	11.0	20.9	9	16.3	20'	16.0
Canal Bank (CV50D)	134 + 00	2 - 48" RCP's	11.0	21.0	9	16.3	20'	16.0
Old Cow Pen Slough (CVOS1)	80 + 65	2 - 4' X 6' BC's	10.5	19.3	140	14.2	—	—
Cow Pen Slough (CVCPS4)	69 + 50	84" RCP	6.5	15.5	112	13.8	—	—
Borrow Road @ Cow Pen Slough (CVBR)	—	72" RCP	13.7	23.0	72	18.9	—	—

There will also be a culvert constructed, without a weir box, on the north leg of the Cow Pen Slough where the access road crosses just before reaching the solid waste disposal complex site. A 72-inch RCP will be provided at this location (see Table 3-11). A similar 72-inch RCP cross culvert structure will be constructed further upstream where the borrow road will cross the north leg of the slough.

As previously discussed, stormwater runoff from the access road will be collected in a swale system, either an interior median swale or side swales, which will be elevated above the seasonal high groundwater table. These swales will be constructed with sections on level planes and controlled by ditch blocks to store 1/2-inch or more of runoff from the contributing road section. Table 3-12 provides a summary of the pertinent data related to each section of the roadway stormwater treatment swales. Standard FDOT ditch inlets will be elevated 4 to 6 inches above the swale bottom to provide the required treatment volume (i.e. a linear retention pond). The shallow depth of the treatment volume in the swales, coupled with elevation of the swales above the seasonal high groundwater table, will provide the required percolation and recovery time for the treatment storage volume. Ardaman and Associates has estimated the percolation rates for the soils from the borrow area to be in the ranges of 5 to 50 inches per hour. The native soils beneath the road construction areas will have percolation rates in the range of 6 to 0.6 inches per hour. Assuming 0.6 inches per hour as the limiting percolation rate, recovery of 6 inches of treatment volume will be achieved in 10 hours, thus provides a 3.6 factor of safety to reach the required recovery criteria of 36 hours.

3.5 WETLAND RESTORATION STORMWATER MANAGEMENT FACILITIES

3.5.1 COW PEN SLOUGH WETLAND RESTORATION AREA

The stormwater management facilities design for the Central County Solid Waste Disposal Complex site includes the construction of several control structures which will serve to impound

TABLE 3-12

ROADWAY TREATMENT SWALE DATA SUMMARY

Roadway/STATION	Structure/STA	Road Type	Contributing Length (ft.)	Required Treatment Volume (ft^3)	Design Treatment Volume (ft^3)	Design Treatment Depth (ft.)	25yr - 24 hr Storm Depth (ft.) Typical
69 +13 to 74 + 30	S-1A	SRR	517	1054	1215*	0.5	0.64
69 + 13 to 74 + 30	S-1	SRR	517	1054	1215*	0.5	0.64
74 + 30 to 80 + 00	S-2, 80 + 00	SRR	570	1140	1539	0.3	0.44
74 + 30 to 81 + 00	S-3, 81 + 00	SRR	670	1340	1809	0.3	0.59
81 + 00 to 100 + 00	S-4, 90 + 50	SER	1900	6270	6840	0.4	0.84
100 + 00 to 105 + 55	S-5, 105 + 00	SRR	555	1110	1498	0.3	0.44
100 + 00 to 117 + 00	S-6, 105 + 00	30% SRR	1700	1020 + 3927	6120	0.4	0.78
117 + 00 to 137 + 00	S-7, 127 + 00	SRD	2000	6600	8000	0.4	0.95
137 + 00 to 157 + 00	S-8, 147 + 00	SDR	2000	6600	8000	0.4	0.95
157 + 00 to 175 + 00	S-9, 166 + 00	SDR	1800	5940	7200	0.4	0.95
175 + 00 to 202 + 50	S-10, 185 + 00	SDR	2750	9075	9200	0.46	1.00
219 + 00 to 237 + 49	S-12, 219 + 00	SRR	1849	3698	4992	0.3	0.60
223 + 00 to 237 + 49	S-13, 223 + 00	SRR	1449	2898	3912	0.3	0.60

SRR - Standard Rural Road (0.5"/12" x 48' = 2 cf/ft treatment needed)

SDR - Standard Divided Road (0.5"/12" x 80' = 3.3 cf/ft treatment needed)

SER - Standard Super Elevated Road
25 yr - 24 hr Storm depth determined with Basin Runoff Networking (BRN)
* 1/2' Ditch Blocks provide treatment on sloped grade.

surface and groundwater within the County's property to rehydrate and restore the severely drained and impacted wetlands adjacent to the Cow Pen Slough and Cow Pen Slough Canal.

Tables 3-13 and 3-14 list the design data for the control structures that are shown on Sheets 4 and 5 and detailed in the construction plans. The main facility, called the Cow Pen Slough Wetland Restoration Area in Table 3-13, will be created by blocking the south leg of the slough at the south property line with a control structure consisting of a 72-inch RCP and a 20-foot long weir box. The crest elevation of this box will be set at 15.0 feet and a 90-degree V-notch will be cut into it with a vertex elevation of 14.0 feet. At the crest elevation of 15.0 feet, there will be 237 acres of impounded surface water and the control structure will be discharging 2.5 cfs exclusively to the downstream Cow Pen Slough. Above elevation 15.0 feet, discharge from the restoration area will be split between this structure and the previously described outfall to the Cow Pen Slough Canal.

To provide relative assurance that the Cow Pen Slough Wetland Restoration Area will be able to mimic the desirable attributes of a natural slough system, a daily water balance model of the wetland system was developed and a 5-year historical record was simulated. The structure of the model is shown in Figure 3-2. The components that were quantified on a daily basis are precipitation, evapotranspiration, direct runoff and groundwater inflow, lateral seepage to the Cow Pen Slough Canal, and direct discharge from the restoration area to the slough and canal.

The resultant change in storage is translated to a calculation of stage in the model. Daily precipitation data for water years 1982-1986 were obtained from published records for the Myakka River State Park gage. Monthly average pan evaporation data were converted to daily evapotranspiration (ET) values using a 0.8 pan coefficient.[1] The same monthly ET distribution was used for all water years. The volumes of precipitation input and ET output were calculated by multiplying the surface area of the restoration area, calculated every day as a function of

[1] Constructed Wetlands for wastewater Treatment, Hammer 1989

TABLE 3-13

COW PEN SLOUGH WETLAND RESTORATION AREA
DESIGN DATA SUMMARY

FACILITY: Wetland Mitigation Bank Area (Node 50)

CONTRIBUTING AREA: Cow Pen Slough   ACREAGE: 4,100ac (incl. mitig. area)

ELEVATION, ft.	AREA, ac	VOLUME, ac-ft.	NOTES
6.5	1.4	0	Existing Channel Bottom
12.0	5.5	19.0	Begin Littoral Zone
14.0	120.3	144.8	Bleeddown Invert
15.0	236.9	323.4	Weir Crest Elev. (Normal Pool)
16.0	379.7	631.7	
17.5	412.1	1027.6	Top of Containment Berm

LITTORAL ZONE: ------   ACREAGE: ------

INITIAL ELEVATION: 15.0

25-YEAR PEAK INFLOW, cfs: --

25-YEAR PEAK OUTFLOW, cfs: 192 (all outfalls combined)

25-YEAR PEAK ELEVATION, ft.: 16.3

STORAGE VOLUME NORMAL POOL TO SEASONAL
HIGH WATER ELEVATION: ----- ac. ft., ----- inches

DISCHARGE CULVERT INTO FACILITY: -----   INVERT ELEV: -----

OUTFALL STRUCTURE: 72" RCP w/Weir Box

BLEED DOWN: 90 degree V-notch   INVERT ELEV: 14.0

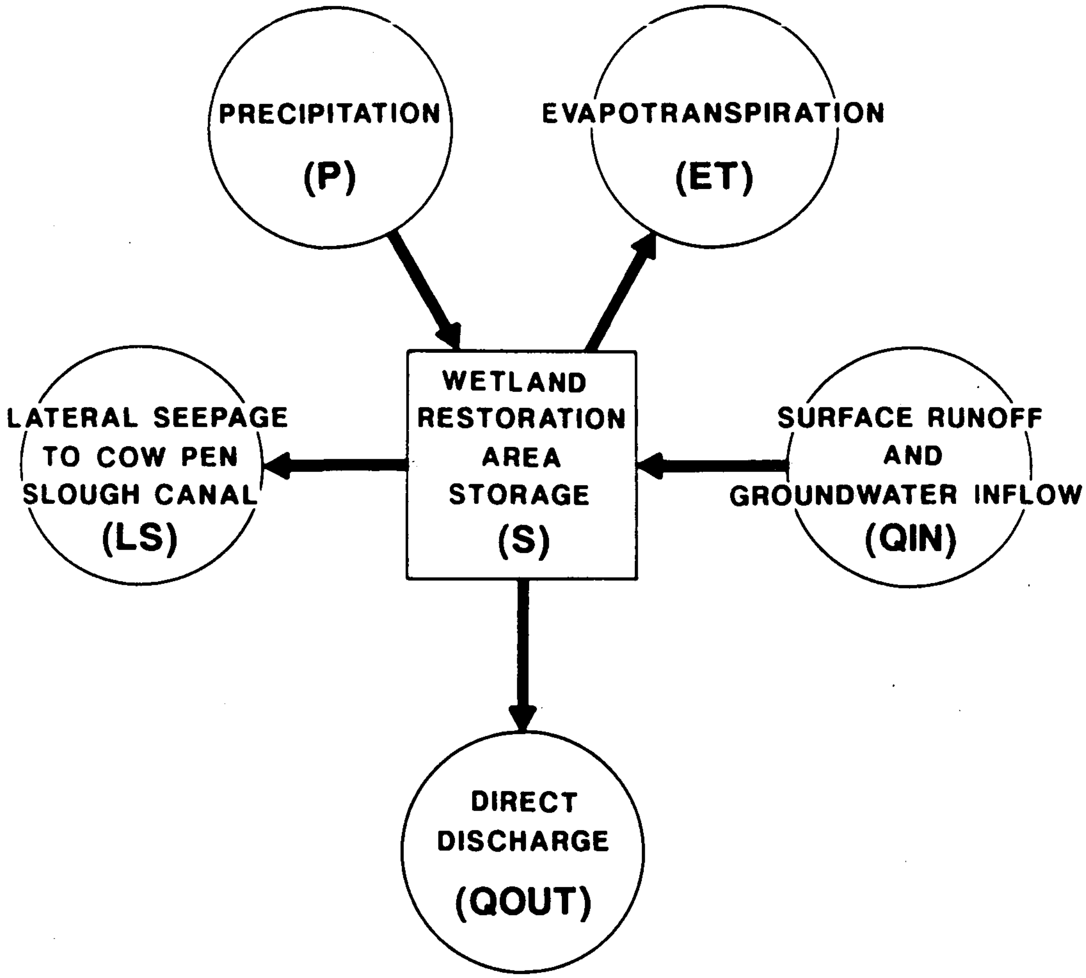
OVERFLOW: Weir Box, L=20'   CREST EVEL: 15.0

DISCHARGE PIPE: 72" RCP   INVERT ELEV:9.0

TABLE 3-14

STORMWATER MANAGEMENT FACILITIES
WETLAND RESTORATION STRUCTURES

Location	Culvert(s)	Invert Elev., ft.	Weir Length ft.	Weir Crest Elev., ft.	25-Year Storm Peak	25-Year Storm Peak Elev., ft.
South Leg Cow Pen Slough at Property Line (CV50)	72"RCP	9.0	90° V-notch 20'	14.0 15.0	94	16.3
North Leg Cow Pen Slough (WCHKDM)	———	———	90° V-notch 20'	17.0 17.5	82	18.7
Cow Pen Slough Canal West Spoil Bank (D2)	24" CMP	8.7	14'	15.3	20	16.0
Cow Pen Slough Canal East Spoil Bank (D4)	24" CMP	9.0	14'	16.0	27	17.2
Cow Pen Slough Canal East Spoil Bank (D5)	24" CMP	9.1	14'	17.0	18	17.5
Cow Pen Slough Canal West Spoil Bank (D6)	24" CMP	10.3	14'	17.0	8	17.3
Cow Pen Slough Canal West Spoil Bank (D7)	24" CMP	5.1	14'	15.3	19	16.0
Cow Pen Slough Canal West Spoil Bank (D8)	24" CMP	8.5	14'	15.3	19	16.0
Cow Pen Slough Canal West Spoil Bank (D9)	24" CMP	8.8	14'	15.3	15	15.8



$$\Delta S = QIN + P - ET - LS - QOUT$$

CDM
environmental engineers, scientists, planners & management consultants

COW PEN SLOUGH WETLAND RESTORATION
AREA WATER BALANCE MODEL SCHEMATIC

FIGURE 3-2

stage, by the respective daily values in inches. For water years 1984-1986, the Howard Creek daily steamflow data, published in U.S.G.S. Water Resources Data - Florida, provided the basis for calculating direct surface water and groundwater inflow to the restoration area. This gage was not used for the first two years because it did not begin operation until 1984, but it is actually the most appropriate due to its proximity to the site and smaller (20 sq. mi.) drainage area. For water years 1982-1983, it was necessary to use the Horse Creek gage, which is further away and encompasses a larger drainage area (42 sq. mi.). The water balance direct runoff component was calculated as a directly proportional area-to-area ratio of the gage data use. Under the stormwater management plan for the solid waste disposal complex, there will be a total of 3932 acres of drainage area contributing to the restoration area, including the restoration area itself, but not including borrow pits. The daily calculated surface area of the wetland restoration area was subtracted from this value to calculate surface water and groundwater inflows.

The dewatering effects of the Cow Pen Slough Canal were quantified using an analysis by Ardaman & Associates. The Dupuit approximation was used to estimate seepage rates from the wetland restoration area west into the canal for variable separation distances. Seasonal operation of the downstream canal sluice gate regulates the canal water surface to an elevation of 7.0 feet during the wet season (June - September) and an elevation of 11.0 feet during the October - May dry season. Ardaman determined a hydraulic conductivity value of 8.9 feet per day from both rising and falling head piezometer tests, and used this value to derive seepage rates as a function of water surface elevation in the wetland, distance of separation from the canal, and season. This relationship was used in the daily water balance model using the daily calculated wetland elevation, the seasonal canal elevation, and an average distance of separation from the canal of 100 feet over 3400 linear feet of seepage flow plane. The depth of the confining layer was assumed at elevation -15.0 feet.

The direct discharge component of the model is a quantification of the flow versus stage relationship for the two outfall structures, one to the south leg of the slough and one to the canal, as previously described. The design of the outfall structures was conceived with the intention of reproducing the hydrologic function of the original slough system and meeting hydroperiod criteria specified by environmental scientists at Biological Research Associates, Inc. (BRA). The original slough seasonal high pool elevation is estimated by BRA to be 15.4, using the palmetto fringe as an indicator, and the normal pool is estimated at an elevation of 14.9 feet. BRA indicated a desirable slough system hydroperiod variation would show occasional but brief excursions above elevation 15.4 feet during the wet season, general maintenance of the 14.9 feet normal pool elevation during the wet season, and drying out periods during the extended dry weather periods.

Figure 3-3 shows the results of the daily water balance model for the proposed Cow Pen Slough Wetland Restoration Area for the 1982-1986 water years. This plot of water of water surface elevation shows all of the above desirable attributes for the restored slough system. The temporary extreme low elevations experienced in mid-1986 is reflective of significant drought conditions at that time. Actual observations of other slough systems at that time revealed significant drying out of these natural systems.

Upstream of the wetland restoration area, a check dam with a weir crest length of 20 feet will be constructed on the north leg of the Cow Pen Slough (see Table 3-14). The weir crest will be set at elevation 17.5 and a 90-degree V-notch with a vertex elevation of 17.0 will be cut into its face. The purpose of this structure is two-fold. First, the impoundment of water upstream will serve to rehydrate Wetland 71, which has been severely impacted by the channelized slough cutting through it. Secondly, the rate of seepage from Borrow Pit 1 will be reduced, thus limiting any further dewatering impacts to adjacent wetlands with its construction.

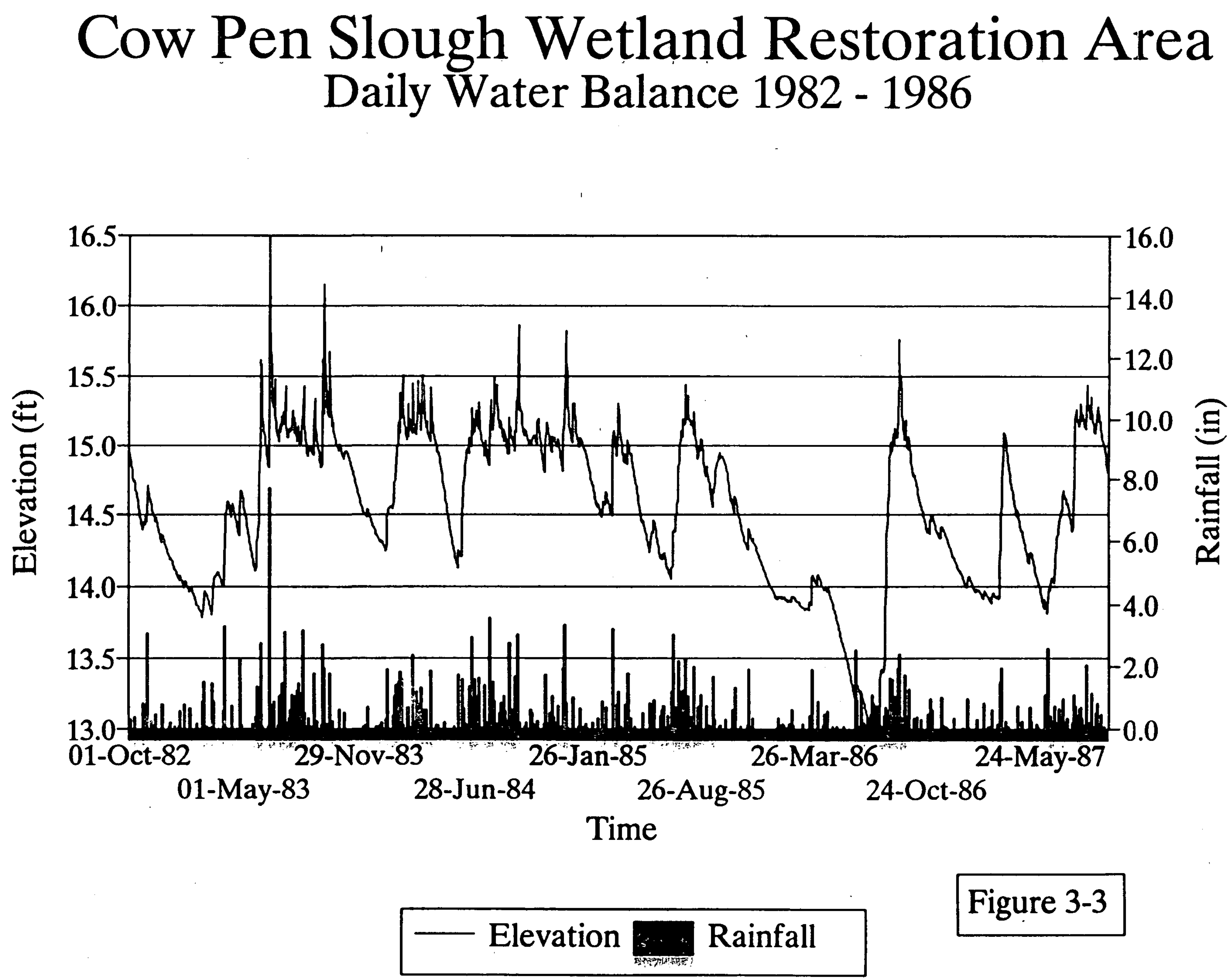

Cow Pen Slough Wetland Restoration Area
Daily Water Balance 1982 - 1986
Elevation (ft)
Rainfall (in)
16.5
16.0
15.5
15.0
14.5
14.0
13.5
13.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
01-Oct-82
01-May-83
29-Nov-83
28-Jun-84
26-Jan-85
26-Aug-85
26-Mar-86
24-Oct-86
24-May-87
Time
Elevation
Rainfall

Figure 3-3

3.5.2 COW PEN SLOUGH CANAL WETLAND RESTORATION AREAS

Table 3-14 also lists replacement inlet structures for seven (7) existing culverts which discharge from adjacent wetlands through the canal spoil banks. The locations of these culverts are shown on Sheets 4 and 5 and in the construction plans. The intent of this action is to prevent the direct discharge from these wetlands to the canal except at an elevation which exceeds the historical seasonal high water elevation. The existing flashboard risers will be replaced with FDOT Type 'D' ditch bottom inlets with grate elevations as listed in Table 3-14. By so doing, the wetlands will retain runoff and groundwater inflows for a greater period of time and should therefore return to a better condition in terms of wetland quality.

3.6 DESIGN 25-YEAR STORM EVENT RESULTS

The results of the ADICPR model simulations of project conditions for the 25-year design storm event are listed in the model outputs in Appendix B. The 25-year peak flood elevations for the individual wetlands and solid waste disposal complex site stormwater management facilities have also been previously listed in Tables 3-2 and 3-3, respectively.

Table 3-15 presents a key summary comparison of 25-year flood elevations and peak flows for existing and project conditions. This table demonstrates how the stormwater management facilities design for the solid waste disposal complex will assure that adverse offside impacts of flooding will not occur. At the northern property line, where the north leg of the Cow Pen Slough enters the site, the 25-year peak flood elevation is not influenced by the downstream control structures to be constructed on the site maximum flood elevation (20.53 feet existing versus 20.55 feet project condition). The 25-year peak discharge of the north leg to the Cow Pen Slough Canal will be reduced from the existing 101 cfs to 82 cfs with the construction of the proposed facilities. At the southern property line, the 25-year peak discharge to the south leg of Cow Pen Slough will be reduced from 148 cfs to 94 cfs. It can also be seen in Table 3-15 that both peak flows and flood elevations will also be reduced at the points where the solid

TABLE 3-15

SUMMARY COMPARISON OF 25-YEAR STORM PEAK FLOWS AND ELEVATIONS EXISTING VERSUS PROJECT CONDITIONS

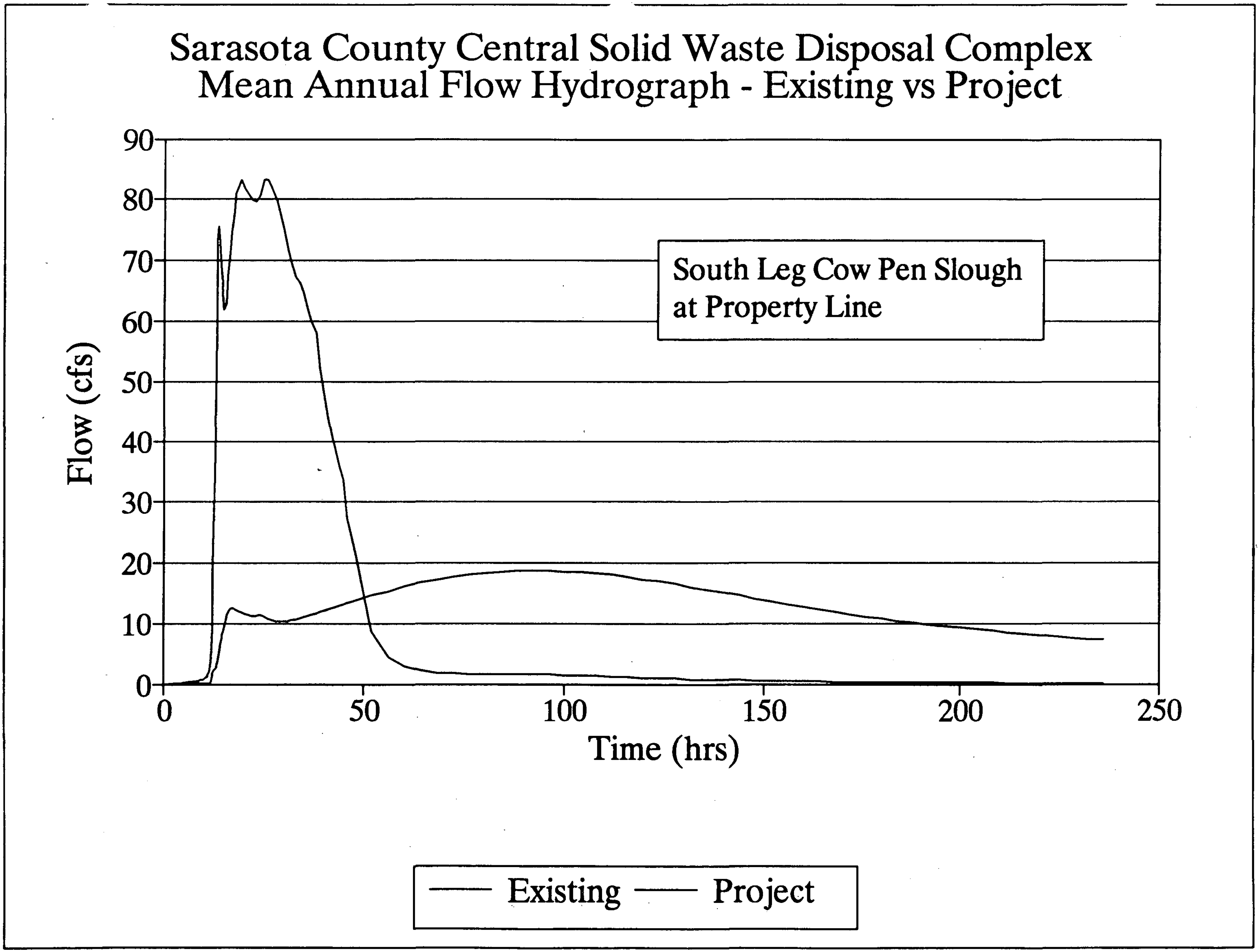
Location	Existing Condition Peak Elev., ft.	Existing Condition Peak Discharge, cfs	Project Condition Peak Elev., ft.	Project Condition Peak Discharge, cfs
Upper Cow Pen Slough at Northern Site Boundary (Node 61)	20.53	70	20.55	70
Upper Cow Pen Slough at New Access Road Crossing (Node 45A)	16.70	104	17.05	98
Upper Cow Pen Slough at Discharge to Cow Pen Slough Canal (Node 49)	15.84	101	16.26	82
Lower Cow Pen Slough at Southern Site Boundary (Node 50)	15.68	148	16.26	94
Lower Cow Pen Slough at Access Road Crossing (Node CPS4)	14.07	118	13.77	112
Old Cow Pen Slough at Access Road Crossing (Node OS1)	14.51	170	14.24	140

waste disposal complex site access road will cross the south leg of Cow Pen Slough and the old slough channel.

3.7 MEAN ANNUAL STORM EVENT RESULTS

One design criteria for the proposed solid waste disposal complex stormwater management facilities design is that the predevelopment volume of discharge from the site for the mean annual (2.33-year) storm event should not be exceeded. The ADICPR models for existing and project conditions were run for a 10-day period following a 2.33-year, 24-hour duration storm event of 4.5 inches. Figures 3-4, 3-5, and 3-6 show comparison plots of discharge at the two control structures (one to south leg Cow Pen Slough and one to Cow Pen Slough Canal).

Several things become apparent upon examination of these plots. First, the proposed control structures and wetland restoration area will do much to attenuate the peak discharge from the site as compared to its current channelized state. Secondly, the south leg of the Cow Pen Slough will benefit greatly from the extended base flow through the south control structure which will relieve the stagnant water conditions downstream of the site. There is also a clear shift in the volume of flow at each structure; the south leg of the slough will receive more than it currently does. However, as can be seen in Figure 3-6, the combined discharge volume at the two discharge points is reduced due to the amount of storage provided on the site.



