DESIGN REPORT FOR THE INTERIM WASTE CONTAINMENT FACILITY
AT THE
NIAGARA FALLS STORAGE SITE

MAY 1986

Prepared for
UNITED STATES DEPARTMENT OF ENERGY
OAK RIDGE OPERATIONS OFFICE
Under Contract No. DE-AC05-81OR20722

By
Bechtel National, Inc.
Advanced Technology Division
Oak Ridge, Tennessee

Bechtel Job No. 14501

MASTER

0123456789

DISCLAIMER

This report was prepared as an account of work sponsored by an agency of the United States Government. Neither the United States Government nor any agency Thereof, nor any of their employees, makes any warranty, express or implied, or assumes any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, apparatus, product, or process disclosed, or represents that its use would not infringe privately owned rights. Reference herein to any specific commercial product, process, or service by trade name, trademark, manufacturer, or otherwise does not necessarily constitute or imply its endorsement, recommendation, or favoring by the United States Government or any agency thereof. The views and opinions of authors expressed herein do not necessarily state or reflect those of the United States Government or any agency thereof.

DISCLAIMER

Portions of this document may be illegible in electronic image products. Images are produced from the best available original document.

ABSTRACT

Low-level radioactive residues from pitchblende processing and thorium- and radium-contaminated sand, soil, and building rubble are presently stored at the Niagara Falls Storage Site (NFSS) in Lewiston, New York. These residues and wastes derive from past NFSS operations and from similar operations at other sites in the United States conducted during the 1940s by the Manhattan Engineer District (MED) and subsequently by the Atomic Energy Commission (AEC).

The U.S. Department of Energy (DOE), successor to MED/AEC, is conducting remedial action at the NFSS under two programs: on-site work under the Surplus Facilities Management Program and off-site cleanup of vicinity properties under the Formerly Utilized Sites Remedial Action Program. On-site remedial action consists of consolidating the residues and wastes within a designated waste containment area and constructing a waste containment facility to prevent contaminant migration. The service life of the system is 25 to 50 years. Near-term remedial action construction activities will not jeopardize or preclude implementation of any other remedial action alternative at a later date. Should DOE decide to extend the service life of the system, the waste containment area would be upgraded to provide a minimum service life of 200 years.

This report describes the design for the containment system. Pertinent information on site geology and hydrology and on regional seismicity and meteorology is also provided. Engineering calculations and validated computer modeling studies based on site-specific and conservative parameters confirm the adequacy of the design for its intended purposes of waste containment and environmental protection.

TABLE OF CONTENTS

	<u>Page</u>
Acronyms	xii
Abbreviations	xiii
1.0 Introduction and Summary	1
1.1 Background	1
1.2 Objective and Scope	1
1.3 Summary	4
2.0 Description and Status of the NFSS	5
2.1 Site Description	5
2.1.1 Site Conditions Prior to Remedial Action	6
2.1.2 Completed and Continuing Remedial Action	6
2.1.3 Site Conditions After Completion of Remedial Action	10
2.2 Waste Containment Area	10
2.3 Environmental Conditions	13
2.3.1 Geology and Soils	13
2.3.2 Groundwater Hydrology	17
2.3.3 Surface Water Hydrology	18
2.3.4 Meteorology	22
2.3.5 Seismicity	23
3.0 Design Requirements and Assumptions	24
3.1 Radiological Criteria	24
3.2 Design Basis	24
3.3 Post-Closure Monitoring and Surveillance	29

	<u>Page</u>
4.0 Containment Facility Design	32
4.1 Sidewalls and Bottom	32
4.1.1 Sidewalls	32
4.1.2 Bottom	33
4.1.3 Waste Placement	34
4.2 Cap	34
4.2.1 Description of the Cap	34
4.2.2 Side Slope Stability	40
4.2.3 Surface and Cap Drainage	41
4.2.4 Shrinkage, Swelling, and Frost Action	42
4.2.5 Radiation Barrier	44
4.2.6 Inspection and Maintenance	44
5.0 Analysis of Contaminant Migration	46
5.1 Scope of Analysis	46
5.2 Methodology	46
5.3 Model Definition and Input Data	47
5.3.1 Groundwater Flow Simulation	47
5.3.2 Contaminant Transport Simulation	53
5.4 Sensitivity Analysis Results	53
5.4.1 Pressure Head	56
5.4.2 Contaminant Transport	62
5.5 Base Case Results	77
5.5.1 Infiltration of Precipitation	83
5.5.2 Contaminant Transport	84
5.6 Conclusions	86

	<u>Page</u>
6.0 Risk/Failure Mode Evaluation	89
6.1 Containment Facility	89
6.1.1 Quality Assurance Assessment	89
6.1.2 Inspection and Maintenance	90
6.1.3 Environmental Monitoring	91
6.1.4 Performance Monitoring	91
References	93
Appendix A Seismicity of the NFSS Region	A-1
Appendix B Theoretical Basis for Analysis of Contaminant Migration	B-1
Appendix C Determination of the Probable Maximum Flood Level at the Niagara Falls Storage Site	C-1
Appendix D Typical Technical Specifications for NFSS Waste Containment Facility Earthwork Construction: Cap, Dike, and Cutoff Wall	D-1
Appendix E Design Drawings for NFSS Waste Containment Facility: Cap, Dike, and Cutoff Wall, Years 1982 Through 1985	E-1

LIST OF FIGURES

<u>Figure</u>	<u>Title</u>	<u>Page</u>
1-1	The Regional Context of the Niagara Falls Storage Site	2
2-1	Diagram of the NFSS Showing Structures and Features of the Site Prior to Remedial Action	7
2-2	The NFSS Following Completion of Remedial Action	11
2-3	Surface Water Drainage Pathways at the NFSS	20
4-1	Plan of the Completed Cover	36
4-2	Section G and Detail I of the Cover	37
4-3	Sections of the Cover	38
5-1	Generalized Geologic Section for the NFSS	48
5-2	GS2 Finite Element Mesh Dike Sections	49
5-3	FLUMP Model Finite Element Mesh	50
5-4	GS2 Model Finite Element Mesh	54
5-5	Pressure Head as a Function of Cap Permeability	59
5-6	Pressure Head as a Function of Cap Thickness	60
5-7	Pressure Head Contours Resulting from a 15-cm (0.5-ft) Constant Head Boundary Condition (upper drawing), and an 8.4-cm/yr (3.3-in./yr) Constant Infiltration Rate Boundary Condition (lower drawing)	61
5-8	Pressure Head Contours for Constant Head Boundary Conditions	63
5-9	Pressure Head Contours for 50 and 200 Years	64

<u>Figure</u>	<u>Title</u>	<u>Page</u>
5-10	Uranium Migration as a Function of K_d , Adsorption Coefficient, for $K_d = 0.0$ and 1.0 ml/g	65
5-11	Uranium Migration as a Function of K_d , Adsorption Coefficient, for $K_d = 0.0$ and 10.0 ml/g	66
5-12	Radium Migration as a Function of K_d , Adsorption Coefficient, for $K_d = 0.0$ and 50.0 ml/g	67
5-13	Radium Migration as a Function of K_d , Adsorption Coefficient, for $K_d = 0.0$ and 500.0 ml/g	68
5-14	Contaminant Migration as a Function of Cap Permeability, K , for $K = 10^{-6}$ and 10^{-7} cm/s	70
5-15	GS2 Model Finite Element Mesh with Two Channels ($K = 10^{-2}$ cm/s) Partially Penetrating the Gray Clay	71
5-16	GS2 Model Finite Element Mesh with Two Channels ($K = 10^{-2}$ cm/s) Completely Penetrating the Gray Clay	72
5-17	GS2 Model Finite Element Mesh with One Channel ($K = 10^{-2}$ cm/s) Completely Penetrating the Gray Clay	73
5-18	Uranium Migration for Gray Clay with No Channels (upper drawing), and for Two Channels Partially Penetrating the Clay (lower drawing)	74
5-19	Uranium Migration for Gray Clay with No Channels (upper drawing), and for Two Channels Completely Penetrating the Clay (lower drawing)	75
5-20	Uranium Migration for Gray Clay with No Channels (upper drawing), and for One Channel Completely Penetrating the Clay (lower drawing)	76
5-21	Uranium Migration for Two Channels Penetrating the Gray Clay (upper drawing) and for Two Channels Completely Penetrating the Brown and Gray Clays (lower drawing)	78

5-22	Uranium Migration at 195 Years (upper drawing) and 1113 Years (lower drawing) $K_d = 1.0$ ml/g	79
5-23	Radium Migration at 195 Years (upper drawing) and 1113 Years (lower drawing) $K_d = 50.0$ ml/g	80
5-24	Uranium Migration at 1113 Years for $b_{cap} =$ 1.5 m (5.0 ft) (upper drawing), and $b_{cap} =$ 0.9 m (3.0 ft) (lower drawing) $K_d = 1.0$ ml/g	81
5-25	Radium Migration at 1113 Years for $b_{cap} =$ 1.5 m (5.0 ft) (upper drawing), and $b_{cap} =$ 0.9 m (3.0 ft) (lower drawing) $K_d = 50.0$ ml/g	82
5-26	Uranium Migration for Base Case Analysis at 190 Years (upper drawing) and 1113 Years (lower drawing)	85
5-27	Uranium Migration for Base Case Analysis at 190 Years (upper drawing) and with Two Channels Penetrating Brown and Gray Clays (lower drawing)	87
A-1	Tectonic Elements of the Niagara Region	A-2
A-2	Seismic Events of Magnitude > 3.0 Within 320 km (200 mi) of the NFSS (1823-1975)	A-4
A-3	475-Year Peak Dynamic Surface Acceleration Within 320 km (200 mi) of the NFSS	A-10
A-4	Maximum Historical Earthquake Intensity (Modified Mercalli Scale) Within 320 km (200 mi) of the NFSS (Through 1965)	A-12
C-1	NFSS Drainage Pathway Location Plan	C-2
C-2	NFSS Regional Channel Network for PMF Routing	C-8

LIST OF TABLES

<u>Table</u>	<u>Title</u>	<u>Page</u>
2-1	Locations of Residues and Contaminated Materials Before and After the Completion of Remedial Action	14
3-1	Summary of Residual Contamination Guidelines for the NFSS Waste Containment Design	25
3-2	NFSS Waste Containment System Design Requirements	27
5-1	Input Data to Groundwater Flow Model	51
5-2	Input Data to Contaminant Transport Model	55
5-3	Sensitivity Analysis for Groundwater Flow	57
5-4	Sensitivity Analysis for Contaminant Transport	58
A-1	Chiburis and Basham Data Base of Earthquakes of Magnitude 3.0 or Greater Occurring Within 320 km (200 mi) of the NFSS	A-7
C-1	PMF at the NFSS	C-4
C-2	Parameters Used for Hydrograph Generation for Sixmile and Twelvemile Creeks	C-5
C-3	PMF Routing: Network Parameters	C-8
C-4	PMF Levels at the NFSS	C-9

ACRONYMS

AEC	Atomic Energy Commission
ASTM	American Society for Testing and Materials
BNI	Bechtel National, Inc.
DOE	Department of Energy
EIS	Environmental Impact Statement
EPA	Environmental Protection Agency
FUSRAP	Formerly Utilized Sites Remedial Action Program
LOIC	Loss of Institutional Control
LOOW	Lake Ontario Ordnance Works
MED	Manhattan Engineer District
NEPA	National Environmental Policy Act
NFSS	Niagara Falls Storage Site
NOAA	National Oceanic and Atmospheric Administration
PMC	Project Management Contractor
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
SFMP	Surplus Facilities Management Program
UBC	Uniform Building Code

ABBREVIATIONS

cfs	cubic feet per second
cm	centimeter
cm/s	centimeters per second
cm/yr	centimeters per year
dpm	disintegrations per minute
dps	disintegrations per second
fps	feet per second
ft	foot
g	gravity
H	horizontal
ha	hectare
in.	inch
K	hydraulic conductivity
K _d	adsorption coefficient
kg	kilogram
kg/m ²	kilograms per square meter
kg/m ³	kilograms per cubic meter
km	kilometer
km/h	kilometers per hour
lb	pound
lb/ft ²	pounds per square foot
lb/ft ³	pounds per cubic foot
m	meter
m ²	square meter
m ³	cubic meters
mg/cm ²	milligrams per square centimeter
μR/h	microroentgens per hour
mi	mile
min	minute
mrad/h	millirads per hour
ml/g	milliliters per gram
mph	miles per hour
mrem/yr	millirem per year
mR/h	milliroentgens per hour
m.s.l.	mean sea level
pCi/g	picocuries per gram
pCi/l	picocuries per liter
pCi/m ² /s	picocuries per square meter per second
V	vertical
yd ³	cubic yards
yr	year

1.0 INTRODUCTION AND SUMMARY

1.1 BACKGROUND

The Niagara Falls Storage Site (NFSS) is a U.S. Department of Energy (DOE) surplus facility located in Lewiston Township, Niagara County, New York (Figure 1-1). The 77-ha (191-acre) site is a small portion of the original Lake Ontario Ordnance Works (LOOW) and was formerly used for uranium transshipment and radioactive waste storage. It is currently being managed for DOE by the Advanced Technology Division of Bechtel National, Inc., (BNI), Oak Ridge, Tennessee, as part of the DOE Surplus Facilities Management Program (SFMP) established to manage and plan the ultimate disposition of surplus DOE-owned facilities. Portions of the former LOOW site and other vicinity properties are within the jurisdiction of the SFMP's companion DOE program, the Formerly Utilized Sites Remedial Action Program (FUSRAP), which evaluates former Manhattan Engineer District/Atomic Energy Commission (MED/AEC) sites and conducts remedial action activities where appropriate.

Several types of radioactive residues and wastes are currently stored at the NFSS (Ref. 1). Some areas of the site also became contaminated from previous burial and spills of contaminated materials, and from radionuclide migration along drainage pathways.

Cleanup efforts under FUSRAP or SFMP typically involve handling relatively large volumes of materials (e.g., earth and building rubble) contaminated with low levels of radioactivity. Temporary storage of such materials is frequently necessary pending long-term disposition.

1.2 OBJECTIVE AND SCOPE

This report describes the design and technical requirements for a low level radioactive waste containment system at the NFSS. It is not intended to stand alone; rather it is one of several DOE

FIGURE 1-1 THE REGIONAL CONTEXT OF THE NIAGARA FALLS STORAGE SITE

documents prepared to support remedial action at the site and as an information source for a future DOE decision on the final disposition of the site (Refs. 1-7).

The purposes of the remedial actions are to provide for more stable and improved storage conditions, to minimize radiation exposure to workers and the public, and to keep the site in compliance with DOE Orders and other applicable federal and state regulations while not jeopardizing or precluding any long-term option for the site. Justification for, and alternatives to, these remedial actions are discussed in the draft Action Description Memorandum prepared in 1983 (Ref. 23).

The design is in conformance with the Design Criteria for Formerly Utilized (MED/AEC) Sites (Ref. 3), which is based on DOE and Environmental Protection Agency (EPA) requirements as they apply to the NFSS (Refs. 8-9 and 13-22).

The containment facility, which is being constructed as part of remedial action at the site, is designed to have a minimum expected service life of 25 years. The design of the facility is described in detail in Section 4.0. The primary purposes of the waste containment facility are to exclude surface water from the contained waste so that it does not become saturated and consequently subject to leaching; to provide a barrier to radiation and radon emanation; and to prevent pollution of the groundwater by minimizing or eliminating radionuclide leaching if moisture infiltration does occur (Ref. 3).

Section 5.0 includes an evaluation of the effects of severe (probable maximum) events that must be considered to ensure that proper design features are incorporated into the design of the short-term containment facility. Criteria have been developed to determine the potential impact of each of the following occurrences on the safety and stability of the containment structure:

- o Seismic events
- o Precipitation and flooding
- o Contaminant migration

1.3 SUMMARY

The design presented for the containment facility fulfills DOE requirements for waste containment, environmental protection, and personnel radiation protection. The construction materials and methods specified meet or exceed established criteria and standards and are consistent with the passive design concept assumed for the containment facility. The containment facility is designed to meet all requirements, subject to the conditions stated in this report, for a minimum of 25 years. Construction of the waste containment facility commenced in 1981 and is scheduled to be completed by the fall of 1986. On- and off-site remedial action will also be completed by fall 1986.

Since the cap and sidewalls will prevent significant seepage of moisture into the wastes, leaching of contaminants through the underlying clay bottom of the containment structure into a saturated groundwater zone will be insignificant. Modeling studies and sensitivity analyses for contaminant migration via groundwater (presented in Section 5.0 of this report) indicate that radionuclide migration is minimal for the 25- to 50-year design life of the system with no migration of contaminants, either laterally or vertically (through the clay bottom into a saturated groundwater flow zone). Radiological monitoring of air and water will be continued as necessary to verify the effectiveness of the containment system.

If the NEPA process results in a decision calling for final disposal of the waste materials at NFSS, this report will be supplemented to address the upgraded waste containment facility that would be required for long-term disposal of waste at the site.

2.0 DESCRIPTION AND STATUS OF THE NFSS

2.1 SITE DESCRIPTION

The current 77-ha (191-acre) NFSS is a small portion of the original LOOW established by the Department of the Army in the early 1940s. A 611-ha (1511-acre) portion of the original 3,035-ha (7,500-acre) LOOW tract was used by the wartime MED for the storage and transshipment of radioactive materials. As a result of these operations, some portions of the former LOOW other than the present NFSS are also contaminated. In addition, some of the radioactive materials from other sites have been subject to water and wind erosion while stored at the NFSS over the years. Consequently, radioactive materials have migrated off-site, chiefly through the on- and off-site drainage ditches. The radioactively contaminated areas adjacent to or near the NFSS are known as the vicinity properties.

The ground surface at the site is generally level, sloping gently to the northwest and lying at elevations between 97 and 99 m (317 and 323 ft) m.s.l. Access to the site is controlled by a 2-m- (7-ft-) high fence that encloses the DOE property. The facility is presently maintained by BNI, which was chosen by DOE as the Project Management Contractor (PMC) for the NFSS project in 1981.

The facility is bordered on three sides by privately owned property. To the west, property is owned by the Niagara Mohawk Power Company. To the north and northeast, the site is bordered by industrial land owned by Chemical Waste Management, Inc. (formerly SCA Chemical Services, Inc.) To the east, property is owned by S. Washuta, an individual presently developing a landfill on portions of his property. This landfill is intended for the disposal of municipal and non-hazardous industrial wastes generated primarily within Niagara County (Ref. 24). The southern boundary of the site borders property owned by the U.S. Department of Labor. Portions of these properties, and others located at the former LOOW, were contaminated. Cleanup of these vicinity properties, which has

been under way since the beginning of the 1982 construction season, falls under the DOE FUSRAP Program.

2.1.1 Site Conditions Prior to Remedial Action

The status of the radioactive contamination at the NFSS prior to the beginning of remedial action was reported by Battelle in 1981 (Ref. 7). Some remedial action had been performed at the site prior to 1981 as described in Reference 2; in 1981 contamination was present on several on-site land areas, in residues stored in various locations on the site, and in the on- and off-site drainage areas contaminated as a result of erosion and migration of radioactive materials. In addition, several vicinity properties located adjacent to or near the NFSS were known to be contaminated in excess of DOE guidelines.

These contaminated areas are described in Reference 7. Figure 2-1 shows the features of the NFSS and the structures at the site prior to initiation of remedial action.

2.1.2 Completed and Continuing Remedial Action

Remedial action at NFSS, conducted in accordance with the DOE Niagara Falls Storage Site Project Management Plan (Ref. 2), requires the control of radionuclides on the site by consolidating the residues and wastes within an engineered waste containment area.

This consolidation will provide for more stable and improved storage conditions, will minimize exposure of workers and the public to radiation, and will assure that compliance with DOE Orders and all other applicable federal and state regulations is maintained, but will not jeopardize or preclude any long-term option for the site. Justification for remedial action and alternatives to such action are discussed in the Action Description Memorandum (Ref. 23).

FIGURE 2-1 DIAGRAM OF THE NFSS SHOWING STRUCTURES AND FEATURES OF THE SITE PRIOR TO REMEDIAL ACTION

Over the past twelve years, various remedial action activities have been conducted at the NFSS. In 1972, approximately 11,500 m³ (15,000 yd³) of contaminated material were excavated from off-site areas and placed on the R-10 pile at the site. During 1981, approximately 340 m³ (450 yd³) of contaminated material were removed from another area immediately off-site and stored at the NFSS.

In 1982, BNI and its subcontractors completed work on two major remedial actions at the NFSS: stabilization of the R-10 residues, including construction of the north portion of the dike/cutoff wall and the upgrading and sealing of Buildings 413 and 414 to contain the L-50 residues. The site water treatment facility and the vehicle decontamination facility (necessary to prevent contaminated materials from being transported from contaminated to clean areas) were also developed. These facilities will continue to be used throughout remedial action activities.

During 1983, on- and off-site cleanup continued, Building 412 was demolished, and development of the waste containment area continued. The latter included southward extensions of the dike/cutoff wall surrounding the R-10 pile to enclose Buildings 410, 411, 413, 414, and 415. Work was also performed to prepare for the transfer of residues from Building 434 and their dewatering and stabilization in Buildings 410 and 411 within the waste containment area.

The 1983 cleanup decontaminated all on-site contaminated areas originally identified by Battelle (except the organics storage area), approximately 1500 m (4,800 ft) of the West Ditch, and the Central Ditch from its on-site origin to its off-site intersection with Balmer Road -- approximately 2100 m (6900 ft). Approximately 41,300 m³ (54,000 yd³) of contaminated material were excavated and placed in the waste containment area north of Building 411.

Work was also completed during 1983 on the southward extension of the dike/cutoff wall, except for a portion of the west wall left

open as future access for placing contaminated materials inside the containment area. The dike and cutoff wall are described in greater detail in Subsection 2.2.

Inside the south end of the 4.0-ha (10.0-acre) waste containment area, work in 1983 concentrated on preparing the area for future storage of contaminated materials. In addition, work was performed on several of the buildings in the waste containment area. Building 411 was prepared for the transfer, dewatering, and stabilization of residues; Building 410 was prepared for storage of water during slurry transfer operations in Building 411; Building 412 was demolished to permit construction of the southward extension of the dike/cutoff wall; and Building 415 was partially demolished to preclude future interference with the cover. Additionally, underground piping in the vicinity of Buildings 409, 410, and 411 was removed or plugged [see Drawing No. 15-DD019-C-06(2) in Appendix E].

Several major remedial actions were accomplished in 1984 and 1985. Residue transfer and dewatering operations continued, and Building 411 became a storage facility for dewatered, consolidated residues. Cap construction commenced in 1984 with the placement of a 3-ft-thick clay blanket on the northern portion of the waste containment structure, and continued with the placement of clay on the southern portion of the structure during 1985. Decontaminated water from residue and dewatering operations was periodically released from the site in accordance with New York SPDES (State Pollution Discharge Elimination System) permitting procedures.

Further demolition and cleanup work during 1984 and 1985 included completing the demolition of Buildings 410 and 415, total demolition of Buildings 434 (after removal of K-65 residues) and 423, earthwork to restore the water pond area near Building 434, construction of temporary ponds for water treatment, decontamination of Building 430, and disassembly and disposal of the K-65 slurry pipeline.

Other on- and off-site work completed by the end of 1985 includes cleanup of remaining vicinity properties and most on-site contaminated areas not previously cleaned up. Combustible organic materials (both contaminated and uncontaminated) collected during cleanup activities were buried in a location separate from the waste containment area.

Remedial actions scheduled for 1986 include completion of on-site cleanup (decontamination or demolition of Building 409 and removal of Ponds 3, 4, 5, and other on-site holding ponds); completion of the southern portion of the cap (i.e., placement of the topsoil, surface soil, remaining 2 ft of the clay layer, and vegetative cover); and site demobilization activities (such as cleanup of the decontamination facility area and demobilization of water treatment equipment). Monitoring wells and moisture detection system instrumentation will be installed.

2.1.3 Site Conditions After Completion of Remedial Action

After the completion of remedial action, the residues and radioactively contaminated soils, sediments, and building rubble will have been placed and compacted within the confines of the engineered waste containment area located in the southwest portion of the NFSS (Figure 2-2). The K-65 residues will have been transferred from Building 434 and deposited in Building 411 (within the containment facility).

Various options for the ultimate configuration of the site are being studied; details will be finalized on the basis of the Record of Decision to be issued in 1986.

2.2 WASTE CONTAINMENT AREA

The waste containment area is enclosed within a dike and cutoff wall, each constructed of compacted clay. The dike will have a minimum width of 2.4 m (8 ft) and has been constructed to a height

FIGURE 2-2 THE NFSS FOLLOWING COMPLETION OF REMEDIAL ACTION

of 1.5 m (5 ft) above grade. It rests upon the cutoff wall, which has a minimum width of 3.6 m (12 ft) and extends a minimum of 0.5 m (1.5 ft) into the gray clay unit described in Subsection 2.3.1. The permeability of the gray clay was determined by field and laboratory tests. Field-tested permeabilities ranged from 9.7×10^{-9} to 2×10^{-7} cm/s (Refs. 6 and 25). Laboratory-tested permeabilities ranged between 3.2×10^{-9} and 1.4×10^{-7} cm/s, depending upon compaction. With 95 percent compaction, the average laboratory permeability was 1.35×10^{-8} cm/s (Ref. 25). The dike and cutoff wall function as a barrier to radionuclide migration in both groundwater and surface water. Both utilize clay selected and compacted to achieve a permeability of 10^{-7} cm/s or less.

