APPENDIX 4A – SECTION 1
PHASE I CRITICAL SYSTEMS DESIGN REPORT

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TABLES

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ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
Affiliate	CH2M HILL, Inc.
AFI	Air freeze index
ALR	Action leakage rate
AOS	Apparent opening size
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
AWWA	American Water Works Association
bgs	Below ground surface
CDN	Composite drainage net
CDR	Conceptual Design Report
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
CFR	Code of Federal Regulations
CH2M HILL	CH2M HILL Hanford Group, Inc.
Design Report	IDF Phase I Critical Systems Design Report
DOE	U.S. Department of Energy
DBVS	Demonstration Bulk Vitrification System
Ecology	Washington State Department of Ecology
EPA	U.S. Environmental Protection Agency
FH	Fluor Hanford, Inc.
FLA	Full load amperage
FS	Factor of safety
FVNR	Full Voltage Non-Reversing
GCL	Geosynthetic clay liner
GFCI	Ground fault circuit interrupters
gpm	Gallons per minute
GRI	Geosynthetic Research Institute
HDPE	High-density polyethylene
HEC	Hydraulic Engineering Circular-1
HELP	Hydrologic Evaluation of Landfill Performance (Model)
HF	Hanford Facility
HMS	Hanford Meteorological Station
HVAC	Heating, ventilating, and air conditioning
I/O	Input/output
ICDF	INEEL CERCLA Disposal Facility (Idaho Falls, ID)
IDF	Integrated Disposal Facility (Hanford)
IEEE	Institute of Electrical and Electronic Engineers
IES	Integrated Engineering Software, Inc.
ILAW	Immobilized low-activity waste
INEEL	Idaho National Environmental Engineering Laboratory
LAN	Local area network
LCRS	Leachate collection and removal system
LDS	Leak detection system
LERF	Liquid Effluent Retention Facility (Hanford)
LLW	Low-level waste
MBPS	Megabits per second
MCC	Motor control center
MLLW	Mixed low-level wastes
NEC	National Electrical Code
NFPA	National Fire Protection Association
OIU	Operator interface unit

ORP	Office of River Protection
PC	Performance category
PICS	Process Instrumentation and Control Systems
PLCs	Programmable logic controllers
PNNL	Pacific Northwest National Laboratory
psi	Pounds per square inch
PVC	Polyvinyl chloride
QA	Quality Assurance
QC	Quality Control
RAP	Response Action Plan
RCRA	Resource Conservation and Recovery Act of 1976
RF	Radio frequency
RGS	Rigid galvanized steel
RPP	River Protection Project
SCADA	Supervisory control and data acquisition
SDR	Standard dimension ratio
SOW	Statement of work
SPT	Standard Penetration Testing
SSCs	Systems, structures, and components
STI	Soil Technology, Inc. (Bainbridge Island, Washington)
THW	Thermoplastic, vinyl insulated building wire; flame retardant, moisture and heat resistant, 75°C, dry and wet locations
TSD	Treatment Storage and Disposal facility
TRU	Transuranic waste (concentrations of transuranic radionuclides greater than or equal to 100nCi/g of the waste matrix)
UBC	Uniform Building Code
UPS	Uninterrupted power supply
USCS	Unified Soil Classification System
WAC	Washington Administrative Code
WSDOT	Washington State Department of Transportation
WTP	Waste Treatment and Immobilization Plant (Hanford)

1.0 INTRODUCTION

1.1 Purpose

The purpose of the Integrated Disposal Facility (IDF) is to develop the capability for near surface disposal of Immobilized Low-Activity Waste (ILAW) waste packages from the River Protection Project–Waste Treatment and Immobilization Plant (RPP-WTP). The IDF is essential in meeting the overall U.S. Department of Energy, Office of River Protection (ORP) mission to store, retrieve, treat, and dispose of the highly radioactive Hanford tank waste in an environmentally sound, safe, and cost-effective manner. The IDF will also provide capacity for disposal of mixed low-level waste (MLLW) and low-level (LLW) from the DBVS. The detailed design for the IDF Phase I Critical Systems landfill will finalize the design process for the:

- X Landfill liner system
- X Leachate removal system
- X Leak detection system (LDS)

The IDF detailed design also involves completing all design work required for an operable landfill and supporting the Resource Conservation and Recovery Act of 1976 (RCRA) Part B permitting for the IDF.