Sarasota County Central Solid Waste Disposal Complex
Mean Annual Flow Hydrograph - Existing vs Project
South Leg Cow Pen Slough
at Property Line
Flow (cfs)
90
80
70
60
50
40
30
20
10
0
0
50
100
150
200
250
Time (hrs)
Existing
Project

Figure 3-4

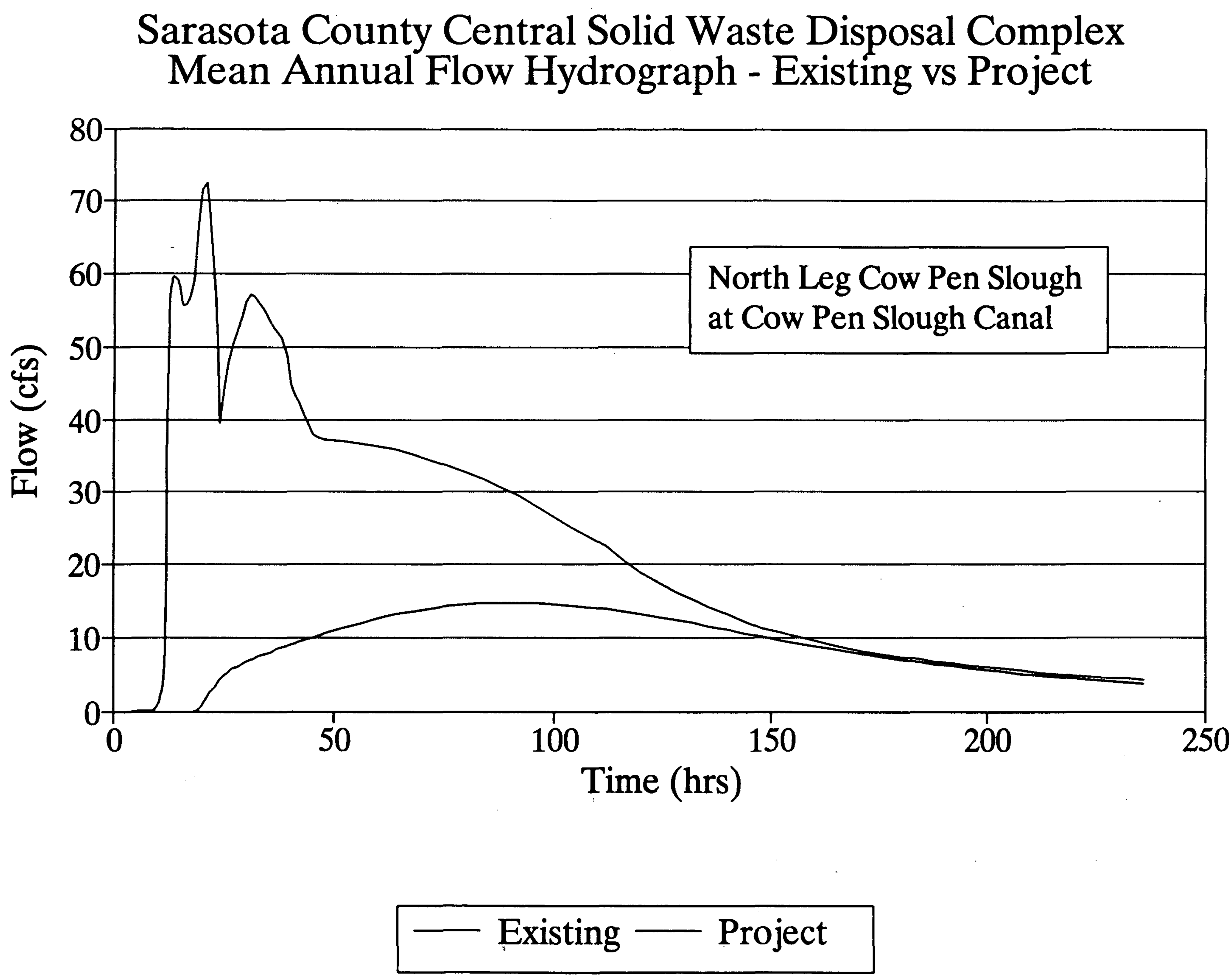

Sarasota County Central Solid Waste Disposal Complex
Mean Annual Flow Hydrograph - Existing vs Project
North Leg Cow Pen Slough
at Cow Pen Slough Canal
Flow (cfs)
Time (hrs)
0
10
20
30
40
50
60
70
80
0
50
100
150
200
250
Existing
Project

Figure 3-5

Sarasota County Central Solid Waste Disposal Complex
Mean Annual Flow Hydrograph - Existing vs Project

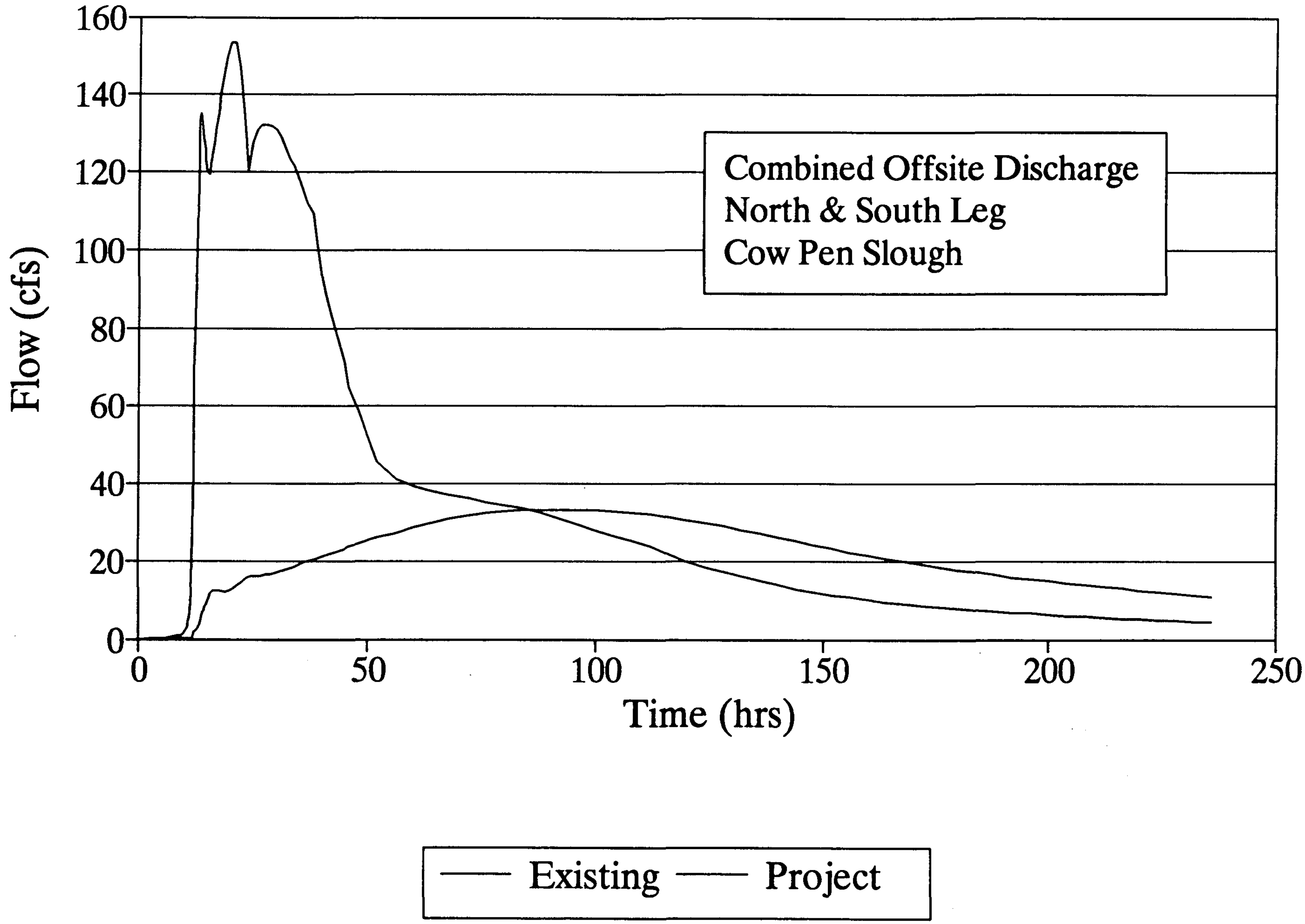


Figure 3-6

PRMT1
4/92

TECHNICAL APPENDIX C
DESIGN CALCULATIONS

TECHNICAL APPENDIX C

Design Calculations

1. Disposal Requirements

2. Service Life Calculations

3. Borrow/Cover Material Calculations

4. Leachate Collection System

5. HELP Model Results

6. Leachate flow estimates, pump and treatment plant sizing

7. Cost Estimate Breakdown

1. Disposal Requirements

WASTE STREAM PROJECTIONS (TONS/YEAR) MAH
SCWSP.WR1

YEAR	PROJECTED FUNCTIONAL POPULATION MEDIAN	SW RAW MEDIAN (T/Y) (UNCOMP)	SW 30% WT REDUCT. (T/Y) (UNCOMP)	SW VOLUME IN-PLACE (CY)	SW CUMULATIVE TOTALS (CY)
1990	------	480,000	336,000	560,000	
1995	380,379	537,000	375,900	626,500	
2000	418,480	587,000	410,900	684,833	
2005	454,235	637,000	445,900	743,167	
2010	487,949	678,000	474,600	791,000	
2015	521,098	724,000	506,800	844,667	
2020	551,207	766,000	536,200	893,667	
2025	588,050	817,000	571,900	953,167	
2030	619,500	861,000	602,700	1,004,500	
2035	651,000	905,000	633,500	1,055,833	
2040	682,550	949,000	664,300	1,107,167	
2015	521,098	724,000	506,800	844,667	15,526,000
2016	527,030	733,000	513,100	855,167	16,381,167
2017	532,963	741,000	518,700	864,500	17,245,667
2018	538,896	749,000	524,300	873,833	18,119,500
2019	544,829	757,000	529,900	883,167	19,002,667
2020	551,207	766,000	536,200	893,667	19,896,333
2021	558,481	776,000	543,200	905,333	20,801,667
2022	565,754	786,000	550,200	917,000	21,718,667
2023	573,028	797,000	557,900	929,833	22,648,500
2024	580,301	807,000	564,900	941,500	23,590,000
2025	588,050	817,000	571,900	953,167	24,543,167
2026	594,240	826,000	578,200	963,667	25,506,833
2027	600,430	835,000	584,500	974,167	26,481,000
2028	606,620	843,000	590,100	983,500	27,464,500
2029	612,810	852,000	596,400	994,000	28,458,500
2030	619,500	861,000	602,700	1,004,500	29,463,000
2031	625,695	870,000	609,000	1,015,000	30,478,000
2032	631,890	878,000	614,600	1,024,333	31,502,333
2033	638,085	887,000	620,900	1,034,833	32,537,167
2034	644,280	896,000	627,200	1,045,333	33,582,500
2035	651,000	905,000	633,500	1,055,833	34,638,333
2036	657,200	914,000	639,800	1,066,333	35,704,667
2037	663,400	922,000	645,400	1,075,667	36,780,333
2038	669,600	931,000	651,700	1,086,167	37,866,500
2039	675,800	939,000	657,300	1,095,500	38,962,000
2040	682,550	949,000	664,300	1,107,167	40,069,167
TOTALS (1995-2040)		34,345,000	24,041,500	40,069,167 -----CHECK	

WASTE STREAM PROJECTIONS (TONS/YEAR)

MAH
SCWSP.WR1

YEAR	PROJECTED FUNCTIONAL POPULATION MEDIAN	SW RAW MEDIAN (T/Y) (UNCOMP)	SW 38% WT REDUCT. (T/Y) (UNCOMP)	SW VOLUME IN-PLACE (CY)	SW CUMULATIVE TOTALS (CY)
1990	------	480,000	297,600	496,000	
1995	380,379	537,000	332,940	554,900	
2000	418,480	587,000	363,940	606,567	
2005	454,235	637,000	394,940	658,233	
2010	487,949	678,000	420,360	700,600	
2015	521,098	724,000	448,880	748,133	
2020	551,207	766,000	474,920	791,533	
2025	588,050	817,000	506,540	844,233	
2030	619,500	861,000	533,820	889,700	
2035	651,000	905,000	561,100	935,167	
2040	682,550	949,000	588,380	980,633	
2015	521,098	724,000	448,880	748,133	13,751,600
2016	527,030	733,000	454,460	757,433	14,509,033
2017	532,963	741,000	459,420	765,700	15,274,733
2018	538,896	749,000	464,380	773,967	16,048,700
2019	544,829	757,000	469,340	782,233	16,830,933
2020	551,207	766,000	474,920	791,533	17,622,467
2021	558,481	776,000	481,120	801,867	18,424,333
2022	565,754	786,000	487,320	812,200	19,236,533
2023	573,028	797,000	494,140	823,567	20,060,100
2024	580,301	807,000	500,340	833,900	20,894,000
2025	588,050	817,000	506,540	844,233	21,738,233
2026	594,240	826,000	512,120	853,533	22,591,767
2027	600,430	835,000	517,700	862,833	23,454,600
2028	606,620	843,000	522,660	871,100	24,325,700
2029	612,810	852,000	528,240	880,400	25,206,100
2030	619,500	861,000	533,820	889,700	26,095,800
2031	625,695	870,000	539,400	899,000	26,994,800
2032	631,890	878,000	544,360	907,267	27,902,067
2033	638,085	887,000	549,940	916,567	28,818,633
2034	644,280	896,000	555,520	925,867	29,744,500
2035	651,000	905,000	561,100	935,167	30,679,667
2036	657,200	914,000	566,680	944,467	31,624,133
2037	663,400	922,000	571,640	952,733	32,576,867
2038	669,600	931,000	577,220	962,033	33,538,900
2039	675,800	939,000	582,180	970,300	34,509,200
2040	682,550	949,000	588,380	980,633	35,489,833
TOTALS		34,345,000	21,293,900	35,489,833 -----CHECK	

2. Service Life Calculations

Phase I Volume Calcs Revised 5-13-92
JAB

Volume of Phase I Area (At EL 33)
2000' × 1386' = 2,772,000 sq ft
= 63.6 ac

Lift NO. 1 8 ft dee ave
4:1 inside side slopes
bottom Area = 2559,392 sq ft
Average Area 2665,696 sq ft
× 8 ft = $\boxed{789,836\ CY}$

Lift No 2 8 ft deep 5:1 side slopes
1920 × 1306 = Top area = 2,507,520 sq ft
Average Area = 2639,760 sq ft
× 8 ft = $\boxed{782,151\ CY}$

Lift No. 3 10 ft 5: side slopes
1820 × 1206 = Top Area = 2,194,920 sq ft
Average Area = 2,351,220 sq ft
× 10 ft = $\boxed{870,822\ CY}$

Lift No 4+5 After 20 ft terrace
bottom Area = 1780 × 1166
= 2075,480 sq ft
Fill 20' Top area = 1580 × 966
= 1,526,280
Average Area = 1800,880 sq ft
× 20 = $\boxed{1,333,985\ CY}$

CAMP DRESSER & McKEE CLIENT ______ JOB NO. ______ COMPUTED BY JAB
PROJECT ______ DATE CHECKED ______ DATE 5-13-92
DETAIL ______ CHECKED BY ______ PAGE NO. ______

Lifts No. 6+7 Bottom Area after 20' Terrace

= 1540 × 926 = 1,426,040 sq ft

Top area = 1340 × 726
= 972,840 sq ft

Average = 1,199,440 sq ft

×20 ft = $\boxed{888,474\ CY}$

Lifts No. 8+9 Bottom Area after 20' Terrace

= 1300 × 686
= 891,800 sq ft

Top Area = 1100 × 486
= 534,600 sq ft

Average = 713,200 sq ft

× 20 ft = $\boxed{528,296\ CY}$

Total Volume = $\boxed{5,193,564\ CY}$
Gross

Less 20% for Cover

Net Volume = $\boxed{4,154,851\ CY}$

PROJECT Walton Tract. DATE CHECKED ______ DATE 6/10/92
DETAIL ______ CHECKED BY ______ PAGE NO. SUMMARY

Revised Volume Calculations
For Final Closure Plan
Class I · Class III Landfill

Total Volume for Class I Landfill

1. 110'	593813 cu. yd.
2. 110' - 90'	3815849 cu. yd.
3. 90'	600253 cu. yd.
4. 90' - 70'	2949520 cu. yd.
5. 70'	464399 cu. yd.
6. 70' - 50'	1933620 cu. yd.
7. 50'	281048 cu. yd.
9. 50' - 24.2'	657233 cu. yd.
10. Grading Center of Landfill	
Subbase + Grading Side Slopes + Grading to ℄	17461413 cu. yd.
11. Subbase for Entire Landfill	1663541 cu. yd.
Total	30420689 cu. yd.

Total Volume for Class III Landfill

1. 70'	159340 cu. yd.
2. 70' - 50'	711343 cu. yd.
3. 50'	113766 cu. yd.
4. 50' - 21.5'	337126 cu. yd.
5. Grading Center of Landfill	132898 cu. yd.
6. Subbase for Center	1166546 cu. yd.
Total	2621019 cu. yd.

Total for Class I · Class III Landfills 33041708 cu. yd.

CAMP DRESSER & McKEE CLIENT ______ JOB NO. ______ COMPUTED BY JAB
PROJECT ______ DATE CHECKED ______ DATE 5-14-92
DETAIL ______ CHECKED BY ______ PAGE NO. ______

Disposal Needs (Total MSW)

Year	NET MSW* CY/YR	Cumulative Total CY
1995	554,900	
*96	565,290	1,120,190
*97	575,580	1,695,720
*98	585,920	2,281,640
*99	596,260	2,877,900
2000	606,600	3,484,500
*01	616,927	4,101,427
*02	627,253	
*03	637,580	
*04	647,906	
2005	658,233	

*Assumes 38% wt reduction and 1200 lb/CY

Phase I to last through 2001 or $\boxed{\text{6 years}}$
Assuming filling begins in 1995

Total Build out

Total Gross Landfill Volume

Class I 30,421,000 CY

Class III 2,621,000 CY

33,042,000 CY

Net volume with cover material subtracted

20% for Class I
15% for Class III

= $\boxed{\text{26,565,000 CY}}$

Looking at Disposal Requirement table
@ 30% wt reduction: With assumed
Starting date of January 1995
Capacity is reached begining of 2027 OR $\boxed{\text{31 years}}$

3. Borrow/Cover Material Calculations

CAMP DRESSER & McKEE CLIENT ______ JOB NO. ______ COMPUTED BY JAB
PROJECT ______ DATE CHECKED ______ DATE 1-20-92
DETAIL Borrow Needs Phase I CHECKED BY ______ PAGE NO. ______

Class I Phase I | Class I ≈ 5,039,259 CY | Class III 1,002,222 CY

as of 10-21-91 = 5,217,963 CY

Phase I Area 1-20-91

5 cells @ 400' × 1500' 68.87 acres

Assume fill = 1 ft + Ave 3 ft = 4 ft

$\boxed{444,500 \text{ CY}}$ + 2 ft protective cover $\boxed{222,222 \text{ CY}}$

Class I
Landfill Perimeter Road / ditch

5' × 50' + 52.9 sqft + 31.7 sqft + 37.5 + 184 sqft + 37.5

= 593.6 cuft / LF = 22.0 CY/LF

@ 1800' + 2000' + 1800' = 5600 LF

22 × 5600 = $\boxed{123,200 \text{ CY}}$

Class III Perimeter Road / ditch

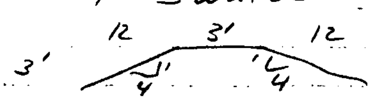
22 CY/LF @ 2200' + 3000' = $\boxed{114 400 \text{ CY}}$

Access Road

Typical section (std)

44 @ 3' + 26' @ 2'
132 ft² + 52 ft²

+ Swales



15 × 3 = 45 × 2 = 90 ft²

274 cuft / LF = 10 CY/LF

@ 3300' + 1400 + 13,749 + 1500 + 1600 = 12,544 LF

@ 10 CY/LF = $\boxed{215,500 \text{ CY}}$

Class III LF 400' × 1000' × 2' = $\boxed{29,630 \text{ CY}}$

CAMP DRESSER & McKEE | CLIENT ______ | JOB NO. ______ | COMPUTED BY JAB
PROJECT ______ | DATE CHECKED ______ | DATE 1-21-92
DETAIL ______ | CHECKED BY ______ | PAGE NO. ______

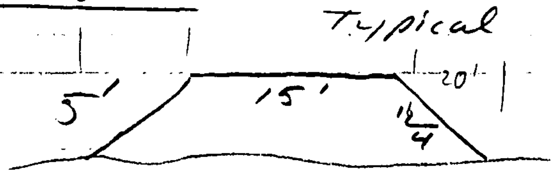
<u>Compost Area</u>

3000 x 1850' = 5,550,000 sq ft

@ Ave 2 ft = $\boxed{411,111 \text{ CY}}$

<u>Pond Berms</u>

Typical section



6.5 CY/LF

3400 + 3400 + 3000 + 3600 + 2000 + 2200 + 800 = 18,400 LF @ 6.5 = $\boxed{119,600 \text{ CY}}$

Initial Construction 1,457,441

+ 10% = $\boxed{1,619,000 \text{ CY}}$

<u>Phase I Operations</u>

5,217,963 Class I @ 15% = 782,695 CY

Class III = 579,773 CY x 10% = 57,977 CY

840,672

+ 10% $\boxed{934,080}$

Combined Initial $\boxed{2,553,000 \text{ CY}}$

+ 222,222 CY = Protective layer

$\boxed{2,775,220 \text{ CY}}$

Say 3 million CY

CAMP DRESSER & McKEE CLIENT ____ JOB NO. ____ COMPUTED BY TAB
PROJECT ____ DATE CHECKED ____ DATE 1-21-92
DETAIL ____ CHECKED BY ____ PAGE NO. ____

Ultimate Landfill Capacity Borrow Needs

Class I Gross Volume = 31,377,000 CY

@ 20% fill = $\boxed{6,275,400 \text{ CY}}$

Class III Gross Volume 3,129,000 CY

@ 15% fill = $\boxed{469,350 \text{ CY}}$

Base Areas

Class I 3000 × 3800 + 1000 × 1100

= 12,500,000 sq ft @ 4 ft = $\boxed{1,851,851 \text{ CY}}$

Liner @ 2 ft ⇒ $\boxed{925,926 \text{ CY}}$

Class III:

2400 × 1000 = 2,400,000

@ 2' = $\boxed{177,800 \text{ CY}}$

Perimeter Road/ditch

15,500' + 6750 = @ 22 CY/LF

= $\boxed{489,500 \text{ CY}}$

Access Road $\boxed{215,500 \text{ CY}}$

Compost Area $\boxed{411,111 \text{ CY}}$

Pond: Berms $\boxed{119,600 \text{ CY}}$

Total Borrow 10,936,038 CY + 10%

$\boxed{12,151,153 \text{ CY}}$

If compost is used for cover material

− 31,377,000 CY × 10% = 3,137,700 CY
− 3,129,000 × 7.5% = 234,675 CY

$\boxed{8,778,778 \text{ CY}}$

CAMP DRESSER & McKEE CLIENT ______ JOB NO. ______ COMPUTED BY ______
PROJECT ______ DATE CHECKED ______ DATE 1-22-91
DETAIL ______ CHECKED BY ______ PAGE NO. ______

Borrow Estimates (Mike Taylor)

Borrow Pit	#1	6,136,000 CY		
	#2	2,545,000 CY		
Stormwater pond	1	341,000 CY		
	2	286,000 CY	}	946,000 CY
	3	261,000 CY		
	4	58,000 CY		

Borrow Pit #1 and SWP = 7,082,000

Total Required 12,151,000 CY
Cover needs 6,744,000 CY
Str. Fill 5,407,000

YW Compost 300 TPD @ 400 lb/CY = 1500 CY/day
@ 50% volume reduction 750 CY/day
233,250 CY/year

6,744,000 CY / 30 YRS = 224,800 CY/year

Gross capacity 34,500,000 CY

CAMP DRESSER & McKEE CLIENT ______ JOB NO. ______ COMPUTED BY JAB
PROJECT ______ DATE CHECKED ______ DATE 5-15-92
DETAIL ______ CHECKED BY ______ PAGE NO. ______

Revised Structural Fill requirements *

Roadway 657,215 CY *

Pond Berms 93,340 CY *

Class I base fill 1,603,250 CY *
Ave depth 3.75 ft
@ 265 acres

Class I 12" protective layer (not imported) 427,533 CY *

~~Class III LF~~

Base Area 108,900 CY *
Ave 1.5 ft
@ 45 acres

Previous Calculations

Perimeter Road/Ditch 489,500 CY

Compost Area 411,111 CY

Total 3,790,849 CY

CAMP DRESSER & McKEE CLIENT ______ JOB NO. ______ COMPUTED BY 7AB
PROJECT ______ DATE CHECKED ______ DATE 5-15-92
DETAIL ______ CHECKED BY ______ PAGE NO. ______

Revised Borrow Quantites

Stormwater Ponds

No. 1	96,303
2	100,557
3	28,467
4	35,419
5	63,588
6	32,807
7	70633
Subtotal	427,774 CY

Borrow Lakes

1	3,597,295
2	2,768,028 CY
3	543,900 CY

Total Borrow 7,336,997 CY

Construction Borrow Needs 3,790,850 CY
* Revised
Remaining for Cover 3,546,148 CY

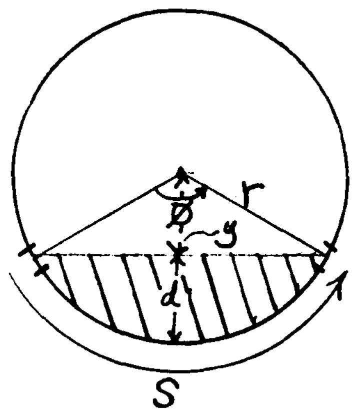
Phase I Borrow Lake 2,636,404 CY

Phase I Construction Borrow Needs 1,697,070 CY * revised

Remaining for Cover 939,334 CY ✓

4. Leachate Collection System

LEACHATE FLOW AND DESIGN SLOPE CALCULATIONS



$y = r \sin Ø$

D = <u>8" pipe</u>

y = 4 in (sin 30°)
y = 2 in
d = 2 in

<u>10" pipe</u>

y = 5 in (sin 30°)
y = 2.5 in
d = 2.5 in

d/D = 0.25 (ratio is used in the following pages)

Wetted Area $= \frac{1}{2}(r^2)(Ø - \sin Ø)$ (Ø is in radians)

Hydraulic Radius $= r_h = \frac{A}{S}$

Wetted Perimeter $= S = rØ$

Radius of pipe = r

<u>8-inch pipe</u>

r = 4"

Ø = 2.0943 radians (120°)

$A = \frac{1}{2}(4)^2(2.0943 - 0.8660)$

$A = 9.83\ in^2$

S = (4 inches)(2.0943) = 8.3772 in

$r_h = \frac{9.83\ in^2}{8.3772\ in} = 1.17\ in$

<u>10-inch pipe</u>

r = 5"

Ø = 2.0943 radians (120°)

$A = \frac{1}{2}(5)^2(2.0943 - 8.660)$

$A = 15.35\ in^2$

S = (5 inches)(2.0943) = 10.4715 in

$r_h = \frac{15.35\ in^2}{10.4715\ in} = 1.47\ in$

V_f = Velocity required to transport sediment in full pipe
R = Hydraulic Radius
n = Manning's Coefficient
B = Constant that starts sediments in motion
S = Specific Gravity of particle
D_g = Particle diameter

Using MOP FD-5, Eq. 5.40(b)

$$V_f = \frac{1.486}{n} R^{1/6} \; [B(S-1)D_g]^{1/2}$$

$R = 1.17$ in $= 0.0975$ ft, $n = 0.013$, $B = 0.42$

$S = \frac{115}{62.4} = 1.84$ (for SW or SP Soil Classification)

$D_g = 0.00015\text{m} \times 3.281 \text{ ft/m} = 0.00049$ ft
(Particle sze passing 100 sieve)

$$V_f = \frac{1.468\ (0.0975)^{1/6}}{0.013} \quad [0.42\ (1.84 - 1)(0.00049)]^{1/2}$$

$V_f = 1.022$ ft/s

Using Fig. 5-22 (attached)

V_s = Velocity required to transport sediment in partially full pipe

$\frac{V_s}{V_f} = 0.92$, $V_s = (0.92)(1.022) = 0.94$ ft/s

Using MOP FD-5, Fig. 5-19 (attached)

Slope $= 0.003$
Manning n $= 0.013$
D $= 8"$
$Q_f = 0.97$ mgd (full pipe flow) on peak day
$V_f = 2.1$ ft/s (full pipe flow)

Q = Flow rate in partially full pipe = 0.15 $\frac{\text{cfs}}{\text{cell}}$ (peak day)