The design for the containment facility includes an engineered earthen drainage cover (cap) designed to minimize water infiltration, radon emanation, gamma radiation, erosion, and frost heave damage. A vegetative cover of shallow-rooted grasses will be established to minimize erosion. The upper layer will be compacted and contoured, as will the underlying layers, to promote runoff. The maximum elevation of the cover will be 105 m (345 ft) m.s.l. The cap will be approximately 137 m (450 ft) wide and 302 m (990 ft) long, measured between the outer edges of the dike at its intercept with ground level. The distance from the contained waste to the existing NFSS property boundaries is 75 m (250 ft) to the west, 134 m (440 ft) to the south, 440 m (1450 ft) to the east, and 425 m (1400 ft) to the north.

The cover may require some maintenance during its service life to repair damage from burrowing animals, frost heave, or other natural causes. Maintenance requirements are discussed in detail in Section 4.0.

The residues and wastes to be stored within the waste containment area are:

- o The residues and wastes presently on-site

- o Contaminated soils, sediments, and sands from vicinity properties
- o Contaminated building rubble
- o Miscellaneous debris, equipment and other materials

The locations of all of these materials prior to remedial action and after completion thereof are shown in Table 2-1. Where possible, residues were placed in concrete buildings located within the waste containment area.

2.3 ENVIRONMENTAL CONDITIONS

2.3.1 Geology and Soils

Site stratigraphy and hydrology were first described by Acres American in a 1981 report (Ref. 6). The report does not describe environmental conditions at the site prior to 1981, although it does contain a review of pertinent literature. The stratigraphy and groundwater regime of the site have also been reported by BNI (Ref. 5). Unless otherwise noted, these reports form the basis for the following brief discussion of site stratigraphy and hydrology.

Site Stratigraphy

The site stratigraphy includes 12.2 to 15.2 m (40 to 50 ft) of unconsolidated deposits overlying a thick sequence of sedimentary rock. The unconsolidated deposits are glacially derived sediments that include glaciofluvial sands and gravel, dense tills, and glaciolacustrine clays. These deposits are underlain by shale, siltstone, and mudstone of the Queenston Formation.

Six major stratigraphic units are identified within the interval from zero to 27.4 m (90 ft) below ground surface. In order of increasing depth, these units are: surficial soils and fill, brown clay, gray clay, sand and gravel, red silt, and the Queenston Formation. These are described in more detail below.

TABLE 2-1
LOCATIONS OF RESIDUES AND CONTAMINATED MATERIALS
BEFORE AND AFTER THE COMPLETION OF REMEDIAL ACTION

Material	Before Remedial Action	After Remedial Action
K-65 residues	Building 434	Bays A, C, and D of Building 411--residues dewatered and covered with clay; contaminated Building 434 rubble stored inside the waste containment area
L-30 residues	Building 411, covered with water	Bays B, C and D of Building 411-- residues dewatered and covered with clay
L-50 residues	Buildings 413, 414	Buildings 413, 414-- residues are covered with clay
F-32 residues	Recarbonation pit, covered with water	Bays B and C of Building 411--residues dewatered and covered with clay
Middlesex sands	Building 410, second level and lower level	Lower level of Building 410
R-10 residue and spoils pile	Soil surface north of Building 411	Soil surface beneath clay cover, north of Building 411, inside the waste containment area
Contaminated sediments from Central and West Drainage Ditches	On-site and off-site portions of the Central and West Drainage Ditches	Inside the waste containment area
Contaminated soils	At various locations within the NFSS	Inside the waste containment area
Contaminated demolition rubble from Buildings 410, 411, 412, 415, and 434	--	Inside the waste containment area
Contaminated soil/rubble from vicinity properties	Nearby vicinity properties	Inside the waste containment area

Surficial Soils and Fill

The surficial soil at the site is generally a loosely compacted to medium-dense brown or yellowish silt with organic matter usually present in the upper 15 cm (6 in.) (root zone). Gravel and sand are distributed throughout this unit. The thickness of the surficial soils varies between zero and 1.5 m (5 ft), averaging 0.3 to 0.6 m (1 to 2 ft). The unit is generally dry to moist and is occasionally saturated. It is described as fill where man-made materials (e.g., glass and bricks) and obvious signs of disturbance have been encountered.

Brown Clay Unit

The brown clay, which is thought to be a ground moraine, underlies the surficial soils. This unit was modified during submergence beneath glacial Lake Iroquois. Some lamination of the clay and deposition of sandy or gravelly zones within the clay resulted from this modification (Ref. 26).

Brown or reddish brown clay is predominant in this unit, and sand and silt are present in the form of seams, pockets, and lenses. Lenses composed of sandy gravel, gravelly sand, or silt are common within the basal portion of the unit. Beneath the site, the brown clay varies in thickness from about 1.5 to 7.6 m (5 to 25 ft). The consistency of the clay within the unit ranges from hard to medium soft. The clay is not saturated and exhibits low to medium plasticity with increasing depth. Isolated gravel-sized fragments are encountered within the clay portion of the unit. In general, the unit varies in classification from a clayey silt to a silty clay.

The sand, gravel, and silt lenses common within the basal portion of the brown clay average from about 0.3 to 1.5 m (1 to 5 ft) in thickness, with abrupt changes in the thickness and lateral extent of the lenses. The sediments in these lenses are damp to saturated and vary from loosely compacted to dense. More extensive sand and

gravel deposits which are 6.1 m (20 ft) thick also occur as random deposits from high energy (high velocity stream flow) deposition in stream channels.

Gray Clay Unit

The naturally occurring, highly impermeable gray clay unit forming the bottom of the waste disposal cell, apparently of lacustrine (freshwater lake) origin, was deposited in the deeper portions of glacial Lake Iroquois (Ref. 26). Some post-depositional erosion of the unit occurred, as is evidenced by the channels along its upper surface, which are filled with coarser-grained sediments of the overlying brown clay unit.

In general, this unit is the thickest unconsolidated unit on-site, varying from about 2.7 to 8.8 m (9 to 29 ft) in thickness. It is characterized by gray, fairly plastic clay that occasionally grades vertically to a silt and clay mixture. Within the unit, pockets of fine- to medium-grained sand exist. Isolated fine to coarse gravels are dispersed throughout the deposit.

Sand and Gravel Unit

Sands and gravels become the predominant constituents of the gray clay unit near the base of the unit; locally, they are clean and lighter gray in color (Ref. 6).

Red Silt Unit

The red silt is distinguished from the sand and gravel base of the overlying gray clay unit by its color and high density. Typically, it is composed of clayey, gravelly silt with lesser amounts of sand. Gravel is dispersed throughout this unit and consists of both rounded gravel and angular fragments of bedrock. Sand and gravel become the predominant constituents near the base of the unit (Ref. 26). The unit is dry to moist, overconsolidated, and its degree of

compaction ranges from medium to very dense. The moisture content of the unit is not seasonally variable. The thickness of the unit varies across the site; in some locations the unit is absent.

Queenston Formation

The bedrock unit at the site consists of reddish brown shale, siltstone, and mudstone of the Queenston Formation. Within this formation, occasional lenses of green siltstone and shale occur. The upper 1 m (3 ft) or so of the bedrock is slightly to moderately weathered.

2.3.2 Groundwater Hydrology

Three water-bearing zones exist at the site: the intermittent sand, gravel, and silt lenses found in the brown clay unit between elevations of 91.5 and 96.6 m (300 and 317 ft) m.s.l. (upper aquifer); the sand and gravel unit at the base of the gray clay unit, typically between elevations of 82.3 and 91.4 m (270 and 300 ft) m.s.l. (lower aquifer); and the upper part of the Queenston Formation (bedrock aquifer). The top elevation of the latter varies from 83.8 to 86.9 m (275 to 285 ft) m.s.l. beneath the NFSS (Ref. 6).

Upper Aquifer

For purposes of this discussion, the sand zones in the brown clay unit are referred to as the upper aquifer. These zones are discontinuous, however, and do not actually form an aquifer in the usual sense. The sand, gravel, and silt lenses and pockets within the brown clay unit range from dry to saturated. Those that contain water yield only small volumes. There are no known continuous lenses across the NFSS. More detailed information can be found in the boring logs and cross sections presented in the Geologic Report, Niagara Falls Storage Site (Ref. 5).

Lower Aquifer

The sand and gravel that form the base of the gray clay unit are less than 3 m (10 ft) thick and form the lower aquifer, the top of which lies between elevations 82.3 and 91.4 m (270 and 300 ft) m.s.l. The lithology and thickness of this aquifer vary across the site; in several locations the aquifer is absent. Facies changes involving clay, silt, sand, and gravel are common. The proportion of the aquifer comprising gravel ranges from zero to about 40 percent (Ref. 6).

Measurements of water levels in wells constructed during the Acres American, Inc. exploration program indicate that the aquifer is confined. Water level measurements taken in July 1981 show that the piezometric surface lies at somewhere between 95.4 and 96.3 m (313 and 316 ft) m.s.l., which is 7.6 to 9.1 m (25 to 30 ft) above the top of the aquifer. Seasonal water level fluctuations are shown on the hydrographs in the Geologic Report, Niagara Falls Storage Site (Ref. 5).

Bedrock Aquifer

The top of the Queenston Formation is weathered and fractured. Groundwater movement in this formation is primarily in this weathered zone. Water levels recorded by BNI in April 1983 indicate that the groundwater is confined by the red silt unit (where it exists) or by the gray clay unit (where the red silt unit is absent). The bedrock aquifer has a piezometric surface about 11.3 m (37 ft) above the top of the aquifer. Water levels in wells drilled into the bedrock beneath the NFSS coincide with the piezometric contours of the lower aquifer. Thus, at least locally, the lower aquifer and bedrock aquifer are hydraulically connected (Ref. 6).

2.3.3 Surface Water Hydrology

During the construction of the LOOW in the 1940s, a system of ditches was installed to drain surface waters to the Central

Drainage Ditch. Sixmile Creek, which originally flowed through the NFSS, was diverted at a point just outside the southwest corner of the NFSS boundary into the Southwestern Drainage Ditch. Surface waters from the south of the NFSS, which originally flowed eastward into Twelvemile Creek, were diverted to the South-31 Ditch. Figure 2-3 depicts the surface water drainages at the site. An inactive slurry pond is located southwest of and adjacent to the NFSS. There is an abandoned sewage treatment plant approximately 250 ft northwest of the site. Both facilities are shown on Figure 2-3.

Currently, runoff from the NFSS is diverted into three major drainage ditches: the Central Drainage Ditch, the West Ditch, and the South-31 Ditch. The West and South-31 Ditches discharge directly into the Central Drainage Ditch, which flows northward into Fourmile Creek and ultimately into Lake Ontario (Ref. 6). Sixmile and Twelvemile Creeks do not receive runoff from the NFSS.

On the site, the Central Drainage Ditch is approximately 3 to 4 m (10 to 15 ft) deep, 3 to 6 m (10 to 20 ft) wide at the base, and 12 to 15 m (40 to 50 ft) wide at the top. It is about 5 km (3 mi) long from its origin at the south end of the NFSS to its confluence with Fourmile Creek. The West Ditch, approximately 1.6 km (1 mi) in length, runs parallel to the Central Drainage Ditch and drains the western portion of the NFSS before it joins the Central Drainage Ditch. The only major off-site contributor to flow across the site is the South-31 Ditch, which flows into the Central Drainage Ditch just south of Building 411. The South-31 Ditch drains an off-site area of approximately 81 ha (200 acres) and is about 427 m (1,400 ft) in length (Ref. 6).

The 100-year flood level is estimated to be approximately 96.5 m (317 ft) m.s.l. in the vicinity of the R-10 pile. This estimate is based on the proposed remedial action plan and the present topographic condition. However, the estimated 100-year flood-peak discharges and subsequent flood level may change as a result of

FIGURE 2-3 SURFACE WATER DRAINAGE PATHWAYS AT THE NFSS

development of the sanitary landfill by S. Washuta (Modern Landfill, Inc.) in the South-31 drainage area. This development will likely alter the hydrologic response of the watershed so as to increase the peak of the 100-year storm runoff.

The Probable Maximum Flood (PMF) is the most severe flood that is reasonably possible or can reasonably be predicted to occur (Ref. 27). The PMF level for the NFSS has been estimated to be 98.4 m (323 ft). The PMF level at the site would remain above 97.7 m (320.7 ft) for a duration of 300 min during the passage of the PMF due to a 6-hour Probable Maximum Precipitation (PMP) event. An all-season 6-hour PMP value of 61.9 cm (24.0 in.) was estimated for the site using methodology developed by the U.S. National Weather Service. The methodology used to develop the estimated PMF and PMP values is described in Appendix C.

A flood peak of $1,146 \text{ m}^3/\text{s}$ ($40,500 \text{ ft}^3/\text{s}$) due to combined runoff to Sixmile and Twelvemile Creeks was calculated. The flood would produce a velocity near the dike of 1.5 m/s (5 ft/s).

Figure 2-2 shows the location of the waste containment area at completion of remedial action. A dike with a crest elevation of 99.1 m (325.25 ft) m.s.l. surrounds the containment structure. Because the probability of occurrence of the PMF is approximately 10^{-6} , the likelihood of a flood ever overtopping the dike is extremely remote.

The all-season, site-specific PMP was derived using the U.S. National Weather Service Hydrometeorological Reports 51 and 52 (Refs. 28 and 29). The PMP was established as 24.0 in. for a 6-hour storm. The seasonal variation of the 6-hour PMP was derived from Hydrometeorological Report 53 (Ref. 30).

The 6-hour PMP was distributed into 5-minute increments based on a triangular distribution following the procedure given in the U. S. Army Corps of Engineers program, HEC-1 (Ref. 31). The distribution

preserves the maximum 5-min, 15-min, 30-min, and the 1-hour PMP values derived from Hydrometeorological Report 51 (Ref. 28).

2.3.4 Meteorology

The climate in the extreme western portion of New York is continental in nature with moderately cold winters and brief warm summers. The NFSS lies within the middle latitude westerlies, so that weather systems generally move eastward and northeastward through the region.

Temperature extremes in the NFSS area are mollified slightly by the proximity of Lake Ontario and Lake Erie. Temperature data collected for Lewiston, New York from 1936 to 1972 indicate a mean monthly maximum of 3.6°C (38.5°F) for the winter months (December through March) and 24.5°C (76.1°F) for the summer months (June through September). Annual mean precipitation is 74 cm (29 in.). Precipitation is fairly evenly distributed over the year, with the maximum average monthly rainfall occurring in late summer and early autumn (Ref. 32). Precipitation and evapotranspiration are discussed further in Subsection 5.5.1. The annual snowfall (occurring primarily between November and March) is approximately 236 cm (93 in.).

The effect of snow on the PMP was also investigated. Average snowfall in the Buffalo-Niagara area varies from 56 cm (22 in.) in January to 8 cm (3 in.) in April with an annual average depth of 236 cm (93 in.) (Ref. 32). The contribution of snow to the PMP during the winter months would not create a PMP event more critical than the all-year PMP.

The prevailing wind direction (obtained from a meteorological station at the site) is from the southwest. Wind speeds are predominantly in the 13- to 18-km/h (7- to 10-knot) range.

The air temperature freezing index at the site is not well defined by published maps because of the moderating influence of the Great Lakes. The mean freezing index is approximately 500 degree-days. The index for the coldest year in 30 years is approximately 1,100. Extrapolation of available data indicates a freezing index of 1,300 for the coldest year in 1,000 years.

2.3.5 Seismicity

The seismicity of the site is described in detail in Appendix A. The peak ground acceleration corresponding to a 500-year return period is 0.1 g. The peak ground acceleration for a 2,400-year return period is approximately 0.15 g. The area falls within Zone 3 as delineated by the Uniform Building Code (UBC). Seismic criteria applicable to the design of the waste containment area are listed in Section 3.0.

3.0 DESIGN REQUIREMENTS AND ASSUMPTIONS

3.1 RADIOLOGICAL CRITERIA

The radiological guidelines for performance of the waste containment structure are based on multiple sources and consolidated in Appendix C of Design Criteria for Formerly Utilized (MED/AEC) Sites (Ref. 3), which incorporates DOE Orders and EPA regulations as well as codes, standards, and guidelines established by industry or by other federal entities (References 8-9 and 13-22, 33). The cap performance objectives apply to the short-term construction/operational/site closure phase and the long-term, post-closure phase, and are based on site acceptance criteria as well as design criteria. Current DOE guidelines are summarized in Table 3-1.

3.2 DESIGN BASIS

The design basis for the containment facility includes design requirements for the bottom, sidewalls, and cap of the facility. The design requirements for the cap assume the 25- to 50-year service life currently contemplated; the requirements for the bottom, dike, and cutoff walls of the facility support a long-term service life (200 to 1,000 years). These design requirements are defined in Table 3-2.

The cap consists of a low permeability clay blanket (to promote surface water runoff and to prevent radon emanation and gamma radiation from the wastes) overlain by a layer of soil, which is in turn overlain by a layer of topsoil. The soil layers are compacted to a lesser degree than the clay. The topsoil cover will be planted with shallow-rooted grass. Slopes are designed to promote surface drainage and runoff, thereby preventing saturation and prolonged wet conditions. The topsoil and grass will also provide an insulating cover to limit the depth of frost penetration into the clay blanket (although they will not prevent freezing of the surface layer or clay blanket), restrict shrinkage and swelling of the blanket, and

TABLE 3-1

SUMMARY OF RESIDUAL CONTAMINATION GUIDELINES FOR THE NFSS WASTE CONTAINMENT DESIGN

Page 1 of 2

SOIL (LAND) GUIDELINES (MAXIMUM LIMITS FOR UNRESTRICTED USE)

<u>Radionuclide</u>	<u>Soil Concentration (pCi/g) above background^{a,b,c}</u>
Radium-226	5 pCi/g, averaged over the first 15 cm of soil below the surface; 15 pCi/g when averaged over any 15-cm-thick soil layer below the surface layer.
Radium-228	
Thorium-230	
Thorium-232	
Other radionuclides	Soil guidelines will be calculated on a site-specific basis using the DOE manual developed for this use.

STRUCTURE GUIDELINES (MAXIMUM LIMITS FOR UNRESTRICTED USE)Airborne Radon Decay Products

Generic guidelines for concentrations of airborne radon decay products shall apply to existing occupied or habitable structures on private property that are intended for unrestricted use; structures that will be demolished or buried are excluded. The applicable generic guideline (40 CFR 192) is: In any occupied or habitable building, the objective of remedial action shall be, and reasonable effort shall be made to achieve, an annual average (or equivalent) radon decay product concentration (including background) not to exceed 0.02 WL.^d In any case, the radon decay product concentration (including background) shall not exceed 0.03 WL. Remedial actions are not required in order to comply with this guideline when there is reasonable assurance that residual radioactive materials are not the cause.

External Gamma Radiation

The average level of gamma radiation inside a building or habitable structure on a site to be released for unrestricted use shall not exceed the background level by more than 20 μ R/h.

Indoor/Outdoor Structure Surface Contamination

<u>Radionuclide^f</u>	<u>Allowable Surface Residual Contamination^e</u> <u>(dpm/100 cm²)</u>		
	<u>Average^{g,h}</u>	<u>Maximum^{h,i}</u>	<u>Removable^{h,i}</u>
Transuranics, Ra-226, Ra-228, Th-230, Th-228 Pa-231, Ac-227, I-125, I-129	100	300	20
Th-Natural, Th-232, Sr-90, Ra-223, Ra-224 U-232, I-126, I-131, I-133	1,000	3,000	220

TABLE 3-1
(continued)

Page 2 of 2

Indoor/Outdoor Structure Surface Contamination (continued)

<u>Radionuclide</u> ^f	<u>Allowable Surface Residual Contamination^e</u> (dpm/100 cm ²)		
	<u>Average</u> ^{g,h}	<u>Maximum</u> ^{h,i}	<u>Removable</u> ^{h,j}
U-Natural, U-235, U-238, and associated decay products	5,000 α	15,000 α	1,000 α
Beta-gamma emitters (radionuclides with decay modes other than alpha emission or spontaneous fission) except Sr-90 and others noted above	5,000 β - γ	15,000 β - γ	1,000 β - γ

^aIn the event of occurrence of mixture of radionuclides, the fraction contributed by each radionuclide to its limit shall be determined, and the sum of these fractions shall not exceed 1.

^bThese guidelines represent unrestricted-use residual concentrations above background averaged across any 15-cm thick layer to any depth and over any contiguous 100-m² surface area.

^cLocalized concentrations in excess of these limits are allowable provided that the average over 100 m² is not exceeded.

^dA working level (WL) is any combination of short-lived radon decay products in 1 liter of air that will result in the ultimate emission of 1.3×10^5 MeV of potential alpha energy.

^eAs used in this table, dpm (disintegrations per minute) means the rate of emission by radioactive material as determined by correcting the counts per minute observed by an appropriate detector for background, efficiency, and geometric factors associated with the instrumentation.

^fWhere surface contamination by both alpha- and beta-gamma-emitting radionuclides exists, the limits established for alpha- and beta-gamma-emitting radionuclides should apply independently.

^gMeasurements of average contamination should not be averaged over more than 1 m². For objects of less surface area, the average shall be derived for each such object.

^hThe average and maximum radiation levels associated with surface contamination resulting from beta-gamma emitters should not exceed 0.2 mrad/h and 1.0 mrad/h, respectively, at 1 cm.

ⁱThe maximum contamination level applies to an area of not more than 100 cm².

^jThe amount of removable radioactive material per 100 cm² of surface area should be determined by wiping that area with dry filter or soft absorbent paper, applying moderate pressure, measuring the amount of radioactive material on the wipe with an appropriate instrument of known efficiency. When removable contamination on objects of surface area less than 100 cm² is determined, the activity per unit area should be based on the actual area and the entire surface should be wiped. The numbers in this column are maximum amounts.

TABLE 3-2
NFSS WASTE CONTAINMENT FACILITY DESIGN REQUIREMENTS

Page 1 of 2

Item No.	Description	Cap	Dike and Cutoff Walls	Bottom	Remarks
1.	Design Service Life	25-50 years	200-1000 years	200-1000 years	
2.	Safety Factor for Sidewalls				
	Slope Stability:				
	Static conditions	1.5	1.5		
	Earthquake	1.0	1.0	--	
3.	Surface Drainage Slope				
	Top Surface	5-10 %	--	--	
	Side Slopes	Max. 3.0 H to 1.0 V	--	--	
4.	Surface Erosion Protection	Shallow-rooted grass	--	--	
5.	Intrusion Barrier Required	No	--	--	
6.	Frost Penetration	48 in.	--	--	Assume bare ground
7.	Radon Barrier Required	Yes (20 pCi/m ² /s)	--	--	
8.	Radiation Barrier Required	Yes (500 mrem/yr)	--	--	
9.	Component Construction	Topsoil/clay	Clay	Natural clay strata	
10.	Clay Permeability (range of value)	10 ⁻⁷ cm/s	Approx. 10 ⁻⁷ cm/s	Approx. 10 ⁻⁷ cm/s	
11.	Clay Adsorption Coefficient				
	Natural Uranium	5 ml/g	5 ml/g	5 ml/g	
	Radium-226	500 ml/g	500 ml/g	500 ml/g	
12.	Inspection and Maintenance Required	Yes (design life)	No	No	
13.	Earthquake Pseudostatic Coefficient	0.1 g	0.1 g	--	
14.	DOE Derived Concentration Guide for Radionuclide Migration (Groundwater concentration in drinking water)				
	Natural Uranium	--	600 pCi/l	600 pCi/l	
	Radium-226, -228	--	100 pCi/l	100 pCi/l	

TABLE 3-2
(continued)

Page 2 of 2

Item No.	Description	Cap	Dike and Cutoff Walls	Bottom	Remarks
15.	Temperature Extremes	-29° to 34°C (-20° to 94°F)	-29° to 34°C (-20° to 94°F)	--	
16.	Rainfall per Year	74 cm (29 in.)	--	--	
17.	Wind Speed and Direction	(80 mph) southwest	--	--	
18.	Annual Deep Infiltration Rate	2.54 cm (1.0 in.)	--	--	
19.	Design Flood Plain Elevation	Elevation 96.6 m (317 ft) m.s.l. per 100 Years	Probable Maximum Flood (PMF) 98.4 m (323 ft.)		
20.	Groundwater Elevation (high)	--	--	Elevation 96 m (315 ft) m.s.l. (Exclusive of PMF)	
21.	Snowfall per Year	2.4 (93 in.)	--	--	
22.	Internal Cap Drainage Layer	None	--	--	
23.	Waste Containment	Minimize settlement (95% compaction)	90-95% compaction	--	
24.	Shrinkage, Swelling, and Expansion	Yes (3 to 5% in volume)	Yes (3 to 5% in volume)	No	
25.	Migration Limits	Not to exceed EPA primary drinking water standards in off-site groundwater	Not to exceed EPA primary drinking water standards in off-site groundwater	Not to exceed EPA primary drinking water standards in off-site groundwater	
26.	Buffer Zone (measured from lateral limit of waste)	30.5 m (100 ft)	--	--	
27.	Groundwater Hydraulic Gradient	--	--	0.0015	

promote evaporation and transpiration of surface water.

The dikes and cutoff walls constructed around the waste material provide an adsorption barrier against horizontal radionuclide migration. The sidewalls penetrate the gray clay unit that serves as the bottom of the containment system.