Q_f = flow rate in full flow condition.

n_f = Manning roughness factor in pipe flowing full = 0.013

D = Diameter of pipe = 8"/12'/ft = 0.67', 0.83' (10" pipe)

S = slope of pipe = 0.003

For 8" pipe

$$\frac{Q_8}{Q_{f8}} = \frac{Qn_f\ (4)^{5/3}}{1.486\ \pi\ D^{8/3}\ S^{1/2}}$$

$$\frac{Q_8}{Q_{f8}} = \frac{(0.15)(0.013)(4)^{5/3}}{1.486\ \pi\ (0.67)^{8/3}\ (0.003)^{1/2}}$$

$$\frac{Q_8}{Q_{f8}} = \frac{0.0197}{0.0878} = 0.22 \qquad \therefore Q_{f8} = 1.50 \text{ cfs} = 969{,}408 \text{ gpd}$$

For 10" pipe

$$\frac{Q_{10}}{Q_{f10}} = \frac{(0.15)(0.013)(4)^{5/3}}{1.486\ \pi\ (0.83)^{8/3}\ (0.003)^{1/2}}$$

$$\frac{Q}{Q_{f10}} = 0.13 \quad \therefore Q_{f10} = 1.15 = 745{,}700 \text{ gpd}$$

Using FIG. 5-19 from MOP FD-5:

For 8" pipe at 969,400 gpd $\rightarrow V_{f8} = 4.3$ fps

For 10" pipe at 745,700 gpd $\rightarrow V_{f10} = 2.1$ fps

Using FIG. 5-22 from MOP FD-5:

For 8" pipe and calculating V_s = velocity in partially full pipe

$$\frac{V_{s8}}{V_{f8}} = 0.92$$

$V_{s8} = (0.92)(4.3) = 3.96$ fps

For 10" pipe

$$\frac{V_{s10}}{V_{f10}} = 0.92$$

$V_{s10} = (0.92)(2.1) = 1.93$ fps

For a slope of 0.002 and 8" pipe:

$$\frac{Q_8}{Q_{f8}} = \frac{(0.15)(0.013)(4)^{5/3}}{1.486\ \pi\ (0.67)^{8/3}\ (0.002)^{1/2}} = 0.28$$

$\therefore Q_{f8} = 0.55$ cfs $= 355{,}450$ gpd

Using Fig 5-19

$V_{f8} = 1.6$ fps

For a slope of 0.002 and 10" pipe:

$$\frac{Q_{10}}{Q_{f10}} = \frac{(0.15)(0.013)(4)^{5/3}}{1.486\ \pi\ (0.83)^{8/3}\ (0.002)^{1/2}} = 0.16$$

$\therefore Q_{f10} = 0.97$ cfs $= 623{,}625$ gpd

Using Fig 5-19

$V_{f10} = 1.7$ fps

Using Fig 5-22

$$\frac{V_{s8}}{V_{f8}} = 0.92$$

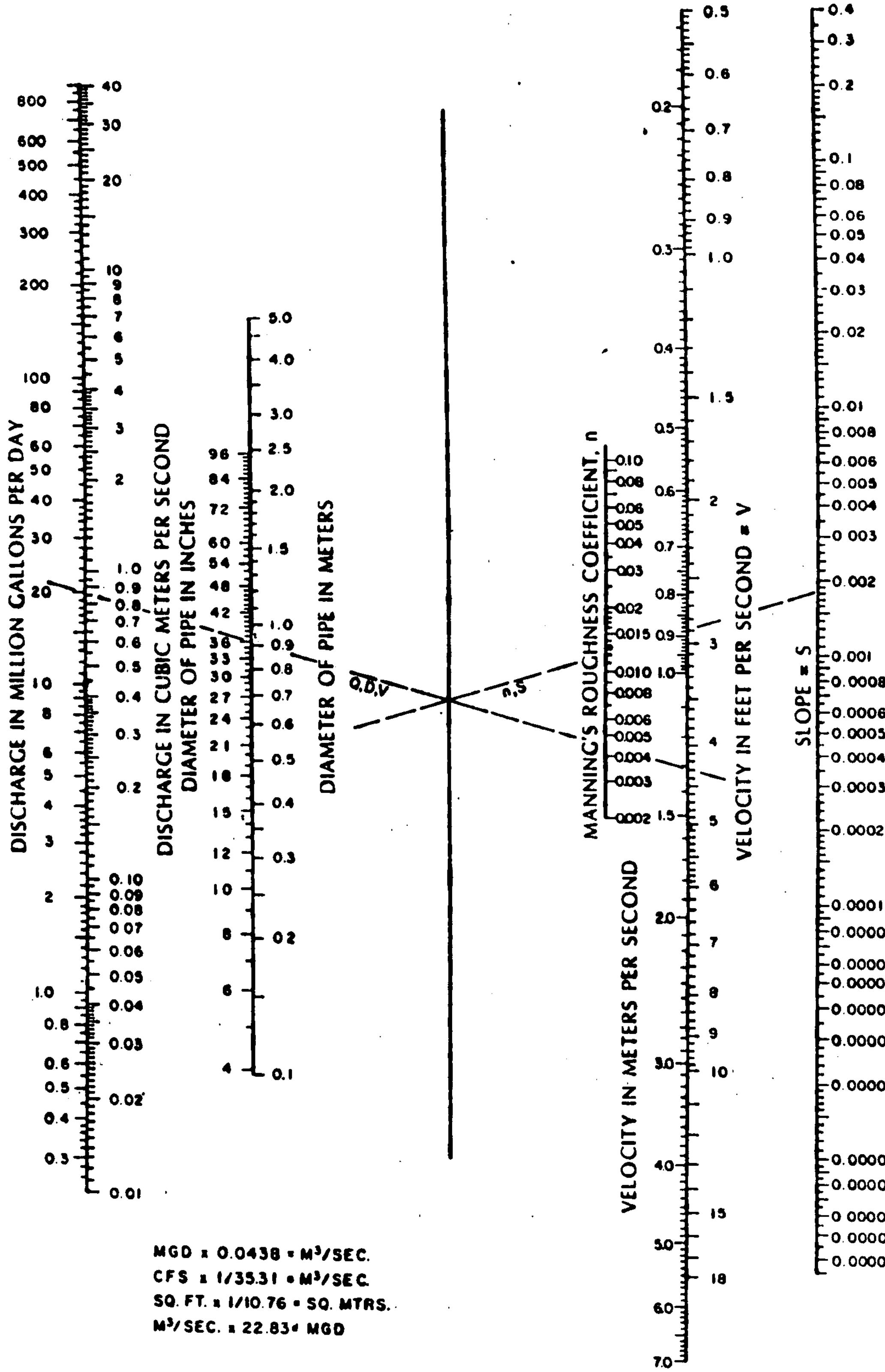
$V_{s8} = (0.92)(1.6\text{ fps}) = 1.47$ fps

$$\frac{V_{s10}}{V_{f10}} = (0.92)$$

$V_{s10} = (0.92)(1.7\text{ fps}) = 1.56$ fps

$\therefore$ A slope of 0.2% would carry sufficient velocity to transport sediment in either an 8-inch or 10-inch leachate collection pipe.

Best Available Copy



MANNING'S FORMULA: ENGLISH $V = \frac{1.486}{n} R^{2/3} S^{1/2}$
METRIC $V = \frac{1}{n} R^{2/3} S^{1/2}$

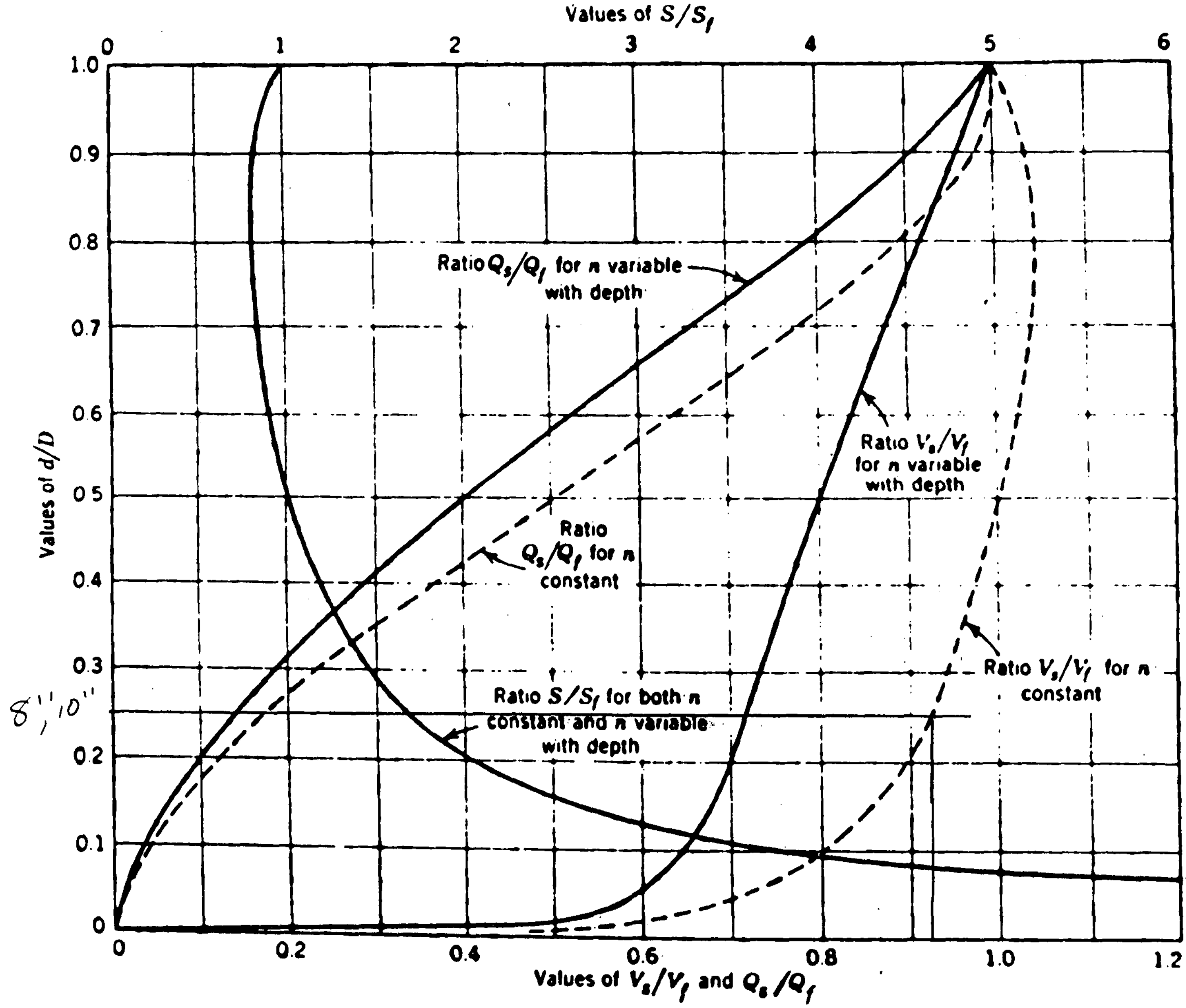


Fig. 5-22. Hydraulic elements of circular sewers that possess equal self-cleansing properties at all depths.

d/D to be greater than 0.5 requires no increase in slope because S_s is less than S_f. For smaller flows, the invert slope S_o must be increased to S_s to avoid a decrease in self-cleaning.

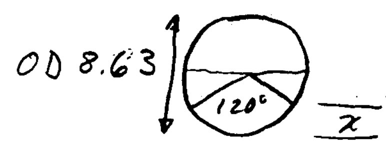
Example 5-2. A 25.4-cm (10-in.) sewer is to discharge 0.0113 m³/sec (0.4 cfs) at a velocity equivalent in self-cleansing action to that of the same size sewer flowing full at 0.61 m/sec (2.0 fps). Find the velocity of flow and the required slope for n_f = 0.013.

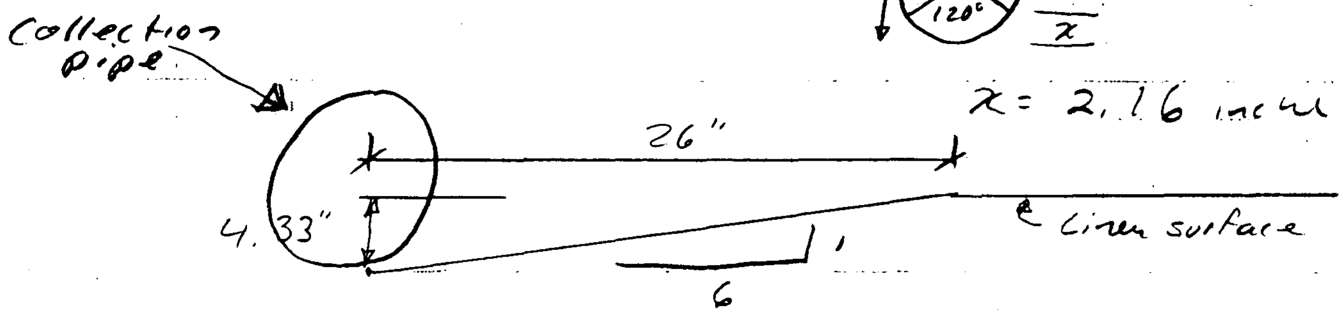
(a) From Eq. 5.40, with B = 0.8, grit of up to 0.12 mm (0.005 in.) will be transported effectively in a 25.4-cm (10-in.) sewer flowing full at 0.61 m/sec (2

Flow though pipe orifice
During max head condition

$Q = A\,C_d\sqrt{2gh}$ $\quad C_d = 0.61$ ("Fluid Mechanics" Daugherty Franzini)
$\qquad\qquad\qquad\quad A = 0.00136\ ft^2$ (Diam 1/2")

h = head on orifice





x = 2.16 inch

h = head on liner + 4.33" − 2.16"
@ 0.1" head on liner
h = 2.27" = 0.189 ft

$Q = 0.00136\ ft^2 \times 0.61 \sqrt{2\,(32.2\ ft/s^2)\ 0.189\ ft}$

$Q = .0029\ ft^3/sec$

Leachate flow under "high" head conditions
965.4 ft³/day/Acre
for Typical Cell = 11.59 acres
Total Q = 0.13 ft³/sec
Pipe length = 1216 ft with four orifice/ft
No. Orifices = 4,864
Flow per Orifice = 0.13 cfs / 4,864 = 0.00003 ft³/sec ✓

Pipe thickness Determination
Wall Crushing method

$$S_A = \frac{(SDR - 1)\,P_t}{2}$$

where S_A = Compressive Stress (psi)
SDR = Standard Dimension Ratio
P_t = External pressure (psi)

Saftey factor = 1500 psi / SA
where 1500 psi = Compressive Yield Strength

1) Calculate P_t with the following conservative assumptions:
a) Unit wt of fill 74 lb/ft^3 (2000 lb/CY)
b) Depth of fill 150 ft (permitting for 100)

P_t = 11,100 lb/ft^2 = 77.1 psi

2) Calculate SA with Assumed SDR
$SA = \left(\frac{17-1}{2}\right)$ 77.11 psi = 616.80 psi

3) Saftey factor = 1500/616.8 = 2.43

Pipe Buckling Method need $P_{cd} < P_t$
for $S_A = 616.8$ psi

$P_{cd} = 0.8\sqrt{E' \times P_c}$ = critical buckling pressure

where P_c = Hydrostatic Critical-collapse differential pressure

E' = soil modulus

$$P_c = \frac{2.32\ E}{(SDR)^3}$$

where E = tensile modulus of elasticity from Chart 25

@ 50 years $E = 17{,}000$

$$P_c = \frac{2.32\,(17{,}000)}{(17)^3} = 8.03$$

$E' = P_t / \varepsilon_s$

where ε_s is found on Chart 26

for 90% Standard Density $P_t = 11{,}100$ psf

$\varepsilon_s \approx .02$

$E' = 555{,}000 = 3{,}854$ psi

$P_{cd} = 0.8\sqrt{3{,}854 \times 8.03} = 140.7$ psi

SF = 140.7 / 77.1 = 1.82 ✓ 1.0 min
2.0 desired

Ring Deflection Check

Modified Iowa Formula

$$\text{Ring Deflection} = \% \frac{\Delta}{D_p} = \frac{DBP\,(100)}{0.149\,P_s + .061E'}$$

where Δ = Deflection

D_p = Pipe O.D.

Values for well compacted

B = Bedding Constant = .09

$P = P_t =$ = 77.1 psi

Table X1.1 ASTM F 714 → P_s = Pipe Stiffness = 150

E' = modulus of Soil Reaction = 2500

D = Deflection lag Factor = 1.75

$\% \frac{\Delta}{D_p} \leq 4.2$ at saftey factor of 2

$$\frac{\Delta}{D_p} = \frac{1.75\,(0.09)\;77.1 \times 100}{0.149\,(150) + 0.061(2500)} = 6.95$$

SF = 8.4 / 6.95 = 1.21 @ 150 ft

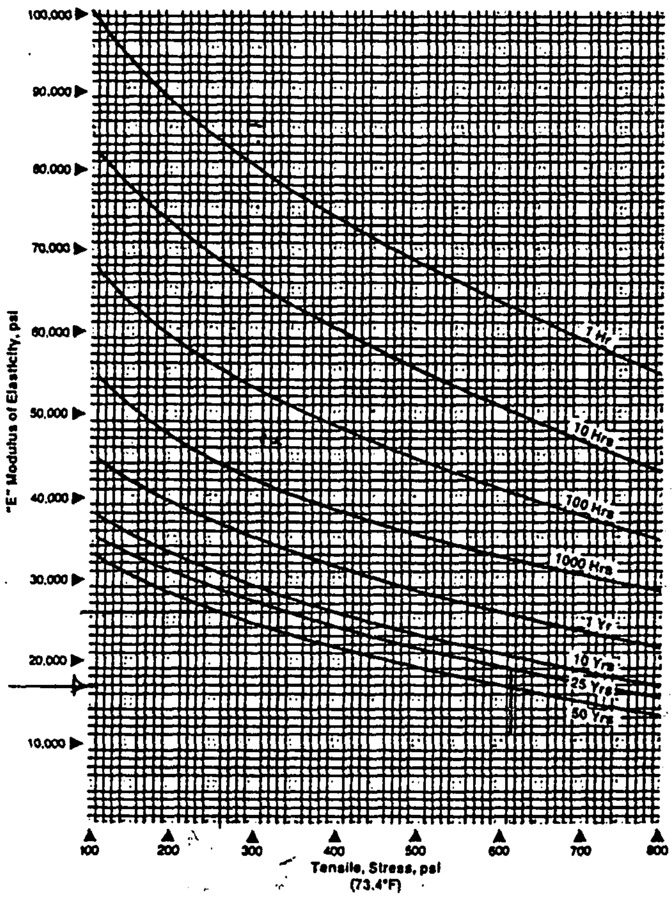
@ 100 ft P_t = 51.4 psi

$$\frac{\Delta}{D_p} = 4.63 \checkmark$$

4/5

Chart 25

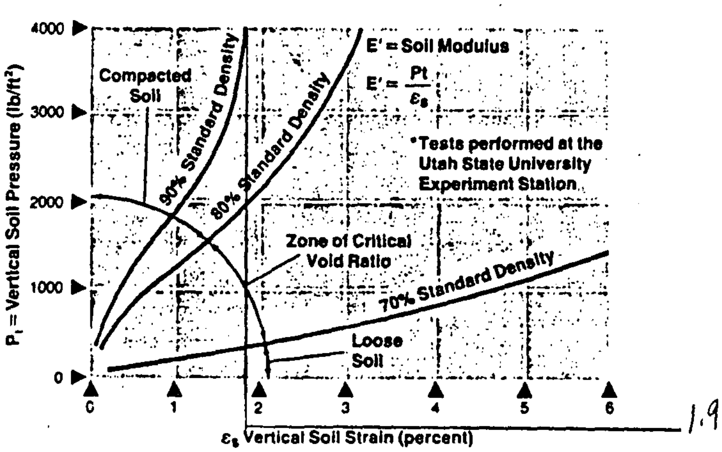
Time Dependent Modulus of Elasticity for Polyethylene Pipe vs. Stress Intensity (73.4°F)



NOTE: The short term modulus of elasticity of Driscopipe per ASTM D 638 is approximately 100,000 psi. Due to the cold flow (creep) characteristic of the pipe material, this modulus is dependent upon the stress intensity and the time duration of the applied stress.

Chart 26

Plot of Vertical Stress-Strain Data for Typical Trench Backfill (Except Clay) from Actual Tests*



EXAMPLE

Find: E' @ 2000 PSF and 80% Density

Formula: $E' = P_t/\varepsilon_s$

Calculations: E' = 2000 PSF / .018 = 111111 PSF = 771 psi

Note: The curves shown on this chart are sample curves for a granular soil. If other types of soil are used for backfill, such as clay or clay loam, curves should be developed from laboratory test data for the material used. Soil pressures greater than 4000 psf may be examined by extrapolating the slope of the curve or by generating curves by testing at those higher soil pressures. Probable error of curves is about half the distance between adjacent lines.

5/5

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X1. DEFLECTION CONTROL IN UNPRESSURIZED POLYETHYLENE PIPING SYSTEMS

X1.1 Control of deflection is achieved primarily through control of the earthwork surrounding buried systems. Practice D 2321 should be followed to achieve this control. All dimensions of pipe specified in this specification may be successfully installed if this practice is followed.

X1.2 When polyethylene pipe is to be installed by insertion into older existing pipes or is to be laid where no support from the surrounding environment is possible. Practice F 585 should be followed in making a selection of appropriate dimension ratio pipe from this specification.

X1.3 The appropriate degree of deflection in buried piping may be calculated using the modified Spangler formula.

$$X = \frac{D_e K W_c}{0.149\ PS + 0.061\ E'}$$

where:

X = deflection (horizontal or vertical), in. (or mm),
K = bedding constant, dependent on the support the pipe receives from the bottom of the trench (dimensionless),
D_e = deflection lag factor (dimensionless),
W_c = vertical load per unit of pipe length, lbf/in. (or N/m) of pipe,
PS = pipe stiffness = $4.472E/(SDR-1)^3$ where E is the flexural modulus of its pipe material (see Section 4 of this specification), psi (or kPa), and
E' = modulus of soil reaction, depending on soil strength and degree of compaction, psi (or kPa).

NOTE X1.1—Pipe stiffness (PS) may also be determined by measurement for datum at a constant 5 % deflection by Test Method D 2412. See appendix to Test Method D 2412 for correction of this test value to other deflection levels.

X1.4 For purposes of this calculation, the pipe stiffness values given in Table X1.1 may be used. For specific data on particular products, consult the manufacturer's literature.

TABLE X1.1 Pipe Stiffness Ranges for Specified Materials and DR's, psi

DR	41	32.5	26	21	17	11
Modulus, Cell Classification						
3	2–6	6–11	11–23	22–45	71–87	179–358
4	6–8	11–16	23–31	45–61	87–120	358–492
5	8–11	16–23	31–46	61–89	120–175	492–716

X2. ALLOWABLE DEFLECTION LIMITS

X2.1 Research reports, including case histories supporting the following information are on file at ASTM Headquarters:

X2.2 When said support is achieved, polyethylene pipes made to this specification may deflect or otherwise distort without kinking or buckling, and remain structurally stable up to 20 % or more of the vertical diameter. However, the lower the DR, the lower is the amount of deflection which should be permitted to ensure that long-term structural integrity is maintained. The pipe manufacturer should be consulted for the safe value for the particular pipe material involved. In the absence of specific data on a particular pipe material. Table X2.1 provides safe values for conventional polyethylene pipe materials. These values provide a safety factor of at least two against loss of structural integrity.

X2.3 If there is *no* external support around the pipe, structural integrity of the pipe is likely to be lost due to buckling if deflection exceeds 10 %. For selection of proper DR, see Practice F 585.

TABLE X2.1 Allowable Deflection of Buried Polyethylene Pipe, Short Term, %

DR	Allowable Deflection
41	10.9
32.5	8.6
26	6.5
21	5.0
17	4.0
11	3.3

X2.4 When polyethylene piping is subject to live external loading at buried depths of less than 4 ft (1200 mm), special precautions to ensure strong supporting soil conditions should be taken.

X2.5 Polyethylene pipes having high DR's will require more careful handling in storage, transport, and installation to avoid inducing pre-installation deflection. Kinking of pipe should be considered destructive damage and sections which have been kinked should not be installed, even though no leakage is observed.

5. HELP Mode Results

```
 ************************************************************************
 ************************************************************************

SARASOTA COUNTY SOLID WASTE DISPOSAL COMPLEX
RUN1, 5 FEET SOLID WASTE, NO COVER, COMPOSITE LINER
2/13/92

************************************************************************
************************************************************************


                                BARE GROUND

                                 LAYER  1
                                 --------

                        VERTICAL PERCOLATION LAYER
  THICKNESS                                 =      60.00 INCHES
  POROSITY                                  =       0.5200 VOL/VOL
  FIELD CAPACITY                            =       0.2942 VOL/VOL
  WILTING POINT                             =       0.1400 VOL/VOL
  INITIAL SOIL WATER CONTENT                =       0.2934 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY          =       0.000199999995 CM/SEC


                                 LAYER  2
                                 --------

                        VERTICAL PERCOLATION LAYER
  THICKNESS                                 =      24.00 INCHES
  POROSITY                                  =       0.5010 VOL/VOL
  FIELD CAPACITY                            =       0.2837 VOL/VOL
  WILTING POINT                             =       0.1353 VOL/VOL
  INITIAL SOIL WATER CONTENT                =       0.3154 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY          =       0.000190000006 CM/SEC


                                 LAYER  3
                                 --------

                        VERTICAL PERCOLATION LAYER
  THICKNESS                                 =       0.06 INCHES
  POROSITY                                  =       0.3000 VOL/VOL
  FIELD CAPACITY                            =       0.2940 VOL/VOL
```

```
WILTING POINT                          =       0.1400 VOL/VOL
INITIAL SOIL WATER CONTENT             =       0.1400 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =       0.400000005960 CM/SEC


                              LAYER  4
                              --------

                       LATERAL DRAINAGE LAYER
THICKNESS                              =       0.22 INCHES
POROSITY                               =       0.8100 VOL/VOL
FIELD CAPACITY                         =       0.0450 VOL/VOL
WILTING POINT                          =       0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT             =       0.0628 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =       9.000000000000 CM/SEC
SLOPE                                  =       2.00 PERCENT
DRAINAGE LENGTH                        =     200.0 FEET


                              LAYER  5
                              --------

         BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER
THICKNESS                              =      18.00 INCHES
POROSITY                               =       0.4300 VOL/VOL
FIELD CAPACITY                         =       0.3663 VOL/VOL
WILTING POINT                          =       0.2802 VOL/VOL
INITIAL SOIL WATER CONTENT             =       0.4300 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =       0.000000100000 CM/SEC
LINER LEAKAGE FRACTION                 =       0.00001000


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

SCS RUNOFF CURVE NUMBER                =       89.04
TOTAL AREA OF COVER                    =    43560. SQ FT
EVAPORATIVE ZONE DEPTH                 =       10.00 INCHES
POTENTIAL RUNOFF FRACTION              =        0.100000
UPPER LIMIT VEG. STORAGE               =        5.2000 INCHES
INITIAL VEG. STORAGE                   =        1.4247 INCHES
           SOIL WATER CONTENT INITIALIZED BY PROGRAM.


                         CLIMATOLOGICAL DATA
                         -------------------

DEFAULT RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND
SOLAR RADIATION FOR      TAMPA                FLORIDA


MAXIMUM LEAF AREA INDEX                   = 0.00
```

START OF GROWING SEASON (JULIAN DATE) = 0
END OF GROWING SEASON (JULIAN DATE) = 367

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
59.80	60.80	66.20	71.60	77.10	80.90
82.20	82.20	80.90	74.50	66.70	61.30

****

MONTHLY TOTALS FOR YEAR 74

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	2.77	3.03	2.92	2.03	3.24	12.46
	8.83	9.59	8.45	3.10	1.97	2.37
RUNOFF (INCHES)	0.097	0.017	0.068	0.027	0.010	0.934
	0.206	0.147	0.403	0.153	0.002	0.008
EVAPOTRANSPIRATION	2.056	2.734	2.327	2.503	2.635	4.219
(INCHES)	6.541	5.961	4.953	2.425	0.446	3.208
TERAL DRAINAGE FROM	0.7134	0.5669	0.4982	0.4226	0.3811	0.4209
LAYER 4 (INCHES)	3.8263	3.3180	4.5613	1.9515	1.2145	0.8035
ERCOLATION FROM	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
LAYER 5 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON	0.00	0.00	0.00	0.00	0.00	0.00
LAYER 5 (INCHES)	0.02	0.02	0.03	0.01	0.01	0.01
STD. DEV. OF DAILY HEAD	0.00	0.00	0.00	0.00	0.00	0.00
ON LAYER 5 (INCHES)	0.01	0.00	0.01	0.00	0.00	0.00

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ANNUAL TOTALS FOR YEAR 74

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	60.76	220559.	100.00

RUNOFF	2.074	7528.	3.41
EVAPOTRANSPIRATION	40.008	145228.	65.85
LATERAL DRAINAGE FROM LAYER 4	18.6783	67802.	30.74
PERCOLATION FROM LAYER 5	0.0000	0.	0.00
CHANGE IN WATER STORAGE	0.000	0.	0.00
SOIL WATER AT START OF YEAR	32.94	119569.	
SOIL WATER AT END OF YEAR	32.94	119569.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 74 THROUGH 74

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.77	3.03	2.92	2.03	3.24	12.46
	8.83	9.59	8.45	3.10	1.97	2.37
STD. DEVIATIONS	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.00
RUNOFF						
TOTALS	0.097	0.017	0.068	0.027	0.010	0.934
	0.206	0.147	0.403	0.153	0.002	0.008
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	2.056	2.734	2.327	2.503	2.635	4.219
	6.541	5.961	4.953	2.425	0.446	3.208
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000

LATERAL DRAINAGE FROM LAYER 4

TOTALS	0.7134	0.5669	0.4982	0.4226	0.3811	0.4209
	3.8263	3.3180	4.5613	1.9515	1.2145	0.8035
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION FROM LAYER 5

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 74 THROUGH 74

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	60.76	(0.000)	220559.	100.00
RUNOFF	2.074	(0.000)	7528.	3.41
EVAPOTRANSPIRATION	40.008	(0.000)	145228.	65.85
LATERAL DRAINAGE FROM LAYER 4	18.6783	(0.0000)	67802.	30.74
PERCOLATION FROM LAYER 5	0.0000	(0.0000)	0.	0.00
CHANGE IN WATER STORAGE	0.000	(0.000)	0.	0.00

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PEAK DAILY VALUES FOR YEARS 74 THROUGH 74

	(INCHES)	(CU. FT.)
PRECIPITATION	8.00	29040.0
RUNOFF	0.775	2814.7
LATERAL DRAINAGE FROM LAYER 4	0.2659	965.4
PERCOLATION FROM LAYER 5	0.0000	0.0
HEAD ON LAYER 5	0.1	
SNOW WATER	0.00	0.0

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.5182

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1368

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FINAL WATER STORAGE AT END OF YEAR 74

LAYER	(INCHES)	(VOL/VOL)
1	17.61	0.2935
2	7.57	0.3154
3	0.01	0.1400
4	0.01	0.0627
5	7.74	0.4300
SNOW WATER	0.00	

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 SARASOTA COUNTY SOLID WASTE DISPOSAL COMPLEX
 RUN2, 20 FEET SOLID WASTE, 12-INCH COVER, COMPOSITE LINER
 2/13/92

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                              BARE GROUND

                               LAYER  1
                               --------

                      VERTICAL PERCOLATION LAYER
  THICKNESS                              =     12.00 INCHES
  POROSITY                               =      0.4096 VOL/VOL
  FIELD CAPACITY                         =      0.2466 VOL/VOL
  WILTING POINT                          =      0.1353 VOL/VOL
  INITIAL SOIL WATER CONTENT             =      0.1705 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY       =      0.000009500000 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             =      0.2946 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY       =      0.000199999995 CM/SEC


                               LAYER  3
                               --------

                      VERTICAL PERCOLATION LAYER
  THICKNESS                              =     24.00 INCHES
  POROSITY                               =      0.5010 VOL/VOL
  FIELD CAPACITY                         =      0.2837 VOL/VOL
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WILTING POINT	=	0.1353 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2853 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000190000006 CM/SEC

LAYER 4

VERTICAL PERCOLATION LAYER

THICKNESS	=	0.06 INCHES
POROSITY	=	0.3000 VOL/VOL
FIELD CAPACITY	=	0.2940 VOL/VOL
WILTING POINT	=	0.1400 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1400 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.400000005960 CM/SEC

LAYER 5

LATERAL DRAINAGE LAYER

THICKNESS	=	0.22 INCHES
POROSITY	=	0.8100 VOL/VOL
FIELD CAPACITY	=	0.0450 VOL/VOL
WILTING POINT	=	0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0501 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	9.000000000000 CM/SEC
SLOPE	=	2.00 PERCENT
DRAINAGE LENGTH	=	200.0 FEET

LAYER 6

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	18.00 INCHES
POROSITY	=	0.4300 VOL/VOL
FIELD CAPACITY	=	0.3663 VOL/VOL
WILTING POINT	=	0.2802 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4300 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000000100000 CM/SEC
LINER LEAKAGE FRACTION	=	0.00001000

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	91.61
TOTAL AREA OF COVER	=	43560. SQ FT
EVAPORATIVE ZONE DEPTH	=	10.00 INCHES
POTENTIAL RUNOFF FRACTION	=	0.100000

UPPER LIMIT VEG. STORAGE = 4.0960 INCHES
INITIAL VEG. STORAGE = 1.5425 INCHES
SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

DEFAULT RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND
SOLAR RADIATION FOR TAMPA FLORIDA

MAXIMUM LEAF AREA INDEX = 0.00
START OF GROWING SEASON (JULIAN DATE) = 0
END OF GROWING SEASON (JULIAN DATE) = 367

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
59.80	60.80	66.20	71.60	77.10	80.90
82.20	82.20	80.90	74.50	66.70	61.30

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MONTHLY TOTALS FOR YEAR 74

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	2.77	3.03	2.92	2.03	3.24	12.46
	8.83	9.59	8.45	3.10	1.97	2.37
RUNOFF (INCHES)	0.598	0.244	0.658	0.270	0.209	5.010
	2.611	1.984	3.199	0.941	0.082	0.183
EVAPOTRANSPIRATION	2.192	2.734	2.131	2.104	2.709	4.442
(INCHES)	7.627	6.102	6.078	2.417	1.247	2.743
LATERAL DRAINAGE FROM	0.2030	0.1812	0.1972	0.1868	0.1884	0.1777
LAYER 5 (INCHES)	0.1792	0.1782	0.1778	0.1936	0.1938	0.2028
PERCOLATION FROM	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
LAYER 6 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

G. DAILY HEAD ON	0.00	0.00	0.00	0.00	0.00	0.00
LAYER 6 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00

STD. DEV. OF DAILY HEAD	0.00	0.00	0.00	0.00	0.00	0.00
ON LAYER 6 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00

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ANNUAL TOTALS FOR YEAR 74

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	60.76	220559.	100.00
RUNOFF	15.990	58043.	26.32
EVAPOTRANSPIRATION	42.526	154370.	69.99
LATERAL DRAINAGE FROM LAYER 5	2.2596	8202.	3.72
PERCOLATION FROM LAYER 6	0.0000	0.	0.00
CHANGE IN WATER STORAGE	-0.016	-57.	-0.03
SOIL WATER AT START OF YEAR	87.35	317073.	
SOIL WATER AT END OF YEAR	87.33	317016.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 74 THROUGH 74

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.77	3.03	2.92	2.03	3.24	12.46
	8.83	9.59	8.45	3.10	1.97	2.37
STD. DEVIATIONS	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.00
RUNOFF						
TOTALS	0.598	0.244	0.658	0.270	0.209	5.010

	2.611	1.984	3.199	0.941	0.082	0.183
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000

EVAPOTRANSPIRATION

TOTALS	2.192	2.734	2.131	2.104	2.709	4.442
	7.627	6.102	6.078	2.417	1.247	2.743
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000

LATERAL DRAINAGE FROM LAYER 5

TOTALS	0.2030	0.1812	0.1972	0.1868	0.1884	0.1777
	0.1792	0.1782	0.1778	0.1936	0.1938	0.2028
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION FROM LAYER 6

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 74 THROUGH 74

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	60.76	(0.000)	220559.	100.00
RUNOFF	15.990	(0.000)	58043.	26.32
EVAPOTRANSPIRATION	42.526	(0.000)	154370.	69.99
LATERAL DRAINAGE FROM LAYER 5	2.2596	(0.0000)	8202.	3.72
PERCOLATION FROM LAYER 6	0.0000	(0.0000)	0.	0.00
CHANGE IN WATER STORAGE	-0.016	(0.000)	-57.	-0.03

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PEAK DAILY VALUES FOR YEARS 74 THROUGH 74

	(INCHES)	(CU. FT.)
PRECIPITATION	8.00	29040.0
RUNOFF	1.513	5491.3
LATERAL DRAINAGE FROM LAYER 5	0.0066	23.8
PERCOLATION FROM LAYER 6	0.0000	0.0
HEAD ON LAYER 6	0.0	
SNOW WATER	0.00	0.0

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.3989

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1346

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FINAL WATER STORAGE AT END OF YEAR 74

LAYER	(INCHES)	(VOL/VOL)
1	2.05	0.1706
2	70.68	0.2945
3	6.85	0.2852
4	0.01	0.1400
5	0.01	0.0501
6	7.74	0.4300
SNOW WATER	0.00	

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SARASOTA COUNTY SOLID WASTE DISPOSAL COMPLEX
RUN3, 50 FEET SOLID WASTE, 12-INCH COVER, COMPOSITE LINER
2/13/92

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BARE GROUND

LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4096 VOL/VOL
FIELD CAPACITY	=	0.2466 VOL/VOL
WILTING POINT	=	0.1353 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1705 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000009500000 CM/SEC

LAYER 2

VERTICAL PERCOLATION LAYER

THICKNESS	=	600.00 INCHES
POROSITY	=	0.5200 VOL/VOL
FIELD CAPACITY	=	0.2942 VOL/VOL
WILTING POINT	=	0.1400 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2943 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000199999995 CM/SEC

LAYER 3

VERTICAL PERCOLATION LAYER

THICKNESS	=	24.00 INCHES
POROSITY	=	0.5010 VOL/VOL
FIELD CAPACITY	=	0.2837 VOL/VOL

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WILTING POINT                          =      0.1353 VOL/VOL
INITIAL SOIL WATER CONTENT             =      0.2849 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =      0.000190000006 CM/SEC


                        LAYER  4
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               VERTICAL PERCOLATION LAYER
THICKNESS                              =      0.06 INCHES
POROSITY                               =      0.3000 VOL/VOL
FIELD CAPACITY                         =      0.2940 VOL/VOL
WILTING POINT                          =      0.1400 VOL/VOL
INITIAL SOIL WATER CONTENT             =      0.1400 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =      0.400000005960 CM/SEC


                        LAYER  5
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                LATERAL DRAINAGE LAYER
THICKNESS                              =      0.22 INCHES
POROSITY                               =      0.8100 VOL/VOL
FIELD CAPACITY                         =      0.0450 VOL/VOL
WILTING POINT                          =      0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT             =      0.0501 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =      9.000000000000 CM/SEC
SLOPE                                  =      2.00 PERCENT
DRAINAGE LENGTH                        =    200.0 FEET


                        LAYER  6
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         BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER
THICKNESS                              =     18.00 INCHES
POROSITY                               =      0.4300 VOL/VOL
FIELD CAPACITY                         =      0.3663 VOL/VOL
WILTING POINT                          =      0.2802 VOL/VOL
INITIAL SOIL WATER CONTENT             =      0.4300 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =      0.000000100000 CM/SEC
LINER LEAKAGE FRACTION                 =      0.00001000


                GENERAL SIMULATION DATA
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SCS RUNOFF CURVE NUMBER                =      91.61
TOTAL AREA OF COVER                    =   43560. SQ FT
EVAPORATIVE ZONE DEPTH                 =      10.00 INCHES
POTENTIAL RUNOFF FRACTION              =       0.100000
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PPER LIMIT VEG. STORAGE = 4.0960 INCHES
INITIAL VEG. STORAGE = 1.5425 INCHES
SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

DEFAULT RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND
SOLAR RADIATION FOR TAMPA FLORIDA

MAXIMUM LEAF AREA INDEX = 0.00
START OF GROWING SEASON (JULIAN DATE) = 0
END OF GROWING SEASON (JULIAN DATE) = 367

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
59.80	60.80	66.20	71.60	77.10	80.90
82.20	82.20	80.90	74.50	66.70	61.30

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MONTHLY TOTALS FOR YEAR 74

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	2.77	3.03	2.92	2.03	3.24	12.46
	8.83	9.59	8.45	3.10	1.97	2.37
RUNOFF (INCHES)	0.598	0.244	0.658	0.270	0.209	5.010
	2.611	1.984	3.199	0.941	0.082	0.183
EVAPOTRANSPIRATION	2.192	2.734	2.131	2.104	2.709	4.442
(INCHES)	7.627	6.102	6.078	2.417	1.247	2.743
LATERAL DRAINAGE FROM	0.1997	0.1797	0.1976	0.1895	0.1939	0.1856
LAYER 5 (INCHES)	0.1897	0.1892	0.1852	0.1954	0.1916	0.1991
PERCOLATION FROM	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
LAYER 6 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

G. DAILY HEAD ON	0.00	0.00	0.00	0.00	0.00	0.00
LAYER 6 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00

STD. DEV. OF DAILY HEAD	0.00	0.00	0.00	0.00	0.00	0.00
ON LAYER 6 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00

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ANNUAL TOTALS FOR YEAR 74

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	60.76	220559.	100.00
RUNOFF	15.990	58043.	26.32
EVAPOTRANSPIRATION	42.526	154370.	69.99
LATERAL DRAINAGE FROM LAYER 5	2.2961	8335.	3.78
PERCOLATION FROM LAYER 6	0.0000	0.	0.00
CHANGE IN WATER STORAGE	-0.052	-189.	-0.09
SOIL WATER AT START OF YEAR	193.21	701370.	
SOIL WATER AT END OF YEAR	193.16	701180.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 74 THROUGH 74

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.77	3.03	2.92	2.03	3.24	12.46
	8.83	9.59	8.45	3.10	1.97	2.37
STD. DEVIATIONS	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.00
RUNOFF						
TOTALS	0.598	0.244	0.658	0.270	0.209	5.010

	2.611	1.984	3.199	0.941	0.082	0.183
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000

EVAPOTRANSPIRATION

TOTALS	2.192	2.734	2.131	2.104	2.709	4.442
	7.627	6.102	6.078	2.417	1.247	2.743
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000

LATERAL DRAINAGE FROM LAYER 5

TOTALS	0.1997	0.1797	0.1976	0.1895	0.1939	0.1856
	0.1897	0.1892	0.1852	0.1954	0.1916	0.1991
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION FROM LAYER 6

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 74 THROUGH 74

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	60.76	(0.000)	220559.	100.00
RUNOFF	15.990	(0.000)	58043.	26.32
EVAPOTRANSPIRATION	42.526	(0.000)	154370.	69.99
LATERAL DRAINAGE FROM LAYER 5	2.2961	(0.0000)	8335.	3.78
PERCOLATION FROM LAYER 6	0.0000	(0.0000)	0.	0.00
CHANGE IN WATER STORAGE	-0.052	(0.000)	-189.	-0.09

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PEAK DAILY VALUES FOR YEARS 74 THROUGH 74

	(INCHES)	(CU. FT.)
PRECIPITATION	8.00	29040.0
RUNOFF	1.513	5491.3
LATERAL DRAINAGE FROM LAYER 5	0.0064	23.4
PERCOLATION FROM LAYER 6	0.0000	0.0
HEAD ON LAYER 6	0.0	
SNOW WATER	0.00	0.0

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.3989

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1346

****

****

FINAL WATER STORAGE AT END OF YEAR 74

LAYER	(INCHES)	(VOL/VOL)
1	2.05	0.1706
2	176.52	0.2942
3	6.84	0.2848
4	0.01	0.1400
5	0.01	0.0501
6	7.74	0.4300
SNOW WATER	0.00	

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SARASOTA COUNTY SOLID WASTE DISPOSAL COMPLEX
RUN4, GOOD GRASS, CLOSED CELL, COMPOSITE LINER
2/13/92

****
****

GOOD GRASS

LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	18.00 INCHES
POROSITY	=	0.5010 VOL/VOL
FIELD CAPACITY	=	0.2837 VOL/VOL
WILTING POINT	=	0.1353 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1447 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000797999965 CM/SEC

LAYER 2

VERTICAL PERCOLATION LAYER

THICKNESS	=	0.06 INCHES
POROSITY	=	0.3000 VOL/VOL
FIELD CAPACITY	=	0.2940 VOL/VOL
WILTING POINT	=	0.1400 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1354 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.400000005960 CM/SEC

LAYER 3

BARRIER SOIL LINER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4300 VOL/VOL
FIELD CAPACITY	=	0.3663 VOL/VOL

```
WILTING POINT                          =      0.2802 VOL/VOL
INITIAL SOIL WATER CONTENT             =      0.2900 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =      0.000000100000 CM/SEC


                        LAYER  4
                        --------

          VERTICAL PERCOLATION LAYER
THICKNESS                              =     12.00 INCHES
POROSITY                               =      0.4096 VOL/VOL
FIELD CAPACITY                         =      0.2466 VOL/VOL
WILTING POINT                          =      0.1353 VOL/VOL
INITIAL SOIL WATER CONTENT             =      0.2838 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =      0.000009500000 CM/SEC


                        LAYER  5
                        --------

          VERTICAL PERCOLATION LAYER
THICKNESS                              =   1200.00 INCHES
POROSITY                               =      0.5200 VOL/VOL
FIELD CAPACITY                         =      0.2942 VOL/VOL
WILTING POINT                          =      0.1400 VOL/VOL
INITIAL SOIL WATER CONTENT             =      0.2837 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =      0.000199999995 CM/SEC


                        LAYER  6
                        --------

          VERTICAL PERCOLATION LAYER
THICKNESS                              =     24.00 INCHES
POROSITY                               =      0.5010 VOL/VOL
FIELD CAPACITY                         =      0.2837 VOL/VOL
WILTING POINT                          =      0.1353 VOL/VOL
INITIAL SOIL WATER CONTENT             =      0.3469 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =      0.000190000006 CM/SEC


                        LAYER  7
                        --------

          VERTICAL PERCOLATION LAYER
THICKNESS                              =      0.06 INCHES
POROSITY                               =      0.3000 VOL/VOL
FIELD CAPACITY                         =      0.2940 VOL/VOL
WILTING POINT                          =      0.1400 VOL/VOL
INITIAL SOIL WATER CONTENT             =     29.4544 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =      0.400000005960 CM/SEC
```

LAYER 8

LATERAL DRAINAGE LAYER

THICKNESS	=	0.22 INCHES
POROSITY	=	0.8100 VOL/VOL
FIELD CAPACITY	=	0.0450 VOL/VOL
WILTING POINT	=	0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT	=	2.8277 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	9.000000000000 CM/SEC
SLOPE	=	2.00 PERCENT
DRAINAGE LENGTH	=	200.0 FEET

LAYER 9

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	18.00 INCHES
POROSITY	=	0.4300 VOL/VOL
FIELD CAPACITY	=	0.3663 VOL/VOL
WILTING POINT	=	0.2802 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4300 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000000100000 CM/SEC
LINER LEAKAGE FRACTION	=	0.00001000

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	74.98
TOTAL AREA OF COVER	=	43560. SQ FT
EVAPORATIVE ZONE DEPTH	=	22.00 INCHES
UPPER LIMIT VEG. STORAGE	=	9.0360 INCHES
INITIAL VEG. STORAGE	=	2.6340 INCHES

SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

DEFAULT RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND
SOLAR RADIATION FOR TAMPA FLORIDA

MAXIMUM LEAF AREA INDEX	=	3.30
START OF GROWING SEASON (JULIAN DATE)	=	0
END OF GROWING SEASON (JULIAN DATE)	=	367

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
59.80	60.80	66.20	71.60	77.10	80.90
82.20	82.20	80.90	74.50	66.70	61.30

****

MONTHLY TOTALS FOR YEAR 74

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	2.77	3.03	2.92	2.03	3.24	12.46
	8.83	9.59	8.45	3.10	1.97	2.37
RUNOFF (INCHES)	0.046	0.000	0.009	0.000	0.000	3.647
	0.146	0.046	0.610	0.172	0.000	0.000
EVAPOTRANSPIRATION	2.234	3.079	2.558	2.997	3.010	4.502
(INCHES)	8.157	7.399	6.478	2.736	1.607	2.965
PERCOLATION FROM	0.0000	0.0000	0.0000	0.0000	0.0000	2.6237
LAYER 3 (INCHES)	0.4829	1.7990	3.4386	0.0000	0.0000	0.0000
LATERAL DRAINAGE FROM	0.1131	0.1042	0.1172	0.1147	0.1196	0.1165
LAYER 8 (INCHES)	0.1218	0.1247	0.1250	0.1360	0.1373	0.1466
RCOLATION FROM	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
LAYER 9 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON	0.00	0.00	0.00	0.00	0.00	0.14
LAYER 3 (INCHES)	0.02	0.06	0.14	0.00	0.00	0.00
STD. DEV. OF DAILY HEAD	0.00	0.00	0.00	0.00	0.00	0.61
ON LAYER 3 (INCHES)	0.05	0.14	0.29	0.00	0.00	0.00
AVG. DAILY HEAD ON	0.00	0.00	0.00	0.00	0.00	0.00
LAYER 9 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00
STD. DEV. OF DAILY HEAD	0.00	0.00	0.00	0.00	0.00	0.00
ON LAYER 9 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00

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ANNUAL TOTALS FOR YEAR 74

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	60.76	220559.	100.00
RUNOFF	4.675	16971.	7.69
EVAPOTRANSPIRATION	47.722	173230.	78.54
PERCOLATION FROM LAYER 3	8.3443	30290.	13.73
LATERAL DRAINAGE FROM LAYER 8	1.4768	5361.	2.43
PERCOLATION FROM LAYER 9	0.0000	0.	0.00
CHANGE IN WATER STORAGE	6.886	24997.	11.33
SOIL WATER AT START OF YEAR	360.92	1310143.	
SOIL WATER AT END OF YEAR	367.81	1335140.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 74 THROUGH 74

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.77	3.03	2.92	2.03	3.24	12.46
	8.83	9.59	8.45	3.10	1.97	2.37
STD. DEVIATIONS	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.00
RUNOFF						
TOTALS	0.046	0.000	0.009	0.000	0.000	3.647
	0.146	0.046	0.610	0.172	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	2.234	3.079	2.558	2.997	3.010	4.502

	8.157	7.399	6.478	2.736	1.607	2.965
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000

PERCOLATION FROM LAYER 3

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	2.6237
	0.4829	1.7990	3.4386	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

LATERAL DRAINAGE FROM LAYER 8

TOTALS	0.1131	0.1042	0.1172	0.1147	0.1196	0.1165
	0.1218	0.1247	0.1250	0.1360	0.1373	0.1466
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION FROM LAYER 9

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 74 THROUGH 74

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	60.76	(0.000)	220559.	100.00
RUNOFF	4.675	(0.000)	16971.	7.69
EVAPOTRANSPIRATION	47.722	(0.000)	173230.	78.54
PERCOLATION FROM LAYER 3	8.3443	(0.0000)	30290.	13.73
LATERAL DRAINAGE FROM LAYER 8	1.4768	(0.0000)	5361.	2.43
PERCOLATION FROM LAYER 9	0.0000	(0.0000)	0.	0.00
CHANGE IN WATER STORAGE	6.886	(0.000)	24997.	11.33

****

****

PEAK DAILY VALUES FOR YEARS 74 THROUGH 74

	(INCHES)	(CU. FT.)
PRECIPITATION	8.00	29040.0
RUNOFF	3.647	13238.6
PERCOLATION FROM LAYER 3	1.9868	7211.9
HEAD ON LAYER 3	3.3	
LATERAL DRAINAGE FROM LAYER 8	0.0048	17.4
PERCOLATION FROM LAYER 9	0.0000	0.0
HEAD ON LAYER 9	0.0	
SNOW WATER	0.00	0.0

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.4051

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1350

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FINAL WATER STORAGE AT END OF YEAR 74

LAYER	(INCHES)	(VOL/VOL)
1	2.63	0.1460
2	0.01	0.1354
3	3.48	0.2900
4	3.47	0.2894
5	347.12	0.2893
6	8.40	0.3500
7	1.77	29.4544
8	0.62	2.8277
9	0.31	0.0173
SNOW WATER	0.00	

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6. Leachate Flow Estimates

Pump and Treatment Plant Sizing

CAMP DRESSER & McKEE CLIENT ______ JOB NO. ______ COMPUTED BY JAB
PROJECT ______ DATE CHECKED ______ DATE 5-12-92
DETAIL ______ CHECKED BY ______ PAGE NO. ______

Treatment Plant Sizing

Average daily flow 49,212 gpd

Peak day - substitute Run 1 peak
= 91,305 gpd

Overflow storage = 521,326
+ 8 inches of rainfall over storage pond
155,833
= 677,159 gal ≈ 700,000 gal

Drain (recirculate) storage pond in 15 days
700,000 / 15 = 46,667 gpd

Add to Average daily flow = 95,879 gpd

Plant size $\boxed{100,000\text{ gpd.}}$

Effluent Irrigation Sizing

Plant Flow 100,000 gpd = 13,370 cuft/day

Assume 1/2" per week

13,730 ft³ x 7 days / 0.0417 ft / 43560 sqft/ac
= 51.56 ac say $\boxed{60\text{ acres}}$

Effluent storage 50 days storage
= 5,000,000 gal.

@ 2 ft depth = 334,225 sq ft = 7.7 acres

CAMP DRESSER & McKEE CLIENT JOB NO. COMPUTED BY JAB
PROJECT DATE CHECKED DATE 2-17-92
DETAIL CHECKED BY PAGE NO.

Revised Leachate Flow estimates

Run		
4	Clay Cap, Good Grass	110 gpd/acre
3	50 ft Solid Waste (Intermediate Operation)	171 gpd/acre
2	20 ft Solid Waste (early operation)	168 gpd/acre
1	Open Cell Condition	2,095 gpd/acre

WORST-CASE Conditions

Run	Area (ac)	Average Daily Contributing Flow (gpd)
4	60.0	6,600
3	71.1	12,158
2	19.2	3,226
1	9.6	20,112
Total		$\boxed{42,096 \text{ gpd}}$

Peak Flow (substitute 19,774 gpd/acre for open cell)

189,830 + 21,984 = $\boxed{211,814 \text{ gpd}}$

Check 25 yr 24 hour storm = 8 inches

8" @ 9.6 acres = 2,085,304 gallons

Assume 6" retained in soil layer (1' head)

1,563,978 gallons

Difference 521,326 gallons/24 hrs

☆ 5A4SW peak day 7,221 gpd/ac @ 9.6 acres = 69,322 gpd

21,984

91,305

CAMP DRESSER & McKEE CLIENT ______ JOB NO. ______ COMPUTED BY JAB
PROJECT ______ DATE CHECKED ______ DATE 4-30-92
DETAIL ______ CHECKED BY ______ PAGE NO. ______

Leachate pumping requirement

must discharge peak flow

of 521,326 gallons within 48 hrs

521,326 / 2 / 1440 = 181 gpm

Select pump capable of minimum flow

of 181 gpm through discharge system

Following pages document selection

of Sure Pump model TSP 27-3

CAMP DRESSER & McKEE CLIENT ____ JOB NO. ____ COMPUTED BY TMP
PROJECT ____ DATE CHECKED ____ DATE 4/24/92
DETAIL ____ CHECKED BY ____ PAGE NO. 1

CALC. SHEET

1. CALCULATE VELOCITY IN PIPES

$Q(GPM/CFP) = VEL(ft/s) \cdot AREA(ft^2)$

$V = Q/A$

VALUES FOR Q ARE GIVEN FROM 0-1000 GPM AS POSSIBLE PUMP FLOWS.