The containment facility is designed to function as a passive system to the extent possible. However, as discussed in Subsection 3.3, routine inspection and maintenance will be required to monitor for conditions which might compromise the integrity of the containment facility if not corrected.

3.3 POST-CLOSURE MONITORING AND SURVEILLANCE

The post-closure site monitoring and surveillance program will be conducted in accordance with DOE Order 5480.1A (Ref. 9). Elements of the post-closure monitoring and surveillance program are summarized below. A detailed description of the program can be found in the draft **Closure/Post-Closure Plan for the Interim Waste Containment Facility at the Niagara Falls Storage Site** (Ref. 10) and the draft **Environmental Monitoring Plan for the Niagara Falls Storage Site and the Interim Waste Containment Facility** (Ref. 11).

Regularly scheduled environmental monitoring of both surface and subsurface areas will be conducted, as will monitoring of the integrity of the containment structure. Regular inspection of the fence and routine maintenance will be performed, and security patrols will be conducted on a regularly scheduled basis.

Environmental monitoring conducted for the waste containment area will be based on cumulative monitoring data and other relevant data available at completion of the cover. Monitoring devices to detect radon emanation and external radiation will be placed along the perimeters of the containment area. Specific locations will include those with localized concentrations of contaminated wastes (e.g., buried Building 411) and other areas to be determined on the basis

of meteorological data. Groundwater monitoring measures will include placement of monitoring wells in both the upper and lower alluvial aquifers around the perimeter of the cutoff wall. Additional wells will be placed at the site boundary, and samples will be taken from the nearest private well on a regularly scheduled basis. Initially, sampling will be performed on a quarterly basis. The frequency of sampling might later be reduced to annually, depending on sampling results. Air and groundwater monitoring plans will be finalized at a later date.

During quarterly surveillance activities, the health physics program will be minimal. However, during the maintenance program, a greater emphasis will be placed on health physics, and individual radiation exposure and radiological survey data will be collected.

During surveillance, the environmental monitoring program will provide data to determine whether the site is in compliance with applicable regulations with regard to exposure rates both on the site and in adjoining areas, and with respect to the effectiveness of the waste containment barriers. Environmental monitoring would be performed by a subcontractor; analytical services support would be performed at an off-site location. Monitoring data would be submitted to the responsible DOE Operations Office. Environmental monitoring data would be respectively collected, analyzed, and reported on the following bases:

- o Exposure rates: continual monitoring, quarterly analysis, annual reporting
- o Air: continual monitoring (for radon-222), quarterly analysis, annual reporting
- o Water: continual monitoring of groundwater and surface water for radium-226 and uranium-238, quarterly analysis, annual reporting

A detailed description of the activities that will be performed as part of the maintenance and surveillance program to monitor the performance of the containment system can be found in the **Report on**

the Performance Monitoring System for the Interim Waste Containment at the Niagara Falls Storage Site (Ref. 12). The performance monitoring system will detect evidence that some failure mode is occurring or has occurred, thereby providing for corrective action to be taken before the integrity of the containment structure is threatened.

A summary of the major activities associated with post-closure monitoring and surveillance follows:

- o Monitoring for saturation of the waste
- o Monitoring of the cap for subsidence, cracks, weathering, or other integrity problems
- o Inspection and maintenance of the fence around the containment facility and of the vegetative cover on the cap
- o Upkeep of roads and accesses to sampling stations
- o Upkeep of support facilities
- o Replacement of security fence (approximately every 25 years)

Repairs and maintenance will be conducted as required in order to meet performance standards established for the facility.

Inspections will include visual checks (which will be made during mowing of the grass cover) for surface erosion, shrinkage cracks, animal burrows, and deep-rooted vegetation. The top of the containment system will be visually checked for depressions and monitored by using survey methods to establish subsidence trends. Aerial photographs will be used to identify areas of vegetation stress and localized subsidence or ponding. The cover will be inspected each fall and each spring (after the last frost), and will be recompactd or reworked if biotic intrusion, settlement, or freeze-thaw effects are found to have caused damage.

Access to the waste containment area will be restricted by security fences, which will be inspected quarterly. A local law enforcement agency will patrol the site.

4.0 CONTAINMENT FACILITY DESIGN

4.1 SIDEWALLS AND BOTTOM

4.1.1 Sidewalls

The sidewalls of the containment system consist of a compacted clay dike and cutoff wall constructed around the waste containment area. The dike has a minimum width of 2.4 m (8 ft) and extends approximately 1.5 m (5 ft) above the original grade. It rests on the cutoff wall, which has a minimum width of 3.6 m (12 ft) and extends at least 0.5 m (1.5 ft) into the gray clay unit described in Subsection 2.3.1. The height of the cutoff wall beneath the dike ranges between 3 and 7 m (10 and 22 ft), varying with changes in the elevation of the top of the gray clay. In general, the cutoff wall is not centered beneath the dike; rather, its location varies according to subsurface conditions.

The construction of the cutoff walls is based on the results of data collected during geologic investigations of the dike foundations conducted by BNI in 1982. The investigations showed that extensive, discontinuous sand deposits existed within the brown clay unit beneath the area where the dikes would be placed. These deposits were determined to be isolated lenses. Even though the sand lenses are not continuous, they could provide pathways for contaminant migration from non-engineered containment areas (Ref. 5). For this reason, a clay cutoff wall was constructed beneath the dike to provide an engineered barrier to migration of contaminants from the containment area. The dike and cutoff wall were constructed around the north end of the containment area in 1982 and around the south end in 1983. The earthwork specification for the 1983 construction is provided in Appendix D to this document. Design drawings for both phases of the work are provided in Appendix E.

The dikes and cutoff walls function as adsorption barriers to radionuclide migration from groundwater and surface water entering

the waste containment area. They were constructed of clay placed in loose lifts of 20 cm (8 in.) and compacted to greater than 90 percent of maximum dry density for the cutoff wall and 95 percent for the dike [American Society for Testing and Materials (ASTM) test procedure D 1557]. The actual degree of compaction, measured by field density testing, averaged nearly 94 percent for the cutoff wall around the northern half of the containment area and nearly 97 percent for the dike and the southern portion of the cutoff wall. The specifications for material properties of the clay and for compaction requirements were established to achieve a permeability of 10^{-7} cm/s for the dike and the cutoff wall. At the time of construction, the dike and cutoff wall both exhibited permeabilities of this order (Ref. 25).

The optimum width of the cutoff wall was based on two main considerations: results of a one-dimensional migration analysis, and the requirement for accessibility to construction equipment compacting the clay in the cutoff wall. The results of the migration analysis showed that a 3.6 m (12 ft) thick cutoff wall is more than adequate for the 25- to 50-year design life of the containment system. These findings were confirmed by a more sophisticated numerical migration analysis of the complete containment system, which is presented in Section 5.0 of this report.

4.1.2 Bottom

The bottom of the waste containment system consists of 1.8 to 7 m (6 to 23 ft) of naturally occurring brown clay in most areas, underlain by 3.3 to 8.8 m (11 to 29 ft) of gray clay. Because of the existence of sand deposits in the brown clay, the gray clay is considered to be the primary migration barrier. It has a permeability that ranges from 9.7×10^{-9} to 2×10^{-7} cm/s.

The number of holes drilled through the gray clay unit was kept to a minimum to avoid compromising its integrity. The unit appears to be

continuous in the on-site areas investigated (Refs. 5 and 6). Borings drilled during BNI investigations revealed only traces of sand and gravel in the upper 1.2 to 1.5 m (4 to 5 ft) of the gray clay, and an occasional small sand and silt lens. Additional geologic characterization of the gray clay layer will be performed during the process of drilling and installation of monitoring wells for the monitoring and surveillance program. These data will be presented in an addendum to the BNI geological report (Ref. 5) when they become available (late 1986 or early 1987).

The combination of the brown clay unit and more uniform gray clay unit will function as an effective barrier to vertical radionuclide migration. Their effectiveness is documented in Section 5.0, where the effect of possible holes or discontinuities in the gray clay is also assessed.

4.1.3 Waste Placement

Waste materials were placed in the containment structure in a manner that will prevent subsidence of the completed structure. Specifically, waste materials were placed in layers and compacted to 90 percent of maximum dry density. Rubble materials were deposited in layers and the voids grouted with fillcrete to create a consolidated, dense mass. Voids between machinery and piping were also grouted to prevent subsidence. Contaminated organic materials were not placed within the waste containment facility; rather, they were buried in a separate area to the north of the containment structure where subsidence is not a consideration.

4.2 CAP

4.2.1 Description of the Cap

The top layer of the cap consists of 0.5 m (1.5 ft) of loosely compacted soil, which acts as a protective cover for the low permeability clay blanket underlying it. The 0.9-m- (3-ft-) thick clay blanket, which is the principal barrier against moisture and

radioactive contaminant migration, is compacted to achieve a permeability of approximately 10^{-7} cm/s. No specific barrier to deep-rooted plants and burrowing animals is provided, since it is assumed that any damage caused by either of these will be detected and repaired in the course of periodic inspection and maintenance activities. Additional features of the cap are discussed in the following sections. The earthwork specification and the design drawings for the portion of the cap constructed from 1982 through 1985 are provided in Appendices D and E, respectively. As-built drawings of the final configuration of the cap will be provided in a supplemental design report to be issued after the end of the 1986 construction season. Figures 4-1 through 4-3 are conceptual design sketches of the completed cap.

Surface Layer

The top layer consists of 0.15 m (0.5 ft) of topsoil and 0.3 m (1.0 ft) of soil. Its purposes are to support the growth of shallow-rooted grass, to provide a protective cover for the clay blanket, and to furnish additional resistance to infiltration of surface water. These protective features are designed to augment the effectiveness of the cover, and are not considered in determining the required thickness of the underlying clay blanket.

The top layer and vegetation will absorb that part of normal rainfall which does not leave the surface as runoff, and will permit the absorbed water to reenter the atmosphere by means of evaporation and transpiration. It will be graded so as to minimize infiltration and promote runoff. Very little moisture is expected to percolate into the clay blanket. The topsoil and vegetation will also serve as insulation but will not prevent freezing of the surface layer or clay blanket. Infiltration of precipitation is discussed in detail in Subsection 5.5.1.

FIGURE 4-1 PLAN OF THE COMPLETED COVER

FIGURE 4-2 SECTION G AND DETAIL I OF THE COVER

FIGURE 4-3 SECTIONS OF THE COVER

Low Permeability Clay Blanket

The 0.9-m- (3-ft-) thick clay blanket provides the primary barrier against radiation and infiltration of surface water. To function properly, it must remain intact and free from damage that might compromise its integrity.

The computer simulation study reported in Section 5.0 indicates that a 0.75-m- (2.5-ft-) thick blanket of clay with a permeability of 10^{-7} cm/s will prevent saturation of the waste for at least 50 years. The study was based on the conservative assumption that the blanket would be subjected to continuous ponding to a depth of 15 cm (6 in.). An estimate of infiltration from expected rainfall indicates that a very small quantity of water would penetrate through an intact blanket. The estimated infiltration would not saturate the waste in a 50-year period.

A 0.9-m- (3-ft-) thick clay blanket has been provided to ensure that the integrity of the cover is maintained during construction. It will consist of medium- to low-plasticity clays with the following additional properties:

o Liquid limit	30 to 50 percent
o Plasticity index	10 to 20 percent
o Maximum particle size	3/4 in.
o Minimum amount passing
no. 200 sieve	70 percent

The clay will be devoid of organic matter, gravel, rocks and other deleterious materials. Its moisture content will be slightly greater than optimum, and it will be compacted to the degree necessary to achieve a permeability equal to or less than 10^{-7} cm/s. It has been demonstrated that a density equivalent to 95 percent of the maximum density determined by the ASTM D 1557 test procedure will achieve the required permeability. The field

permeability test performed on in-place clay cap materials in September of 1985 indicated a permeability of 5×10^{-7} cm/s. Field density tests will be performed on clay cap materials during the winter of 1986 to determine the effect of frost penetration on in-place density.

4.2.2 Side Slope Stability

Maximum side slopes of 33 percent (3.0H to 1.0V) will be constructed in areas where there are lateral restraints such as would be imposed by the proximity of the Central Drainage Ditch. Twenty percent slopes will be constructed in areas where flatter slopes would not interfere with the existing ditch, road, or structures.

The stability of the side slopes under static and seismic forces was assessed by performing analyses of 20 percent (5.0H to 1.0V), 25 percent (4.0H to 1.0V), and 33 percent (3.0H to 1.0V) slopes for heights of 3 and 6 m (10 and 20 ft). The static conditions were analyzed using the methods presented in Reference 34. Seismic conditions were checked by pseudostatic methods using the Simplified Bishop Method.

The following parameters were used for the compacted clay:

Shear strength:

-effective cohesion

$$c' = 976 \text{ kg/m}^2 \text{ (200 lb/ft}^2\text{)}$$

-effective angle of internal
friction

$$\phi = 25^\circ$$

Total unit weight

$$W = 2160 \text{ kg/m}^3 \text{ (135 lb/ft}^3\text{)}$$

The internal friction shear strength parameter is based on correlations of angles of internal friction for clay soils with plasticity indices for clay soils. The plasticity indices used range from 10 to 20 percent as specified for clay material. The cohesion parameter was assumed to be conservative for compacted

clay. The total unit weight was calculated from data reported for the existing dikes.

The results of the stability analyses for the design static and seismic events are as follows:

Height of Slope	Factor of Safety			
	Static		Earthquake (0.1g)	
	3m (10 ft)	6m (20 ft)	3m (10 ft)	6m (20 ft)
Slope Inclination				
3H to 1V (33%)	2.8	2.3	≥ 2.0	2.0
4H to 1V (25%)	3.1	2.7	≥ 2.0	≥ 2.0
5H to 1V (20%)	3.4	3.2	≥ 2.0	≥ 2.0

The factors of safety for slopes with 3- and 6-m (10- and 20-ft) heights and 3H to IV inclination are greater than the 1.5 criterion for the static condition specified in Table 3-2 and are considered satisfactory. The factor of safety for the 6-m (20-ft) high, 3H to 1V slope under earthquake conditions is greater than the 1.0 criterion and is considered satisfactory. Flatter slopes would have a higher factor of safety than the 3H to 1V slope given the same construction methods and materials.

In addition to the pseudostatic analyses, Newmark's method (Ref. 34) was employed to determine slope stability. Peak ground accelerations of 0.1 g and 0.15 g were used and were found to be of insufficient magnitude to initiate slope deformation.

4.2.3 Surface and Cap Drainage

The top of the containment facility will be graded to a 5- to 10-percent slope to promote runoff without excessive retention or erosion. Surface drainage will be collected in a perimeter ditch at the toe of the slope; the ditch will also be sloped to promote proper drainage. No settling of the waste is anticipated; however, if settling and consequently ponding or retention of runoff occurs, the surface would be regraded to provide positive drainage.

Placement of a thin layer of sand or sand and gravel to permit drainage between the topsoil and the clay blanket was considered. However, it was determined not to be required because only a very small quantity of moisture is expected to percolate from the topsoil and soil layers to the clay blanket.

The installation of an impervious synthetic membrane below the clay was also considered but was determined not to be required since the clay is expected to provide an adequate barrier against the entry of water and the emanation of radon from buried radium-bearing wastes. Furthermore, practical considerations would dictate that at least 30 cm (12 in.) of protective loose clay material be placed on the membrane before reasonably sized equipment could operate safely without jeopardizing its integrity. The placement of clay in thin layers and compaction could then proceed on top of this protective lift. The loose clay lift layer would not meet compaction and 10^{-7} cm/s permeability criteria. It could also act as a porous medium in which moisture could accumulate above the impervious, membrane, thereby increasing the potential freeze problem and possible development of ice lenses. Both would detract from the quality and effectiveness of the clay blanket.

Surface slopes covered with a good growth of shallow-rooted grass will be adequately resistant to surface erosion as the result of rainfall. A slope of 20 percent (5.0H to 1.0V) is considered to be sufficient protection against surface erosion provided that a complete ground cover of grass is maintained on the slope. Slopes of 33 percent (3.0H to 1.0V), which may exist on the sides of the cap, may be expected to experience some gullying, and maintenance will consequently be required.

4.2.4 Shrinkage, Swelling, and Frost Action

Some seasonal shrinkage and cracking of the topsoil are anticipated due to desiccation. The cracks are not expected to extend far enough into the clay layer to jeopardize the integrity of the

cover. All cracks are expected to close when the soil is moistened and during the winter season. If cracking becomes excessive and extends too far into the clay, measures will be taken to repair the cracks as part of the maintenance program.

The freezing index for the coldest year in 30 years (Ref. 35) is approximately 1,100. This value is considered appropriate for use in evaluating the cap. Assuming an all-clay profile, the calculated depth of freezing based on procedures described in Reference 35 is 1.1 m (3.6 ft) for bare ground and 0.7 m (2.3 ft) for ground covered with turf. Generalized charts in Reference 36 indicate 0.6 m and 0.5 m (2 ft and 1.5 ft) of frost penetration in bare ground and in an area containing turf, respectively. The UBC indicates 1.2 m (4 ft) of frost penetration in this area. While it is not certain that the clay blanket would freeze during a period of extremely low temperatures, for purposes of evaluation it is assumed that it would do so.

Depending on water content at the time of freezing, the soils are expected to expand 3 to 5 percent. This freezing could result in some permanent reduction in unit weight of the compacted clay blanket and some increase in water content over a period of time. Neither frost action nor ice lensing in the blanket is anticipated due to the absence of a saturated groundwater zone in proximity to the blanket and the low susceptibility of clay to the formation of ice lenses. The impermeable nature of the clay inhibits transfer of moisture toward a frost boundary. Some minor ice lensing may develop in the surface layer.

The surface layer on the top of the waste pile, sloped at 5 to 10 percent, is expected to be stable under repeated cycles of freezing and thawing. Some minor surface creep may occur in the 5H to 1V slopes and would be expected on the 3H to 1V slopes. Such movements are not expected to compromise the integrity of the system and can be compensated for by periodic maintenance.

4.2.5 Radiation Barrier

The primary radiological concern raised by the buried radium-bearing wastes is the possibility for emanation of radon gas through the cover materials. Calculations for the cap thickness were made on the basis of the proposed materials and configuration discussed in Section 4.2.1, using the methodology specified in ORO-832 (Ref. 37) and 40 CFR 192 (Ref. 21). The results indicate that the rate of radon emanation from the waste alone would be $0.061 \text{ pCi/m}^2/\text{s}$ through the cap surface. By comparison, radon emanation from the "clean" topsoil surface layer due to naturally occurring radium (and radon) in the topsoil would be $0.24 \text{ pCi/m}^2/\text{s}$, or four times that attributable to the buried waste. The total rate, $0.301 \text{ pCi/m}^2/\text{s}$, would be well below the DOE/EPA limit of $20 \text{ pCi/m}^2/\text{s}$. The cap thickness required to reduce the rate of radon emanation from the buried waste to $20 \text{ pCi/m}^2/\text{s}$ was calculated to be 13.7 cm (5.4 in.). However, a cap consisting of 0.5 m (1.5 ft) of soil underlain by a 0.9-m- (3-ft-) thick clay blanket has been provided to ensure that the rate of radon emanation from the buried contaminated waste is negligible (far below the allowable limit). Gamma radiation through the cap will likewise be attenuated to a very low level. Erosion will not reduce the 25- to 50-year service life of the containment facility, since any damage from erosion, settling, frost heaving, cracking, or biotic intrusion will be repaired.

4.2.6 Inspection and Maintenance

Systematic surface and subsurface inspection of containment performance will be provided along with repair and maintenance actions as required to ensure an effective cap life of 25 to 50 years (Ref. 12). The surface inspection will include a walkover survey with visual checks for erosion, shrinkage cracks, animal burrows, and deep-rooted vegetation. Grid surveys and aerial photography will be used to assist in detecting surface deformation. The surface of the cap will also be checked visually for depressions, and low areas will be surveyed to detect subsidence

trends. Action will be taken promptly to correct deficiencies and minimize the probability of recurrence.

Vibrating wire pressure transducers and pneumatic pressure transducers (Ref. 12) will be used to monitor variations in subsurface moisture conditions within the waste containment area. Monitoring wells will be installed around the containment facility and in the upper and lower aquifers. The groundwater in these wells will be sampled to verify that no contamination is migrating from the containment facility.

5.0 ANALYSIS OF CONTAMINANT MIGRATION

A detailed analysis of the long-term probability of contaminant migration via groundwater was performed using numerical techniques. These techniques permit a more realistic evaluation of the contamination problem and of the proposed remedial action than do traditional analytical methods, because they accommodate the inclusion of nonhomogeneous materials, irregular boundaries, and various types of boundary conditions among the factors considered. This section describes the scopes of the numerical analyses, the methodology used, and the results of the studies.

5.1 SCOPE OF ANALYSIS

The numerical study was performed to evaluate the effectiveness of the waste containment system. The study analyzes both the short- and long-term potential for contaminant migration given the known hydrogeologic conditions of the site and the specific geometrical and physical properties of the cap to be constructed over the waste materials.

The numerical analysis was performed in two phases. First, a parametric study was performed to determine the minimum and maximum values for the cap thickness, taking into account uncertainties in the input parameters such as hydraulic conductivity, adsorption coefficient, and boundary conditions. Second, a base case analysis was performed to evaluate the selected cap design on the basis of the most representative geologic and climatic conditions at the site over an 1113-year period -- far in excess of the 25- to 50-year design life of the waste containment facility.

5.2 METHODOLOGY

This numerical contaminant migration modeling study addressed the transport of contaminants via groundwater. Since the waste materials are initially unsaturated, the first step of the analysis was to determine whether these materials would eventually become

saturated as a result of surface infiltration. The second step was the analysis of contaminant transport from the waste containment area into the underlying geologic formations.

In both steps, numerical methods were used to solve the partial differential equations describing groundwater flow and contaminant transport. The mathematical formulation of the problems and the numerical models used for their solutions are described in detail in Appendix B.

5.3 MODEL DEFINITION AND INPUT DATA

The numerical analysis of groundwater flow and contaminant transport at the NFSS was based on the generalized geologic section shown in Figure 5-1. This section was derived from data contained in geologic reports (Refs. 5 and 6). It is oriented along the generally north-south direction of the regional hydraulic gradient and is considered representative of the conditions prevailing at the site. The finite element mesh dike sections used in connection with the GS2 model (Appendix B) are shown in Figure 5-2.

5.3.1 Groundwater Flow Simulation

The numerical model used to solve the groundwater flow equations, referred to as FLUMP, is described in Appendix B.

The finite element mesh used in conjunction with the FLUMP model, similar to that used in the GS2 finite element mesh dike sections, is shown in Figure 5-3. This mesh is designed to approximate closely the geological section illustrated in Figure 5-1. Five different geologic units are presented in the model, which also includes the waste materials, clay caps, dikes, and cutoff walls. The hydraulic properties of the various materials used in the model are listed in Table 5-1; these values are based on site-specific field and laboratory data. Properties of the waste and engineered materials were assigned a range of representative values and were the subject of a sensitivity study (described in Section 5.4). An elevation of 94.5 m (310 ft) m.s.l. was used for the dike and cutoff

FIGURE 5-1 GENERALIZED GEOLOGIC SECTION OF THE NFSS

FIGURE 5-2 GS2 FINITE ELEMENT MESH DIKE SECTIONS

FIGURE 5-3 FLUMP MODEL FINITE ELEMENT MESH

TABLE 5-1
INPUT DATA TO GROUNDWATER FLOW MODEL

<u>HYDRAULIC CONDUCTIVITIES</u>	<u>K (cm/s)</u>
Queenston Formation	5.9×10^{-6}
Queenston Aquifer	3×10^{-5}
Red Silt	H: 1×10^{-6} V: 5×10^{-8}
Gray Clay	H: 1×10^{-5} V: 5×10^{-7}
Brown Clay	H: 2×10^{-5} V: 2×10^{-7}
Waste	3×10^{-5}
Clay Cap	10^{-6} to 10^{-7}
Dikes/Cutoff Walls	1×10^{-7}

SPECIFIC YIELD

A range of 0.05 to 0.2 was used. This is a dimensionless unit ($L^3/L^2/L$).

H = Horizontal groundwater flow direction

V = Vertical groundwater flow direction

walls of the west dike, and an elevation of 91.9 m (301.5 ft) m.s.l. was used for the north dike.

Permeability and moisture content data for the unsaturated zone were not available at the time of the study, and were therefore assumed to be identical to those measured in the saturated zone. This is a conservative assumption since the permeability of unsaturated material is known to be significantly lower than that of saturated material, with a corresponding decrease in groundwater velocity and rate of contaminant migration.

A value of 96 m (315 ft) m.s.l., representative of the water levels measured at the site, was used for the initial water table elevation. Seasonal water level fluctuations are shown on the hydrographs in the Geologic Report, Niagara Falls Storage Site (Ref. 5). The PMF event described in Subsection 2.3.3 was not considered to be significant for this analysis because it is a highly unlikely event and would be of very short duration if it did occur.

Constant hydrostatic pressure is maintained along the left and right model boundaries, with the water table elevation at 96 m (315 ft) m.s.l. The constant value at each node was equal to the elevation at that node. Constant head boundary conditions are also specified at the nodes located above the waste containment area. In the analyses described later in this section, a constant infiltration rate was specified above the containment area. The area located between the waste pile and the left and right model boundaries was treated as a no-flow boundary in most of the parametric runs. As a check, a run was made using a constant infiltration rate throughout the upper model boundary, and the results were compared with those obtained when only the top of the waste pile is subjected to a constant infiltration rate. Although the pressure distribution away from the pile is different in the two simulations, the contaminant migration is not affected by this change.

For the base case calculation, an infiltration rate of 3.6 cm/yr (1.4 in./yr) was used throughout the top of the finite element model.

5.3.2 Contaminant Transport Simulation

The numerical code used to solve the mass transport equations, referred to as GS2, is described in Appendix B.

The finite element mesh used for the contaminant transport simulation is shown in Figure 5-4. The different material types present in the model are also indicated in this figure. The mesh contains 1653 nodes and 1548 elements. The hydraulic properties used in this model are the same as those specified for the groundwater flow simulation (Table 5-1). Additional parameters used in the mass transport equation are given in Table 5-2. The GS2 finite element mesh dike sections are shown in Figure 5-2.

The boundary conditions are such that the concentration is equal to 1 (100 percent) within the waste pile. (An alternative set of boundary conditions was used in some runs with only the nodes located at the bottom of the pile held constant; the long-term effect was found to be negligible.) Due to the heterogeneous nature of the wastes to be placed in the containment area, accurate estimates of radionuclide concentrations that would result from saturated conditions are not possible. However, the results of this modeling study show that the concentration is irrelevant for the purposes of this report.

5.4 SENSITIVITY ANALYSIS RESULTS

A sensitivity analysis was performed to study the response of the model to differing input parameters. This method is useful in determining the range of the pressure head and contaminant concentration distributions when uncertainties in the geohydrologic properties are taken into account. It also serves to compare the

FIGURE 5-4 GS2 MODEL FINITE ELEMENT MESH

TABLE 5-2
INPUT DATA TO CONTAMINANT TRANSPORT MODEL

DISPERSIVITY*

Longitudinal	9.1 m (30 ft)
Transverse	0.9 m (3 ft)

DISTRIBUTION COEFFICIENT, K_d (ml/g)

Uranium	1.0
	10.0
Radium	50.0
	500.0

*The dispersivity values are within the range of measured regional dispersivities. Table 4 in Reference 38 presents longitudinal and transverse dispersivities measured in various types of aquifers, with values ranging from 3 m (9.8 ft) to 200 m (656 ft). In addition, using larger dispersivity values is conservative since the contaminant front would spread more and reach the property boundaries sooner.

merits of different engineering designs. The input data corresponding to the various cases considered in the sensitivity study are shown in Tables 5-3 and 5-4, and the results are illustrated and discussed in the following subsections. In all the figures, the symbols K_{cap} and b_{cap} correspond to the permeability (or hydraulic conductivity) and thickness of the clay cap, respectively; λ represents the radioactive decay constant, and K_d refers to the adsorption coefficient.

5.4.1 Pressure Head

Figures 5-5 and 5-6 illustrate the effects of changing the permeability and thickness of the clay cap, respectively. In Figure 5-5, the cap permeability was decreased from 10^{-6} cm/s (lower drawing) to 10^{-7} cm/s (upper drawing). In Figure 5-6, the cap permeability is maintained at 10^{-7} cm/s, and the cap thickness is decreased from 1.5 m (5 ft) (upper drawing) to 0.76 m (2.5 ft) (lower drawing). It is seen that the saturation of the waste is retarded by using lower permeability or thicker clay caps.

Figure 5-7 shows the effect of changing the boundary condition specified for the top of the waste pile from 15 cm (0.5 ft) of constant head (upper drawing) to a constant infiltration rate of 8.4 cm/yr (3.3 in./yr) (lower drawing). The infiltration rate of the 15 cm (0.5 ft) constant head case is calculated directly in the FLUMP model. The pressure distributions obtained in each case were very similar, although the constant head conditions appeared to result in slightly greater saturation at a given time. For this reason, the constant head boundary conditions were used throughout the subsequent mass transport simulation of the sensitivity analysis. Subsequent analyses performed to establish the base case (discussed in Subsection 5.5) showed that the boundary conditions are approximately three times more severe than for the most conservative case used in the base case analysis.

TABLE 5-3
SENSITIVITY ANALYSIS FOR GROUNDWATER FLOW

Run	K_{cap} (cm/s)	b_{cap} m (ft)	B.C. m (ft)	Time (yrs)
A1	10^{-7}	1.5 (5.0)	0.15 (0.5)	50
B1	10^{-7}	1.5 (5.0)	1.5 (5.0)	50
C1	10^{-6}	1.5 (5.0)	0.15 (0.5)	50
D1	10^{-7}	0.8 (2.5)	0.15 (0.5)	50
E1	10^{-7}	1.5 (5.0)	8.4 cm/yr (3.3 in./yr)	50
F1	10^{-7}	1.5 (5.0)	0.15 (0.5)	200
G1	10^{-6}	3.0 (10.0)	0.15 (0.5)	50

B.C. = Boundary Conditions (constant head or constant flux)

K_{cap} = Cap hydraulic conductivity

b_{cap} = Cap thickness

NOTES: 1. Run G1 is not shown.
2. B.C. for Run E1 is expressed in cm (in.)/yr (see Fig. 5-7).

TABLE 5-4
SENSITIVITY ANALYSIS FOR CONTAMINANT TRANSPORT

Run	K _{cap} (cm/s)	b _{cap} m (ft)	K _d (ml/g)	Lambda (1/yr)	Time (yr)	Remarks
A2	10 ⁻⁷	1.5 (5)	500.0	0	195	
B2	10 ⁻⁶	1.5 (5)	50.0	0	195	
C2	10 ⁻⁶	1.5 (5)	1.0	0	195	
D2	10 ⁻⁶	1.5 (5)	10.0	0	195	
E2	10 ⁻⁶	1.5 (5)	0.0	0	195	
F2	10 ⁻⁶	1.5 (5)	1.0	4.33x10 ⁻⁴	195	2 channels partially penetrating gray clay
G2	10 ⁻⁶	1.5 (5)	1.0	4.33x10 ⁻⁴	195	2 channels through gray clay
H2	10 ⁻⁶	1.5 (5)	1.0	4.33x10 ⁻⁴	195	1 channel through gray clay
I2	10 ⁻⁶	1.5 (5)	1.0	1.55x10 ⁻¹⁰	1113	
J2	10 ⁻⁶	0.9 (3)	1.0	1.55x10 ⁻¹⁰	1113	
K2	10 ⁻⁶	0.9 (3)	50.0	4.33x10 ⁻⁴	1113	
L2	10 ⁻⁶	1.5 (5)	500.0	0	195	
M2	10 ⁻⁶	1.5 (5)	50.0	4.33x10 ⁻⁴	1113	
N2	10 ⁻⁷	1.2 (4)	5.0	1.55x10 ⁻¹⁰	1113	2.54 cm/yr (1.0 in./yr) infiltration along upper boundary
N2'	10 ⁻⁷	1.2 (4)	5.0	1.55x10 ⁻¹⁰	190	

N2 and N2' are different time steps of the same run.

K_{cap} = permeability, or hydraulic conductivity

b_{cap} = thickness of the clay cap

K_d = adsorption coefficient

Lambda = radioactive decay constant

FIGURE 5-5 PRESSURE HEAD AS A FUNCTION OF CAP PERMEABILITY

FIGURE 5-6 PRESSURE HEAD AS A FUNCTION OF CAP THICKNESS

FIGURE 5-7 PRESSURE HEAD CONTOURS RESULTING FROM A 15-CM (0.5-FT) CONSTANT HEAD BOUNDARY CONDITION (UPPER DRAWING), AND AN 8.4-CM/YR (3.3-IN./YR) CONSTANT INFILTRATION RATE BOUNDARY CONDITION (LOWER DRAWING)

Figure 5-8 also shows the effect of a drastic change in boundary conditions on the degree of saturation. In this case, the constant head value specified at the top of the waste pile was increased by a factor of 10, from 15 cm to 1.5 m (0.5 ft to 5 ft).

Finally, Figure 5-9 shows the changeable nature of the pressure distribution: a partially saturated state is indicated at the end of a 50-year period (upper drawing), and a fully saturated condition is expected at the end of a 200-year period (lower drawing).

5.4.2 Contaminant Transport

As a reference case for the contaminant transport simulation, the first analysis considered the case of a dissolved contaminant traveling with the velocity of groundwater, which is equivalent to ignoring the retarding effect of adsorption by the geologic materials. Based on analyses of samples taken from the site, it appears unrealistic to assume that no retardation will occur; however, the numerical results of the reference case serve to illustrate the importance of the adsorption coefficient. Figures 5-10 through 5-13 show the results obtained using various retardation factors (designated by K_d) on uranium and radium, successively. In Figures 5-10 and 5-11, the upper drawings depict the extent of migration without retarding factors, while the lower drawings show the results obtained for uranium with increasingly higher adsorption coefficients. The range of adsorption values selected for uranium ($K_d = 1$ and 10 ml/g) corresponds to the range of laboratory results obtained for core BH-77 samples at most depths (Ref. 39). [Very high adsorption values were obtained from the 137-m (45-ft) depth at core BH-77.]

FIGURE 5-8 PRESSURE HEAD CONTOURS FOR CONSTANT HEAD BOUNDARY CONDITIONS

FIGURE 5-9 PRESSURE HEAD CONTOURS FOR 50 AND 200 YEARS

FIGURE 5-10 URANIUM MIGRATION AS A FUNCTION OF K_d , ADSORPTION COEFFICIENT, FOR $K_d = 0.0$ AND 1.0 ML/G

FIGURE 5-11 URANIUM MIGRATION AS A FUNCTION OF K_d , ADSORPTION COEFFICIENT,
 FOR $K_d = 0.0$ AND 10.0 ML/G

FIGURE 5-12 RADIUM MIGRATION AS A FUNCTION OF K_d , ADSORPTION COEFFICIENT, FOR $K_d = 0.0$ AND 50.0 ML/G

FIGURE 5-13 RADIUM MIGRATION AS A FUNCTION OF K_d , ADSORPTION COEFFICIENT, FOR $K_d = 0.0$ AND 500.0 ML/G

The effect of adsorption on the migration of radium was considered and is illustrated in Figures 5-12 and 5-13. High adsorption coefficients ranging from 660 to 18,000 ml/g were obtained from laboratory tests (Ref. 39). Figure 5-12 shows the extent of contaminant migration without adsorption (upper drawing) and the results obtained using an adsorption coefficient equal to 50 ml/g (lower drawing). Figure 5-13 shows a similar comparison using $K_d = 500$ ml/g, a conservative lower limit for radium. Both Figures 5-12 and 5-13 show that retardation effects are important and that radium does not migrate significantly beyond the waste pile.

Figure 5-14 shows the extent of contaminant migration assuming cap permeabilities equal to 10^{-6} cm/s and 10^{-7} cm/s, respectively; both cases assume a K_d of 500 ml/g. In this figure, the lower drawing depicts the less permeable cap, and the upper drawing shows the more permeable cap. Lowering the cap permeability was seen to be an effective means of limiting the extent of contaminant migration; however, the extent of contaminant migration was found to be more strongly correlated with the magnitude of adsorption coefficients than with cap permeability for the range of values considered in this study.

Because of the concern regarding the possible effects of permeable stream channels in the aquifer, the model was used to study this hypothetical situation. For this study, the permeability of selected elements was increased to 10^{-2} cm/s as shown in Figures 5-15, 5-16, and 5-17. As shown in the figures, these simulated channels were 15 x 15 m (50 x 50 ft) wide (in the plane of the model). The results of these simulations are shown in Figures 5-18 through 5-20. In each of these figures, the upper drawing corresponds to the reference case of uranium migration with $K_d = 1$ ml/g and no channels, and the lower drawing corresponds to uranium migration with the same K_d in a medium where the channels have been introduced. It is seen that the long-term effects of introducing channels into the gray clay are not significant.

FIGURE 5-14 CONTAMINANT MIGRATION AS A FUNCTION OF CAP PERMEABILITY, K , FOR $K = 10^{-6}$ AND 10^{-7} CM/S

GS2 MODEL FINITE ELEMENT MESH

TWO CHANNELS PARTIALLY PENETRATING GRAY CLAY

FIGURE 5-15 GS2 MODEL FINITE ELEMENT MESH WITH TWO CHANNELS ($K = 10^{-2}$ CM/S) PARTIALLY PENETRATING THE GRAY CLAY

FIGURE 5-16 GS2 MODEL FINITE ELEMENT MESH WITH TWO CHANNELS ($K = 10^{-2}$ CM/S) COMPLETELY PENETRATING THE GRAY CLAY

GS2 MODEL FINITE ELEMENT MESH

ONE CHANNEL COMPLETELY PENETRATING GRAY CLAY

FIGURE 5-17 GS2 MODEL FINITE ELEMENT MESH WITH ONE CHANNEL ($K = 10^{-2}$ CM/S) COMPLETELY PENETRATING THE GRAY CLAY

FIGURE 5-18 URANIUM MIGRATION FOR GRAY CLAY WITH NO CHANNELS (UPPER DRAWING), AND FOR TWO CHANNELS PARTIALLY PENETRATING THE CLAY (LOWER DRAWING)

FIGURE 5-19 URANIUM MIGRATION FOR GRAY CLAY WITH NO CHANNELS (UPPER DRAWING), AND FOR TWO CHANNELS COMPLETELY PENETRATING THE CLAY (LOWER DRAWING)

FIGURE 5-20 URANIUM MIGRATION FOR GRAY CLAY WITH NO CHANNELS (UPPER DRAWING) AND FOR ONE CHANNEL COMPLETELY PENETRATING THE CLAY (LOWER DRAWING)

Figure 5-21 includes a modification of Run G2. Hydrologic parameters and boundary conditions for Run G3 are the same as for Run G2, except that permeable channels extend through both the gray clay and brown clay in Run G3. The results obtained in Run G3 are shown on the lower drawing of Figure 5-21. The shape of the isoconcentration lines clearly reflects discontinuities in the flow field introduced by the permeable channels. However, the changes in contaminant concentrations are relatively small. After 190 years of simulation time, the maximum change in the position of the 10 percent isoconcentration lines resulting from extending the channels through the brown clay is less than 20 ft.

Figures 5-22 and 5-23 show the extent of contaminant migration at 195 years (upper drawing) and 1113 years (lower drawing). (These specific time intervals correspond to the time steps selected automatically by the computer program.) Figure 5-22 corresponds to uranium migration with $K_d = 1$ ml/g, and Figure 5-23 is for radium with $K_d = 50$ ml/g.

In Figures 5-24 and 5-25, the results obtained for cap thicknesses of 1.5 m (5.0 ft) and 0.9 m (3.0 ft) are compared for uranium and radium, respectively. It is seen that neither cap thickness significantly affects the long-term extent of contaminant migration. Consequently, a thickness of 1.2 m (4 ft) was used as the base case in subsequent calculations.

5.5 BASE CASE RESULTS

The sensitivity analysis was followed by the base case analysis, which incorporated representative geologic and climatic conditions at the site. The analysis assumed a cap thickness of 1.2 m (4 ft) and a permeability of 10^{-7} cm/s. The geologic units and material properties are shown in Table 5-1 and in Figures 5-3 and 5-4. The geometry and material properties of the dike/cutoff walls shown were similar, although the west dike cutoff wall terminated slightly above the gray clay in the model (a conservative feature). The adsorption coefficient was 5 ml/g, which represents a median

FIGURE 5-21 URANIUM MIGRATION WITH TWO CHANNELS PENETRATING THE GRAY CLAY (UPPER DRAWING) AND FOR TWO CHANNELS COMPLETELY PENETRATING BROWN AND GRAY CLAYS (LOWER DRAWING)

FIGURE 5-22 URANIUM MIGRATION AT 195 YEARS (UPPER DRAWING) AND 1113 YEARS (LOWER DRAWING) $K_d = 1.0$ ML/G

FIGURE 5-23 RADIUM MIGRATION AT 195 YEARS (UPPER DRAWING) AND 1113 YEARS (LOWER DRAWING) $K_d = 50.0$ ML/G

FIGURE 5-24 URANIUM MIGRATION AT 1113 YEARS FOR $b_{cap} = 1.5 \text{ M (5.0 FT)}$ (UPPER DRAWING), AND $b_{cap} = 0.9 \text{ M (3.0 FT)}$ (LOWER DRAWING) $K_d = 1.0 \text{ ML/G}$

FIGURE 5-25 RADIUM MIGRATION AT 1113 YEARS FOR $b_{cap} = 1.5 \text{ M}$ (5.0 FT) (UPPER DRAWING), AND $b_{cap} = 0.9 \text{ M}$ (3.0 FT) (LOWER DRAWING) $K_d = 50.0 \text{ ML/G}$

value for uranium. The base case was analyzed only for uranium, since the sensitivity analysis showed that radium would not migrate significantly beyond the pile even with the most conservative K_d assumption of 500 ml/g (Figure 5-13).

5.5.1 Infiltration of Precipitation

The base case analysis of infiltration of the clay layer of the cap by precipitation was modeled on the basis of mean annual precipitation and potential evapotranspiration. When precipitation falls on the surface of the cap, some is lost as runoff and some is intercepted and detained by vegetation and small depressions in the surface. Precipitation that does not run off either enters the soil as infiltration or evaporates from the surface. All or some of the water that infiltrates the surface is transpired by plants; some may eventually reach the top of the clay layer. The amount that could reach the clay depends on the capacity of the soil to store moisture, the existing soil moisture, and the potential for evapotranspiration. To estimate the possibility of infiltration, several assumptions were necessary. Assuming the surface of the cap is sloped at 5 to 10 percent and is covered with vegetation, it is estimated that 18 to 22 percent of the precipitation falling on the waste containment facility will leave the cap as runoff. The layers above the clay were assumed to have a moisture storage capacity of 12.7 cm (5 in.). Potential evapotranspiration was estimated at 69 cm/yr (27 in./yr), and mean annual precipitation was found to be 74 cm (29 in.) for the site, with a standard deviation of 10.9 cm (4.3 in.). The precipitation data for the NFSS for the 47-year period from 1936 to 1982 were developed using data from two National Oceanic and Atmospheric Administration (NOAA) gauges: Lewiston 1N (1936-1972) and Buffalo AP (1936-1982). The Lewiston 1N data were extended from 1973 to 1982 based on Buffalo AP data, and the resulting 1936-1982 data were assumed to be applicable to the NFSS. Original data were obtained from the Weather Bureau and NOAA publications (Refs. 40-42).

In wet years, water balance would indicate that the potential evapotranspiration of 69 cm/yr (28.5 in./yr) is fully satisfied and that water is available for infiltration (assuming that at least 18 percent of the precipitation leaves the containment facility as runoff). However, when annual precipitation is less than or equal to the average, actual evapotranspiration is limited by the amount of water available and is less than the potential maximum. Under these dry conditions no infiltration occurs.

Methodology developed by Thornthwaite (Ref. 43) was used to model the soil moisture balance on a monthly basis and to calculate a mean annual deep infiltration rate. Two separate sets of calculations were made in analyzing deep infiltration. In the first set, mean monthly precipitation and mean potential evaporation were calculated, producing an estimated average deep infiltration rate of 2.77 cm/yr (1.09 in./yr). In the second set of calculations, 1000 years of annual precipitation values were generated using stochastic methods that ensured that the annual mean and standard deviations of the generated precipitation values were essentially the same as those in the historical record. These generated precipitation data were then examined using the Thornthwaite approach. In many dry years, the unavailability of water would preclude deep infiltration. The 1000 year calculations (Ref. 44) yielded an average deep infiltration rate of 3.7 cm/yr (1.47 in./yr).

The base case analysis was performed using the estimated 2.54 cm/yr (1.0 in./yr) deep infiltration value since the constant head condition of 15 cm (6 in.) of water used to arrive at this value yielded much more conservative results over the long term.

5.5.2 Contaminant Transport

The base case results are shown in Figure 5-26. In this figure, the upper drawing corresponds to the results obtained after 190 years of simulation time, and the lower drawing corresponds to those obtained after 1113 years. The positions of the isoconcentration lines are very similar for these two periods, which demonstrates that the rate

FIGURE 5-26 URANIUM MIGRATION FOR BASE CASE ANALYSIS AT 190 YEARS (UPPER DRAWING) AND 1113 YEARS (LOWER DRAWING)

of migration is very slow. One thousand years after placement of the wastes, the concentration of uranium in groundwater is negligible (0 percent isoconcentration line) at a distance of less than 30.5 m (100 ft) from the waste pile.

The effect of extending permeable channels through the brown and gray clays was further investigated under base case conditions. The results (Run N3) are shown in the lower drawing of Figure 5-27; the base case results without permeable channels (Run N2'), shown in the upper drawing, are similar to those shown in the upper drawing in Figure 5-26. The effect of the channels is seen to be insignificant after 190 years (simulated time).

5.6 CONCLUSIONS

A numerical analysis was performed to study the likelihood of contaminant migration from the NFSS given a range of material properties and boundary conditions. Results corresponding to data considered to be the most representative of actual field conditions, as determined by field and laboratory work, can be summarized as follows:

- o Long-term results are almost identical for cap thicknesses of 0.9 m (3 ft) and 1.2 m (4 ft) (both with a permeability factor of 10^{-7} cm/s).
- o The extent of contaminant migration is very sensitive to the magnitude of the adsorption coefficient, K_d , used in the calculations.
- o Permeable channels may occur in the gray clay, but model simulations assuming 15-m- (50-ft-) long, 15-m- (50-ft-) wide channels did not indicate significantly modified lateral or vertical contaminant migration.
- o For base case calculations, the permeability and thickness of the cap were as stated above, and an infiltration rate of 2.54 cm/yr (1 in./yr) was specified at the ground surface. Using $K_d = 5$ mg/l, the concentration of uranium in groundwater is negligible (0 percent isoconcentration line) at a distance of less than 30.5 m (100 ft) from the waste pile more than 1,000 years after emplacement of the waste materials.

FIGURE 5-27 URANIUM MIGRATION FOR BASE CASE ANALYSIS AT 190 YEARS (UPPER DRAWING) AND WITH TWO CHANNELS PENETRATING BROWN AND GRAY CLAYS (LOWER DRAWING)

- o The sensitivity analysis indicates that the 0.9-m (3-ft) thickness specified for the clay blanket is sufficient to control infiltration for the 25- to 50-year design life of the cap

Assuming a 0.9-m- (3-ft-) thick clay blanket, numerical analysis indicates that there would be no migration of contaminants from the site during the 25- to 50-year design life of the cap. Furthermore, analyses of contaminant migration over longer periods of time (up to 1113 years) with cap thickness ranging from 0.8 m (2.5 ft) to 3.0 m (10 ft) have been performed to allow evaluation of design features required to ensure proper safety under extreme design conditions.

6.0 RISK/FAILURE MODE EVALUATION

6.1 CONTAINMENT FACILITY

6.1.1 Quality Assurance Assessment

A quality assurance assessment of the cap was performed in accordance with project instructions. The assessment identified potentially significant quality failures and evaluated the potential consequences of such failures. Results of the assessment identified the following construction errors as potential threats to the long-term integrity of the containment facility:

- o Improper compaction or consolidation of contaminated waste material
- o Improper compaction of clay in the cutoff wall, dike, or cap
- o Failure to fill voids inside the storage area

A quality failure in any of these activities could increase the susceptibility of the containment facility to moisture infiltration and to subsidence and cracking of the cap, thereby permitting surface water to enter the waste. Either or both of these effects could ultimately result in increased radionuclide migration from the containment area.

The quality assurance assessment identified each of the potential quality failures as significant. However, the probability of failure for each item was judged to be low. This judgment was based on the knowledge that a reliable design would be developed and strenuously tested, and that continuing testing and inspection of the facility would ensure appropriate quality and early detection and correction of conditions which could jeopardize the integrity of the containment facility.

Earthwork operations would be continuously inspected by a geotechnical engineer or qualified soils technician to verify

the quality of materials and construction methods. Field and laboratory testing were performed as frequently as deemed necessary by the quality inspector to verify compliance with specifications.

The Geotechnical Post-Construction Report (Ref. 25) details the materials and placement methods used for construction activities from 1982 through 1985, as well as the quality standards maintained for both materials and methods. Corresponding details regarding the placement of topsoil and clay scheduled for 1986 will be incorporated into a supplement to this report.

Quality assurance assessment of the clay bottom of the containment facility was not required since the clay layer is naturally occurring and not an engineered structure. The cutoff walls extend into this layer and surround the containment area. All data indicate that the clay layer is continuous beneath the storage pile; however, discontinuities in the form of hypothetical stream channels were evaluated in the waste containment analysis discussed in Section 5.0 and found not to be critical.

6.1.2 Inspection and Maintenance

The design criteria include a requirement for the containment facility to undergo a systematic quarterly inspections as discussed in Subsection 4.2.6. The inspection will include a visual check for erosion, shrinkage cracks, animal burrows, and deep-rooted vegetation. Action to correct deficiencies will include placement of backfill where required and periodic mowing of the vegetative cover to prevent deep-rooted vegetation from becoming established.

The cap will be monitored by visual inspection and by periodical surveillance of the elevations of set reference points to identify possible settlement trends. The cap will be repaired in areas where significant differential settling is detected.

6.1.3 Environmental Monitoring

The current and planned environmental monitoring programs are more fully described in Reference 11. Groundwater, surface water, air, and sediment will be sampled and analyzed for parameters that are known to be present or that are likely to be present given the materials stored at the site.