AREA CALCULATED USING $A = \pi R^2$
THE VALUE FOR R WAS CHOSEN AS 4" DIA FOR SUMP PIPING AND 6" DIA. FOR FORCE MAIN.

$$AREA = 3.1416 * \left(\frac{4}{2}\right)^2 = 12.5664\ IN^2 \cdot \left(\frac{1\ ft}{12\ in}\right)^2$$

$$= \underline{\underline{0.09\ ft^2}}$$

$$VEL(ft^3/s) = 25\ GPM \cdot \frac{0.133666\ FT^3}{GAL} \cdot \frac{1\ M}{60\ s} \Big/ 0.09\ ft^2$$

$$= \underline{\underline{0.62\ ft/s}}$$

2. CALCULATE HEAD LOSS DUE TO FITTINGS IN SUMP PIPING

	k VALUES	# OF FITTINGS	
STANDARD ELBOW 90°	0.51	4	2.04
CHECK VALVE	2.0	2	4.0
STANDARD TEE (THRU BRANCH)	1.02	2	2.04
GATE VALVE (FULLY OPEN)	0.14	2	0.28
FLEX HOSE	$C = \frac{1.486}{n} \cdot R^{1/6}$ $n \approx 0.03$		1.54
MISC.			2.0
			≈ 12

$$H_L = \frac{k \cdot VEL^2}{2 \cdot g} \quad \text{WHERE } g = 32.2\ ft/s^2$$

$$= \frac{12 \cdot (1.28)^2}{2 \cdot 32.2} \quad @\ 50\ GPM \Rightarrow \underline{\underline{0.30\ ft}}$$

CAMP DRESSER & McKEE

CLIENT ______ JOB NO. ______ COMPUTED BY TMP

PROJECT ______ DATE CHECKED ______ DATE 4/24/92

DETAIL ______ CHECKED BY ______ PAGE NO. 2

3. CALCULATION OF HEAD LOSS DUE TO FRICTION IN PIPE

A. REFERENCE NOMOGRAPH ENCLOSED FOR H_L / 1000 ft

B. ASSUMPTION

2.0 ft OF STATIC HEAD

C. 35 LF OF PIPING USED FOR 4" DIA. SUMP PIPING

$$H_L\ (\text{FRICTION}) = \frac{35\ \text{LF}}{1000\ \text{FT}} \cdot 3.50 \text{ @ } 50\ \text{GPM}$$

$$= 0.1225$$

4. TOTAL DYNAMIC HEAD (ft) @ 50 GPM

$$H_L\ (\text{FITTINGS}) + H_V + H_S = 0.30 + 0.12 + 2.0$$

$$= 2.42\ \text{ft}$$

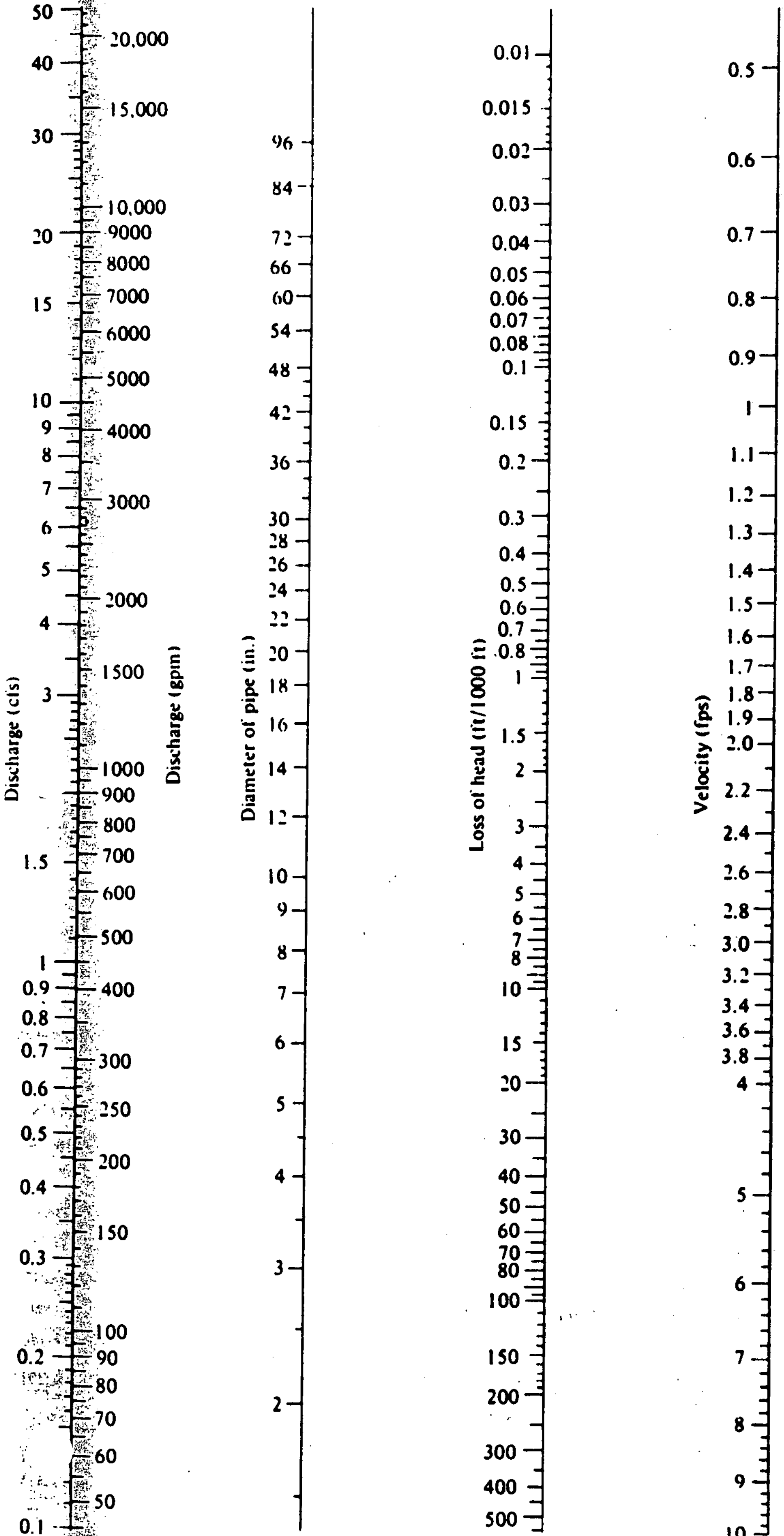
CALCULATIONS FOR 4" DIA & 6" DIA ARE SIMILIAR
ADDITIONAL INFORMATION FOR 6" PIPE IS GIVEN BELOW

L.F. = 9000

K VALUES

STANDARD ELBOW 90°	= 0.45	× 9	=	4.05
STANDARD ELBOW 45°	= 0.24	× 3	=	0.72
MISC.			=	1.00
				≈ 6.0

THE TDH FOR THE SUMP PIPING IS ADDED TO THE FORCE MAIN PIPING AND PLOTTED AGAINST FLOW (GPM) TO ESTABLISH A SYSTEM CURVE. FROM PUMP CAPACITY CURVES GIVEN BY THE MANUFACTURER, A C PUMP WAS CHOSEN (TSP 27-3).

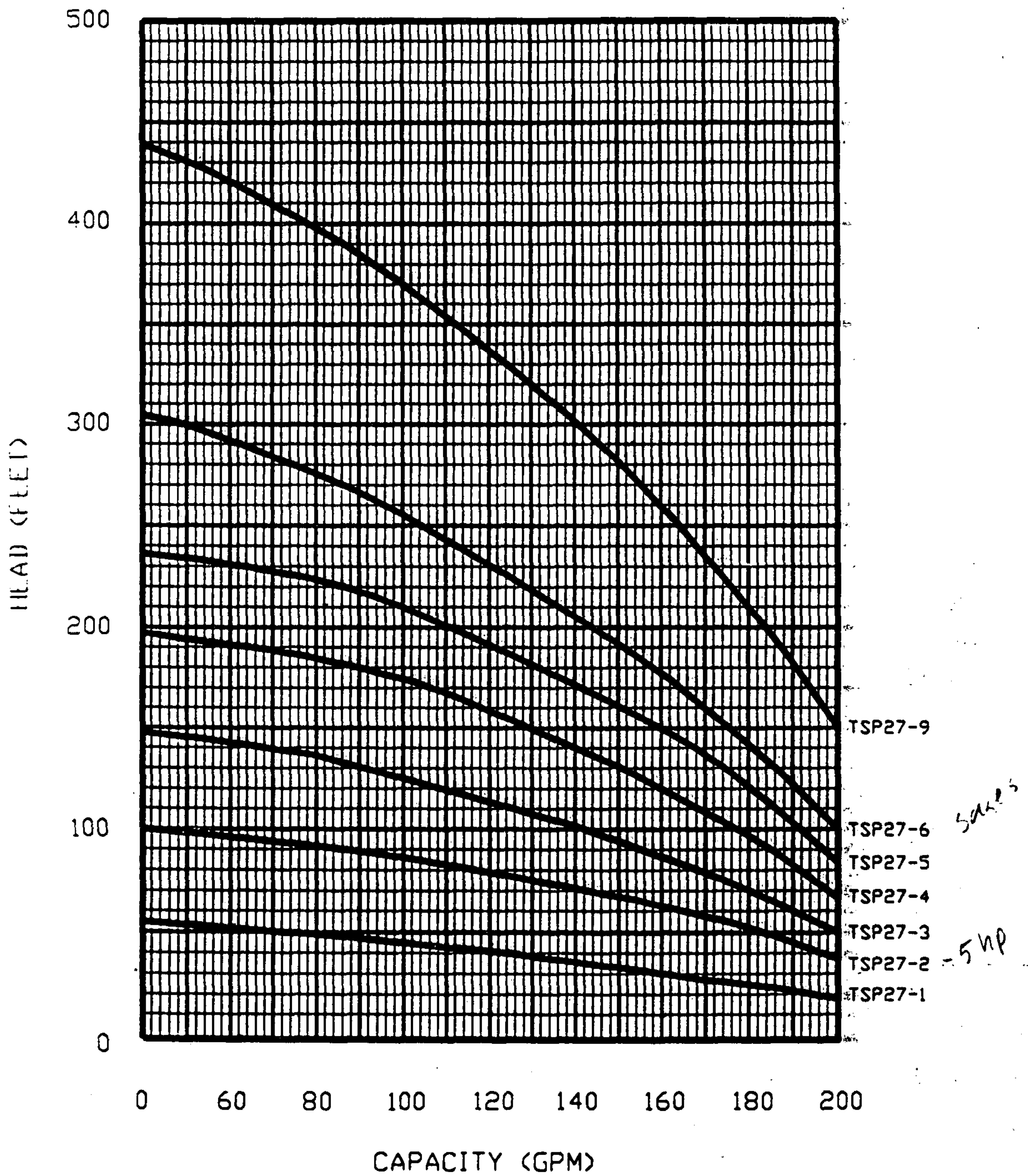


...ure 6.6 Nomograph for the Hazen–Williams formula with $C = 100$.

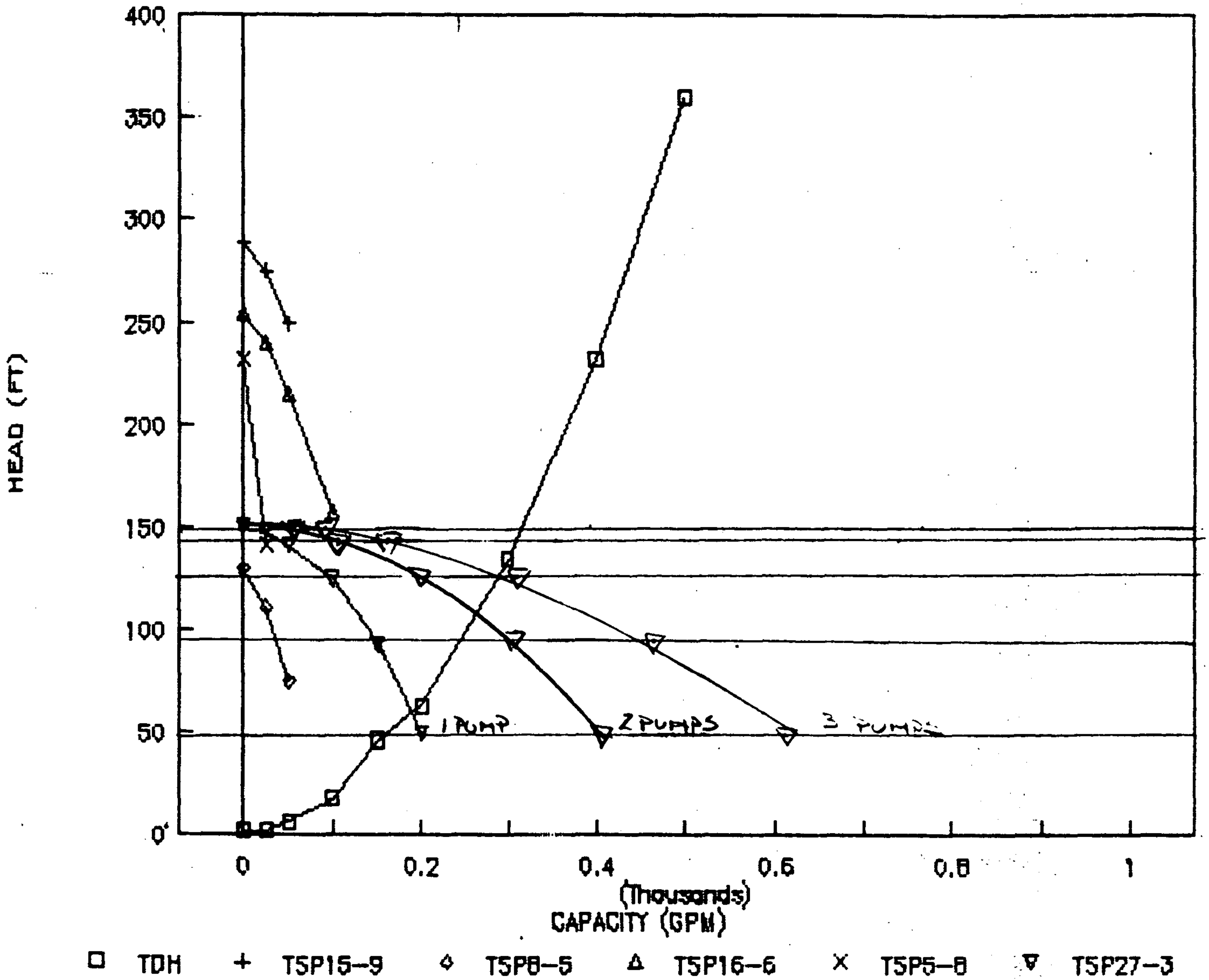
LEACHATE COLLECTION SYSTEM TOTAL DYNAMIC HEAD

FLOW (ft^3/s)	(GPM)	4 " PIPE TDH (ft)	6" PIPE TDH (ft)	TOTAL TDH (ft)
0.00	0	2.00	2.0	2.0
0.06	25	2.08	2.0	2.1
0.11	50	2.43	6.5	7.0
0.22	100	3.67	16.5	18.2
0.33	150	5.68	42.8	46.4
0.45	200	8.47	56.5	62.9
0.67	300	16.43	120.1	134.5
0.89	400	27.38	206.4	231.8
1.11	500	41.98	320.0	360.0
1.34	600	58.84	429.3	486.2
1.56	700	79.00	610.9	687.9
1.78	800		792.7	792.7
2.00	900		956.7	956.7
2.23	1000		1139.0	1139.0

TSP27 PERFORMANCE CURVES



LEACHATE COLLECTION SYSTEM



TSP 27-3 => THREE PUMPS IN PARALLEL

7. Cost Estimate Breakdown

CAMP DRESSER & McKEE CLIENT ______ JOB NO. ______ COMPUTED BY VAB
PROJECT ______ DATE CHECKED ______ DATE 5/22/92
DETAIL ______ CHECKED BY ______ PAGE NO. ______

Class I and Class III Landfill Construction Cost Estimate Breakdown

Class I Landfill Phase I (Approx. 60 acres)

Site Prep and Earthwork	$5,350,000
Liner System	$6,355,000
Leachate collection and pumping system	$236,000
Cover Material Stockpile	$3,660,000
Subtotal	$15,601,000

Class III Landfill (Phase I)

Site Prep + Earthwork	$1,831,000
Leachate Treatment System (includes effluent Irrigation + forcemain)	$1,538,000

Stormwater Ponds

Site Prep + Earthwork + Culverts	$820,000

Access Roadway

Site Prep + Earthwork	$4,975,000
Pavements	1,930,000
Culverts	472,000
	$7,377,000

CAMP DRESSER & McKEE CLIENT ______ JOB NO. ______ COMPUTED BY JAB
PROJECT ______ DATE CHECKED ______ DATE 5/22/92
DETAIL ______ CHECKED BY ______ PAGE NO. ______

Miscellaneous

Monitoring wells	$40,000
Scale Facilities	200,000
Fencing	376,000
Site prep	310,000
	926,000

RAW Total	$27,093,000
20% Contigency	$5,419,000
Total Estimate	$\boxed{\$32,512,000}$

CAMP DRESSER & McKEE CLIENT ______ JOB NO. ______ COMPUTED BY JAB
PROJECT ______ DATE CHECKED ______ DATE 6-4-92
DETAIL ______ CHECKED BY ______ PAGE NO. ______

Closure Cost Estimate

18" Clay Cap
750,200 Cu Yd @ 7.00 per Cu Yd = 5,251,000

18" Soil Cover
750,200 Cu Yd @ 4.00 per Cu Yd = 3,001,000

Letdown Structures
16,900 LF @ $100 per LF includes inlets = 1,690,000
50 Structures @ 20,000 each = 1,000,000

99 Landfill Gas Wells
Ave 80 ft each @ 16,000 = 1,584,000
see attached calc.

Gas Piping 16,000 LF @ 24.40 per LF = 390,400

Gas Flaring Units
2 units @ 200,000 each = 400,000

Final Vegitation @ $0.50/SY 750,200

Subtotal $14,066,600
20% Contingency 2,813,320

$16,880,000

CAMP DRESSER & McKEE CLIENT ______ JOB NO. ______ COMPUTED BY JAB
PROJECT ______ DATE CHECKED ______ DATE 6-4-92
DETAIL ______ CHECKED BY ______ PAGE NO. ______

Landfill Gas Wells

Pilot hole 80' @ $20/ft	= 1600
Auger hole 80' @ 100/ft	= 8,000
Screen	630
Gravel Pack 240 ft³ @ $5/ft³	= 1200
Caps & Collars	= 1250
Valve Box	= 750
Misc.	= 2500
	$\boxed{15,930 \text{ per}}$

TECHNICAL APPENDIX D
LINER SPECIFICATIONS

SECTION 02775

CLAY BASE LINER

PART 1 - GENERAL

1.01 SCOPE OF WORK

A. Furnish all labor, transportation, materials, supervision, equipment, and incidentals necessary to provide and install a clay base liner as shown on the Drawings comprised of a homogeneous clay soil such that the in-situ hydraulic conductivity meets the requirements of the specifications, and is free of cracks, fissures, clods, foreign objects and materials and is placed, compacted, and graded in accordance with the Drawings and as specified herein.

1.02 QUALIFICATIONS

A. The work shall be performed by a firm (or firms) that has experience in processing and installation of a clay liner. This firm(s) shall demonstrate proven experience by completing the bid questionnaire (Appendix A). This questionnaire shall be submitted with the bid documents.

B. The supplier shall show evidence of an adequate supply of material which is relatively homogeneous within a designated mine area which is properly permitted by the appropriate local agencies, water management districts, and state and federal agencies including as a minimum, conformance with the reclamation rules of the Florida Department of Natural Resources Ch. 16C-38 or 16C-39.

1.03 SUBMITTALS

A. Before furnishing and installing the clay liner, the Contractor shall submit the following to the Engineer for approval:

1. A signed affidavit from the supplier, with permit numbers and type, that the clay source is in full compliance with applicable standards and regulations.

2. A description of the installation procedure and a schedule to include clay installation and a list of installation equipment.

3. A Quality Control/Quality Assurance Plan for the clay cover installation.

4. A 10-pound sample of clay from the clay supplier.

PART 2 - PRODUCTS

2.01 MATERIALS

A. The liner material shall be provided by one supplier. The Contractor shall provide the material submittal as part of the bid. The submittal shall include the source of the material, applicable testing information with certification, written assurance that adequate quantities are available to complete the work and a list of similar projects with references, in which the material supplier has provided a clay liner which demonstrated satisfactory performance.

B. The material used for liner construction shall be a predominately natural calcium montmorillonite-sand blend, and shall be a stable, commercially available product.

C. The material used for liner construction shall have a minimum of 25 percent by dry weight of material passing the U.S. No. 200 standard sieve size (ASTM D-1140) and 100 percent passing the 1.5-inch sieve. The clay shall be free from roots and organic materials. It shall classify as a sandy clay in accordance with the Unified Soil Classification System and shall be free from roots, rocks, clods and organic materials. The material shall be delivered to the site at a moisture content as needed or slightly drier than that required to permit effective compaction as specified herein.

D. The material for liner construction shall have a plasticity index in the range of 20 to 80 percent.

E. The material used for liner construction shall be relatively uniform in character and, after processing, shall be capable of achieving an in-place saturated hydraulic conductivity not greater than 1×10^{-7} cm/sec.

PART 3 - EXECUTION

3.01 SUBBASE PREPARATION

A. Preparation of the subbase shall be as specified in Section 02220.

B. The Engineer will inspect and approve in writing the subbase before installation of the clay liner can proceed. It shall be the Contractor's responsibility to properly prepare and maintain the subbase in a smooth, uniform and compacted condition during installation of the liner.

C. If the Contractor damages the compacted subbase, he shall restore, recompact and have the Owner's Testing Service retest the subbase prior to installing the liner. All costs related to the retest and restoration of the subbase shall be paid for by the Contractor.

D. The Contractor shall provide adequate protection for the clayey materials delivered to the site from inclement weather conditions and any traffic that may occur near the storage area.

3.02 APPLICATION

A. The selected materials or products used for liner processing shall be uniformly mixed and homogenized to provide an in-place saturated hydraulic conductivity not greater than the value specified in paragraph 4.04 of this section.

B. Field test strip(s) shall be constructed onsite as directed in paragraph 4.03.

C. The placement moisture content shall be wet of the modified Proctor optimum moisture content (ASTM D-1557), and no greater than the standard Proctor optimum moisture content (ASTM D-698) plus 5 percent.

D. Clayey soil materials shall be disced, harrowed and kneaded as necessary to break down all clods and produce a uniform material that is free of clods. If the Engineer is not satisfied that all clods have been broken down, then the Contractor shall remove or rework the clayey material to the satisfaction of the Engineer. Clayey materials which have been contaminated with clusters of rock or gravel, sand lenses or other deleterious material shall be removed and replaced with uncontaminated clayey materials.

E. The clayey materials shall be uniformly compacted to no less than 95 percent of the modified Proctor dry density (ASTM D-1157) corresponding to the molding moisture content. This minimum density shall be uniformly obtained throughout the entire thickness of the liner. The liner shall be constructed in 6-inch compacted lifts to assure achievement of the specified compaction in the lower part of the liner. Each 6-inch compacted lift shall be constructed from approximately 9 inches of uncompacted material. The surface of the lower lift shall be scarified prior to placement of the upper lift. The direction of equipment movement during spreading and compaction of the upper lift shall be perpendicular to that carried out on the lower lift. To achieve the specified compaction the Contractor shall use a non-self-propelled compactor with ten inch (10") steel kneading feet, in spreading and kneading the materials. A smooth wheel compactor such as Caterpillar CS553 shall be used with sufficient number of passes to smooth the final surface of the liner. This type of compactor shall be used only for smoothing the surface. It shall not be used for achieving the specified compaction. Rubber tired rollers and vibratory rollers will not be allowed for use in compaction of the clayey materials.

The final minimum clay liner thickness shall be 18-inches with a hydraulic conductivity of no greater than 1×10^{-7} cm/sec.

F. WATER FOR COMPACTION

1. The Contractor shall provide water as required to guarantee constructability and protection of the in-place and stored clayey soil.

2. The water shall be of potable quality.

3. Prior to laying down the clayey soil the Contractor shall inspect the subgrade to ensure that is has been sufficiently wetted to prevent excessive absorption of moisture from the clayey soil.

4. Should the clayey soil be stockpiled for any length of time the contractor shall slope the stockpile and condition it with a track vehicle or light roller to prevent oversaturation.

5. Should the material become oversaturated, the Contractor shall spread and dry the material as needed to adjust the moisture to the proper level.

G. The compacted liner shall not be less than the specified thickness after any finish grading. Areas not meeting the thickness requirements shall be augmented with additional material if the finished grade meets the requirements for finished grades as indicated on the Drawings. The added material shall be reworked within the clay liner to ensure homogeneity and proper bonding. This shall be done by scarification of the surface prior to addition of the new material. As a minimum, the top 4 inches of liner shall be wetted, kneaded, compacted and reworked with the additional material to obtain the required thickness.

H. Work shall be limited to an area that can be completed in one working day. Completion shall be defined as follows:

1. Spreading and compaction of the clayey materials for the required thickness (in 6-inch lifts).

2. Placement of the synthetic liner over the compacted clay liner to protect it from desiccation, mechanical damage, etc. Final installation of the synthetic liner including field seaming, shall proceed following achievement of satisfactory results of field tests.

I. After installation of the clay liner, it shall not be allowed to desiccate. In order to prevent the formation of desiccation cracks, periodic wetting may be required until the liner is protected from the elements. Should desiccation cracks develop, the clay liner shall be rewetted, rehomogenized, and recompacted per these specifications to the depth of any such cracks.

J. During construction the Contractor shall make all necessary provisions to deal with inclement weather conditions. The Contractor shall be fully responsible for control of stormwater during installation of the liner system and for moisture control and protection of the clay liner.

K. The Contractor shall provide onsite supervision during all periods of installation. At no time shall any work be performed by the Contractor without The Contractor's supervisor on-site.

L. Installation of the clay liner shall be closely coordinated with all other work. All completed liner shall be covered with liner material daily. Liner not covered within 12 hours after installation will be inspected by the Engineer and the Contractor. If required by the Engineer this uncovered area will be removed or reworked as determined by the Engineer.

M. For any area not passing the specified tests, the Engineer will designate the area to be replaced or reworked. The Engineer will have the authority to determine if a failed area shall be removed.

N. Prior to commencing the work, the Contractor shall submit for approval his proposed method for liner construction and proposed method of construction for sealing adjacent to and around any appurtenant structures, e.g., outlet pipes, which penetrate the liner if different than shown on the drawings.

PART 4 - QUALITY ASSURANCE PROGRAM

4.01 TESTING LABORATORY

The Owner will retain an independent soil testing laboratory who has specific triaxial type permeability equipment to provide test results in a timely manner. The Contractor shall coordinate and schedule all tests as required by these plans and specifications.

4.02 WORKMANSHIP

The Contractor is responsible for correction of portions of the work which do not conform to the specified project requirements at no cost to the Owner.

4.03 TEST STRIP

A. A field test strip of a dimension of no less than 50' x 150' and 18 inches in thickness shall be constructed on site using the same equipment and processing procedures that will be used during full scale construction. If any elements of the construction including the materials, equipment or processing procedures are changed, this procedure must be repeated. The test strip shall be constructed over a prepared surface within the landfill area. If the initial test strip does not give the specified results, additional strips shall be performed until the specified results are obtained. Placement moisture content and compaction shall be recorded to determine the effort required to achieve the specified hydraulic conductivity. If test results indicate that the test strips meet the technical specifications provided herein, the test strips may be used as part of the liner provided that they are adequately protected from drying as specified in paragraph 3.02H and equipment damage during installation and provided that they are adequately tied into the surrounding liner. The tie-in shall consist of feathered overlapping edges.

B. Four triaxial type permeability tests will be performed on the test strip in accordance with ASTM D5084 as modified below. All tests will be taken within the middle 50 feet of the test strip at locations selected by the Engineer. These tests will be performed on 3-inch diameter (O.D.) undisturbed samples obtained from a Shelby tube or drive cylinder, trimmed if needed, encapsulated within a flexible latex membrane and mounted in triaxial type permeameters. The test specimen shall be consolidated under an effective stress of 5-10 psi, and permeated under a back pressure of 80-90 psi to achieve saturation. The hydraulic gradient used for hydraulic conductivity measurements shall be established in the laboratory which will provide for accelerated testing and final results within seven (7) days. The maximum hydraulic gradient shall not exceed 90. The hydraulic gradient shall be increased slowly in increments with careful observations of the test sample for consolidation, piping, etc. Test samples that are consolidated by more than 5 percent shall be voided. The inflow and outflow from the samples shall then be monitored and the coefficient of permeability calculated for each recorded flow increment. The tests will continue until steady state flow is achieved as evidenced by values of inflow and outflow that do not differ by more than 10 percent for the last 6 readings, and by stable values of the coefficient of permeability.