Initially, groundwater wells will be monitored quarterly to ensure that any migration of radionuclides via groundwater is detected. If the monitoring results are below DOE or other institutional guidelines and are not highly variable, annual monitoring may be deemed adequate. On the other hand, if monitoring results exceed DOE or other institutional guidelines, a plan will be developed to locate the discharge site and to define the contaminated groundwater plume. Implementation of that plan will lead to a determination of the activities required to ensure containment of contaminants on site.

6.1.4 Performance Monitoring

The purpose of monitoring the performance of the containment structure is to determine whether or not the main engineered elements of the structure are performing satisfactorily. The performance monitoring program is distinct from the environmental monitoring program and will be conducted over a shorter period of time (Ref. 12 and Subsection 3.3).

The key objective of the performance monitoring program is the early detection of trends that could be indicative of developing weaknesses in the containment structure so that corrective action can be taken before the integrity of the containment structure is compromised. Deviation of the monitoring data from baseline values will prompt activities to locate and repair the weakness in the

structure if it has indeed been weakened. If examination of the structure indicates that the monitoring data have been misleading as to its condition, modifications to the methods of collecting data may be implemented.

REFERENCES

1. Bechtel National, Inc. Engineering Evaluation of Alternatives for the Disposition of Niagara Falls Storage Site, Its Residues and Wastes, DOE/OR/20722-1, Oak Ridge, TN, April 1984.
2. U.S. Department of Energy. Niagara Falls Storage Site Project Management Plan, Oak Ridge, TN, September 1985.
3. U.S. Department of Energy. Design Criteria for Formerly Utilized Sites Remedial Action Program (FUSRAP) and Surplus Facilities Management Program (SFMP), 14501-00-DC-01, Rev. 1, Oak Ridge, TN, February 1986.
4. U.S. Department of Energy. Draft Environmental Impact Statement: Long-Term Management of the Existing Radioactive Wastes and Residues at the Niagara Falls Storage Site, Washington, DC, August 1984.
5. Bechtel National, Inc. Geologic Report, Niagara Falls Storage Site, Oak Ridge, TN, June 1984.
6. Acres American, Inc. Hydrologic and Geologic Characterization of the DOE-Niagara Falls Storage Site, Buffalo, NY, September 1981.
7. Battelle Columbus Laboratories. A Comprehensive Characterization and Hazard Assessment of the DOE-Niagara Falls Storage Site (Vol. 1, 2, and 3), BMI-2074 (Revised), Columbus, OH, June 1981.
8. U.S. Department of Energy. Order 1540.1, "Materials Transportation and Traffic Management," May 2, 1983.
9. U.S. Department of Energy. Order 5480.1A, "Environmental Protection, Safety, and Health Protection Program for DOE Operations," August 13, 1981 and supplement June 22, 1982.

10. Bechtel National, Inc. Closure/Post-Closure Plan for the Interim Waste Containment Facility at the Niagara Falls Storage Site, DOE/OR/20722-85, Oak Ridge, TN, May 1986.
11. Bechtel National, Inc. Environmental Monitoring Plan for the Niagara Falls Storage Site and the Interim Waste Containment Facility, DOE/ORO/20722-54, Oak Ridge, TN, April 1986.
12. Bechtel National, Inc. Report on the Performance Monitoring System for the Interim Waste Containment at the Niagara Falls Storage Site, DOE/ORO/20722-71, Oak Ridge, TN, October 1985.
13. U.S. Department of Energy. Order 5480.2, "Hazardous and Radioactive Mixed Waste Management," December 13, 1982.
14. U.S. Department of Energy. Order 5481.1A, "Safety Analysis and Review System," August 13, 1981 and supplement June 4, 1982.
15. U.S. Department of Energy. Order 5482.1A, "Environmental Protection, Safety, and Health Protection Appraisal Program," April 19, 1982.
16. U.S. Department of Energy. Order 5483.1, "Occupational Safety and Health Program for Government-Owned Contractor-Operated Facilities," April 13, 1979.
17. U.S. Department of Energy. Order 5484.1, "Environmental Protection, Safety, and Health Protection Information Reporting Requirements," February 24, 1981 and supplement May 5, 1982.
18. U.S. Department of Energy. Order 5484.2, "Unusual Occurrence Reporting System," August 13, 1981 and supplement July 8, 1982.
19. U.S. Department of Energy. Order 5820, "Radioactive Waste Management" (Draft), July 29, 1983.

20. U.S. Code of Federal Regulations. 40 CFR 141, "National Interim Primary Drinking Water Standards," Washington, DC, July 1985.
21. U.S. Code of Federal Regulations. 40 CFR 192, "Health and Environmental Protection Standards for Uranium and Thorium Mill Tailings," Washington, DC, July 1985.
22. U.S. Code of Federal Regulations. 40 CFR 173, "Shippers -- General Requirements for Shipments and Packagings," Washington, DC, July 1985.
23. U.S. Department of Energy. "Action Description Memorandum for Niagara Falls Storage Site Proposed Interim Remedial Actions for FY-1983-85 Accelerated Program" (Draft), prepared by Argonne National Laboratory, Argonne, IL, April 1983.
24. Modern Landfill, Inc. Environmental Impact Statement (Draft) for the Modern Landfill, Inc. Sanitary Landfill, Model City, NY, June 1982.
25. Bechtel National, Inc. Geotechnical Post-Construction Report, Niagara Falls Storage Site, Lewiston, New York, Oak Ridge, TN, January 1984.
26. Kindle, E. M. and F. B. Taylor. Description of the Niagara Quadrangle, NY, U.S. Geologic Survey, Geological Atlas, Folio 190, U.S. Geological Survey, Washington, DC, 1913.
27. Newton, D. W. "Realistic Assessment of Maximum Flood Potential", ASCE Journal of Hydraulic Engineering, Vol. 109, No. 6, June 1983.
28. National Oceanic and Atmospheric Administration, Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No. 51, June 1978.

29. National Oceanic and Atmospheric Administration, Application of Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No. 52, August 1982.
30. National Oceanic and Atmospheric Administration, U. S. National Weather Service, Seasonal Variation of 10-Square-Mile Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No. 53, April 1980.
31. U. S. Army Corps of Engineers, The Hydrologic Engineering Center, HEC-1, Flood Hydrograph Package, Users Manual, September 1981.
32. National Oceanic and Atmospheric Administration. Climatic Atlas of the United States, Washington, DC, June 1968 (reprinted 1983).
33. U.S. Nuclear Regulatory Commission. Guidelines for Decontamination of Facilities and Equipment Prior to Release for Unrestricted Use or Termination of Licenses for Byproduct, Source, or Special Nuclear Material, Washington, DC, July 1982.
34. Duncan, J. M. and A. L. Buchignani. An Engineering Manual for Slope Stability Studies, Department of Civil Engineering, Institute of Transportation and Traffic Engineering, University of California, Berkeley, CA, March 1975.
35. U.S. Army Corps of Engineers. Pavement Design for Frost Conditions, EM 1110-1-306, May 1962.
36. Departments of the Army and the Air Force, Arctic and Subarctic Construction. Calculation Methods for Determination of Depths of Freeze and Thaw in Soils, Army Technical Manual (TM) 5-852-6, Air Force Manual (AFM) 88-19, Washington, DC, January 1966.

37. U.S. Department of Energy. Pathways Analysis and Radiation Dose Estimates for Radioactive Residues at Formerly Utilized MED/AEC Sites, ORO-832, Oak Ridge, TN, March 1983.
38. Anderson, Mary P. "Using Models to Simulate the Movement of Contaminants through Groundwater Flow Systems," Critical Reviews in Environmental Control, Volume 9, Issue 2, 1979.
39. Kelmers, A. D. and F. G. Seeley. "Geotechnical Studies for the Formerly Utilized Sites Remedial Action Program (FUSRAP) Site Management; Chemical Support," Progress Reports for the period March 16 - April 15, 1983, ORNL-6083, Oak Ridge National Laboratory, Oak Ridge, TN, November 1984.
40. National Oceanic and Atmospheric Administration. Climatology of the United States, No. 11-19, Climatic Summary of the United States - Supplement for 1931 through 1952, Washington, DC, Undated.
41. National Oceanic and Atmospheric Administration. Climatology of the United States, No. 86-26, Climatic Summary of the United States - Supplement for 1951 through 1960, Washington, DC, Undated.
42. National Oceanic and Atmospheric Administration. Climatological Data, New York, monthly publications, 1960 through 1982.
43. Thornthwaite, C. W. and J. R. Mather. Instruction and Tables for Computing Potential Evapotranspiration and the Water Balance, Drexel Institute of Technology, Laboratory of Climatology, Vol. X, No. 3, 1957.
44. U.S. Environmental Protection Agency. Use of the Water Balance Method for Predicting Leachate Generation from Solid Waste Disposal Sites, EPA/530/SW-168, Washington, DC, October 1975.

APPENDIX A

SEISMICITY OF THE NFSS REGION

SEISMICITY OF THE NFSS REGION

1.0 TECTONIC PROVINCES

The principal tectonic divisions within 320 km (200 mi) of the NFSS are shown in Figure A-1.

The NFSS lies within the Central Stable Region. This region has been described by Eardley (Ref. 1) as one comprising a veneer of sediments overlying Precambrian crystalline rocks that have been formed into arches, basins, and other structures primarily as a result of Paleozoic epeirogenic activity. The Central Stable Region extends from the eastern Appalachian Mountain Chain to the western Rocky Mountains, and from the Canadian Shield in the north to the onlapping Cretaceous and Tertiary sediments of the Coastal Plain in the south.

To the north of the site (at about 160 km (100 miles) at its closest approach), the Canadian Shield is characterized by a vast expanse of Precambrian rock. Its upland surfaces are uniform in height over large areas and represent an old erosion surface. The extensive surface rises 305 to 610 m (1000 to 2000 ft) above sea level north of the St. Lawrence River and Lake Superior.

Approximately 260 km (160 mi) east-northeast of the NFSS, the Adirondack uplift exposes shield-type Precambrian rocks flanked to the north and south by younger sedimentary rocks. The uplift is a zone of high-angle faulting along the southeastern edge of the Canadian Shield. This roughly 120-km (75-mi) radius circular zone contains many faults. A west-trending fault and graben zone penetrates the shield and separates the Adirondack Uplift from the Canadian Shield.

North of the Adirondack Uplift lies the western part of the St. Lawrence lowlands. The rocks in the lowlands are Paleozoic limestones, dolomites, and sandstones. It is a rift valley in which major block faulting has occurred (Ref. 2).

FIGURE A-1 TECTONIC ELEMENTS OF THE NIAGARA REGION

Finally, the Central Appalachian Fold Belt is located about 160 km (100 mi) southeast of the Niagara site. This wide fold belt encompasses the southern part of New York and the western parts of Pennsylvania and West Virginia. In the site region, the Central Appalachian Fold Belt is made up of relatively undeformed Paleozoic sedimentary rocks above a deeply buried and little-deformed basement.

The Central Stable Region is generally considered to be tectonically stable, and earthquakes within this region have generally been of moderate intensity (modified Mercalli VI or VII) or less. An exception to this rule exists near the NFSS. Here a small seismic area in western New York and adjoining Ontario has suggested control by an unidentified westward-trending structure approximately parallel to the strike of the south-dipping Paleozoic rocks of the region (Ref. 3). Earthquake intensities are low except for an earthquake near Attica, New York at the eastern end of the area. This earthquake and some lesser ones nearby may be related to a buried north-trending fault that displaces the lower part of the Paleozoic sequence and probably the underlying basement. It is represented by a west-dipping monocline in the surface rocks known as the Clarendon-Linden structure. The configuration of this Attica seismic zone is not well defined. A diffuse configuration (suggested by the historic seismicity shown in Figure A-2) taken from Tera Corporation (Ref. 4) is shown in Figure A-1. A possible alternate interpretation would restrict the most damaging earthquakes of this area to the vicinity of Attica itself and, thus, exclude the NFSS.

The other principal region of recorded earthquakes near the site region is located in northeastern New York and adjoining parts of Quebec. It extends northwestward beyond Ottawa and northeastward beyond Quebec City. The region has an exceptionally long history of recorded earthquakes, dating from the 17th century, and several of these [beyond the 320-km (200-mi) radius site region, and therefore, not discussed in the historical seismicity section below nor shown in Figure A-1] have been rated as intensity IX or higher on the

FIGURE A-2 SEISMIC EVENTS OF MAGNITUDE ≥ 3.0 WITHIN 320 KM (200 MI) OF THE NFSS (1823-1975)

basis of contemporary but obviously rather limited accounts. The western limit of this zone occurs in the St. Lawrence Lowland part of the site region.

Earthquake activity in the remaining site region portions of the Canadian Shield, Adirondack Uplift, and Central Appalachian Fold Belt has been scattered and of moderate to small size. This activity does not contribute significantly to the earthquake potential of the NFSS relative to the Attica seismic zone and the St. Lawrence Lowland area.

2.0 HISTORICAL SEISMICITY

A compilation was made of all earthquakes of magnitude 3.0 (Richter scale) or greater occurring within 320 km (200 mi) of the NFSS. A number of sources were reviewed. Ultimately, a data base for the U.S. portion of the site region (Chiburis) and another for the Canadian part of the site region (Basham) were combined to give a complete and homogeneous data base. Both the Chiburis and Basham data sets appear in a seismic hazard analysis study of Tera Corporation (Ref. 5).

The complete Chiburis catalog contains northeastern U.S. earthquakes from 1534 through 1977. In-depth investigations of particular earthquakes were performed. Many of the original records were reread and reinterpreted to produce the instrumental local magnitudes contained in the catalog. When only intensity reports were available, an equivalent magnitude was estimated as discussed below.

The Basham catalog also contains contributions from Smith (Refs. 6 and 7) for the period 1534 through 1959. These data are supplemented by annual compilations of earthquakes published by the Earth Science Branch of the Canadian Department of Energy, Mines, and Resources from 1960 through 1975. All earthquakes with magnitudes greater than or equal to 4.0 were reviewed and calculated. Earthquakes (in the magnitude range of 4.0 to 5.5 in

source references) for which no instrumental information existed were assigned magnitudes based on the felt area of the event (Ref. 8). Magnitudes of some larger earthquakes were calculated using a distance-intensity-magnitude relation developed for eastern Canada. Other earthquakes were assigned magnitudes based on epicentral intensity (I_o) using the Gutenberg and Richter formula $M = 2/3 I_o + 1$ (Ref. 9). This last procedure was the one also adopted by Chiburis. For all larger events, a review of the macroscopic information was used to assign intensities representative of the overall effects.

The combination of the Chiburis and Basham data sets for the NFSS area is listed in Table A-1 and shown in Figure A-2. A review of Table A-1 indicates that the largest earthquake within 320 km (200 mi) of the site was the August 12, 1929 magnitude 5.8 event at Attica, New York. This earthquake occurred approximately 65 km (40 mi) from the NFSS. Other near-site events occurred on October 23, 1857 [distance about 30 km (18 mi), magnitude 5.0]; April 27, 1954 [distance about 23 km (14 mi), magnitude 4.1]; and a series of smaller earthquakes (November 27, 1982; November 12, 1927; March 7, 1897; April 27, 1954; August 22, 1958; and November 27, 1974) at distances of between 15 and 23 km (9 and 14 mi).

The earthquakes near Attica have been related to the Clarendon-Linden structure as discussed above; other earthquakes in the site region have not been related to known geologic structures. Some of these may be due to stress concentrations in the crust resulting from glacial unloading of the region.

TABLE A-1
CHIBURIS AND BASHAM DATA BASE OF EARTHQUAKES OF MAGNITUDE 3.0 OR
GREATER OCCURRING WITHIN 320 KM (200 MI) OF THE NFSS

No.	Date			Time (GMT)			Depth (Mi.)	Location		ML	IMM	(Mi.)
	Year	Mo.	Day	Hr.	Min.	Sec.		Lat. (°N)	Long. (°W)			
1	1823	05	30					41.50	81.00	3.7	IV	157
2	1840	09	10					43.20	79.85	4.4	V	45
3	1847	01	08	20	00			44.00	78.00	3.0	III	73
4	1847	01	14					44.20	78.20	3.0	III	79
5	1852	12	15					43.30	78.20	3.0	III	38
6	1853	03	12	07				43.70	75.50	5.0	VI	176
7	1853	03	13	10				43.10	79.40	4.4	V	24
8	1857	10	23	20	15			43.20	78.60	5.0	VI	18
9	1869	04	09	13	00			42.70	80.80	3.0	III	100
10	1873	04	30					43.30	79.90	3.7	IV	48
11	1873	07	06	14	30			43.00	79.50	5.0	VI	31
12	1877	05	02					43.90	78.85	3.0	III	49
13	1877	12	18	06	00			45.60	76.80	3.0	III	197
14	1877	12	18	11	00			45.60	76.80	4.33	V	197
15	1879	08	21	08	00			43.20	79.20	3.7	IV	13
16	1882	11	27	23	30			43.00	79.25	3.7	IV	20
17	1883	04	01	06	00			43.25	79.85	3.0	III	45
18	1885	09	04	14	40			44.30	77.90	3.0	III	92
19	1886	08	19	08	00			43.60	79.60	3.0	III	43
20	1887	02	19					45.35	80.00	3.7	IV	157
21	1888	01	11	09	00			45.70	77.20	3.67	IV	193
22	1888	07	10					44.40	77.00	3.0	III	128
23	1897	03	07					43.10	79.20	3.7	IV	14
24	1906	06	27					41.40	81.60	4.4		184
25	1907	01	10	10	00			41.20	77.10	3.67	IV	167
26	1907	01	25	06	00			44.10	79.10	3.7	IV	63
27	1907	11	14	05	00	00		45.47	76.68	3.7	IV	193
28	1910	02	25					43.20	79.80	3.7	IV	43
29	1912	03	27	12	52			43.20	79.70	4.3	V	38
30	1914	02	10	18	31			44.98	76.92	5.5	VIII	159
31	1921	09	27	04	32			42.10	80.20	3.0	III	99
32	1925	04	07	20	18	00		43.03	76.13	3.0	III	143
33	1925	05	23					43.40	77.10	3.0	III	94
34	1926	08	23	16	40			45.70	77.20	3.67	IV	193
35	1927	03	29	20	30	00		43.03	76.13	3.0	III	143
36	1927	03	31	21	00	00		43.00	76.10	3.0	III	144
37	1927	03	31	21	30			43.00	76.10	3.0	III	144
38	1927	11	12					43.10	79.06	3.7	IV	9
39	1929	08	12	06	00			42.20	77.20	3.0	III	113
40	1929	08	12	08	45			42.87	78.35	3.0	III	38
41	1929	08	12	11	24	48		42.87	78.35	5.8	VIII	38
42	1929	12	02	22	14			42.80	78.30	3.6	IV	43
43	1929	12	03	12	50			42.80	78.30	3.6	IV	43
44	1930	02	16	12	17			42.80	80.50	3.0	III	83
45	1931	04	22					42.90	78.90	3.6	IV	21

TABLE A-1
(continued)

No.	Date			Time (GMT)			Depth (MI.)	Location		ML	IMM	(MI.)
	Year	Mo.	Day	Hr.	Min.	Sec.		Lat. (°N)	Long. (°W)			
46	1932	01	21					41.10	81.60	3.7		199
47	1932	12	29					44.00	75.90	3.0	III	162
48	1934	10	29	20	07			42.20	80.20	4.3	V	94
49	1935	07	13					40.50	78.50	5.0	VI	188
50	1936	08	26	08	55			41.40	80.40	3.0	III	145
51	1938	07	15	22	46			40.40	78.20	4.33	V	197
52	1939	01	14	08	10	16		43.25	79.85	3.3		45
53	1939	02	24	00	20			42.87	78.28	3.0	III	41
54	1943	03	09	03	26			42.20	80.90	5.5		121
55	1944	02	26					42.90	78.80	4.33	V	22
56	1945	04	15	13	15			43.00	76.40	3.0	III	129
57	1945	04	15	14	20			43.00	76.40	3.0	III	129
58	1945	04	15	15	30			43.00	76.40	3.0	III	129
59	1946	03	20	02	01			44.33	75.92	3.0	III	170
60	1946	10	28	20	36	06		41.50	76.60	3.6		168
61	1946	11	10	11	41	23.1		42.87	77.45	3.1		79
62	1951	12	03	07	02			41.60	81.40	3.6		167
63	1952	11	20					42.92	76.57	3.0	III	122
64	1952	12	25	04	28	32.5		43.85	80.97	3.6		111
65	1954	02	01	00	37	50		43.03	76.65	3.3		117
66	1954	04	27	02	14	08		43.10	79.20	4.1		14
67	1955	05	26	18	09	23		41.48	81.73	4.3		185
68	1955	06	29	01	17	40		43.37	79.63	3.0		36
69	1955	08	16	07	35			42.89	78.28	4.3	V	40
70	1957	06	29	11	25			42.90	81.30	4.2		120
71	1957	08	21	02	40	33		44.80	76.17	3.0		177
72	1958	01	24	17	10			44.98	81.25	3.5		168
73	1958	05	01	22	46	31		41.48	81.73	4.3		185
74	1958	07	22	01	46	40		43.00	79.50	4.3		31
75	1958	08	04	20	25	58		43.13	80.00	3.9		53
76	1958	08	22	14	25	05		43.00	79.00	3.6		14
77	1962	03	27	06	35	05		43.00	79.33	3.0	V	24
78	1963	01	30	14	50			44.00	75.90			162
79	1963	02	27	06	00	00		43.20	79.57	3.0		31
80	1963	05	19	19	14	18		43.50	75.23	3.5	III	188
81	1963	05	20					43.20	75.20	3.5	IV	189
82	1964	02	13	19	46	42		40.40	78.20	5.2		197
83	1965	07	16	11	06	57	18	43.04	78.08	3.1	IV	45
84	1965	08	27	01	55	56	18	43.00	78.07	3.1		47
85	1966	01	01					42.80	78.30	3.0	III	43
86	1966	01	01	11	29	20	5	42.85	78.28	3.0		42
87	1966	01	01	13	23	38	10	42.90	78.20	4.7	VI	43
88	1967	06	13	19	08	54		42.90	78.20	3.9	VI	43
89	1974	11	27	10	28			43.30	79.10	3.3		10
90	1975	06	30	20	15	23	10	43.40	79.77	3.0	III	43

3.0 SEISMIC PROBABILITIES

In recent years, several procedures have been developed that allow formal determination to be made of earthquake probabilistic design parameters (Refs. 10 and 11), and a number of studies have been performed incorporating these procedures (Refs. 12, 13, and 14). In typical seismic risk studies of this kind, the region of interest is divided into seismic sources from which future earthquakes are considered equally likely to occur at any location. For each source, the rate of occurrence of earthquakes larger than a threshold level is estimated. This parameter is termed the source activity rate. The sizes of successive events for each source are assumed to be independent and exponentially distributed; the slope of the log number versus frequency relationship is estimated from the relative frequency of difference sizes of events observed in the historical data. This slope, often termed the b value (Ref. 15), is determined either for each seismic source individually or for all sources in the region jointly. Finally, the maximum possible size of earthquake for each source zone is determined using judgement and the historical record (Ref. 16).

Probabilities of peak dynamic accelerations have been evaluated for the NFSS area in several recent studies. Algermissen and Perkins (Ref. 12) calculate accelerations with a 10 percent expectation during a 50-year period (approximately equivalent to the 475-year acceleration). Their results are shown in Figure A-3.

Interpolation between the 0.04g acceleration contour and the 0.09g point in the figure shows that the site has about a 0.07g acceleration at this probability level.

Additional studies have been performed by Milne and Davenport (Ref. 17), Donovan et al (Ref. 18), and Basham et al (Ref. 19). Donovan et al find an approximately 0.1g 475-year effective peak acceleration at the NFSS. Milne and Davenport find the 100-year effective peak acceleration to be 0.03g for the site. Basham et al

FIGURE A-3 475-YEAR PEAK DYNAMIC SURFACE ACCELERATION WITHIN 320 KM (200 MI) OF THE NFSS

determine that the historical seismicity, not geology, controls the risk for Attica. They calculate the 1000-year acceleration as approximately 0.12g.

The principal differences in these derived accelerations arise from the different periods (100, 475, and 1000 years) upon which they are based. The longer time periods result in higher accelerations. Differences can also be the result of different characterizations of the source zones used in the analysis.

4.0 MAXIMUM INTENSITY AND MAGNITUDE

Several estimates of maximum site intensity (and magnitude associated with a near-site earthquake of this maximum intensity) are possible. These are maximum historical intensity, maximum probable intensity (at a specified probability), and maximum potential intensity.

A characterization of maximum historical intensities for the site region through 1965 appears in Figure A-4. This figure is taken from a U.S. Coast and Geodetic Survey study (Ref. 20). The NFSS lies within an area of Modified Mercalli intensity V. A review of events through 1980 indicates that no earthquakes since 1965 have resulted in a higher intensity at the site. A comparison of Figures A-1, A-2, and A-4 indicates that earthquakes within the Attica seismic zone (in particular the 1929 Attica earthquake) govern the maximum historical site intensity. A reinterpretation by Fox and Spiker of the isoseismals of the Attica earthquake suggests a reduction of the epicentral intensity from VIII to VII (Ref. 21). Although not directly addressed by Fox and Spiker, this might imply a reduction in the NFSS intensity as well.