C. The liner thickness shall be determined from four locations from test strip.

D. A minimum of two random samples of the clayey soil material delivered to the site shall be tested for moisture content, particle size analysis (hydrometer) and Atterberg limits.

E. At least 16 samples of the materials from the test strip shall be recovered for fines content determination (percent passing a No. 200 sieve).

4.04 TESTING DURING CONSTRUCTION

A. Grain size analyses or percent fines determinations shall be conducted on samples from each 22,000 square feet of each lift of installed clay liner. Tests shall be conducted in accordance with ASTM D-1140 to determine the percentage by dry weight passing the number 200 standard sieve size. In addition, particle size analysis shall be conducted on samples from each 110,000 sq. feet of each lift of installed clay liner or as determined by the Engineer. Tests shall be in accordance with ASTM D-422, to determine the percent clay.

B. Atterberg limits (ASTM D-4318) shall also be performed on samples from each 22,000 square feet of each lift of installed clay liner.

C. Field density and moisture content shall be determined on each 22,000 square feet of each lift of installed clay liner. Nuclear and Speedy determinations shall not be used unless a product-specific correlation has been established at the job site with more accurate methods, i.e., drive cylinder (ASTM D-2937), sand cone (ASTM D-1558) and oven drying (ASTM D-2216) methods.

D. Depth measurements to determine the thickness of the liner layer shall be conducted every 11,000 square feet of installed clay liner.

E. Hydraulic conductivity testing shall be performed for the compacted clayey soil at a minimum frequency of one test per 22,000 square feet of completed clay liner. The Engineer shall have the authority to request additional permeability tests in areas which, in his judgement, may be suspect or deficient. Hydraulic conductivity tests shall be conducted in accordance with ASTM D-5084 except as modified in paragraph 4.03B. The maximum allowable hydraulic conductivity shall be 1×10^{-7} cm/sec. The hydraulic gradient used for hydraulic conductivity measurements shall be approved by the Engineer. Pure bentonite powder shall be used to backfill the holes in the clay liner where hydraulic conductivity samples were obtained.

F. Any sample or area tested shall be rejected, removed and replaced if it does not meet the requirements of the technical specifications. Reconstructed areas shall have feathered, overlapping edges that tie into adjacent liner areas.

G. The testing frequency shall be doubled during construction of the first five (5) acres of the clay liner or until construction is judged by the Engineer to be proceeding satisfactorily.

4.05 POST CONSTRUCTION

A. The Contractor shall obtain a maintenance bond for the period of two (2) years from the date of final acceptance by the Owner

END OF SECTION

APPENDIX A

APPENDIX A

BID INFORMATION QUESTIONNAIRE

CLAY LINER SUPPLIER

(Please submit with necessary attachments along with your bid documents)

(Please Type or Print)

1. CORPORATE INFORMATION

COMPANY NAME: ______________________________

ADDRESS: ______________________________

TELEPHONE: (   )   - __________

MAIN OFFICERS:

PRESIDENT ______________________________

SALES MANAGER ______________________________

QUALITY CONTROL MANAGER ______________________________

BRIEF CORPORATE HISTORY ______________________________

____

____

2. PROPOSED BORROW PIT

LOCATION: ______________________________

BRIEF DESCRIPTION OF PIT SIZE, EQUIPMENT AND PERSONNEL: __________

____

____

____

QUANTITY AVAILABLE: ______________________________

SC3B/4

BID INFORMATION QUESTIONNAIRE

CLAY LINER INSTALLER

(Please submit with necessary attachments along with your bid documents)

(Please Type or Print)

1. CORPORATE INFORMATION

COMPANY NAME: ______________________________

ADDRESS: ______________________________

TELEPHONE: (   )   - ________

MAIN OFFICERS:

PRESIDENT ______________________________

SALES MANAGER ______________________________

QUALITY CONTROL MANAGER ______________________________

BRIEF CORPORATE HISTORY ______________________________

____

____

2. COMPANY CAPABILITIES

BRIEF DESCRIPTION OF COMPANY SIZE, EQUIPMENT AND PERSONNEL: ________

____

____

____

____

____

3. LIST OF COMPLETED CLAY LINER OR CAP FACILITIES

( )JECT NAME	1. ____	2. ____
3. ____		
PURPOSE OF FACILITY	____	____
____		
LOCATION OF FACILITY	____	____
____		
OWNER	____	____
____		
DESIGNER	____	____
____		
SOURCE OF CLAY (LOCATION)	____	____
____		
PROJECT MANAGER	____	____
____		
CLAY THICKNESS (INCHES)	____	____
____		
ERED AREA (FT^2)	____	____
____		

PROJECT NAME	4. ____	5. ____
PURPOSE OF FACILITY	____	____
LOCATION OF FACILITY	____	____
OWNER	____	____
DESIGNER	____	____
SOURCE OF CLAY (LOCATION)	____	____
PROJECT MANAGER	____	____
CLAY THICKNESS (INCHES)	____	____
COVERED AREA (FT^2)	____	____

END OF SECTION

SECTION 02776
HIGH DENSITY POLYETHYLENE (HDPE)
60 MIL GEOMEMBRANE LINER

PART 1 - GENERAL

1.01 SCOPE OF WORK

Furnish all labor, materials, equipment and incidentals required to install High Density Polyethylene (HDPE) geomembrane as shown on the Drawings and as specified herein. The work shall include placement, seaming and anchoring of the geomembrane, field quality assurance, and all other work as shown on the Drawings and as specified herein.

1.02 RELATED WORK

A. Site preparation is included in Section 02100.

B. Structural excavation, fill and grading is included in Section 02221.

C. Clay liner is included in Section 02775.

1.03 DEFINITIONS AND RESPONSIBILITIES

A. Contractor

The Contractor is the firm or corporation with whom the Owner has entered into the agreement to construct the Project. The Contractor is responsible for all submittals by the Manufacturer and the Installer as required by the Specifications. The Contractor is also responsible for scheduling and coordinating of the required work with the Manufacturer and the Installer to complete the Project.

B. Manufacturer

The Manufacturer is the firm or corporation responsible for production of the geomembrane material to be used in the Project. The Manufacturer is responsible for any damage to the geomembrane until the material is accepted by the Owner or his field representative upon delivery. The Manufacturer shall produce a consistent product meeting the Project Specifications, and shall provide quality control documentation for the product specified herein.

C. Installer

The Installer is the firm responsible for installation of the geomembrane. The Installer shall be the Manufacturer or an approved Installer trained and certified by the Manufacturer to install the Manufacturer's geomembrane. The Installer shall provide a Field Site Manager responsible for the installation field crew. The Installer shall be responsible for field handling, storing, deploying, seaming, and all other aspects of the geomembrane installation. The Contractor is ultimately responsible for installation of the geomembrane and final acceptance of the project by the Owner.

1.04 QUALIFICATIONS

A. Manufacturer

The Manufacturer of the lining material described hereunder shall have previously demonstrated his ability to produce this geomembrane by having at least five years continuous experience in the manufacture of HDPE geomembrane and successfully manufactured a minimum of 50 million square feet of similar liner material for hydraulic lining installations. The manufacturer must be certified and registered by National Sanitation Foundation Standard 54 (NSF) as meeting all the requirements for manufacturing HDPE geomembrane.

B. Installer

The Installer shall be the Manufacturer or an approved Installer trained and certified by the Manufacturer to install the Manufacturer's geomembrane. Installation shall be performed under the constant direction of a single Field Site Manager (provided by the Manufacturer) who shall remain on site and be in responsible charge, throughout the geomembrane installation, for geomembrane layout, seaming, patching, testing, repairs, and all other activities required by the Installer. The Field Site Manager shall have installed or supervised the installation and seaming of a minimum of two million square feet of HDPE geomembrane liner.

1.05 SUBMITTALS

A. Manufacturer

1. Submittals with Bid Documents

The Contractor shall submit the following information about the Manufacturer with the Bid Documents.

a. Corporate Background

b. Manufacturing Capabilities:

(1) Information on factory size, equipment, personnel, number of shifts per day, and production capacity per shift.

(2) List of material properties and samples of liner with attached certified test results.

(3) Manufacturer's quality control program and manual including description of laboratory facilities.

(4) A list of ten completed facilities totaling a minimum of ten million square feet ft^2, for which the Manufacturer has manufactured an HDPE geomembrane. The following information shall be provided for each facility.

- o Name and purpose of facility, its location and date of installation;
- o Name of owner, project manager, design engineer and installer;
- o Geomembrane thickness and surface area; and
- o Information on performance of the facility.

c. Manufacturer's NSF certification.

d. The origin of the resin to be used in the manufacturing of geomembrane including the suppliers name and production plant, as well as brand name and number.

e. A fingerprint of the Manufacturer's geomembrane properties as listed in Appendix A, Table A1. The purpose of these tests is to identify the Manufacturer's geomembrane product. The results of these tests shall be submitted to the Owner or his representative for approval of the product. Once the product is approved, all HDPE geomembrane to be supplied for the Project shall be manufactured using the same resin type identified through fingerprinting tests.

2. Submittals After Award of Contract

Within 30 calendar days following the effective date of the Agreement, the Contractor shall submit the following information certified by the Manufacturer.

a. Certification that all resin used in the manufacture of HDPE membrane for this Project meets the approved fingerprinting protocol.

b. Copy of quality control certificates in conformance with Sections 2.01 and 2.02 of these Specifications.

c. Certification that the HDPE membrane and extrudate produced for this project have the same properties.

B. INSTALLER

1. Submittals with Bid Documents

The Contractor shall submit the following information about the Installer with the Bid Documents:

a. Background Information

b. Installation Capabilities:

(1) Information on equipment and personnel.

(2) Anticipated average daily production.

(3) A minimum of three field seam samples and a list of minimum values for seam properties.

c. A minimum of five completed facilities totaling 2,000,000 ft^2 for which the installer has installed HDPE geomembrane. The following information shall be provided for each facility:

(1) Name and purpose of facility, its location and date of installation.

(2) Name of owner, design engineer, manufacturer. Name and telephone number of contact person at the facility who can discuss the project.

(3) Thickness of geomembrane and surface area of the installed geomembrane.

(4) Type of seaming, patching and tacking equipment.

(5) A copy of the manufacturer's certification or approval letter.

(6) Resume of the qualifications of the Installation Supervisor and Master Seamer to be assigned to this project.

2. Submittals Prior to Commencement of Installation.

The Contractor shall submit the following information to the Owner, or his representative for approval.

a. Shop Drawings, including:

(1) Proposed panel layout showing the installation layout identifying field seams as well as any variance or additional details which deviate from the engineering drawings.

(2) Details of seaming the HDPE geomembrane, anchoring, connections, penetrations, and other construction details.

b. Installation schedule.

c. A manual that specifically defines the quality control and quality assurance program during installation. The manual shall include daily procedures, welding techniques, field testing procedures, lab testing procedures, specific steps that are to be taken in the event of a failure or defect, personnel requirements, levels of authority, and all other information necessary to ensure a high-quality geomembrane installation.

d. Resume of the Supervisor to be assigned to the Project.

e. Resume of the Master Seamer to be assigned to the Project.

f. A list of personnel performing field seaming operations along with pertinent experience information.

1.06 REFERENCE STANDARDS

A. American Society for Testing and Materials (ASTM)

ASTM D638-89 - Standard Test Methods for Tensile Properties of Plastics

ASTM D746-87 - Standard Test Methods for Brittelnese Temperature of Plastics and Elastomers by Impact

ASTM D751-89 - Standard Test Method for Testing Coated Fabrics

ASTM D792-86 - Standard Test Method for Specific Gravity and Density of Plastics by Displacement

ASTM D1004-66(88) - Standard Test Method for Tear Resistance of Plastic Film and Sheeting

ASTM D1204-84 - Standard Test Method for Linear Dimensional Changes of Nonrigid Thermoplastic Sheeting of Film at Elevated Temperature

ASTM D1603-76 (88) - Standard Test Method for Carbon Black in Olefin Plastics

ASTM D1238-90b - Standard Test Method for Flow Rate of Thermoplastics by Extrusion Plastometer

ASTM D1505-85 (90) - Standard Test Method for Density of Plastics by the Density - Gradient Technique

ASTM D1693-70 (88) - Standard Test Method for Environmental Stress - Cracking of Ethylene Plastics

ASTM D3015-72 (85) - Recommended Practice for Microscopical Examination of Pigment Dispersion in Plastic Compounds

ASTM D4437-84(88) - Standard Practice for Determining the Integrity of Field Seams Used in Joining Flexible Polymeric Sheet Geomembranes

ASTM D3895 - Standard Test Method for Oxidative Induction Time

B. Federal Test Method Standards

FTMS 101 - Test Method For Puncture Resistance for Plastics

C. National Sanitation Foundation

NSF 54 ANNEX - Modifications to ASTM Standard Test Methods Representing Industries Standard (Noted in Appendix B) in joining flexible polymeric sheet geomembranes.

1.07 QUALITY CONTROL AND QUALITY ASSURANCE

In addition to the Manufacturer and Installer requirements for qualifications and certification specified in Section 1.05 - Submittals, the quality assurance consists of conformance testing of the material delivered to the site and field quality control during installation.

A. Conformance Testing

Conformance testing requirements are discussed in Section 2.03 of the Specifications. The purpose of conformance testing is to assure that the supplied material conforms to the Specifications and to the Manufacturer's quality control certificates.

B. Field Quality Control

Field quality control requirements are discussed in Section 3.06 of the specifications. The purpose of field quality control procedures is to assure that the geomembrane has been installed in accordance with the specifications and Manufacturer's recommendations.

C. Quality Control Plan

The forms for geomembrane quality control documentation, included in Appendix C, shall be used for field installation documentation. Alternative forms may be used for documentation as approved by the Owner, or his field representative.

D. Geomembrane Quality Control Documentation

1. Pre-Installation Conference

Prior to commencing work, a pre-installation conference shall be held and the following Project personnel shall be identified by name and recorded in the Project files:

o	Owner's Representative	(OR)
o	Field Site Manager	(FSM)
o	Engineer's Field Representative	(EFR)
o	Installation Supervisor	(IS)
o	Master Seamer	(MS)

o	Contractor's Representative	(CR)
o	Quality Assurance Laboratory	(QAL)
o	Quality Assurance Technician	(QAT)

Two duplicate Project files shall be maintained. One shall be maintained by the QAT and the other shall be maintained by the FSM. At the end of each work week the files shall be updated and checked to assure that copies of all pertinent Project information are included in each file. The QAT shall submit daily copies of the documentations to the EFR.

Blank copies of the following ten Project forms shall be available on site throughout the duration of the project:

Form No.	Title
1	Startup
2	Liner Pre-Delivery
3	Liner Delivery
4	Daily Checklist
5	Geomembrane Panel Placement
6	Onsite Geomembrane Welding Report
7	Damage and Failure Report
8	In-Situ Weld Testing Report
9	Post Installation Checklist
10	Daily Field Log

E. Record Drawings

The Contractor shall furnish Record Drawings and a copy of complete documentation for final installation of the geomembrane.

1.08 MATERIAL DELIVERY AND STORAGE

The geomembrane rolls shall be packaged and shipped by appropriate means to prevent damage of the geomembrane rolls. Off-loading and storage of the geomembrane is the responsibility of the Installer. The Installer shall be responsible for replacing any damaged or unacceptable material at no cost to the Owner.

No off-loading shall be performed unless the Engineer's Field Representative (EFR) is present. Damage during off-loading shall be documented by the EFR and Quality

Assurance Technician (QAT). All damaged rolls must be separated from the undamaged rolls until the proper disposition of that material has been determined by the EFR.

The geomembrane rolls shall be stored so as to be protected from puncture, dirt, grease, water, moisture, mud, mechanical abrasions and excessive heat that may damage the geomembrane material. The rolls shall be stored on a prepared surface (not wooden pallets) and shall not be stacked more than two rolls high.

1.09 MATERIAL WARRANTY

The HDPE geomembrane Manufacturer shall warrant the membrane, on a prorated basis, against manufacturing defects and material degradation under outdoor exposure for a period of twenty years from the date of installation. The Manufacturer shall replace at no expense any material which fails from the above causes within the warranty period. The manufacturer shall furnish a written warranty covering the requirements of this paragraph.

1.10 GUARANTEE

The Contractor shall guarantee the HDPE geomembrane against defects in installation and workmanship for the period of two years commencing with the date of Final Acceptance. The guarantee shall include the services of qualified service technicians and all materials required for the repairs at no expense to the Owner.

PART 2 - PRODUCTS

2.01 MATERIALS

A. General

1. The geomembrane shall be manufactured of new, prime first-quality products designed and manufactured specifically for the purpose of liquid containment in hydraulic structures, and chemically resistant to leachate.

2. The geomembrane material shall be so produced as to be free of holes, blisters, undispersed raw materials, or any sign of contamination by foreign matter.

3. The sheets shall have the NSF label of approval, and shall be manufactured in a minimum 15.0 ft seamless width. Labels on the roll shall identify the thickness, length, width, and manufacturer's lot number.

B. Properties

The geomembrane rolls shall meet the minimum properties listed in Appendix B, Table B1.

C. Other Materials

1. Extrudate welding rods shall be of the same compound as the geomembrane and supplied by the Manufacturer and shall be delivered in the original sealed containers. Each container shall have a label bearing the brand name, Manufacturer's lot number, and complete directions as to proper storage.

2. Boots and shrouds for pipe penetrations shall fit snugly around the pipe. Prefabricated material shall be designed to fit site specific conditions for the intended slope and size of pipe.

2.02 QUALITY CONTROL DOCUMENTATION

Prior to installation commencement of any geomembrane material, the Contractor shall provide to the Owner the following information certified by the Manufacturer for the delivered geomembrane.

1. Origin, identification, and production of the resin (supplier's name, brand name and production plant).

2. Copies of quality control certificates issued by the resin supplier.

3. Manufacturer's certification verifying that the quality of the resin used to manufacture the geomembrane meets the fingerprint properties as shown in Appendix A, Table A1.

4. Each roll delivered to the Project site shall have the following identification information;

    - Manufacturer's name
    - Product identification
    - Thickness
    - Roll number
    - Roll dimensions

5. Quality control certificates, signed by the Manufacturer's quality assurance manager. Each certificate shall have roll identification number, sampling procedures, frequency and test results. At a minimum the following test results shall be provided every 50,000 square feet of manufactured liner in accordance with test requirements specified in Appendix B.

    - Thickness
    - Density
    - Tensile properties
    - Tear resistance
    - Carbon black control
    - Carbon black dispersion

2.03 CONFORMANCE TESTING

Conformance testing shall be performed by an independent Quality Assurance Laboratory (QAL) approved by the owner. A Quality Assurance Technician (QAT) from the QAL shall obtain the samples from the roll, mark the machine direction, and identification number. The following conformance tests shall be conducted at the laboratory.

- Thickness
- Density
- Tensile properties
- Tear resistance
- Carbon black control
- Carbon black dispersion

These conformance tests shall be performed in accordance with Appendix B of the Specifications.

All conformance test results shall be reviewed by the EFR and accepted or rejected, prior to the deployment of the geomembrane. All test result shall meet, or exceed, the property values listed in Appendix B. In case of failing test results, the Manufacturer may request that another sample be retested by the QAL with Manufacturer's technical representative present during the testing procedures. This retesting shall be paid for by the Manufacturer. The Manufacturer may also have the sample retested at two different laboratories approved by the Owner. If both laboratories report passing results, the material shall be accepted. If both laboratories do not report passing results, all geomembrane material from the lot representing the failing sample will be considered out of specification and rejected. The manufacturer reserves the right to obtain additional samples from rolls immediately before and after the failing roll or as directed by the QAT and test it by the QAL at his own expense. If these rolls pass, then only the failing roll will be rejected. If they fail, then the entire lot will be rejected.

PART 3 - EXECUTION

3.01 SUBGRADE PREPARATION

A. Preparation of the subgrade shall be as specified in Section 02221 and 02775.

B. The surface of the subgrade shall be smooth, uniform, free from sudden changes in grade (such as vehicular ruts), rocks, stones, debris, and deleterious materials. During actual placing and seaming of the geomembrane, the subgrade shall be kept free of all standing water. If the subgrade below the geomembrane becomes wet and unstable, it shall be dried and recompacted.

C. Before the geomembrane installation begins, the Contractor and the Installer shall verify and sign off on the following:

1. Lines and grades are in conformance with the design drawings and Specifications.

2. The surface area to be lined has been rolled and compacted, free of irregularities, and abrupt changes in grade.

3.02 ANCHOR TRENCH

A. The anchor trench shall be constructed as shown on the design drawings and specified herein.

B. Slightly rounded corners shall be provided in the trench to avoid sharp bends in the geomembrane.

C. The anchor trench shall be adequately drained to prevent water ponding and softening to adjacent soils. The anchor trench shall be backfilled with local fill material and compacted to 90 percent standard proctor density.

D. If the anchor trench is located in a clay susceptible to desiccation, the amount of trench open at any time shall be limited to one day of geomembrane installation capacity.

3.03 GEOMEMBRANE DEPLOYMENT

A. Weather Conditions

Geomembrane deployment shall not proceed at an ambient temperature below 40°F on above 104°F unless otherwise authorized, in writing, by the Engineer or his field representative. Geomembrane deployment shall not be performed during precipitation, excessive moisture, in an area of ponded water, or excessive winds.

B. Method of Deployment

1. Each panel of the geomembrane shall be rolled out and installed in accordance with the approved shop drawings prepared by the Contractor. The layout shall be designed to keep field joining of the HDPE geomembrane to a minimum and consistent with proper methods of HDPE geomembrane installation.

2. Geomembrane rolls shall be deployed using proper spreader and rolling bars with cloth slings. If a sheet must be redeployed a distance greater than its width, a slip sheet shall be used.

3. The Quality Assurance Technician (QAT) shall inspect each panel, after placement and prior to seaming, for damage and/or defects. Defective or damaged panels shall be replaced or repaired, as approved by the EFR.

4. The Installer shall avoid dragging the geomembrane sheets on rough soil subgrades.

5. All geomembrane shall be anchored as shown on the Drawings and consistent with Manufacturer's recommendations.

6. Personnel working on the geomembrane shall not smoke, wear damaging shoes or involve themselves in any activity that may damage the geomembrane.

7. All edges of the geomembrane shall be properly weighted to avoid uplift due to wind.

8. Vehicular traffic across the geomembrane shall not be allowed.

9. All damage shall be recorded and located in the Record Drawings.

10. When tying into existing geomembrane, all excavation of previously installed liner shall be performed by hand to prevent damage.

11. The geomembrane shall be kept free of debris, unnecessary tools and materials. In general, the geomembrane area shall remain neat in appearance.

C. Liner Boots

1. HDPE boots or shrouds shall be furnished and installed where indicated on the Drawings. The boots shall be of the same material as the geomembrane.

2. All boots or shrouds shall be prefabricated and tested for leaks and certified by the manufacturer.

3. The geomembrane end of the boots shall terminate in a skirt section suitable for welding to the geomembrane. The overlap between the boot and the geomembrane shall be approximately 18 inches. The boot shall be welded to the geomembrane as previously specified herein.

4. Boots and shrouds shall fit snugly around the pipe. Prefabricated material shall be designed to fit site specific condition, for the intended slope and size of pipe.

5. Neoprene sponge rubber gasket shall be used between the boot or shroud and the pipe with a stainless steel clamp. An HDPE sacrificial sheet shall be used between the boot or shroud and the clamp for protection.

6. For pipes larger than four inches in diameter, a second clamp shall be used. The fastener of the second clamp shall be located on the opposite side of the pipe from the first clamp, to compensate for uneven pressure and elongation.

3.04 FIELD SEAMS

1. Individual panels of geomembrane shall be laid out and overlapped by a minimum of three to five inches prior to welding. The area to be welded shall be cleaned and prepared in accordance with the quality control welding procedures.

2. Single or double track hot wedge fusion welder shall be used as a primary welding method.

3. Extrusion welder shall be used for cross seam tees, patches and repairs, and penetration boots.

4. The welding equipment used shall be capable of continuously monitoring and controlling the temperatures in the zone of contract where the machine is actually fusing the geomembrane material so as to ensure that changes in environmental conditions will not affect the integrity of the weld.

5. Any fillet weld that is terminated long enough (could be less than three minutes - depending on temperature) for the extrudate to "set up" (harden) is considered a start-stop weld and must be ground prior to restarting. (ie. starting approximately two inches from the termination point. The weld is feathered to a point.

6. No "fish mouths" will be allowed within the seam area. Where "fish mouths" occur, the material shall be cut, overlapped, and a patch fusion weld shall be applied and vacuum tested. All welds upon completion of the work shall be tightly bonded. Any geomembrane area showing injury due to excessive scuffing, puncture, or distress from any cause shall be replaced or repaired with an additional piece of gemomebrane. The number of patches per 100-foot length shall not exceed five (5). If more than (5) patches per 100-foot length are necessary, then the entire 100-foot length of seam shall be cap stripped. Further welding will cease at this time and the Engineer shall be notified.

7. All seams shall have a seam number that corresponds with the panel layout numbers. The numbering system shall be used in the development of the Record Drawings. Seam numbers shall be derived from the combination of the two panel numbers that are to be welded together.

8. All fusion welded "T" seams (i.e., the result of the geomembrane panels placed perpendicular to each other) shall be double welded where possible. The extrusion process shall be used for the second weld.

9. All extrudate shall be free of dirt, moisture and protected from damage.

10. If an extrusion welder is stopped for longer than one minute, it shall be purged to remove heat-degraded extrudate. All purged extrudate shall be placed on a sacrificial sheet and disposed of.

11. All seams constructed on sloped surfaces shall be vertical seams.

12. All vertical panels placed on sloped surfaces shall extend five (5) feet inward from the toe of slope.

13. To prevent moisture buildup during fusion welding, it may be necessary to place a movable protective layer of plastic directly below each overlap of geomembrane that is to be seamed.

14. If required, a firm substrate shall be provided by using a flat board, slip sheet or similar hard surface directly under the seam overlap to achieve proper support.

15. All seams shall extend a minimum of twelve (12) inches into the anchor trench.

16. All factory seams, field seams, and repair welds shall meet seam strength requirements specified in Appendix B, Table B2.

3.05 SEAMING WEATHER CONDITIONS

A. Normal Weather Conditions

The normal required weather conditions for seaming are:

1. Ambient temperature higher than 40°F and lower than 104°F.

2. No precipitation or other excessive moisture, such as fog or dew.

3. No excessive winds.

These weather conditions shall be fulfilled during seaming process.

B. Cold Weather Conditions

If the ambient temperature is below 40°F, the following conditions shall be met to ensure quality seaming process:

1. Preheating the surface of the geomembrane to achieve normal temperature range.

2. Preheating maybe waived by the EFR if the installer demonstrates that satisfaction welds of equivalent quality may be obtained without preheating at the expected temperature of installation.

3. Preheating devices shall be approved by the Manufacturer.

4. Care shall be taken to assure that surface temperatures are not lowered below the minimum required surface temperature for welding due to winds.

5. Additional destructive tests samples shall be taken at the discretion of the EFR.

6. Test seams, as described in Section 3.06-A, shall be performed under the same ambient temperature conditions as the actual seams.

C. Warm Weather Conditions

1. If the ambient temperature is above 104°F, no seaming of geomembrane shall be permitted unless the installer can demonstrate to the satisfaction of the Engineer or his field representative that geomembrane seam quality is not adversely impacted.

2. Test seams shall be performed under the same ambient temperature conditions as the actual seams.

3. Additional destructive tests shall be taken at the discretion of the EFR.

3.06 FIELD QUALITY CONTROL

A. Start-up Testing

1. A test weld three feet long from each welding machine shall be run upon the beginning of each shift and every four hours thereafter, under the same conditions as exist for the geomembrane welding. The test weld shall be marked with date, ambient temperature, and welding machine number. A tensiometer shall be required to be on-site before and during geomembrane installation for the purpose of testing samples. Specimens of weld 1-inch wide shall be cut from the test weld and tested on site for shear and peel strength in accordance with Appendix B, Table B-2. No welder may start work until the sample weld has been approved by the EFR.