No study was found that treated maximum probable site intensity explicitly. The probabilistic accelerations noted above can be used to derive estimates of equivalent probabilistic intensities. Using the intensity-acceleration relations of Neumann (Ref. 22) and Trifunac and Brady (Ref. 23), the 0.07g to 0.1g 475-year

FIGURE A-4 MAXIMUM HISTORICAL EARTHQUAKE INTENSITY (MODIFIED MERCALLI SCALE) WITHIN 320 KM (200 MI) OF THE NFSS (THROUGH 1965)

acceleration can be associated with a VI to VI-1/2 475-year intensity while the 0.12g 1000-year acceleration can be associated with a VII 1000-year intensity.

Characterization of maximum potential site intensity depends critically on characterization of the Attica seismic source zone. A number of characterizations of this source zone have been published (Refs. 12, 19, and 24). Estimates of maximum potential site intensity from these characterizations range from extreme values of XII (Ref. 24) to VI (implied by the recurrence of the Attica earthquake or a random intensity VI earthquake at or near the site). Preferred estimates are in the VII to VIII maximum potential intensity range.

Estimates of magnitudes associated with these intensities may be derived from magnitude intensity relations (Ref. 25). These imply body wave magnitudes of about 5.3 and 5.8 for intensities of VII and VIII.

APPENDIX A

REFERENCES

1. Eardley, A.J. Structural Geology of North America, Harper & Row, New York, 1962.
2. Fox, Fred L. "Seismic Geology of the Eastern United States," Bulletin of the Association of Engineering Geologists, Vol. VII, Nos. 1 and 2, pp. 21-43, 1970.
3. Hadley, Jarvis B. and James F. Devine. Seismotectonic Map of the Eastern United States, U.S. Geological Survey, Miscellaneous Field Studies MF-620, 1974.
4. Tera Corporation. Seismic Hazard Analysis Solicitation of Expert Opinion, NUREG/CR-1582, Vol. 2, 1981.
5. Tera Corporation. Seismicity Data for the Bayesian Seismic Hazard Questionnaire, 1979.
6. Smith, W.E.T. Earthquakes of Eastern Canada and Adjacent Areas, 1534-1927, Publications of the Dominion Observatory, Ottawa, Vol. 26, pp. 271-301, 1962.
7. Smith, W.E.T. Earthquakes of Eastern Canada and Adjacent Areas, 1928-1959, Publications of the Dominion Observatory, Ottawa, Vol. 32, pp. 87-121, 1966.
8. Nuttli, O.W. and J.E. Zollweg. "The Relation Between Felt Area and Magnitude for Central United States Earthquakes," Bulletin of the Seismological Society of America, Vol. 64, No. 1, pp. 73-85, 1974.
9. Gutenberg, B. and C.F. Richter. "Earthquake Magnitude, Intensity, Energy and Acceleration," Bulletin of the Seismological Society of America, Vol. 46, No. 2, pp. 105-190, 1956.

10. Cornell, C.A. "Engineering Seismic Risk Analysis," Bulletin of the Seismological Society of America, Vol. 58, pp. 1583-1606, 1968.
11. Cornell, C.A. and E. H. Vanmarke. "The Major Influence on Seismic Risk," Proceedings 4th World Conference on Earthquake Engineering, Santiago, Chile, Vol. 1, pp. 69-83, 1969.
12. Algermissen, S.T., and D.M. Perkins. A Probabilistic Estimate of Maximum Ground Acceleration in the Contiguous United States, U.S. Geological Survey Open File Report 76-416, 1976.
13. Cornell, C.A. and H.A. Merz. "Seismic Risk Analysis of Boston," Journal of the Structural Division of the American Society of Civil Engineers, Vol. 10, pp. 2027-2043, 1975.
14. Shah, H.C., C.P. Mortgat, A. Kiremidjian, and T.C. Zsutty. A Study of Seismic Risk for Nicaragua, Report 11, Dept. of Civil Engineering, Stanford University, Stanford, CA, 1975.
15. Richter, C.F. Elementary Seismology, W.H. Freeman and Company, San Francisco and London, 1958.
16. McGuire, R.K. "Effects of Uncertainty in Seismicity on Estimates of Seismic Hazard for the East Coast of the United States," Bulletin of the Seismological Society of America, Vol. 67, pp. 827-848, 1977.
17. Milne, W.G. and A.G. Davenport. "Distribution of Earthquake Risk in Canada," Bulletin of the Seismological Society of America, Vol. 59, No. 2, pp. 729-754, 1969.
18. Donovan, N.C., B.A. Bolt, and R.V. Whitman, Development of Expectancy Maps & Risk Analysis, ASCE Annual Convention and Exposition, Preprint 2805, 1976.

19. Basham, P.W., D.H. Weichert, and M.J. Berry. "Regional Assessment of Seismic Risk in Eastern Canada, Bulletin of the Seismological Society of America, Vol. 69, No. 5, pp. 1567-1602, 1979.
20. U.S. Coast and Geodetic Survey Studies in Seismicity and Earthquake Damage Statistics, Department of Housing and Urban Development, Studies of Natural Disasters, Washington, DC, 1967.
21. Fox, Fred L. and Carlisle T. Spiker. "Intensity Rating of the Attica (N.Y.) Earthquake of August 12, 1929, A Proposed Reclassification," Earthquake Notes, Vol. 48, Nos. 1-2, pp. 37-46, 1977.
22. Neumann, F. Earthquake Intensity and Related Ground Motion, University of Washington Press, Seattle, 1954.
23. Trifunac, M.D. and A.G. Brady. "On the Correlation of Seismic Intensity Scales with the Peaks of Recorded Strong Ground Motion," Bulletin of the Seismological Society of America, Vol. 65, No. 1, pp. 139-162, 1975.
24. Tera Corporation. Seismic Hazard Analysis, A Methodology for the Eastern United States, NUREG/CR-1582, Vol. 2, 1980.
25. Nuttli, Otto W. and Robert B. Herrmann. Credible Earthquakes for the Central United States, Report 12, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, 1978.

APPENDIX B

THEORETICAL BASIS FOR ANALYSIS OF CONTAMINANT MIGRATION

THEORETICAL BASIS FOR ANALYSIS OF CONTAMINANT MIGRATION

This appendix discusses the theoretical formulation of the mass transport problem and the models used for the simulations, including model validation.

1.0 THEORETICAL BACKGROUND

The equation describing the transport of a dissolved contaminant can be written in the form (Ref. 1)

$$\theta R_d \frac{\partial c}{\partial t} - \frac{\partial}{\partial x_\alpha} \left(\theta D_{\alpha\beta} \frac{\partial c}{\partial x_\beta} \right) + \frac{\partial q_\alpha c}{\partial x_\alpha} + \left(R_d \frac{\partial \theta}{\partial t} + \alpha \theta R_d \frac{\partial \psi}{\partial t} + \lambda \theta R_d \right) c + Qc' = 0 \quad (1)$$

with c , solute concentration (ML^{-3})

c' , concentration of the considered ion in the source or sink fluid (ML^{-3})

D , dispersion tensor (L^2T^{-1})

q , volumetric flux of water (LT^{-1})

R_d , $= 1 + K_d \frac{\rho}{\epsilon}$, retardation factor.

K_d , adsorption coefficient (L^3M^{-1})

θ , water content

α , modified coefficient of compressibility ($ML^{-3}T^{-1}$)

ψ , pressure head (L)

λ , decay constant (T^{-1})

The mass transport equation is coupled to the flow equation through the mass-average velocity q , which is obtained from Darcy's law as

$$q_a = -(K_r K_{a\beta}^s \frac{\partial \psi}{\partial x_\beta} + K_r K_{a3}^s) \quad (2)$$

with ψ , pressure head

K_r , relative hydraulic conductivity, and

K^s , hydraulic conductivity at saturation (LT^{-1})

The dispersion coefficient $D_{a\beta}$ in Equation (1) is also related to the velocity field. This coefficient includes the effect of mechanical dispersion and molecular diffusion. In field problems where groundwater velocities are not negligible, the mechanical dispersion term is generally the most significant and can be expressed in the form:

$$D_{a\beta}^M = a_L |V| \delta_{a\beta} + (a_L - a_T) \frac{V_\beta V_a}{|V|} \quad (3)$$

where a_L and a_T are the longitudinal and transverse dispersivities, respectively, and V_a is the component of velocity in the direction x_a .

The pressure head, ψ , in Equation (2) is obtained from the solution of the groundwater flow equation. In the saturated-unsaturated zone, the equation has been derived by Neuman (Ref. 2) in the form:

$$(K + \frac{\theta}{\epsilon} S_s) \frac{\partial \psi}{\partial t} - \frac{\partial}{\partial x_a} (K_r K_{a\beta}^s \frac{\partial \psi}{\partial x_\beta} + K_r K_{a\beta}^s) = Q \quad (4)$$

$$a, \beta = 1, 2, 3$$

with κ , moisture capacity (L^{-1})

$Q = Q_w(x_{wa}) \delta(x - x_{wa})$ strength of source of sink function (T^{-1})

Q_w , well discharge ($L^3 T^{-1}$)

S_s , specific storage (L^{-1})

x_{wa} , coordinate of discharge or recharge point (L)

ϵ , porosity

θ , moisture content

In the unsaturated zone, moisture content and hydraulic conductivity are functions of the pressure head; thus Equation (1) is nonlinear. In the saturated zone, K and θ are constant (although they may vary spatially), and Equation (3) can be simplified to the form:

$$L = S_s \frac{\partial \Psi}{\partial t} - \frac{\partial}{\partial x_\alpha} \left\{ K_{\alpha\beta}^s \left(\frac{\partial \Psi}{\partial x_\beta} + 1 \right) \right\} - Q = 0 \quad (5)$$

In many practical problems, the flow can be assumed to be two-dimensional and only a vertical cross-section of the three-dimensional space needs to be considered. The flow and solution movements are then described by Equations (4) and (1), respectively, where the indices α and β represent the X and Z directions of the two-dimensional space.

The flow and mass transport equations are solved in conjunction with a set of initial and boundary conditions. For the flow equations, these conditions take the form:

$$\Psi(x_\alpha, 0) = \Psi_0 \text{ in } \Omega \quad (6)$$

$$\psi(x_\alpha, t) = \bar{\psi} \quad \text{on } \Gamma_1 \quad (7a)$$

$$K_r (K_{\alpha\beta}^s \frac{\partial \psi}{\partial x_\beta} + K_{\alpha 3}^s) n_\alpha = -v \quad \text{on } \Gamma_2 \quad (7b)$$

where Ω is a three-dimensional domain bounded by a surface

$$\Gamma = \Gamma_1 + \Gamma_2 \quad \text{with outward normal } n.$$

Similarly, the initial conditions required to solve the mass transport equation are expressed in the form:

$$c(x_\alpha, 0) = c_0 \quad \text{in } \Omega \quad (8)$$

Dirichlet, Neumann, or mixed boundary conditions are specified in the form:

$$c(x_\alpha, t) = \bar{c} \quad \text{on } \Gamma_1' \quad (9a)$$

$$n_\alpha \frac{\partial c}{\partial x_\beta} = 0 \quad \text{on } \Gamma_2' \quad (9b)$$

$$(-D_{\alpha\beta} \frac{\partial c}{\partial x_\beta} + q_\beta c) n_\alpha = q_\beta n_\alpha C_0(t) \quad \text{on } \Gamma_3' \quad (9c)$$

$$\Gamma = \Gamma_1' + \Gamma_2' + \Gamma_3'$$

where $C_0(t)$ is the concentration of solute in the influx water.

In summary, the usual solution procedure is to a) calculate the pressure head using Equation (4) or (5) subject to specified initial and boundary conditions, b) calculate the velocities by substituting the known pressure head into Equation (2), c) substitute the velocities into Equation (1) and solve for the concentration c , d) repeat steps a) through c) for the next time interval.

2.0 GROUNDWATER FLOW MODEL

The model used to solve the groundwater flow equations, referred to as FLUMP, was originally developed by Drs. T.N. Narasimhan, S.P. Neuman, and P.A. Witherspoon (Ref. 3). In this model, the groundwater flow equation, Equation (4), is discretized using the Galerkin finite element method. The result is a system of first order linear or quasi-linear differential equations of the form:

$$[A]\{\Psi\} + [D]\left\{\frac{\partial \Psi}{\partial t}\right\} = \{Q\} \quad (10)$$

where $[A]$ is the stiffness matrix, $[D]$ is the capacity matrix, Q is the known forcing function and Ψ is the unknown vector of pressure heads (Ref. 2).

In this model, the problem is formulated so that $[D]$ is a diagonal matrix. This allows one to use large time steps (thereby improving the efficiency) and also avoids numerical instabilities when there is a drastic change in Q . This concept has been implemented in a new approach in which the system of differential equations (10) is discretized in time by finite differences; this procedure allows one to treat the equations either explicitly, implicitly, or by a combination of both schemes^(*).

The iterative procedure used to solve the equations at the so-called implicit nodes is known as the point acceleration method, originally developed by Evans et al (Ref. 4). Iterations are continued until

(*)A formula that expresses one unknown directly in terms of known values is called an explicit formula. Conversely, a method where the calculation of an unknown involves of a set of simultaneous equations is called implicit.

the maximum change in the unknown and the net error in fluid content are less than specified values. If the convergence criteria are not met, the calculations are repeated with a smaller time step.

The size of the time step is controlled automatically in the program by several factors: the lower and upper limits specified by the user, the desired maximum change of the unknown at any node during a time step, the stability criteria, the average number of iterations required for convergence, and the desired interval between printed outputs. At appropriate times, all explicit nodes are tested for stability and reclassified as implicit nodes, if necessary.

The printed output of FLUMP provides information on the nodal values of the dependent variable (ψ) in Equation (10) and its change between time steps, and on the estimated value of the time derivative at discrete time intervals specified by the user. Information on the degree of saturation of the system and the flux across the various boundaries is also provided, which makes it possible to maintain a continuous check on material balance. The program also warns against local instabilities. Should these occur, the mesh can be redesigned locally. An important feature of FLUMP is that this modification can be easily introduced by changing a few cards in the data deck. This is due to the nature of the solution technique, which is independent of the node and element number schemes.

3.0 CONTAMINANT TRANSPORT MODEL

The model used to solve the mass transport equations, referred to as GS2, is based on the Galerkin finite element method and is applicable to two- and three-dimensional problems in saturated and unsaturated media (Ref. 5). As mentioned previously, solution of the groundwater flow equations is a prerequisite to solving the contaminant transport problems. In the model, the pressure head and contaminant concentration are calculated sequentially by solving the following system of equations:

$$[S] \frac{d\{P\}}{dt} + [H] \{P\} + \{F\} = 0 \quad (11)$$

$$[M] \frac{d\{C\}}{dt} + [N] \{C\} + \{R\} = 0 \quad (12)$$

where [S], [H], [M], and [N] are matrices that can be calculated knowing the geometrical and physical properties of the medium, and where {F} and {R} are known forcing functions.

A finite difference scheme is introduced to evaluate the time derivatives in Equations (11) and (12). The symmetric system of algebraic Equation (11) is solved first for the pressure head using the Cholesky method. Then the values of pressure head, known at each nodal point of the finite element model, are used to compute moisture content, velocity, and dispersion coefficients appearing in the mass transport equation. The concentration distribution is then obtained from the solution of Equation (12) using a Gauss elimination procedure with partial pivoting to reduce round-off errors.

4.0 MODEL VALIDATION

The FLUMP and GS2 models have been verified by their authors as well as by Bechtel staff prior to their implementation in the Bechtel library of digital models.

Narasimhan et al (Ref. 3) describe several sample problems that were used to both verify the code and demonstrate the range of its applications. The problems used concerned:

- o The growth of a groundwater ridge under free-surface conditions and the influence of uniform recharge from an infinite strip. This problem has been studied experimentally by Marino (Ref. 6) with a vertical viscous analogue (Hele-Shaw) model and analytically by Hantush (Ref. 7).
- o Infiltration into a vertical column of unsaturated soil. The problem possesses an analytical solution derived by Philip (Ref. 8) and experimental results obtained by Van der Ploeg and Benecke (Ref. 9).
- o Drainage from a saturated-unsaturated system, for which Vauclin (Ref. 10) designed a complex experimental sandbox model.
- o Confined axisymmetric flow to a well with storage. For this problem, the numerical model results were compared with analytical solutions by Papadopoulos and Cooper (Ref. 11) and by Muskat (Ref. 12).

As shown in Narasimhan et al (Ref. 3), the model results are in satisfactory agreement with the analytical solutions.

The model GS2 was checked by simulating a one-dimensional infiltration experiment reported by Warrick et al (Ref. 13). In this experiment, a column of unsaturated soil was wetted by a solution of CaCl_2 , and the pressure and concentration movement with time were monitored. Comparison of model and experimental data is shown in Segol (Ref. 5). The same problem was solved using the three-dimensional version of the GS2 code, and identical results were obtained (this demonstrates that there are no procedural error in the Fortran code). The model was also used to simulate a seepage problem solved by Reeves and Duguid (Ref. 14), using also a Galerkin - Finite element scheme. The results obtained in the two codes were found in satisfactory agreement.

One will also note that Equation (11) provides pressure head at each nodal point, as Equation (10), but the numerical techniques used in the two models are significantly different. This redundancy was used to compare the pressure solutions obtained by each model. It was found that the values are in satisfactory agreement.

Comparing the results obtained with different numerical models is a useful technique for model validation, especially in situations where an analytical solution of the problem is not possible, as in this case.

APPENDIX B
REFERENCES

1. Duguid, J.O. and Mark Reeves. "Material Transport through Porous Media: A Finite-Element Galerkin Model," ORNL-4928, Oak Ridge National Laboratory, Oak Ridge, TN, 1976.
2. Neuman, S. P. "Saturated-Unsaturated Seepage by Finite Elements," Proceedings ASCE, Hydraulics Division, Vol. 99, HY12, pp. 2233-2250, 1973.
3. Narasimhan, T. N., S. P. Neuman, and P. A. Witherspoon. "Finite Element Method in Subsurface Hydrology Using a Mixed Explicit-Implicit Scheme," Water Resources Research, Vol. 14, No. 5, pp. 863-877, 1978.
4. Evans, G.W., R. J. Brousseau, and R. Kierstead. "Instability Considerations for Various Difference Equations Derived from the Diffusion Equation," USRL-4476, Lawrence Livermore Laboratory, Livermore, CA, 1954.
5. Segol, Genevieve. "A Three-Dimensional Galerkin-Finite Element Model for the Analysis of Contaminant Transport in Saturated-Unsaturated Porous Media," Proceedings of the First International Conference on Finite Elements in Water Resources, Princeton University, Princeton, NJ, pp. 2.123-2.144, 1976.
6. Marino, M. A. "Hele-Shaw Model Study of the Growth and Decay of Groundwater Ridges," Journal of Geophysical Research, Vol. 72, No. 4, pp. 1195-1205, 1967.
7. Hantush, M.S. "Growth and Decay of Groundwater Mounds in Response to Uniform Percolation," Water Resources Research, Vol. 3, No. 1, pp. 227-254, 1967.

8. Philip, J. R. "Theory of Infiltration," Advances in Hydroscience, Vol. 5, No. 216, 1969.
9. Van der Ploeg, R. R. and P. Benecke. "Unsteady, Unsaturated, N-dimensional Moisture Flow in Soil: A Computer Simulation Program," Soil Science Society of America Proceedings, Vol. 36, No. 6, pp. 881-885, 1974.
10. Vauclin, M. Etude experimentale et numerique du drainage de nappes a surface libre: Influence de la zone non saturee, Doctoral Thesis, University of Grenoble, Grenoble, France, 1975.
11. Papadopolous, I.S. and H. H. Cooper, Jr. "Drawdown in a Well of Large Diameter," Water Resources Research, Vol. 3, No. 1, pp. 241-244, 1967.
12. Muskat, M. Flow of Homogeneous Fluids Through Porous Media, J. W. Edwards, Ann Arbor, MI, 1946.
13. Warrick, A.W., J.W. Biggar, and D.R. Nielsen. "Simultaneous Salute and Water Transfer for an Unsaturated Soil," Water Resources Research, Vol. 7, No. 5, pp. 1216-1225, 1971.
14. Reeves, M. and J. O. Duguid. "Water Movement through Saturated-Unsaturated Media: A Finite Element Galerkin Model," ORNL-4927, Oak Ridge National Laboratory, Oak Ridge, TN, 1975.

APPENDIX C

DETERMINATION OF THE PROBABLE MAXIMUM FLOOD LEVEL

AT THE NIAGARA FALLS STORAGE SITE

DETERMINATION OF THE PROBABLE MAXIMUM FLOOD LEVEL
AT THE NIAGARA FALLS STORAGE SITE

1.0 SUMMARY

The Probable Maximum Flood (PMF) level at the Niagara Falls Storage Site has been estimated to be 323 ft m.s.l. The PMF level at the site would remain greater than 320.7 ft m.s.l. for 300 min during the passage of the PMF due to a 6-hour Probable Maximum Precipitation (PMP) event. An all-season 6-hour PMP of 24.0 in. was estimated for the site using methodology developed by the U.S. National Weather Service.

Figure C-1 shows the location of the waste containment area. A dike with a crest elevation of 325.25 ft m.s.l. surrounds the containment facility. Because the probability of a PMF occurrence is approximately 10^{-6} , the possibility of water overtopping the dike is extremely remote.

A flood peak of 40,500 cfs due to combined runoff to Sixmile and Twelvemile Creeks was calculated. The flood would produce a velocity near the dike of 5 fps. Riprap with a minimum size of 5 in. will be required to an elevation of approximately 323 ft m.s.l. around the base of any dike or earthen structure to protect against erosion.

2.0 METHODOLOGY

PMF levels at the site were determined by using a 6-hour PMP to develop a corresponding PMF and subsequently routing the PMF through the network of channels and flood plains which exist in the tributary area drainages. In summary, the study consisted of the following steps:

- o Development of a 6-hour PMP using the U.S. National Weather Service Hydrometeorological Reports 51, 52 and 53 (Refs. 1-3).

FIGURE C-1 NFSS DRAINAGE PATHWAY LOCATION PLAN

- o Development of PMF runoff hydrographs for all upstream tributary areas, including the drainages for Sixmile Creek and Twelvemile Creek. The U.S. Army Corps of Engineers Flood Hydrograph Package HEC-1 (Ref. 4) was used to generate the hydrographs.
- o Development of a routing network which represented Sixmile Creek, Twelvemile Creek, and the local drainages, including the West Drainage Ditch, the Central Drainage Ditch, and the S-31 Ditch. Examination of the topography indicated that "O" Street would control flood levels at the site; this factor formed the basis for determining the flood elevation and peak flows at the containment facility site.
- o Sizing of stone for a protective riprap layer.

2.1 Probable Maximum Precipitation (PMP) for the Site

An all-season, site-specific PMP was determined using U.S. National Weather Service Hydrometeorological Reports 51 and 52 (Refs. 1 and 3). The PMP was established as 24.0 in. for a 6-hour storm. PMP amounts for other durations (shown on Table 1) as well as the seasonal variation of the 6-hour PMP were derived from Hydrometeorological Report 53 (Ref. 3).

The effect of snow on the PMP was also investigated. Average snowfall at the site varies from 22 in. in January to 3 in. in April, with an annual average depth of 93 in. (Ref. 4). Relative to winter-month PMP amounts shown in Table C-1, snow contribution would not create a PMP event more critical than the all year PMP.

The 6-hour PMP, calculated using the procedure given in U.S. Army Corps of Engineers program HEC-1 (Ref. 5), was distributed into 5-min increments based on a triangular distribution. The distribution preserves the maximum 5-min, 15-min, 30-min, and 1-hour PMP amounts derived from Hydrometeorological Report 51, as shown in Table C-1.

TABLE C-1
PMP AT NIAGARA FALLS STORAGE SITE
SEASONAL VARIATION PMP (6-hour Duration, 10 mi² Area)

Month	In.
January	5.2
February	5.2
March	5.7
April	8.2
May	12.1
June	21.0
July	24.0
August	24.0
September	22.0
October	15.0
November	9.0
December	6.0

All-Season PMP

Duration (hours)	PMP for 10 mi ² Area (in.)	PMP for 1 mi ² Area (in.)
0.12	4.6	5.5
0.25	7.2	8.7
0.5	10.3	12.5
1.0	13.4	16.3
6.0	24.0	24.0
12.0	27.3	
24.0	29.0	
48.0	32.5	
72.0	34.0	

2.2 PMF Hydrographs for Sixmile Creek and Twelvemile Creek

Using HEC-1, the PMF hydrographs were generated for Sixmile Creek at Pletcher Road and Twelvemile Creek near the S-31 Ditch. The Snyder/Clark methodology was used to generate synthetic unit hydrographs for the basins. The required basin parameters, which were derived using the U.S. Geological Survey topographic map of the Ransonville Quadrangle, are shown in Table C-2. The basin lag time, which is a function of basin size and shape, was derived using the following U.S. Corps of Engineers expression:

$$\text{Lag} = n \left(\frac{L \cdot L_c}{S^{0.5}} \right)^{0.38}$$

where:

Lag = time in hours from the centroid of rainfall to the centroid of runoff

L = main stream distance from outlet to divide, in miles

L_c = stream distance from outlet to a point opposite the basin centroid, in miles

S = weighted channel slope in feet per mile

n = Manning's n value for basin channels

HEC-1 uses Clark's method to optimize the input parameters for the Snyder's method of unit graph generation. A cumulative value for the relationship between the surface area of the basin and the lapse time of runoff was developed for each basin as input to HEC-1.