2. Test seams shall be performed under the same conditions as the actual seams, and shall be at least three feet long, one foot wide after seaming. Test seam for welding shall be cut out of the geomembrane rolls.

B. Nondestructive Seam Testing

The Installer shall perform nondestructive test on all field seams over their full length. The purpose of this test is to assure continuity and integrity of the seams. Vacuum and air pressure tests shall be used for nondestructive testing. The vacuum test shall be used for extrusion welds and single track hot wedge welds. The air pressure test shall bs used for double track hot wedge welds.

1. Vacuum Testing

Equipment for testing single wedge fusion seams and extrusion seams shall be comprised of the following:

a. A vacuum box assembly consisting of a rigid housing, a transparent viewing window, a soft rubber gasket attached to the bottom, port hole or valve assembly and a vacuum gauge.

b. A vacuum tank and pump assembly equipped with a pressure controller and pipe connections.

c. A rubber pressure/vacuum hose with fittings and connections.

d. A plastic bucket and wide paint brush.

e. A soapy solution.

The following procedures shall be followed by the Installer.

a. Excess sheet overlap shall be trimmed away.

b. Clean the window, gasket surfaces and cheek for leaks.

c. Energize the vacuum pump and reduce the tank pressure to approximately 5 psi.

d. Wet a strip of geomembrane approximately 12 inches by 48 inches (length of box) with the soapy solution.

e. Place the box over the wetted area and compress.

f. Close the bleed valve and open the vacuum valve.

g. Ensure that a leak-tight seal is created.

h. For a minimum period of ten seconds, examine the geomembrane through the viewing window for the presence of soap bubbles.

i. If no bubbles appear after 10 seconds, close the vacuum valve and open the bleed valve, move the box over the next adjoining area with a minimum of three inches overlap and repeat the process.

j. All areas where soap bubbles appear shall be marked and repaired in accordance with Section 3.07-F and then retested.

If the seam cannot be tested prior to final installation, the seaming operations shall be observed by the QAT and the FER for uniformity and completeness.

2. Air Pressure Testing (for double track fusion seams only).

The following procedures are applicable to those processes which produce a double seam with an enclosed space.

Equipment for testing double wedge fusion seams shall be comprised of the following:

a. An air pump equipped with pressure gauge capable of generating and sustaining a pressure between 25 and 30 psi and mounted on a cushion to protect the geomembrane.

b. A manometer equipped with a sharp hollow needle, or other approved pressure feed device.

The following procedures shall be followed by the installer.

a. Seal both ends of the seam to be tested.

b. Insert needle or other approved pressure feed device into the tunnel created by the double wedge fusion weld.

c. Energize the air pump to a pressure between 25 and 30 psi, close valve, and sustain pressure for at least five minutes.

d. If loss of pressure exceeds 4 psi, or pressure does not stabilize, locate faulty area, repair in accordance with Section 3.07-F, and retest.

e. Remove needle or other approved pressure feed device and seal.

Destructive seam testing shall be performed in accordance with Section 3.07 of this specification.

3.07 DESTRUCTIVE SEAM TESTING

The purpose of the destructive testing is to evaluate seam strength properties. A minimum of one test sample shall be obtained per 500 feet of performed seam length. The location of samples shall be determined by the EFR. Selection of such locations may be prompted by suspicion of overheating, contamination, or other potential cause that may adversely impact the welds. Field samples shall be taken by the Installer. Testing of field samples shall be performed by the QAT as described herein.

A. Sampling Procedures

1. Samples shall be cut by the Installer at locations chosen by the EFR as the seaming progresses.

2. The seams shall not be covered by another material before they have been tested and accepted by EFR.

3. Upon obtaining each ample, the QAT shall assign a number to the sample and mark it accordingly.

4. Record sample location on layout drawing.

5. Record purpose of the sample, statistical routine or suspicious weld area.

6. Holes in the geomembrane resulting from destructive seam testing shall be immediately repaired in accordance with Section 3.07-F, of these Specifications.

B. Size and Disposition of Samples

Two types of samples shall be taken by the Installer. First, two samples for field testing shall be taken. Each of these samples shall be cut with a 1 inch wide die, with the seam centered parallel to the width. The distance between these two samples shall be 36 inches. If both samples pass the field test described in Section 3.07-C, a sample for laboratory testing shall be taken.

The sample for laboratory testing shall be located between the samples for field testing. The sample for laboratory testing shall be 12 inches wide by 36 inches long with the seam centered lengthwise. The sample shall be cut into three parts and distributed as follows:

1. One portion to the Installer for optional laboratory testing, 12 in x 12 in.

2. One portion for geosynthetic laboratory quality Assurance testing, 12 in x 12 in.

3. One portion to the EFR for archive storage, 12 in x 12 in.

C. Field Testing

The following shall be performed, at the presence of the EFR:

1. The QAT shall cut ten 1-inch wide replicate specimens from the sample to be tested for shear and peel strength, in accordance with the criteria set in Appendix B, Table B-2.

2. The QAT shall test five specimens for shear seam strength and five for peel strength. Four out of the five replicate test specimens shall pass for the seam to be acceptable.

3. Any specimen that fails through the weld or by fusion at the weld sheet interface is a non-FTB (Film Tearing Bond) break and shall be considered a failure. Refer to locus-of-brake diagrams in Appendix B, Figures B1 through B5.

D. Quality Assurance Laboratory Test

1. The Installer shall package and ship destructive test samples to the independent QAL approved by the owner.

2. Laboratory test shall include shear and peel strength tests. The minimum acceptable values obtained in these tests shall be in accordance with Appendix B, Table B-2.

3. At least five specimen shall be tested each for shear and peel strength. A passing test shall meet the minimum required values in a least four of the five specimens tested for each method.

4. The QAL shall provide verbal test results to the EFR no more than 24 hours after they receive the samples. The EFR shall review the laboratory results as soon as they become available.

E. Procedures for Destructive Test Failure

The following procedures shall apply whenever a sample fails a destructive test, whether that test is conducted in the field or by the QAL. The Installer has two options.

1. The Installer can repair the seam between any two passing test locations.

2. The Installer can retrace the welding path to an intermediate location 10 feet from the location of the failed test and take a sample for an additional field test. If this test passes, then the seam shall be repaired between that location and the original failed location. If the test fails, then the process is repeated to establish the zone in which the seam should be repaired.

All acceptable repaired seams shall be bound by two locations from which samples passing laboratory destructive tests have been taken. In cases where repaired seam exceeds 150 ft, a sample taken from the zone in which the seam has been repaired must pass destructive testing. Repairs shall be made in accordance with Section 3.07-F.

The QAT shall document all actions taken in conjunction with destructive test failures.

F. Repair Procedures

Any portion of the geomembrane exhibiting signs of defect, failing a destructive or a nondestructive test shall be repaired. Several procedure exist for the repair of these areas. The final decision as to the appropriate repair procedure shall be made by the EFR.

1. The repair procedures available include:

a. Patching - Used to repair large holes, tears, undispersed raw materials, and contamination by foreign matter.

b. Spot welding or seaming - Used to repair small tears, pinholes, or other minor, localized defects.

c. Capping - Used to repair large lengths of failed seams.

d. Remove bad seam including the material extending at least six inches away from the centerline of the seam and replace with a strip of new material welded in place.

2. For any repair method, the following provisions shall be satisfied:

a. Surfaces of the geomembrane which are to be repaired using extrusion methods shall be abraded no more than one hour prior to the repair.

b. All surfaces shall be clean and dry at the time of the repair.

G. Repair Verification

Each repair shall be numbered and logged by the QAT. Each repair shall be nondestructively tested using the methods described in Section 3.06 as appropriate. Repairs which pass the nondestructive test shall be taken as an indication of an adequate repair. Repairs more than 150 ft long may be of sufficient length to require destructive test sampling, at the discretion of the EFR. Failed test indicate that the repair shall be redone and retested until a passing test results are achieved. The QAT and EFR shall observe all nondestructive testing of repairs. The QAT shall record the number of each repair, date, and test outcome.

3.08 DISPOSAL OF WASTE MATERIAL

A. Upon completion of installation, the Contractor shall dispose of all trash, waste material and equipment used in connection with the performed work and shall leave the premises in a neat and acceptable condition.

END OF SECTION

APPENDIX A

TABLE A1

FINGERPRINTING PROPERTIES FOR HIGH DENSITY POLYETHYLENE (HDPE) GEOMEMBRANE

PROPERTY	TEST METHOD
Density	ASTM D792 or ASTM D1505
Melt Index	ASTM D1238
High Load Melt Index	ASTM D1238
Carbon Black Content	ASTM DI603
Oxidative Induction Time	ASTM 3895
Crystallinity Melting range and point at endotherm maximum	Differential Scanning Calorimetry (DSC)

The above tests shall be performed by the Manufacturer of the HDPE geomembrane for identification of the Manufacturer's product. The above test results shall be submitted to the Engineer for approval of the product. The geomembrane to be supplied for the project shall meet these fingerprinting properties.

APPENDIX B

TABLE B1
MATERIAL PROPERTIES
HIGH DENSITY POLYETHYLENE (HDPE) GEOMEMBRANE
60 MIL NON-TEXTURED SHEET

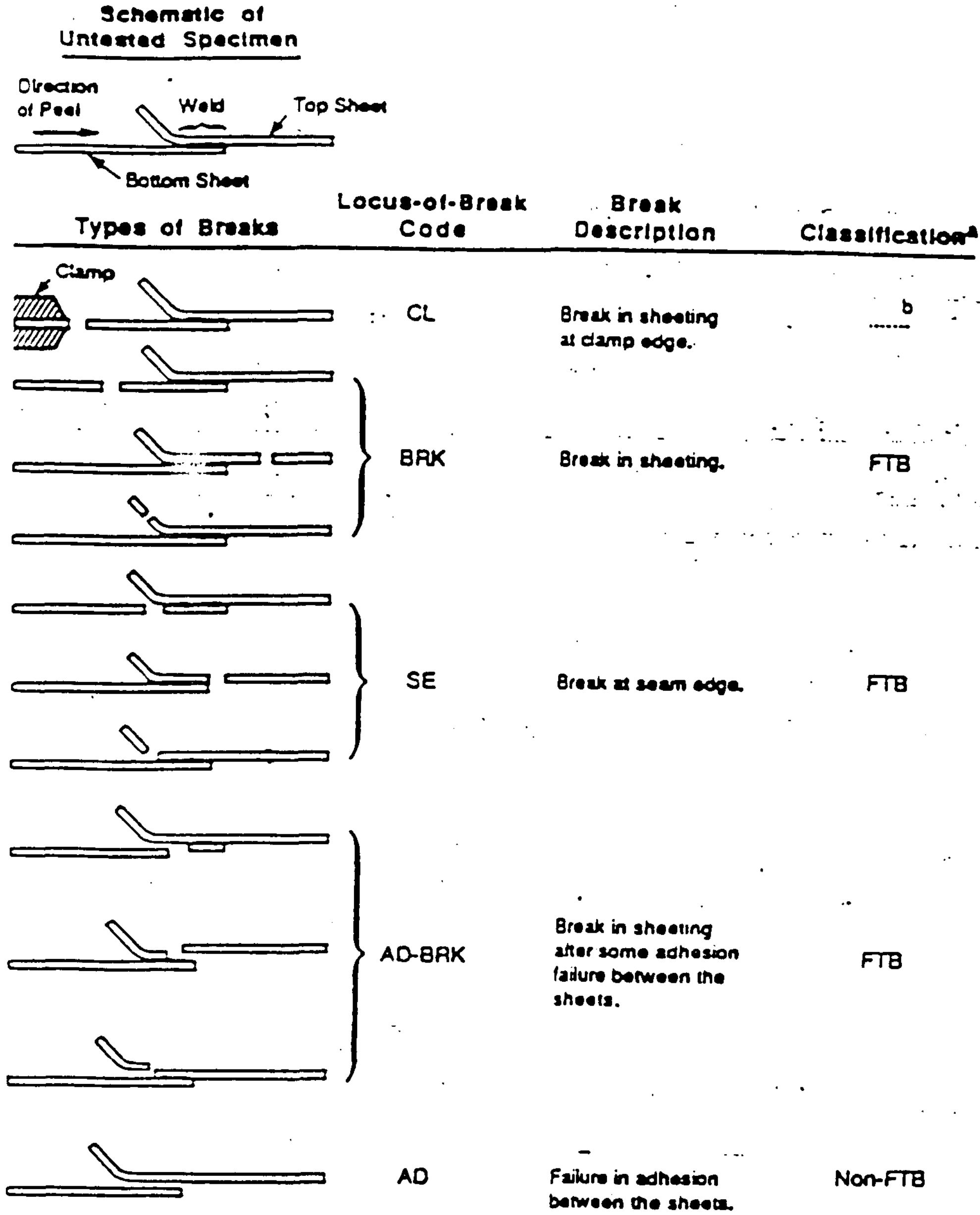
PROPERTY	UNIT	VALUE		TEST METHOD
Thickness	mils	60 min. avg.		[1]ASTM D751
Thickness	mils	54 min.		[1]ASTM D751
Density	g/cc	0.940 min.		ASTM D1505
Tensile Properties (Each Direction)				[1]ASTM D638
1. Yield Strength	lb/in	126 min.	2100 PSI	
2. Break Strength	lb/in	228 min.	3800 PSI	
3. Elongation at Yield	Percent	12 min.		
4. Elongation at Break	Percent	560 min.		
Tear Resistance	lb	39 min.	650 lb/in	ASTM D1004
Puncture Resistance	lb	72 min.	1200 lb/in	FTMS 101 Method 2065
Law Temperature	C	60 max.		ASTM D746
Dimensional Stability	%	±2.0 max.		ASTM D1204 1 hour @ 1080
Environmental Stress Crack	Hours	1500 min.		[1]ASTM D1693
Carbon Black Content	%	2.0 to 3.0		ASTM D1603
Carbon Black Dispersion	N/A	A1, A2, or B1 rating		[1]ASTM D3015

[1]Test methods as modified in Annex A, NSF Standard 54, May 1991.

TABLE B2
FACTORY AND FIELD SEAMS PROPERTIES
HIGH DENSITY POLYETHYLENE (HDPE) GEOMEMBRANE
60 MIL NON-TEXTURED SHEET

PROPERTY	UNIT	VALUE		TEST METHOD
Bonded Shear Strength	lb/in	120 min.	2000 psi	[1]ASTM D4437
Seam Peel Adhesion	lb/in	FTB and 88 min.	FTB and 1470 psi	[1]ASTM D4437

[1]Test methods as modified in Annex A, NSF Standard 54, May 1991.



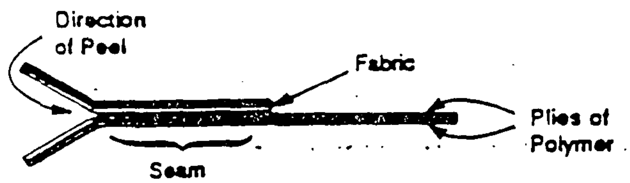
[a] FTB = Film - Tear Bond.

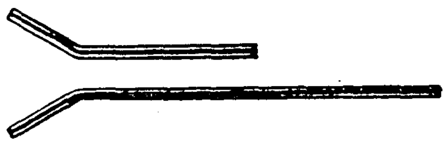
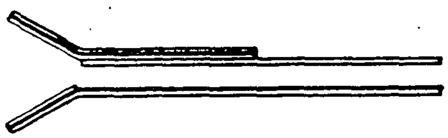
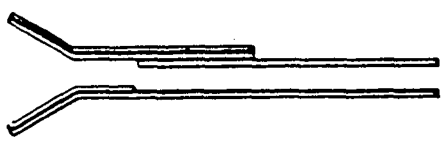
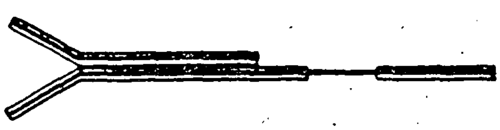
[b] Acceptance of CL - type breaks may depend on whether test values meet a minimum specification value. In general, though, a CL - type break should be considered a "no test". If specimens for a particular sample break consistently at the clamp edge, changes in the testing procedure should be considered, e.g. changing the clamp face, using a dumbell - type specimen.

Locus-of-break codes for dielectric-welded or solvent-welded seams in unreinforced FMS's tested for seam strength in shear and peel modes.

FIGURE β1

Schematic of a Seam of a 3-Ply Fabric-Reinforced FML



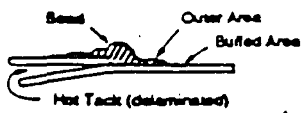
Types of Breaks	Locus-of-Break Code	Break Description	Classification[a]
	AD	Adhesion failure resulting in delamination in the plane of the bond.	Non - FTB
	DEL	Delamination in the plane of the scrim. (Applicable to peel only).	FTB
	AD-DEL	Delamination in the plane of the scrim after some delamination in the plane of the bond. (Applicable to peel only)	FTB
	BRK	Break in the sheet through both the fabric and the plies of polymer. Fabric break may precede break in sheeting.	FTB
	FP	Fabric pullout. Pullout of the threads parallel to the direction of test followed by break in the plies of polymeric sheeting.	No Test

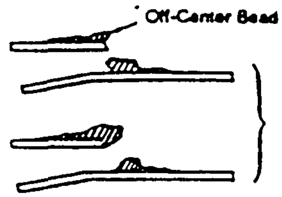
[a] FTB = Film - Tear Bond.

Locus-of-break codes for seams in three-ply fabric-reinforced FML's tested for seam strength in shear and peel modes.

FIGURE B2

Schematic of Untested Specimen



Types of Breaks	Locus-of-Break Code	Break Description	Classification[a]
	AD1	Failure in adhesion. Specimens may also delaminate under the bead and break through the thin extruded material in the outer area.	Non-FTB
	AD2	Failure in adhesion.	Non-FTB
	AD-WLD	Break through the fillet. Breaks through the fillet range from breaks starting at the edge of the top sheet to breaks through the fillet after some adhesion failure between the fillet and the bottom sheet.	Non-FTB[b]
	SE1	Break at seam edge in the bottom sheet. Specimens may break anywhere from the bead/outer area edge to the outer area/buffed area edge. (Applicable to shear only).	FTB
	SE2	Break at seam edge in the top sheet. Specimens may break anywhere from bead/outer area edge to the outer area/buffed area edge.	FTB
	SE3	Break at seam edge in the bottom sheet. (Applicable to peel only).	FTB
	BRK1	Break in the bottom sheeting. A "B" in parentheses following the code means the specimen broke in the buffed area. (Applicable to shear only).	FTB
	BRK2	Break in the top sheeting. A "B" in parentheses following the code means the specimen broke in the buffed area.	FTB
	AD-BRK	Break in the bottom sheeting after some adhesion failure between the fillet and the bottom sheet. (Applicable to peel only).	FTB
	HT	Break at the edge of the hot tack for specimens which could not be delaminated in the hot tack.	No Test

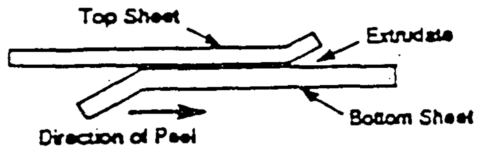
[a] FTB = Film - Tear Bond.

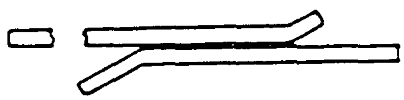
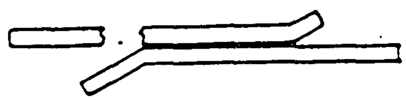
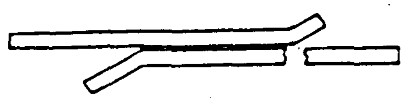
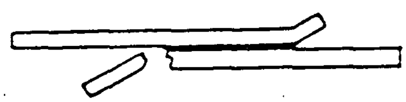
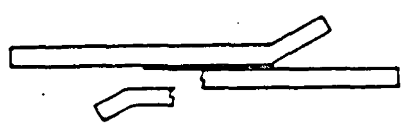
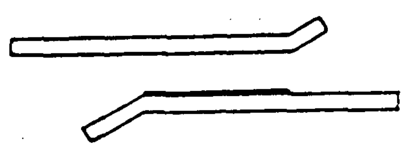
[b] Acceptance of AD-WLD breaks may depend on whether test values meet a minimum specification value and not on classification as a FTB or non-FTB break.

Locus-of-break codes for fillet-extrusion weld seams in semi-crystalline FML's tested for seam strength in shear and peel modes.

FIGURE B3

Schematic of Untested Specimen



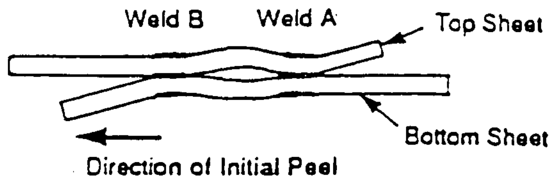
Location of Break	Locus-of-Break Code	Break Description	Classification[a]
	BRK	Break in sheeting outside weld area. Break can be in either the top or bottom sheet	FTB
	SE1	Break in top sheet at seam edge.	FTB
	SE2	Break in bottom sheeting at seam edge	FTB
	SE3	Break in bottom sheeting at seam edge. (Applicable to peel only).	FTB
	AD-BRK	Break in sheeting after some adhesion failure between ex-trudate and surface of the sheeting. Break can be in either the top or bottom sheet.	FTB
	AD	Failure in adhesion between the ex-trudate and the sheeting surface.	Non-FTB

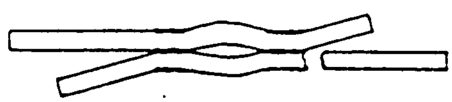
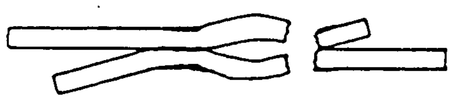
[a] FTB = Film - Tear Bond

Locus-of-break codes for extrusion weld seams in semicrystalline FML's tested for seam strength in shear and peel modes.

FIGURE B4

Schematic of Untested Specimen



Types of Break	Locus-of-Break Code	Break Description	Classification[a]
	AD	Adhesion failure.	Non-FTB
	BRK	Break in sheeting. Break can be in either top or bottom sheet.	FTB
	SE1	Break at outer edge of seam. Break can be in either top or bottom sheet.	FTB
	SE2	Break at inner edge of seam through both sheets.	FTB
	AD-BRK	Break in first seam after some adhesion failure. Break can be in either the top or bottom sheet.	FTB

[a] FTB = Film - Tear Bond

NOT TO SCALE

Locus-of-break codes for dual hot-wedge seams in semicrystalline FML's tested for seam strength in shear and peel modes. In cases where the Weld A fails in adhesion in a peel test, it is recommended that the test be stopped, that the specimen be replaced in the testing machine, and that Weld B be tested by peeling in the direction opposite to that used to Weld A.

FIGURE B5

APPENDIX C

Project Name ____________
Project No. ____________

FORM NO. 1

START-UP

	Date
1. Project start-up meeting held with selected manufacturer/installer.	____________
2. Duplicate project files established.	____________
3. Project personnel recorded.	____________
4. Liner construction drawings accepted.	____________
5. One complete drawing set received onsite.	____________
6. Drawings and specifications log begun by FSM.	____________
7. Verification that all equipment to be used is operating properly to be signed and placed in file by FSM.	____________
8. Liner warranty reviewed and accepted by OR.	____________

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4/8/92

Project Name________

Project No.__________

FORM NO. 2

LINER PRE-DELIVERY

	Date
1. Certificates of compliance with specifications received by EFR (including resin and factory seams).	__________
2. Certificates meet specifications.	
- EFR Signature __________________	__________
3. - FSM Signature __________________	__________
4. Liner fabrication and installation schedule received by EFR.	__________
5. Schedule is accepted as submitted __________	__________
as revised __________	__________
- EFR Signature __________________	__________
- CR Signature __________________	__________
6. Liner shop and installation drawings received by EFR.	__________
7. Copy of installer's affidavit given him by the manufacturer or supplier received by EFR.	__________

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Date__________

FORM NO. 4
(Continued)

DAILY CHECKLIST

Time ______________

7. Destructive testing completed according to specifications. ______________

- No. of failures. ______________ ______________
- No. of repairs. ______________ ______________
- Peel and shear test results recorded. ______________ ______________
- FSM Signature ________________ ______________
- EFR Signature ________________ ______________

8. All SMTs have been thoroughly trained in the use, limitations, adjustment, and repair of each seaming machine.

- FSM Signature ________________ ______________
- Seaming Machine Operator Names:

______________________________ ______to______

______________________________ ______to______

______________________________ ______to______

______________________________ ______to______

______________________________ ______to______

9. Number of rolls installed. ______________

10. Roll Nos. installed. ______________

11. FSM Comments: ______________________________________

____

____

FSM Signature ________________

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4/8/92

Date__________

FORM NO. 4
(Continued)

DAILY CHECKLIST

Time ______

12. EFR Comments: ____

____

____

- Signature ____________________

13. No. of additional comment pages attached.

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4/8/92

Project Name ____________
Project No. ____________
Date ____________

FORM NO. 5

GEOMEMBRANE PANEL PLACEMENT

Panel No.	Roll No.	Subgrade Condition	Length	Width	Visual Inspection Pass/Fail	FSM Initials	EFR Initials

Comments: ____

CORP1T.4/27
4/8/92

Project Name ____________
Project No. ____________
Date ____________

FORM NO. 6

ON-SITE GEOMEMBRANE WELDING REPORT

Welder: ______________________ Weld Type: ______________________

Welding Machine No.: ______________ Liner Thickness: ______________

Seam ID.	Original or Repair	Ambient Temp.	Weather Conditions	Non-Destructive Test Pass/Fail	On-Site Destructive Peel Test Pass/Fail	FSM Initials	EFR Initials

Comments: ____

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4/8/92

Project Name ____________
Project No. ____________
Date/Page ____________

FORM NO. 7

DAMAGE AND FAILURE REPORT

Panel No.	Seam No.	Location of Repair or Damage	Type of Damage	Inspec. Date	FSM Initials	EFR Initials

Comments: ____

Project Name ____________
Project No. ____________
Date/Page ____________

FORM NO. 8

IN-SITU WELD TESTING REPORT

Sample No.	Test	Test Method	Results	QAT Initials	FSM Initials	EFR Initials

Comments: ____

Project Name ______________
Project No. ______________

FORM NO. 9

POST-INSTALLATION CHECKLIST

PRIMARY ______ SECONDARY ______

	Date
1. Measurements of installed liner conducted by FSM and EFR.	______
2. No. of deviations from design drawings:	
No. ______	______
3. Deviations acceptable and as-built drawings modified accordingly (No. ____).	______
4. Deviations unacceptable and liner installation corrected (No. ____).	______
- FSM Signature ________________	______
- EFR Signature ________________	______

CORP1T.4/30
4/8/92

FORM NO. 10
DAILY FIELD LOG

Weather Conditions ______________________ Date ______________

______________________ Project No. ______________

Project Name ______________

Personnel On-site ____

____

Log ____

Installation Supervisor ____

FORM NO. 10
DAILY FIELD LOG

Weather Conditions ____________________ Date ____________

____________________ Project No. ____________

Project Name ____________

Personnel On-site ____________________

____

Log ____________________

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Installation Supervisor ____________________

TECHNICAL APPENDIX E

LEACHATE TREATMENT PLANT DESIGN REPORT

E1.0 INTRODUCTION AND BACKGROUND

E1.1 OBJECTIVE

The basic objective of this technical memorandum is to evaluate the feasibility of the alternatives considered for leachate treatment for the Central County Solid Waste Disposal Complex and present recommendations. A conceptual preliminary design of the selected process is presented to provide an essential basis of information for an effective and economical leachate treatment system.

E1.2 REGULATORY REQUIREMENTS

The preliminary design considerations are based on the Workshop Draft Rule 17-701 Revision Phase II - Solid Waste Facilities Rule of December 3 and 4, 1991. These draft rules are anticipated for promulgation in the near future with few changes. Per the proposed rules, the leachate must be removed from the landfill area. A summary of regulatory requirements of options for leachate systems presented in 17-701 are presented below:

- Leachate shall be collected and treated as necessary to meet surface and groundwater quality standards of Chapter 17-3 and 17-302, F.A.C.

- Leachate may be discharged to an offsite treatment plant. A written contract or agreement with the offsite treatment plant to discharge to the facility is necessary.

- On-site leachate treatment or pretreatment systems are part of the leachate collection and removal system and shall be designed according to the expected leachate characteristics.

- A contingency plan to handle leachate collection, removal, and treatment problems such as interruptions of discharges to a treatment plant is necessary.

E2.0 LEACHATE MANAGEMENT PLAN

E2.1 LEACHATE QUALITY

Landfill leachate is generated as a result of rain water percolation through the open landfill cells. As water seeps through the deposited solid waste, it is retained by the impermeable cell liner. As this water comes into contact with the solid waste, pollutants are extracted from the decaying material. The leachate formed may contain high levels of organic and inorganic pollutants and must therefore be treated prior to discharge. Leachate quality is highly variable and dependent on the landfill location, landfill design, design of the liner and leachate collection system, and operational procedures. Prediction of leachate quality is difficult due to the effects of site specific conditions and the lack of leachate quality data from lined landfills.

In order to estimate the leachate quality for preliminary treatment design, available leachate quality data obtained from similar landfill systems located in Florida were reviewed. The landfills were considered similar based on their liner and leachate collection system in comparison to the anticipated design of the new Sarasota County landfill. The leachate water quality parameters were taken from "An Investigation of Solid Waste Landfills in the South Florida Water Management District", April 1987. The leachate characteristics were gathered based on sampling from seven Class I landfills (synthetically lined or lined with natural impermeable material) and are located in the Southeast region of Florida. Leachate quality summarized in Table E2-1 presents the ranges of leachate water quality parameters from the Florida area landfills.

For preliminary design purposes, the average leachate quality parameters of the Florida landfills were used in the treatment process review. Values of Biochemical Oxygen Demand (BOD) samples were not taken and reported in the leachate data shown in Table E2-1. Review of several local area landfills which had BOD data available represented a Chemical Oxygen Demand (COD) to

TABLE E2-1
FLORIDA SOUTHEAST AREA LANDFILLS
LANDFILL LEACHATE WATER QUALITY PARAMETERS

Parameter	Average	Maximum	Minimum
pH (s.u.)	6.8	7.5	6.1
Specific Conductivity (umho/cm)	7,244	10,450	2,050
Total Solids	4,723	7,912	1,916
Suspended Solids	281	870	25
Dissolved Solids	4,450	7,341	1,874
COD	1,932	3,000	530
*BOD	193	300	53
CN^-	<0.005	0.22	<0.005
F^-	0.22	0.48	0.04
Cl^-	1,036	2,360	112
SO_4^{-2}	72	191	24
TKN -N	655	1,670	22.5
NH_3-N	543	1,340	9.4
Organic N	112	330	13.1
Total P	4.4	9.9	0.2
TOC	710	1,400	189
Aluminum	1.519	4.8	BDL
Antimony	0.041	0.38	BDL
Arsenic	0.168	1.6	BDL
Barium	0.208	0.350	0.155
Beryllium	0.001	0.009	BDL
Cadmium	BDL	0.0005	BDL
Calcium	196	614	15
Chromium	0.192	0.290	BDL
Copper	0.003	0.015	BDL
Iron	9.4	22	1.8
Lead	0.011	0.105	BDL
Magnesium	56	149	0.085
Manganese	0.600	1.8	BDL
Nickel	0.012	0.060	BDL
Potassium	70	345	BDL
Selenium	BDL	BDL	BDL
Silver	BDL	0.015	BDL
Sodium	629	999	250
Strontium	0.915	3.2	BDL
Tin	0.004	0.040	BDL
Titanium	0.059	0.250	BDL
Thallium	0.001	0.005	BDL
Zinc	0.467	1.2	BDL

(Source: An Investigation of Solid Waste Landfills in the South Florida Water Management District. University of Florida, 1987).

Units are reported in mg/l unless otherwise noted.
BDL - Below Detectable Limits
s.u. - standard units
* - BOD values established from a COD:BOD ratio of 10:1

BOD ratio range of 17:1 to 4:1. Based on this information, a COD:BOD conservative ratio of 10:1 is applied for the leachate in preliminary design.

E2.2 LEACHATE QUANTITY

The Environmental Protection Agency's (EPA) Hydrologic Evaluation of Landfill Performance (HELP) model was applied to assist in estimating the amounts of surface runoff, subsurface drainage, and leachate generated by the operation of a landfill. The model assesses the effects of precipitation, surface storage, surface runoff, infiltration, percolation, evapotranspiration, soil moisture storage, and lateral drainage in the active and/or closed landfill.

The quantity of leachate generated and the anticipated depth over the base liner was computed for four scenarios. The liner was assumed to have a leakage fraction of 0.00001 which makes the synthetic liner over the barrier layer essentially impermeable. This assumption maximizes the calculated leachate head over the liner produced from the model runs.

The base liner in all four model runs consisted of a composite liner. The composite liner is generally described as a barrier soil liner at the base with a hydraulic conductivity (HC) of not more than 1×10^{-7} cm/sec and a thickness of not less than 18-inches. The soil layer is covered by a synthetic membrane with a thickness of not less than 60-mils. This layer is overlayed with a synthetic geonet to improve lateral drainage along the membrane. The geonet is then overlaid by a geotextile filter fabric to prevent fines from clogging the geonet openings.

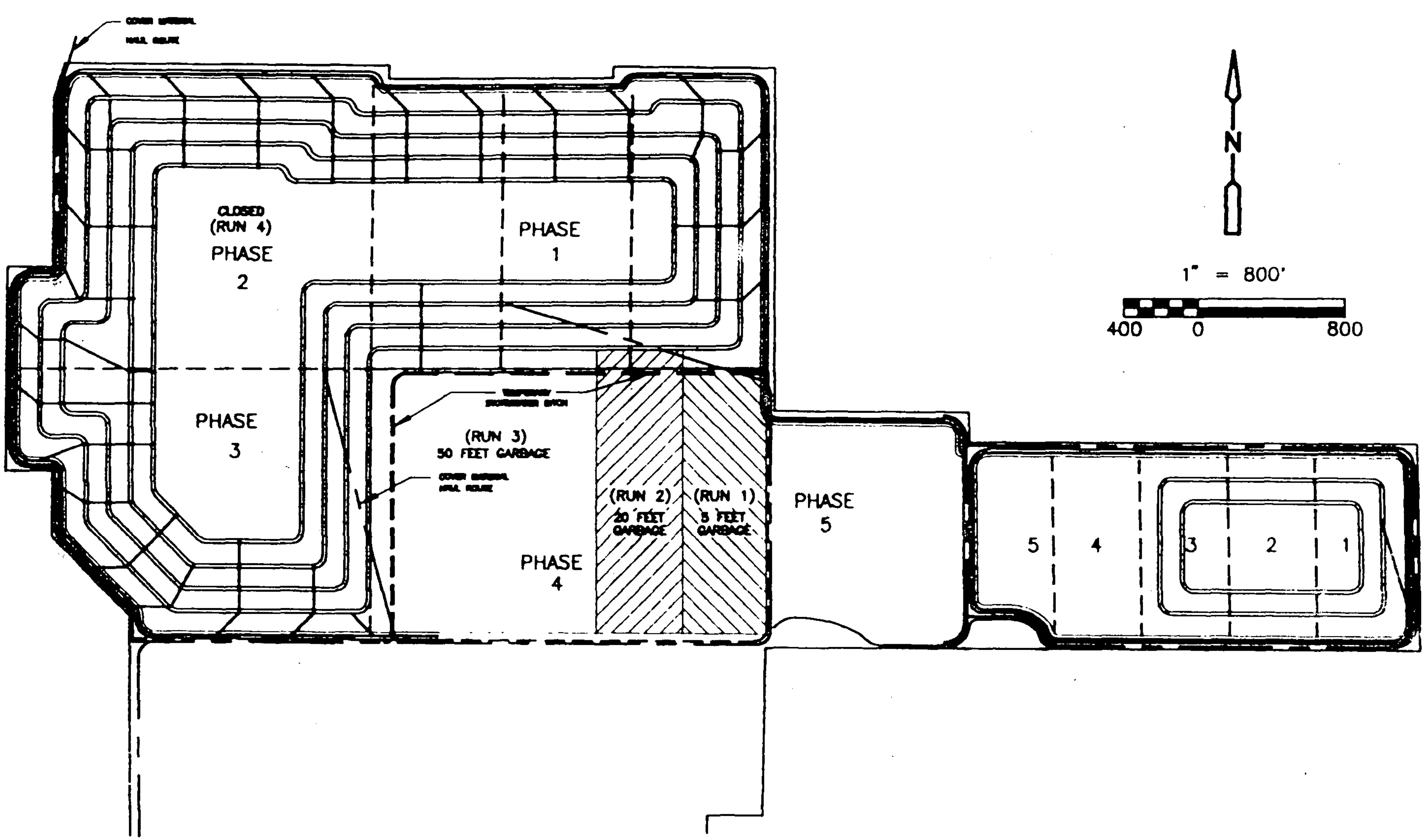
Above the base liner is the leachate collection system (LCS) consisting of a sloped 8-inch perforated pipe, drainage aggregate, and geotextile filter fabric. This system will be designed in such a manner to eliminate accumulation of leachate on the base liner. The LCS will be embedded in a protective layer of 24-inches of uncompacted local sand.

Four separate HELP model runs were performed to estimate leachate depths and generation rates during some common landfill operating scenarios as depicted in Figure E2-1. The purpose of model Run 1 (open cell) is to estimate the maximum possible flow through the lateral drainage layer to properly design the leachate collection and removal system. The purpose of model Run 2 is to simulate the conditions at a relatively young landfill (20 feet refuse). Model Run 3 simulates the conditions in a landfill that has been operating for several years (50 feet refuse), and Model Run 4 is the final capped condition which simulates the leachate generation for the closed landfill. HELP model results are shown in Table E2-2.

A 1.5 safety factor is applied to the new cell leachate flow of 1,390 gal/acre-day to provide a conservative value for leachate quantity generation. The resulting preliminary design value of anticipated leachate flow for an open cell is 2,095 gal/acre-day. Table E2-3 lists the values which are used for leachate quantity estimates in preliminary design. The average day treatment capacity is approximately 50,000 gpd.

E2.3 LEACHATE STORAGE

The leachate treatment regime will require a storage/equalization system in order to maintain 1" or less head on the landfill liner as required by regulations. The storage volume is based on a 25-year, 24-hour storm event of 8-inches of rainfall. A storage capacity of approximately 700,000 gallons will be required for a 13-acre open cell operation. The leachate volume retained in the storage system may be recirculated to an active cell in the landfill. A water balance conducted on the leachate quantities to be treated and a 700,000 gallon storage system to be recirculated in 15-days resulted in a peak plant capacity of 100,000 gpd. The leachate storage system will be located in the maintenance facility area depicted in Figure E2-2.



CENTRAL SOLID WASTE DISPOSAL COMPLEX

LEACHATE QUANTITY PROJECTION ESTIMATES LANDFILL SIMULATION

CDM
environmental engineers, scientists, planners, & management consultants

Figure No. E2-1

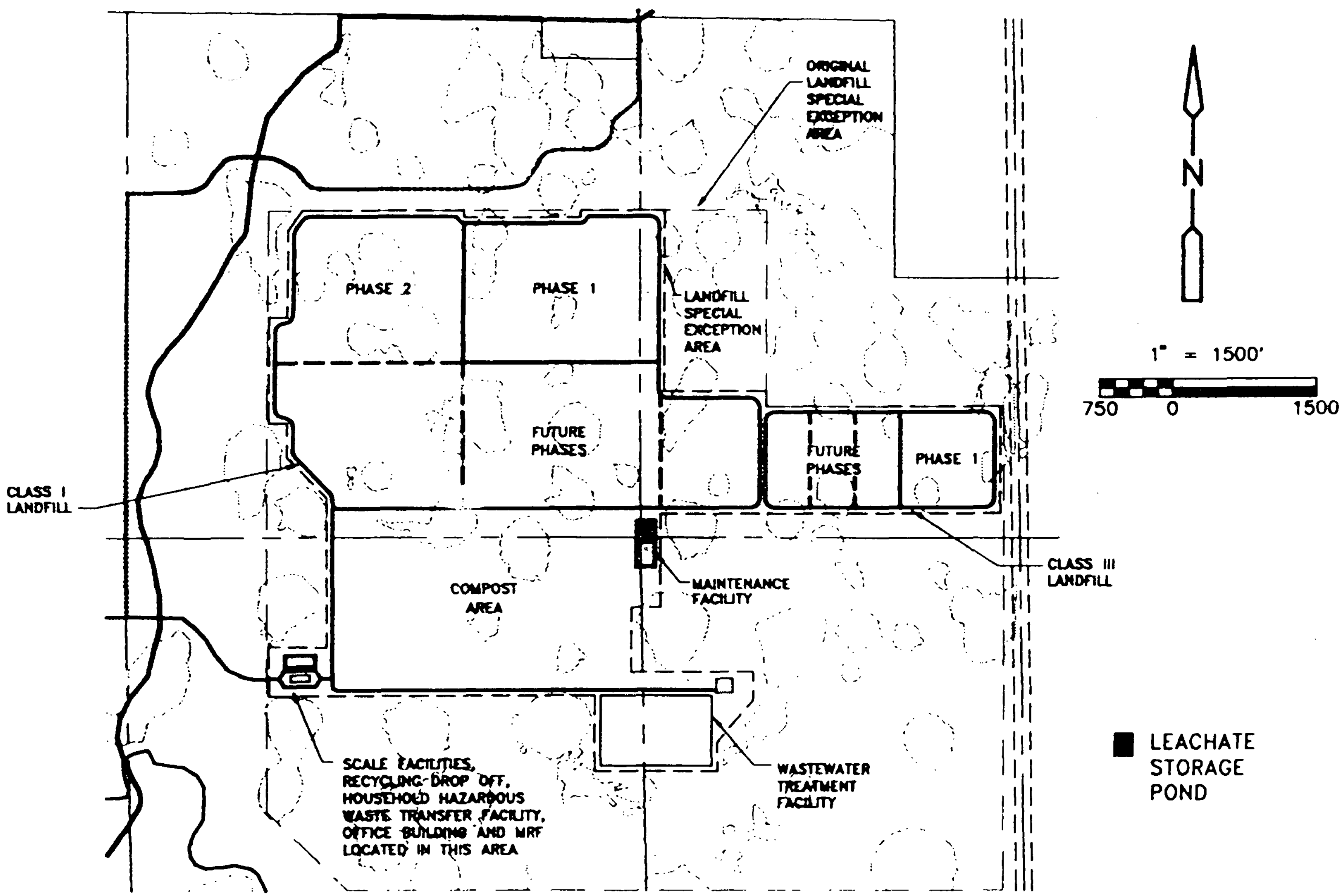
TABLE E2-2

ESTIMATED LEACHATE GENERATION VOLUMES
HELP MODEL SUMMARY RESULTS

	PEAK DAY VOLUME/ACRE-DAY					AVERAGE DAY VOLUME/ACRE-DAY				
	SURFACE RUN-OFF		LEACHATE FLOW			SURFACE RUN-OFF		LEACHATE FLOW		Ave.
RUN CONDITIONS	CF	GAL	CF	GAL	Head In.	CF	GAL	CF	GAL	Head In.
1. NEW CELL CONDITION (open)	2,814.1	21,054	965.4	7,221	0.1	20.6	154	185.8	1,390	<0.1
2. EARLY OPERATING CONDITION (20 feet refuse)	5,491.3	41,075	23.8	178	<0.1	159	1,190	22.5	168	<0.1
3. INTERMEDIATE OPERATING CONDITION (50 feet refuse)	5,491.3	41,075	23.4	175	<0.1	159	1,190	22.8	171	<0.1
4. FINAL CAPPED CONDITION (closed)	13,238.6	99,025	17.4	130	<0.1	46.5	348	14.7	110	<0.1

TABLE E2-3 LEACHATE QUANTITY DESIGN ESTIMATES

RUN	(WORST CASE) SITE SIZE PHASE IV ACRES	SITE TYPE	AVERAGE DAILY FLOW GPD/ACRE	PHASE IV AVG. DAILY GPD	OPEN CELL 25-YR. 24-HR STORM EVENT GALLONS	RETAINED IN SOIL GALLONS	GALLONS/DAY STORAGE VOLUME
1	9.6	OPEN CELL	2,095	27,235	2,823,850	2,117,887	705,962
2	19.2	20 FT. SOLID WASTE	168	3,226			
3	71.1	50 FT. SOLID WASTE	171	12,158			
4	60.0	CLAY CAP, GOOD GRASS	110	6,600			
	163.3			49,219			



SARASOTA COUNTY

CDM
environmental engineers, scientists, planners, & management consultants

CENTRAL COUNTY SOLID WASTE DISPOSAL COMPLEX
LEACHATE STORAGE PONDS

Figure No. E2-2

The preliminary design criteria for the leachate storage system is established in FDER 17-701 as follows:

- Double-lined - 60 mil each (upper and lower) geomembrane
- Leak Detection System
- Compartment Construction (Independent Operation Ability)
- Above Seasonal High Water Table
- Minimum 2-Feet Freeboard at 25-Year, 24-Hour Storm Event

Preliminary design associated with the storage system is based on the following information:

$$\frac{(700{,}000\ \text{gal})}{(7.48\ \text{gal/cf})} = 93{,}583\ \text{cf};\ 3{,}466\ \text{cy - volume}$$

Assume 3-feet depth volume storage with 2-feet freeboard; pond is located above seasonal high groundwater table.

$$\frac{(93{,}583\ \text{cf})}{(3\ \text{ft})} = 31{,}194\ \text{ft}^2$$

2 compartments - inside dimensions
125 ft x 125 ft

E2.4 LEACHATE PRELIMINARY DESIGN CHARACTERISTICS AND TREATMENT OPTIONS

E2.4.1 PRELIMINARY DESIGN PARAMETERS

The preliminary design characteristics used for each option are shown in Table E2-4. Each system evaluation is based on the same flow rates, storage capacity, and raw constituent quality. Capital and operating costs used in the evaluations were obtained from either the EPA Innovative and Alternative Technology Manual or similar recent project costs.

TABLE E2-4

LEACHATE TREATMENT - DESIGN PARAMETERS

FLOW QUANTITIES	
Average Day, gpd	50,000
Peak, gpd	100,000
STORAGE CAPACITY, gallons	700,000
LEACHATE QUALITY (primary design constituents)	
pH, s.u.	6.8
TSS, mg/l	281
TDS, mg/l	4,450
BOD, mg/l	193
COD, mg/l	1,932
Chloride, mg/l	1,036
Sulfate, mg/l	72
TKN, mg/l	655
TP, mg/l	4.4
Iron, mg/l	9.4
Sodium, mg/l	629
Calcium, mg/l	196

E2.4.2 TREATMENT OPTIONS

Successful treatments to acceptable discharge quality are feasible using a combination of chemical and biological treatment unit processes or ultrafiltration/reverse osmosis unit processes. These include:

Chemical/Biological	Ultrafiltration/Reverse Osmosis
o Storage/equalization	o Storage/equalization
o High pH addition/lime precipitation	o pH adjustment
o pH adjustment	o Anti-scalent addition
o Biological Treatment Activated sludge Clarification	o Ultrafilter application o Reverse osmosis application o Degasifier (aeration)
o Filtration (possibly denitrification filters)	o pH adjustment o Disinfection
o Disinfection	o Effluent disposal or wastewater treatment plant (no disinfection)
o Effluent disposal or wastewater treatment plant (no disinfection)	

Three alternatives reviewed for the leachate were as follows:

Option 1: Pretreatment with discharge to municipal wastewater treatment facility.

Option 2: On-Site treatment to reuse standards with on-site irrigation.

Option 3: Ultrafiltration/reverse osmosis to permittable standards with on-site irrigation.

E2.4.3 OPTIONS SUMMARY

Option 1: Pretreatment of the leachate with effluent disposal may not be a feasible method. If high concentrations of chlorides or TDS are present in the leachate, the treatment regime of lime precipitation may not remove them to the levels required by the Venice standards. The possibility exists of negotiating a variance for these constituent levels; however, the Venice Eastside WWTP final effluent disposal is through reuse application. The reuse system can not tolerate the elevated chlorides and TDS levels and, therefore, is an unlikely alternative to pursue for the County.

Option 2: On-site treatment involving primarily lime precipitation, anoxic, and denitrifying filter can treat to on-site irrigation usage, but the chloride levels are considered high for this application and cannot be lowered effectively.

Option 3: Treatment on-site with an ultrafiltration/reverse osmosis membrane application can reasonably treat all anticipated constituents in the leachate to the desired levels of irrigation. UF/RO applications can handle the varied fluctuating levels of the leachate. This option is the overall recommended form of treatment for the leachate.

E2.5 RECOMMENDED TREATMENT
OPTION 3 - ULTRAFILTRATION/REVERSE OSMOSIS TO ON-SITE IRRIGATION

Leachate treatment can be provided on-site by membrane application processes. Recent studies conducted on the Sarasota County Bee Ridge

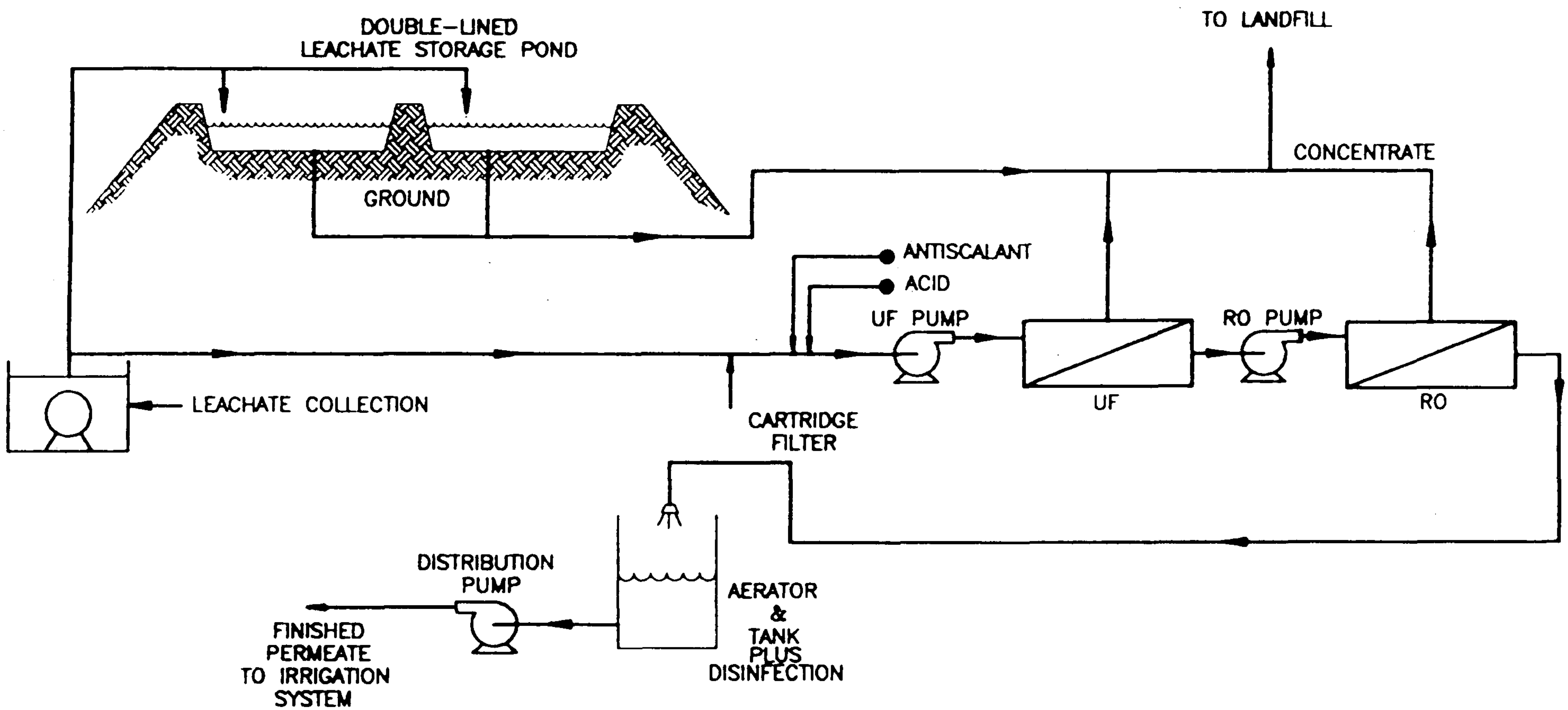
Landfill leachate using ultrafiltration/reverse osmosis (UF/RO) membranes yielded primary and secondary water quality permeate. Although it is not required by regulations to treat leachate to primary and secondary water quality standards, it is required to provide effluent which meets acceptable standards for groundwater, surface water, or reuse quality depending on the type of disposal.

The UF/RO treatment option is depicted in Figure E2-3. The preliminary design of the system is based on leachate feed to the plant directly from the leachate pump stations. As determined in the water balance discussed in earlier sections, a peak flow of 100,000 gpd is required. The UF/RO system capacity is sized for the peak conditions of 100,000 gpd which will produce 70,000 gpd of permeate and 30,000 gpd of concentrate. The 700,000 gallons of leachate storage is still required as previously discussed.

The UF/RO process will require pretreatment involving a cartridge filter, antiscalant and acid addition. The cartridge filter provides protection for the membranes. Antiscalant is added to prevent membrane scalent caused from various constituents such as iron and sulfates. Acid is applied to lower the pH of the feed stream to optimize membrane system operation.

The ultrafilters (UF) are membranes which act as pre-filters to the reverse osmosis membranes. The UF membranes will remove suspended solids and many organic compounds which may foul the reverse osmosis membranes. It is anticipated to have an UF recovery of 93%. The UF pump will boost the feed flow to a sufficient operating pressure to enter the UF membranes.

The UF permeate is fed to the reverse osmosis pump and raised to a sufficient operating pressure for the RO membranes. The reverse osmosis membranes are anticipated to have a 75% recovery with effective removal of many inorganic dissolved solids. The overall system recovery is anticipated for 70% recovery from initial raw leachate feed.



UF/RO TREATMENT
TO IRRIGATION

CDM
environmental engineers, scientists, planners, & management consultants

Figure No. E2-3

The RO permeate will be fed to a degasifier unit which assists in the release of possible CO_2 while raising the pH. The final permeate effluent is sent to a distribution system for onsite irrigation.

E2.5.1 PERMEATE DISPOSAL

Disposal of the permeate from the UF/RO plant will be through on-site irrigation as shown on permit drawing sheet G-6. The annual average for site irrigation is based on 50,000 gpd (70,000 gpd maximum) of permeate generated. The average application rate is estimated at 0.25 in./week yielding a total area of irrigation requirement of 70 acres (maximum). Also a 5 MG effective wet weather storage pond is designed next to the treatment facility. The maximum capacity is established by a one and ten year storm event and is equivalent to 70 days of effluent storage.

E2.5.2 CONCENTRATE RECIRCULATION

Concentrate generated from the UF/RO process will be recirculated to an active landfill cell on an average annual basis of 21,450 gpd. The concentrate may be spray irrigated over active fill areas where eventual percolation to the collection system will occur.

PRMT1
4/92

TECHNICAL APPENDIX F
FINANCIAL RESPONSIBILITY

SARASOTA COUNTY GOVERNMENT
SARASOTA, FLORIDA

Solid Waste Operations Division

[redacted]
Manager
8350 Bee Ridge Road
Sarasota, Florida 34241
(813) 951-5096
FAX (813) 951-5161

June 10, 1992

Department of Environmental Regulation
4520 Oak Fair Boulevard
Tampa, Florida 33610-7347

Gentlemen:

Subject: Financial Responsibility

With respect to Chapter 17-701.076, F.A.C., we wish to offer the following information with regard to our permit application for construction of a Class I and Class III landfill at the Central County Solid Waste Disposal Complex located in Sarasota County, Florida.

This facility is to be owned and operated by the County of Sarasota. The county will provide proof of financial responsibility to the Department. The effective date of the financial document will be sixty days prior to the acceptance of any solid waste at the facility. Such proof shall be that there has been established a landfill management escrow account as a separate, interest bearing, enterprise fund within the Solid Waste Operations Division of the Sarasota County Government. This account will be established and maintained in accordance with Chapter 17-701.076, F.A.C.

Sincerely,

William K. Lederman, P.E.
Engineer IV