TABLE C-2
PARAMETERS USED FOR HYDROGRAPH GENERATION
FOR SIXMILE AND TWELVEMILE CREEKS

Parameter	Sixmile Creek at Pletcher Road	Twelvemile Creek near S-31 Ditch
Drainage Area	5.4 mi ²	3.7 mi ²
Main Channel Length	2.6 mi	3.4 mi
Channel Length (outlet to basin centroid)	1.9 mi	2.0 mi
Weighted Channel Slope	19 ft/mi	27 ft/mi
Roughness Coefficient	0.035	0.035
Lag	0.75 h	0.80 h
Snyder's Peaking Coefficient	0.6	0.6

The basin was assumed to be saturated prior to occurrence of the PMP and no losses were assumed in calculating the overall excess. This procedure guarantees a conservative estimate of peak flow.

The computed peaks for the PMF are 35,300 cfs and 25,200 cfs for Sixmile and Twelvemile Creeks, respectively. Both peak approximately 3.83 hours after beginning of the PMP.

2.3 PMF Routing

The network shown in Figure C-2 was assumed in routing the PMF flows over "O" Street. Sixmile Creek was assumed to flow into the West Drainage Ditch and Twelvemile Creek into the S-31 Ditch. The PMP was applied to each local tributary area without losses to calculate runoff hydrographs for those areas. PMF routing network parameters for individual reaches are shown in Table C-3.

All channels would be overtopped during the PMF event, and the entire area around the site would be inundated. Calculations showed that "O" Street would control the flood levels at the project site.

The final routing included the PMP runoff from the area between Pletcher Road and "O" Street and the inflow hydrographs from the Sixmile and Twelvemile Creeks. The rating for flow over "O" Street was computed using the relationship for flow over a broad-crested weir. The centerline elevation of "O" Street is 319.7 ft m.s.l.

The equation $Q = 3.09 L H^{3/2}$ was used to calculate flow over "O" Street,

where:

Q = flow in cfs

L = length of "O" Street over which flow occurs (3,000 ft)

H = water depth over the crest, in feet

FIGURE C-2 CHANNEL NETWORK AT THE NFSS

TABLE C-3
PMF ROUTING: NETWORK PARAMETERS*

Reach No.	Flow to (Reach No.)	Tributary Area (acres)	Channel Parameters				Flood Plain Slope (%)	Manning's Value
			Length (ft)	Width (ft)	Depth (ft)	Slope (%)		
1	2	2368	5600	15	5	0.15	0.1	0.035
2	9	238	4725	5	5	0.10	0.1	0.035
3	9	61	2750	5	5	0.10	0.1	0.035
5	9	137	2750	5	5	0.10	0.1	0.035
7	13	87	3450	5	5	0.10	0.1	0.035
9	13	81	5040	15	10	0.11	0.1	0.035
10	11	3456	4200	20	7.5	0.25	0.1	0.035
11	13	220	5250	5	10	0.22	0.1	0.035
13	outlet	31	1890	20	10	0.10	0.1	0.035

*See Figure C-2

The PMF flow rate was determined to be 40,500 cfs, and the corresponding velocity at the site was calculated to be 5 fps. The maximum flood elevation at the site was found to be 3.8 ft above the ground elevation of 318.6 ft m.s.l., thereby giving a peak flood level of 323 ft. The flood levels and expected durations at the NFSS are shown in Table C-4.

2.4 Riprap Sizing

The flow velocity near the site could cause erosion of the dike unless riprap protection is provided. For PMF conditions at the site, riprap with a minimum size of 5 in. will provide adequate protection. The riprap was sized using procedures given in the U.S. Army Corps of Engineers Shore Protection Manual (Ref. 6).

TABLE C-4
PMF LEVELS AT THE NIAGARA FALLS STORAGE SITE

Maximum Water Level (Ft)	Duration (min)
320.7	300
321.7	135
322.2	60
322.3	40
322.4	5

APPENDIX C
REFERENCES

1. National Oceanic and Atmospheric Administration. Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No. 51, Washington, DC, June 1978.
2. National Oceanic and Atmospheric Administration. Application of Probable Maximum Precipitation Estimates, United States East of the 150th Meridian, Hydrometeorological Report No. 52, Washington, DC, August 1982.
3. National Oceanic and Atmospheric Administration. U.S. National Weather Service, Seasonal Variation of 10-Square-Mile Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No. 53, Washington, DC, April 1980.
4. Ruffner, James A., and Bair, Frank E. The Weather Almanac, Avon Press, New York 1981.
5. U.S. Army Corps of Engineers. The Hydrologic Engineering Center, HEC-1, Flood Hydrograph Package, Users Manual, September 1981.
6. U.S. Army Coastal Engineering Research Center. Shore Protection Manual, Vol. II, Ft. Belvoir, VA, 1977.

APPENDIX D

TYPICAL TECHNICAL SPECIFICATIONS FOR
NFSS WASTE CONTAINMENT FACILITY
EARTHWORK CONSTRUCTION:
CAP, DIKE, AND CUTOFF WALL

TECHNICAL SPECIFICATION

FOR

EARTHWORK CONSTRUCTION

(CAP)

LEWISTON, NEW YORK

NIAGARA FALLS STORAGE SITE

CONTENTS

	<u>Page</u>
1.0 GENERAL	D-2
2.0 ABBREVIATIONS	D-2
3.0 QUALITY STANDARDS	D-2
4.0 SUBMITTALS	D-3
5.0 MATERIALS	D-3
6.0 FIELD OPERATIONS	D-6

TECHNICAL SPECIFICATION
FOR
EARTHWORK CONSTRUCTION
(CAP)

1.0 GENERAL

This technical specification establishes the quality of workmanship required and defines how quality is measured for earthwork construction for the Waste Containment Area at the Niagara Falls Storage Site.

2.0 ABBREVIATIONS

The abbreviations listed below, where used in this specification, shall have the following meanings:

AASHTO	American Society of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
NYDOT	New York Department of Transportation Standard Specifications, Construction and Materials
USDA	United States Department of Agriculture

3.0 QUALITY STANDARDS

The Subcontractor shall control the quality of items and services to meet the requirements of this specification, applicable codes and standards, and other Subcontract documents. Unless otherwise specified or shown, the latest the following codes and standards of the latest issue at the time of Subcontract Bid shall apply to the extent indicated herein:

ASTM	D 1556	Standard Test Method for Density of Soil in Place by the Sand-Cone Method
ASTM	D 1557	Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-lb. Rammer and 18-in. Drop
ASTM	D 2216	Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures
NYDOT	Section 610	Turf establishment
NYDOT	Section 713	Landscape development materials

4.0 SUBMITTALS

Engineering documentation requirements are summarized on Form 14501-DR, and are augmented by detailed requirements listed herein. Bechtel will determine if documentation is complete as submitted by the Subcontractor, and reserves the right to disapprove any schedules, methods, equipment, plans, drawings, or other submittals that do not meet the Subcontract requirements.

5.0 MATERIALS

5.1 FILL MATERIALS

5.1.1 Fill material for earthwork construction shall be from approved on-site stockpiles or approved off-site borrow source. Designation and approval of an area for obtaining fill material does not indicate that all material within the area is suitable. Only suitable material, as determined by Bechtel, from approved borrow sources shall be used.

5.1.2 All fill material shall be free from brush, roots, peat, sod, ice, frozen soil, refuse, and other deleterious materials.

5.1.3 All clay fill designated on the design drawings shall be clay free of organic material, and gravel or rocks meeting the following requirements:

- o The plasticity index shall be not less than 10% nor more than 20.
- o The liquid limit shall be not less than 30% nor more than 50.
- o The permeability shall not be greater than 10^{-7} cm/sec.
- o Gradation:

<u>Sieve Size</u>	<u>Percent Passing by Weight</u>
3/4 - inch	100
No. 200 mesh	70

5.1.4 The soil for turf establishment is to be loam or silt/loam (USDA Textural Class, AASHTO A-4 or A-5) or other soil submitted to Bechtel for review. The Subcontractor shall be responsible for providing lime if required to facilitate grass growth.

5.1.5 The Subcontractor shall submit the location of proposed borrow sources to Bechtel and provide assistance to Bechtel for obtaining representative samples of the borrow area.

5.2 TURF ESTABLISHMENT

5.2.1 Grass seed and seed mixture shall be as follows:

<u>Name</u>	<u>Percent by Weight of Pure live seed</u>
Kentucky Bluegrass	60
Perennial Ryegrass	20
Fine Fescue	20

The seed shipment shall be accompanied by a weight slip and a certification by the supplier stating the minimum percentage of germination and purity of each kind of seed. The weight slips and supplier certification shall be submitted to Bechtel for review prior to use of the seeds.

5.2.2 Fertilizer shall be a standard quality commercial carrier of plant food elements and shall be in accordance with type No. 2 as specified in NYDOT specifications section 713-03, Fertilizer. A delivery slip shall accompany the shipment of fertilizer indicating the number of bags and weight of each. The composition of the fertilizer shall appear on each bag or the delivery slip and should be reviewed by Bechtel prior to use of the fertilizer.

5.2.3 Mulch and mulch anchoring materials shall be in accordance with acceptable materials listed in NYDOT specifications section 713. The type of mulch and mulch anchoring chosen shall be submitted to Bechtel for review prior to use.

5.3 SEDIMENTATION BARRIERS

Sedimentation barriers shall consist of either baled hay, baled straw, or filter cloth attached to posts on the up stream side. When baled hay or baled straw is used, each bale shall consist of no less than 5 cubic feet of material. When filter cloth is used, it shall be made expressly for use as a sedimentation barrier.

5.4 EROSION CONTROL

Jute mesh or other material approved by Bechtel shall be used to control erosion. Jute mesh and suitable anchoring materials shall be in accordance with NYDOT specifications section 713.

6.0 FIELD OPERATIONS

6.1 GENERAL

6.1.1 Subcontractor shall submit, for Bechtel's review, the proposed plans and methods for all earthwork operations at least three weeks prior to implementation. Submittals shall address proposed borrow sources, fill operations for clay and topsoil, routing of haul trucks, compaction method, construction water control, turf establishment, and sequence of operations.

6.1.2 Subcontractor shall, upon request, submit equipment information for all equipment, other than incidentals, to be used in earthwork operations. Submittals shall address equipment specifications, make, model and proposed use.

- 6.1.3 The Subcontractor shall perform surveys to establish locations, elevations (line and grade), and cross sections for all work required in this Subcontract. Line and grade for all locations shall be established in accordance with the grid coordinate system shown on the design drawings.
- 6.1.4 Existing observation wells, buildings, bench marks, earth dikes, or utilities shall not be damaged. Wells or utilities shall not be removed unless their removal is indicated on the design drawings. Any damage incurred on the above items shall be repaired at the expense of the Subcontractor.

6.2 REMOVAL OF CRUSHED STONE

- 6.2.1 Crushed stone material on the outer side of the dike shall be removed to the lines and grades shown on the design drawings.
- 6.2.2 In the event of over-excavation during removal of the crushed stone material, the over-excavated area shall be restored to the proper lines and grades in the manner specified by Bechtel and at the expense of the Subcontractor.
- 6.2.3 Excavated material shall be stockpiled in the area shown on the design drawings.

6.3 STOCKPILES

All material to be stockpiled shall be placed where shown on the design drawings. Stockpiled material shall be placed to provide natural drainage and a stable embankment. Stockpiled soils shall be compacted with a

minimum of 2 passes of a D-8 Caterpillar or similar method approved by Bechtel. The maximum uncompacted lift thickness of soil shall be 12 inches. Stockpiled crushed stone material need not be compacted.

6.4 SEDIMENTATION BARRIERS

6.4.1 Sedimentation barriers shall be installed to minimize the amount of sediment leaving the work area.

6.4.2 Sediment barriers shall be installed as shown on design drawings or as required to comply with state and federal sedimentation laws.

6.4.3 Filter cloth barriers shall be installed in accordance with the manufacturer's recommendations and shall be anchored by methods reviewed by Bechtel prior to use.

6.4.4 Baled hay or straw shall be laid end to end such that no gap exists between bales and securely fastened to the ground with at least two #4 reinforcing bars per bale. Reinforcing bars shall be a minimum of 2 1/2 feet long.

6.5 FILL - GENERAL REQUIREMENTS

6.5.1 Areas designated on the design drawings as requiring fill will be released to the Subcontractor by Bechtel in satisfactory condition and to the schedule stated in the Subcontract. Contaminated material in the waste containment area will be graded, compacted, and sealed (smooth rolled) by others prior to release. Any required rework of these areas, as determined by Bechtel,

due to the Subcontractor's mistreatment or delay shall be performed by and at the expense of the Subcontractor.

- 6.5.2 Fill shall not be placed on snow, ice, or frozen ground surfaces or when the temperature is 32°F or below.
- 6.5.3 All fill placement shall conform to the specified lines, grades, sections, and elevations shown on the design drawings. Grading tolerances shall be 0.1 foot for all surfaces of fill unless otherwise indicated on the design drawings.
- 6.5.4 Moisture content tests will be performed by Bechtel on fill material in accordance with ASTM D2216. Tests will be taken as deemed necessary with a minimum testing frequency of one test taken for each 200 cubic yards of fill placed or at least one test performed each day fill is placed.
- 6.5.5 Field density tests will be performed, by Bechtel, on fill material in accordance with ASTM D1556. Tests will be taken as deemed necessary with a minimum testing frequency of one test taken for each 500 cubic yards of fill placed.
- 6.5.6 All density and moisture testing performed by Bechtel will be done without cost to the Subcontractor, provided that the Subcontractor shall at its expense furnish labor, equipment, and materials to assist Bechtel in preparing test areas and obtaining samples for testing.

- 6.5.7 After placement but prior to compaction, fill material determined by Bechtel to be outside the moisture content limits stated herein, shall be moisture conditioned in place to within the specified moisture limits. Should it be impractical to condition the fill in place it shall be removed and replaced with material meeting the moisture limits. Such moisture conditioning or removal and replacement shall be by and at the expense of the Subcontractor.
- 6.5.8 Material satisfactorily placed and spread shall be compacted by rolling. Rolling shall be performed systematically on all portions of each area being compacted. Unless otherwise approved, all roller trips shall be made in parallel paths. To ensure complete coverage of the areas to be rolled, each pass of the roller shall overlap the adjacent pass by not less than 2 feet.
- 6.5.9 Earthwork equipment which has become contaminated, as determined by Bechtel, shall be decontaminated by others at facilities provided by Bechtel, prior to its reuse on clean material.

6.6 PLACING AND COMPACTING CLAY FILL MATERIAL

- 6.6.1 All clay fill furnished by the Subcontractor shall be placed and compacted prior to placement and compaction of Bechtel supplied clay fill.
- 6.6.2 Clay material shall be placed in the areas shown on the design drawings in loose lifts not to exceed 8 inches and shall be compacted to 95 percent of the maximum dry density as determined in accordance with ASTM D 1557.

- 6.6.3 Clay material shall be placed 1 percent to 3 percent wet of the optimum moisture content as determined in accordance with ASTM D1557.
- 6.6.4 The first lift of clay material shall be placed in a manner that prevents contamination of equipment. This shall be accomplished by pushing fill ahead of the spreading equipment thereby keeping the equipment on clean fill. Alternate methods of fill placement shall be submitted to Bechtel for approval prior to use.
- 6.6.5 Clay material shall be placed and compacted in a manner that assures cross contamination between the clay fill and existing waste is minimized. Should the top surface of the first lift of clay fill become contaminated or the surface of any other lift of clay become contaminated as determined by Bechtel, the contaminated clay material shall be removed and replaced as directed by Bechtel and at the expense of the Subcontractor.

6.7 TURF ESTABLISHMENT

- 6.7.1 The soil for turf establishment shall be placed, as shown on the design drawings, in two equal lifts and shall be compacted to 85 percent of the maximum dry density determined in accordance with ASTM D 1557.
- 6.7.2 Prior to the placement of the soil for turf establishment, depressions and other cavities in the clay surface greater than 0.1 foot shall be filled and compacted with clay to the level of the surrounding surface in lifts not to exceed 8 inches.

- 6.7.3 Subsequent to placement and compaction, the soil for turf establishment shall be conditioned, fertilized, seeded, and mulched in accordance with NYDOT specifications section 610.
- 6.7.4 Subsequent to placement of mulch, erosion control materials shall be used to prevent erosion of the soil and loss of seed on areas having slopes greater than 30 percent. Placement of materials shall be in accordance with NYDOT specifications section 613.
- 6.7.5 The Subcontractor shall mow seeded areas prior to acceptance by cutting to a height of three inches when growth reaches an average height of six inches or when the growth tends to smother seedlings.
- 6.7.6 Acceptance of established turf shall be after the second mowing.

6.8 CONSTRUCTION WATER CONTROL

- 6.8.1 The Subcontractor shall keep the construction area dewatered during fill activities. Runoff water which accumulates in the Waste Containment Area shall be pumped to the retention ponds.
- 6.8.2 During construction the top surface of the fill shall be sloped to the extent possible so that clean water flows away from the contaminated waste.
- 6.8.3 As soon as practicable, each lift of fill shall be sealed with pneumatic wheeled rollers or steel wheel rollers to minimize water infiltration. The sealed surface shall be scarified a minimum of 3 inches prior to placement of additional lifts.

- 6.8.4 Care shall be taken to ensure that contaminated water from the waste containment area does not flow into or over uncontaminated areas. Areas which become contaminated as a result of inadequate surface drainage control shall be restored in the manner specified by Bechtel. Such restoration shall be by and at the expense of the Subcontractor.

TECHNICAL SPECIFICATION
FOR
EARTHWORK CONSTRUCTION
(DIKE AND CUTOFF WALL)

LEWISTON, NEW YORK
NIAGARA FALLS STORAGE SITE

CONTENTS

	<u>Page</u>
1.0 GENERAL	3
2.0 ABBREVIATIONS	3
3.0 QUALITY STANDARDS	3
4.0 MATERIALS	4
5.0 FIELD OPERATIONS	5

TECHNICAL SPECIFICATION
FOR
EARTHWORK CONSTRUCTION

1.0 GENERAL

This technical specification establishes the quality and workmanship and defines how quality is measured for earthwork construction for the south dike at the Niagara Falls Storage Site.

2.0 ABBREVIATIONS

The abbreviations listed below, where used in this specification, shall have the following meaning:

ASTM American Society for Testing and Materials

NYDOT New York Department of Transportation Standard
 Specifications, Construction and Materials

3.0 QUALITY STANDARDS

Unless otherwise specified or shown, the following codes and standards of the latest issue on the date of proposal shall apply to the extent indicated herein:

ASTM D 1556 Standard Method of Test for
 Density of Soil in Place by the
 Sand Cone Method.

ASTM D 1557 Standard Method of Test for the
 Moisture-Density Relations of
 Soils using a 10-lb. Rammer and
 18-in. Drop.

4.0 MATERIALS

- 4.1 Fill material for earthwork construction shall be from approved offsite borrow source.
- 4.2 Designation and approval of an area for procurement of fill material does not indicate that all material within the area is suitable. Only suitable material from approved offsite borrow sources shall be used.
- 4.3 Material to be used for fill shall be free from brush, roots, peat, sod, ice, frozen soil, refuse, and other deleterious materials.
- 4.4. All fill material shall be radiologically clean as determined by Bechtel and its representative.
- 4.5 Areas requiring clay fill as designated on the design drawings shall be clay or silty clay. Gradation shall be such that over fifty percent (50%) by weight shall pass a 200 mesh sieve. The Subcontractor shall submit a certificate of compliance to Bechtel for the clay prior to placement.
- 4.6 All other fill material shall be uniformly graded earth material with a maximum stone size of 2 inches, unless noted otherwise on the design drawings.

- 4.7 Crushed stone for south dike stabilization as shown on the design drawings shall conform to NYDOT Standard Specifications Section 703-02, size 4. The Subcontractor shall submit a sieve analysis to Bechtel for review prior to shipment of the aggregate.

5.0 FIELD OPERATIONS

5.1 Excavation

- 5.1.1 The Subcontractor shall perform all survey and layout work required for the excavation.
- 5.1.2 The Subcontractor shall not damage or destroy existing observation wells, buildings or utilities. Wells or utilities shall not be removed unless their removal is indicated on the design drawings or directed by Bechtel in writing. Any damage done to the above items shall be repaired by the Subcontractor at his expense.
- 5.1.3 All excavation will be cut accurately to the lines and grades shown on the design drawings. Grading tolerances shall be zero to minus 0.5 feet along the slurry pipeline route and zero to minus 0.1 feet in all other areas of excavation unless noted otherwise.
- 5.1.4 Excavation beyond the lines and grades shown on the design drawings may be directed by Bechtel in order to embed the cutoff wall into the underlying, existing gray clay.

- 5.1.5 Over-excavated areas not previously authorized in writing by Bechtel, shall be restored to the proper elevations in the manner directed by Bechtel and at the expense of the Subcontractor.
- 5.1.6 All roots, stumps, rock, and foreign matter in, or exposed by, excavation shall be removed. Roots, stumps, and other organic material shall be disposed of in areas shown on the design drawings. If unsuitable material is encountered at the elevation indicated, Bechtel will specify the modifications that are deemed necessary. These modifications will be paid for at the stated unit prices.
- 5.1.7 Where the material to be excavated within the limits of this Subcontract is identified by Bechtel as contaminated material, the excavation shall be performed in accordance with Specification No. 14501-15-19-C-07, Excavation of Contaminated Materials. The revised classification for excavation shall be performed at the unit price noted in the conditional pay item.
- 5.1.8 The trench for the clay cutoff wall shall be excavated in sections not to exceed 200 feet. Compacted fill shall cover the trench section and shall extend above normal ground water level before excavation of the next 200 foot trench section may commence.

- 5.1.9 Temporary earth support systems shall be installed by the Subcontractor as required to stabilize steep excavation slopes which may be required in removing piping and utilities. Support systems shall be submitted to Bechtel for review upon request.
- 5.1.10 The Subcontractor shall, at its expense, dewater excavations. Water from dewatering operations shall be conveyed to existing water retention ponds as directed. Surface drainage from surrounding areas shall be diverted from the excavations. Springs or seepage shall be reported immediately to Bechtel and corrected as directed by Bechtel.

5.2 Stockpiling

- 5.2.1 Excavated materials shall be segregated into separate stockpiles for those that meet the requirements for future reuse defined in Section 4.0 and for those that do not meet the requirements for reuse. The reusable materials shall be further separated into stockpiles of soils to be used as clay fill and those classified as fill.
- 5.2.2 Stockpiled material shall be placed to provide natural drainage and stable slopes. The areas surrounding stockpiles shall be protected with straw or hay bales anchored with 2- #4 reinforcement bars per bale with a minimum length of 2 1/2 feet, or an equivalent method of erosion protection.

- 5.2.3 Stockpiled material shall be compacted with a minimum of two passes with the tracks of a D-8 Caterpillar or equal. Maximum stockpile slope shall be 1V to 3H.

5.3 Fill and Backfill

- 5.3.1 Ground surfaces to receive fill shall be scarified and compacted prior to the commencement of fill construction. At the bottom of the cutoff wall, surfaces of exposed clay, if not saturated, shall be disked and compacted to 90 percent of the maximum dry density to a depth of 6 inches prior to clay placement; if saturated, as determined by Bechtel, the initial loose lift of clay backfill shall be 2 feet deep, and compacted as per Section 5.5.3.
- 5.3.2 Materials shall not be placed on snow, ice, or frozen ground surfaces.
- 5.3.3 All fill and backfill shall conform within the tolerances specified, to the lines, grades, sections, and elevations shown on the design drawings. Grading tolerances shall be plus or minus 0.1 feet for all exposed surfaces of fill and backfill, unless otherwise indicated on the design drawings.

5.4 Backfill for Corrugated Steel Culvert

- 5.4.1 Backfill shall be equally placed along both sides of the culvert in uniform horizontal layers, and compacted. Layer thickness shall not exceed 12 inches before compaction.
- 5.4.2 All backfill for culverts shall be compacted to not less than 90 percent of the maximum dry density as determined in accordance with ASTM D 1557.

5.5 Placing and Compacting Fill and Backfill

- 5.5.1 Optimum moisture content will be determined by Bechtel in accordance with ASTM D 1557.
- 5.5.2 After placement of loose fill or backfill, the moisture content of the material shall be adjusted as necessary to obtain the required compaction or the material shall be removed and replaced with material having a moisture content to obtain specified compaction. Such removal and replacement shall be by and at the expense of the Subcontractor.
- 5.5.3 Material for the clay dike and cutoff wall, and the associated fill or backfill as shown on the design drawings, shall be placed in loose lifts not to exceed eight inches and shall be compacted to 95 percent of the maximum dry density. Other areas requiring fill or backfill shall be placed in lifts no greater than 12 inches and shall be compacted to 90 percent of the maximum dry density.

- 5.5.4 Rolling shall be performed systematically on all portions of each area being rolled. Unless otherwise approved, all roller trips shall be made in parallel paths. To ensure complete coverage of the areas to be rolled, each trip of the roller shall overlap the adjacent trip by 2 feet.
- 5.5.5 Immediately adjacent to building walls, and in other areas where mechanized rollers cannot operate, the specified material shall be placed in layers no greater than 3 inches, and shall be compacted by power tampers or vibrator compactors to achieve the same degree of compaction as required by roller compactors.
- 5.5.6 The tests for the specified compaction of fill or backfill shall be made by Bechtel in accordance with ASTM D 1556. Field tests will be taken as required with an average frequency of one test being taken per 200 cubic yards of fill or backfill placed, or at least once every day of fill construction. Additional testing may be performed when deemed necessary by Bechtel. All testing done by Bechtel will be done without cost to the Subcontractor, provided that the Subcontractor shall furnish labor, equipment, and materials to assist Bechtel in preparing test areas, and obtaining samples for testing.
- 5.5.7 Where compaction of existing ground, fill or backfill material does not meet the specified compaction, it shall be reworked as directed by Bechtel.

5.5.8 The embankment top surfaces of the dike shall be crowned at all times during construction to ensure that water will readily drain off.

5.6 Placement of Crushed Stone

Crushed stone shall be placed as shown on the design drawings.

APPENDIX E

DESIGN DRAWINGS FOR NFSS WASTE CONTAINMENT FACILITY:
CAP, DIKE, AND CUTOFF WALL
YEARS 1982 THROUGH 1985

CONTENTS

DIKE AND CUTOFF WALL

<u>Drawing Number</u>	<u>Revision</u>	<u>Title</u>
15-DD01-C-12	1	Dike and Ponds: Plan, Sections, and Details
15-DD19-C-01	0	South Dike Site and Vicinity Plan
15-DD19-C-02	1	South Dike Plan and Sections
15-DD19-C-03	2	South Dike Sections
15-DD19-C-04	2	South Dike Sections and Details
15-DD19-C-05	1	South Dike Soil Profile and EDPM Layout
15-DD19-C-06	2	South Dike Piping Plan and Schedule

CAP

<u>Drawing Number</u>	<u>Revision</u>	<u>Title</u>
301-DD27-C-01	0	Site and Vicinity Plans
301-DD27-C-02	0	Plan
301-DD27-C-03	0	Cross Sections and Detail
301-DD27-C-04	0	Cross Sections and Details
15-DD37-C-01	0	Site and Vicinity Plans
15-DD37-C-02	2	Phase Plan, Section and Detail
15-DD37-C-03	0	Plan and Details
15-DD37-C-04	2	Sections and Detail

18" ASTM C-76
CLASS IV
RCP CULVERT

5700

5800

5900

51000

51100

51200

51300

51400

51500

51600

51700

51800

51900

5200

5300

5400

5500

5600

5700

5800

5900

6000

6100

6200

6300

6400

6500

6600

6700

6800

6900

7000

7100

7200

7300

7400

7500

7600

7700

7800

7900

8000

8100

8200

8300

8400

8500

8600

8700

8800

8900

9000

9100

9200

9300

9400

9500

9600

9700

9800

9900

10000

10100

10200

10300

10400

10500

10600

10700

10800

10900

11000

11100

11200

11300

11400

11500

11600

11700

11800

11900

12000

12100

12200

12300

12400

12500

12600

12700

12800

12900

13000

13100

13200

13300

13400

13500

13600

13700

13800

13900

14000

14100

14200

14300

14400

14500

14600

14700

14800

14900

15000

15100

15200

15300

15400

15500

15600

15700

15800

15900

16000

16100

16200

16300

16400

16500

16600

16700

16800

16900

17000

17100

17200

17300

17400

17500

17600

17700

17800

17900

18000

18100

18200

18300

18400

18500

18600

18700

18800

18900

19000

19100

19200

19300

19400

19500

19600

19700

19800

19900

20000

20100

20200

20300

20400

20500

20600

20700

20800

20900

21000

21100

21200

21300

21400

21500

21600

21700

21800

21900

22000

22100

22200

22300

22400

22500

22600

22700

22800

22900

23000

23100

23200

23300

23400

23500

23600

23700

23800

23900

24000

24100

24200

24300

24400

24500

24600

24700

24800

24900

25000

25100

25200

25300

25400

25500

25600

25700

25800

25900

26000

26100

26200

26300

26400

26500

26600

26700

26800

26900

27000

27100

27200

27300

27400

27500

27600

27700

27800

27900

28000

28100

28200

28300

28400

28500

28600

28700

28800

28900

29000

29100

29200

29300

29400

29500

29600

29700

29800

29900

30000

30100

30200

30300

30400

30500

30600

30700

30800

30900

31000

31100

31200

31300

31400

31500

31600

31700

31800

31900

32000

32100

32200

32300

32400

32500

32600

32700

32800

32900

33000

33100

33200

33300

33400

33500

33600

33700

33800

33900

34000

34100

34200

34300

34400

34500

34600

34700

34800

34900

35000

35100

35200

35300

35400

35500

35600

35700

35800

35900

36000

36100

36200

- NOTES**
1. FOR GENERAL NOTES SEE DRAWING 20-C-600-91
 2. FOR PIPE REMOVAL INFORMATION SEE DRAWING 15-0019-C-06
 3. VALVE PIT FILLED WITH FILLCRETE TO THE TOP
 4. MATERIALS USED FOR SURFACING OF ROADS SHALL BE CRUSHED STONE TYPE 2, TO A MINIMUM OF 3/4" MAXIMUM SIZE, NEW YORK DOT STANDARD SPECIFICATIONS SECTION 304-2 AND 304-3.
 5. GRAVEL SHALL BE PLACED IN LAYERS NOT TO EXCEED 6 INCHES AFTER COMPACTION.
 6. RAMP STABILIZATION GEOTEXTILE FABRIC SHALL BE MINIMUM 500 G/SM SUBMITTED FOR REVIEW. THE FABRIC SHALL BE INSTALLED IN ACCORDANCE WITH MANUFACTURER'S WRITTEN INSTRUCTIONS.
 7. GRAVEL SURFACE SHALL BE GRADDED TO ALLOW FOR PROPER DRAINAGE AND NOT OFF FROM DITCHES OR DRAINAGE AREA.

- LEGEND**
- HYDRANT
 - (U.S. / LINES)
 - SLOPE LINES
 - POWER POLES
 - EXPLORATORY HOLES
 - SLOPE TO DRAIN

- REFERENCE DRAWINGS**
- 15-009-C-01 SITE AND VICINITY PLAN
 - 15-009-C-03 SECTIONS AND DETAILS
 - 15-009-C-04 SECTIONS AND DETAILS
 - 15-009-C-05 SOIL PROFILE AND EROSION LAYOUT
 - 15-009-C-06 PIPING PLAN AND SCHEDULE

ISSUED FOR AS-BUILT	DATE	BY
REVISED AS NOTED	DATE	BY
ISSUED FOR CONSTRUCTION	DATE	BY
NOTED	DATE	BY

**U.S. DEPARTMENT OF ENERGY
FORMERLY UTILIZED SITES
REMEDIAL ACTION PROGRAM**

BECHTEL NATIONAL, INC.
OAK RIDGE, TENNESSEE

**NIAGARA FALLS STORAGE SITE
SOUTH DIKE
PLAN AND SECTIONS**

14501

15-009-C-05 2

NOTES

1. FOR GENERAL NOTES SEE DWG. 15-DD19-C-01
2. FOR EXCAVATION LIMITS SEE DWG. 15-DD19-C-04
3. CRUSHED STONE USED FOR BURE PROTECTION ON DIKE SHALL BE IN ACCORDANCE WITH NEW YORK DOT STANDARD SPECIFICATIONS SECTION 703-02, SIZE 4.

REFERENCE DRAWINGS

- 15-DD19-C-01 SITE AND VICINITY PLAN
- 15-DD19-C-02 PLAN AND SECTIONS
- 15-DD19-C-03 SOIL PROFILE & EPM LAYOUT
- 15-DD19-C-06 PIPING PLAN AND SCHEDULE

SCALE: 1"=5' HORIZ. 1"=5' VERT.

NO.	DATE	REVISION	BY	CHKD.	APP'D.
1		ISSUED FOR AS-BUILT			
2		REVISED AS PER FOR 15-28			
3		ISSUED FOR CONSTRUCTION			

**U.S. DEPARTMENT OF ENERGY
FORMERLY UTILIZED SITES
REMEDIAL ACTION PROGRAM**

BECHTEL NATIONAL, INC.
OAK RIDGE, TENNESSEE

**NIAGARA FALLS STORAGE SITE
SOUTH DIKE
SECTIONS**

BECHTEL JOB NO.	DSE CONTRACT NO.
14501	DE-AC08-81OR20722
DRAWING NO.	REV. DATE
15-DD19-C-03	2

<u>REFERENCE DRAWINGS</u>	
15-0019-C-01	SITE AND VICINITY PLAN
15-0019-C-02	PLAN AND SECTIONS
15-0019-C-06	PIPING PLAN AND SCHEDULE

H
G
F
E
D
C
B
A

SOIL PROFILE ALONG SOUTH DIKE CENTERLINE
SCALE: HOR 1" = 50'
VERT 1" = 5'

EPDM LAYOUT

NOTES

- FOR GENERAL NOTES SEE DRAWING 15-DD19-C-01.
- PRESSURE TREATED TIMBER TO BE TREATED WITH WATER-BORNE PRESERVATIVE (AS TO COMPOUND RATE) IN ACCORDANCE WITH AMERICAN WOOD PRESERVERS BUREAU SPECIFICATION (P-22). RETENTION TO BE AT LEAST 0.5 LBS/C.
- DISPOSAL AREA "A" SHALL BE COVERED WITH A GALVANIZED HEXAGONAL WIRE REINFORCING WITH 2" OPENINGS AND 4" INCHES OF STYAM TO PROTECT THE EPDM MEMBRANE FROM ANY MISCELLANEOUS METALS OR RUBBLE.

LEGEND

REFERENCE DRAWINGS

15-DD19-C-01	SITE AND VICINITY PLAN
15-DD19-C-02	PLAN AND SECTIONS
15-DD19-C-03	SECTIONS
15-DD19-C-06	PIPING PLAN AND SCHEDULE

NO.	DATE	ISSUE DESCRIPTION	BY	CHECKED	APPROVED
1	10/1/88	ISSUED FOR CONSTRUCTION	JAS	CE	10/1/88
2	10/1/88	AS-BUILT	JAS	CE	10/1/88

U.S. DEPARTMENT OF ENERGY
FORMERLY UTILIZED SITES
REMEDIAL ACTION PROGRAM

BECHTEL NATIONAL, INC.
OAK RIDGE, TENNESSEE

NIAGARA FALLS STORAGE SITE
SOUTH DIKE
SOIL PROFILE & EPDM LAYOUT

BECHTEL JOB NO. DE-AC05-81OR20722

14501

DRAWING NO. REV. DATE

15-DD19-C-05

DATE

NO. DATE		ISSUE DESCRIPTION		BY	
SCALE NOTED		DESIGNED: DC		DRAWN: AKP	
U.S. DEPARTMENT OF ENERGY FORMERLY UTILIZED SITES REMEDIATION ACTION PROGRAM					
BECHTEL NATIONAL, INC. ONE RIVER, TORONTO					
NIAGARA FALLS STORAGE SITE INTERIM COVER SITE & VICINITY PLANS					
PROJECT NO.		DOE CONTRACT NO.			
14501		DE-AC05-81OR20722			
DRAWING NO.		REV.		DATE	
301-DD27-C-01		0

H
G
F
E
D
C
B
A

NOTES

- 1) FOR GENERAL NOTES SEE DRAWING 301-DD27-C-01.
- 2) SUBCONTRACTOR SHALL CONSTRUCT AND MAINTAIN AN ACCESS RAMP CAPABLE OF SUPPORTING TWO WAY TRAFFIC.
- 3) SUBCONTRACTOR SHALL MAINTAIN THE HAUL ROAD TO SUPPORT CONSTRUCTION ACTIVITIES.
- 4) SUBCONTRACTOR SHALL CONTROL DRAINAGE TO PREVENT PONDING OF WATER, PREVENT UNCONTAMINATED WATER FROM ENTERING THE RETENTION POND AND TO PREVENT CONTAMINATED WATER FROM FLOWING INTO CLEAN AREAS.
- 5) CONSTRUCTION OF INTERIM COVER SHALL BE COMPLETED IN THREE PHASES AS DISCUSSED IN THE SUBCONTRACT DOCUMENTS. EACH PHASE SHALL INCLUDE THE FOLLOWING AREAS:
 PHASE I FROM EAST SIDE OF HAUL ROAD TO E140
 PHASE II FROM E140 TO E233
 PHASE III FROM E233 TO E428
- 6) TURF ESTABLISHMENT, INCLUDING SOIL CONDITIONING, SEEDING, MULCHING, AND MULCH ANCHORING, SHALL BE COMPLETED WITHIN TWO WEEKS OF TOPSOIL PLACEMENT FOR EACH PHASE OF WORK IN ACCORDANCE WITH TECHNICAL SPECIFICATION 14501-301-27-C-01. LIMITS OF TURF ESTABLISHMENT SHALL INCLUDE ALL AREAS WHERE TOPSOIL IS PLACED.
- 7) SEDIMENTATION BARRIERS SHALL BE PROVIDED IN ACCORDANCE WITH TECHNICAL SPECIFICATION 14501-301-27-C-01.
- 8) DURING CONSTRUCTION ACTIVITIES, SUBCONTRACTOR SHALL REPLACE CRUSHED STONE RAMP WITH MATERIALS SPECIFIED FOR INTERIM COVER.
- 9) PRIOR TO PLACING CLAY MATERIAL ON THE OUTSIDE SLOPES OF THE WEST DIKE, BETWEEN THE ACCESS RAMP, SUBCONTRACTOR SHALL REMOVE CRUSHED STONE SURFACING AND PLACE IN THE STOCKPILE AREA. NO MORE THAN 1 INCH OF CLAY SHALL BE REMOVED WITH THE CRUSHED STONE.
- 10) PRIOR TO BEGINNING CONSTRUCTION ACTIVITIES, SUBCONTRACTOR SHALL LOCATE AND VISIBLY MARK ALL MONITORING WELLS WITHIN THE WORK AREA.
- 11) DURING PHASE II CONSTRUCTION AND PRIOR TO PLACEMENT OF COVER MATERIAL, EPDM SHALL BE REMOVED FROM SPILLWAY AREAS. THE SPILLWAYS SHALL BE FILLED WITH CLAY MATERIAL IN ACCORDANCE WITH SPECIFICATION 14501-301-27-C-01 TO EXISTING TOP OF DIKE ELEVATIONS.

LEGEND

● EXISTING MONITORING WELL

REFERENCE DRAWINGS

301-DD27-C-01 SITE & VICINITY PLAN
 301-DD27-C-03 CROSS SECTIONS & DETAILS
 301-DD27-C-04 CROSS SECTIONS & DETAILS

REVISIONS	DATE	BY	CHKD	APP'D
1	10/1/01	JAG	AKP	
2	10/1/01	JAG	AKP	
3	10/1/01	JAG	AKP	
4	10/1/01	JAG	AKP	
5	10/1/01	JAG	AKP	
6	10/1/01	JAG	AKP	
7	10/1/01	JAG	AKP	
8	10/1/01	JAG	AKP	
9	10/1/01	JAG	AKP	
10	10/1/01	JAG	AKP	

U.S. DEPARTMENT OF ENERGY
 FORMERLY UTILIZED SITES
 REMEDIAL ACTION PROGRAM

BECHTEL NATIONAL, INC.
 OAK RIDGE, TENNESSEE

NIAGARA FALLS STORAGE SITE
 INTERIM COVER
 PLAN

BECHTEL JOB NO. 14501
 DOE CONTRACT NO. DE-AC05-81OR21732

301-DD27-C-02	REV.	DATE
1		

SCALING RULES

301-DD27-C-03

NOTES

(1) FOR GENERAL NOTES SEE DRAWING 301-DD27-C-01.

LEGEND

REFERENCE DRAWINGS

301-DD27-C-01 SITE & VICINITY PLANS
301-DD27-C-02 PLAN
301-DD27-C-04 CROSS SECTION & DETAILS

NO.	DATE	REVISION	BY	CHECKED	DESIGNED	IN CHARGE
1		REVISED AS NOTED				
2		REVISED AS NOTED				
3		ISSUED FOR CONSTRUCTION				

U.S. DEPARTMENT OF ENERGY
FORMERLY UTILIZED SITES
REMEDIAL ACTION PROGRAM

BECHTEL NATIONAL, INC.

OAK RIDGE, TENNESSEE

NIAGARA FALLS STORAGE SITE
INTERIM COVER
CROSS SECTIONS & DETAIL

BECHTEL JOB NO.
14501

DOE CONTRACT NO.
DE-AC05-81OR21722

DRAWING NO. 301-DD27-C-03
REV. DATE 2

SECTION (5786) A
1" = 30' HORIZ
1" = 5' VERT

SECTION (5946) B
1" = 30' HORIZ
1" = 5' VERT

SECTION (5910) C
1" = 30' HORIZ
1" = 5' VERT

SECTION (51010) D
1" = 30' HORIZ
1" = 5' VERT

SECTION (51120) E
1" = 30' HORIZ
1" = 5' VERT

DETAIL (1) (TYP)
1/4" = 1' 0"

H
G
F
E
D
C
B
A

SECTION (E233) F
1" = 30' HORIZ
1" = 5' VERT

DETAIL 1
1/4" = 1'-0"
(TYPICAL FOR WEST DIKE)

DETAIL 2
1/4" = 1'-0"
(TYPICAL FOR NORTH & EAST DIKE)

DETAIL 3
NTS
C-02

SECTION

NOTES

- 1) FOR GENERAL NOTES SEE DRAWING 301-DD27-C-01.
- 2) SUBCONTRACTOR SHALL GRADE SWALE TO DRAIN SOUTH ALONG THE WESTERN EDGE OF THE INTERIM COVER.

LEGEND

REFERENCE DRAWINGS

301-DD27-C-01	SITE & VICINITY PLANS
301-DD27-C-02	PLAN
301-DD27-C-03	CROSS SECTIONS & DETAIL

NO.	DATE	ISSUE DESCRIPTION	BY	CHECKED	APPROVED
1		ISSUED FOR CONSTRUCTION	OC		
2		REVISED AS NOTED	JAG		
3		REVISED AS SHOWN	JAG		

U.S. DEPARTMENT OF ENERGY
FORMERLY UTILIZED SITES
REMEDIAL ACTION PROGRAM

BECHTEL NATIONAL, INC.
OAK RIDGE, TENNESSEE

NIAGARA FALLS STORAGE SITE
INTERIM COVER
CROSS SECTION & DETAILS

BECHTEL JOB NO. 14501
DOE CONTRACT NO. DE-AC05-81OR20722

DRAWING NO. 301-DD27-C-04
REV. 2
DATE

15-DD37-C-03

- ### NOTES
- FOR GENERAL NOTES, SEE DWG. 15-DD37-C-01.
 - CRUSHED STONE MATERIAL ON THE OUTSIDE OF THE DIKE SHALL BE REMOVED WITHIN THE LIMITS SHOWN. SAND REMOVAL SHALL NOT EXCEED A DEPTH INTO THE CLAY DIKE OF 0.2 FEET. STONE MATERIAL TO BE REMOVED SHALL BE STOCKPILED AT THE LOCATION SHOWN ON THIS DRAWING OR AS DIRECTED BY BECHTEL.
 - UPON COMPLETION OF WORK, SUBCONTRACTOR SHALL RESTORE THE LAYDOWN AREA TO ITS ORIGINAL CONDITION.
 - SUBCONTRACTOR SHALL ESTABLISH GRASS ON BANK SLOPES OF CENTRAL DITCH.
 - SURFACE CASING AND RISER CASING SHALL BE CUT 6" MINIMUM BELOW PVC ADAPTER AS SHOWN.
 - THE SUBCONTRACTOR SHALL FURNISH AND INSTALL ALL MATERIALS REQUIRED TO EXTEND EXISTING OBSERVATION WELLS AS SHOWN.
 - 2" SCH 40 PVC RISER CASING EXTENSION SHALL BE COUPLED TO EXISTING RISER CASING AS SHOWN AND IN ACCORDANCE WITH THE MANUFACTURER'S WRITTEN INSTRUCTIONS.
 - SUBCONTRACTOR SHALL REMOVE THE ENTIRE LENGTH OF THE 1/2" PVC PIPE AND DISPOSE OF AT THE STOCKPILE AREA FOR UNCONTAMINATED MATERIAL OR AS DIRECTED BY BECHTEL.
 - SURFACE CASING EXTENSION SHALL BE SCHEDULE 40, 6" STEEL.
 - SUBCONTRACTOR SHALL REPLACE OLD PVC PIPE CAP WITH A NEW PVC PIPE CAP WITHOUT HOSE BARB OR AIR LINE PLUG. THE OLD PVC PIPE CAP SHALL BE TRANSPORTED TO BECHTEL FOR STORAGE.
 - EXISTING SEDIMENT BARRIERS SHALL BE REMOVED AND DISPOSED OF AT THE STOCKPILE AREA FOR UNCONTAMINATED MATERIAL.
 - NEW SEDIMENT BARRIER SHALL BE PROVIDED IN ACCORDANCE WITH TECHNICAL SPECIFICATION M201-15, 37-C-01.
 - REMOVE 24" CULVERT AND TURNOUT GATE AND RESTORE DITCH GRADE. CULVERT IS LOCATED ALONG THE SOUTH SIDE OF "D" STREET AT APPROXIMATELY E1720.
 - ELEVATIONS SHOWN ARE FINISHED GRADE ELEVATIONS OF THE TOP SOIL.
 - PRIOR TO BEGINNING CONSTRUCTION ACTIVITIES, SUBCONTRACTOR SHALL LOCATE AND VISIBLY MARK ALL MONITORING WELLS WITHIN THE WORK AREA.
 - ACCESS WITHIN THESE LIMITS WILL REQUIRE COORDINATION WITH THE ON-SITE REMEDIAL ACTION SUBCONTRACTOR RESPONSIBLE FOR THE POND REMOVAL.
 - THE SOUTHERN RIDGE LINE WILL BE THE MINIMUM ELEVATION TO BE ESTABLISHED AND WILL INCREASE IF ADDITIONAL CONTAMINATED MATERIAL IS EXCAVATED AND BROUGHT TO THE WASTE CONTAINMENT AREA BY OTHERS.

- ### LEGEND
- EXISTING OBSERVATION WELL
 - OBSERVATION WELL TO BE EXTENDED
 - EXISTING SEDIMENT BARRIER (SEE NOTE 10)
 - NEW SEDIMENT BARRIER (SEE NOTE 11)

REFERENCE DRAWINGS

SITE AND VICINITY PLANS	15-DD37-C-01
PHASE PLAN, SECTION 1 DETAIL	15-DD37-C-02
INTERIM CAP - SECTIONS & DETAIL	15-DD37-C-04

ISSUED FOR CONSTRUCTION	DATE	BY	CHECKED	DATE

**U.S. DEPARTMENT OF ENERGY
FORMERLY UTILIZED SITES
REMEDIAL ACTION PROGRAM**

BECHTEL NATIONAL, INC.
OAK RIDGE, TENNESSEE

**NIAGARA FALLS STORAGE SITE
CAP CONSTRUCTION
INTERIM CAP - PLAN AND DETAILS**

CONTROL JOB NO. **14801** SUB CONTRACT NO. **02-ADG-000780722**

DRAWN BY **15-DD37-C-03** DATE **0**

WELL LOCATIONS

WELL NO.	SOUTH	EAST
A-23	51700.1	E194.1
A-42	51048.6	E13.6
A-43	51198.9	E20.0
A-47	5146.4	E402.8
A-50	5159.6	E446.6
A-51	5125.4	E490.2
A-52	5179.7	E300.1
A-54	5137.7	E11.7
A-55	5157.4	E10.5
A-56	5167.1	E118.3
A-57	5165.0	E125.5

NOTES

1. FOR GENERAL NOTES SEE DWG. 15-DD37-C-01.
2. WORK THIS DWG. WITH BASE AND OPTIONAL WORK SCHEDULE FOR INTERIM CAP (SEE DWG. C-02).

LEGEND

	3'-0" CLAY LAYER + MISC. CLAY TO BE PLACED
	1'-0" SOIL MATERIAL TO BE PLACED
	6" TOPSOIL TO BE PLACED
	1'-0" CLAY TO BE PLACED (OPTIONAL WORK)
	1'-0" SOIL MATERIAL TO BE PLACED (OPTIONAL WORK)
	6" TOPSOIL TO BE PLACED (OPTIONAL WORK)
	NOT IN CONTRACT (N.I.C.)

REFERENCE DRAWINGS

SITE AND VICINITY PLANS	15-DD37-C-01
CAP CONSTRUCTION / INTERIM CAP - PLAN	15-DD37-C-02

REVISED PHASE IV AND DELETED PHASE V	15-DD37-C-01
REVISED PHASE IV AND ADDED PHASE V	15-DD37-C-02
ISSUED FOR CONSTRUCTION	15-DD37-C-03
DATE	10/1/88
SCALE	AS SHOWN
DESIGNED	CAS
ENGR.	CRJ

U.S. DEPARTMENT OF ENERGY FORMERLY UTILIZED SITES REMEDIAL ACTION PROGRAM	
BECHTEL NATIONAL, INC. DAR RIDGE, TENNESSEE	
NIAGARA FALLS STORAGE SITE CAP CONSTRUCTION INTERIM CAP - SECTIONS & DETAIL	
BECHTEL JOB NO. 14501	DOE CONTRACT NO. DE-AC05-81OR20722
DRAWING NO. 15-DD37-C-04	REV. DATE 2