This Phase I Critical Systems Design Report (the Design Report) provides documentation of engineering calculations, criteria, and information that have been developed as part of the IDF detailed design for Phase I. Specifically, the Design Report documents the following important design information:

- X Identifies key design requirements for the project (Section 2)
- X Summarizes studies on site conditions and investigations that have been used in the development of detailed design parameters for the critical systems (Sections 3 and 4)
- X Presents detailed engineering analysis performed in the development of the Phase I Critical Systems design and updated during construction implementation (Section 5). Provides system component descriptions, references important construction quality assurance (QA) requirements, and describes important interfaces with non-critical systems (Section 6)

Describes operating provisions that have influenced the development of the design including waste placement requirements, operational interfaces with other Hanford facilities, and leakage response action plan requirements (Section 7).

1.2 Scope

1.2.1 General

CH2M HILL, Inc. (Affiliate) is responsible for production of a cost-effective final design and to produce critical systems detailed design documents and construction specifications to facilitate RCRA permit approval of the IDF. The IDF technical requirements are found in the following documents:

- Immobilized Low-Activity Waste (ILAW) Project Definition Criteria, Revision 1 (RPP-7898)
- System Specifications for ILAW Disposal, Revision 3 (RPP-7307)
- Hanford Environmental Management Specification (DOE/RL-97-55)

Design products are to be prepared in compliance with the technical requirements, as well as with other specific procedures that are dictated by CH2M HILL Hanford Group, Inc. (CH2M HILL) requirements and outlined in the Statement of Work (SOW), *Integrated Disposal Facility Detailed Design Support* (Rev. 2, 2003), described in more detail under Section 2 of this Design Report. The overall design work includes reports, schedules, estimates, and other special services as specified in the SOW. As part of the design effort, the Affiliate will perform the following global tasks:

- Develop a conceptual layout and preliminary design drawings for the IDF. The IDF preliminary layout will depict a single expandable landfill system, with capability for segregation of RCRA regulated and non-regulated waste placement and segregated leachate management systems.

- Develop a detailed design that meets the requirements of the ILAW Project Definition Criteria and the ILAW System Specification.
- Develop the construction specifications for the detailed design.
- Ensure that there is full technical integration between all detailed design reports prepared for the detailed design of the IDF.
- Perform the design activities in accordance with all applicable regulatory requirements.

The design will implement the safety and health protection requirements imposed on the design by the SOW and the technical baseline criteria documents, and will comply with all applicable regulatory requirements for the project. It is important to note that although the design is for identified critical systems of the Phase I IDF, a preliminary safety evaluation was performed for the W-520 Project that identified no safety class items, including criticality safety (*Conceptual Design Report for ILAW Facility*, CH2M HILL, May 2001).

The timely completion of the critical system detail design of the IDF, in compliance with the RCRA permit approval process (Washington Administrative Code [WAC] 173-303-665 and 173-303-806[4][h]), is a critical component of the SOW. Drawings, construction specifications, and reports needed to obtain U.S. Department of Energy (DOE) certification and Washington State Department of Ecology (Ecology) approval of the IDF RCRA Part B permit is the overall goal of the project. The detailed design for the initial Phase I disposal landfill and the critical systems design include the liner system, the leachate collection system, and the LDS. The detailed design will produce an operable landfill design and support the IDF RCRA Part B permitting.

1.2.2 Design Report

The Design Report describes the key facility components and provides the design basis and detailed calculations that support the development of drawings and specifications. Key facility components that are described in the Design Report include:

Facility layout (location, access roads and operational ramps, survey control system)

Landfill geometry (disposal volume total and per disposal unit, disposal unit dimensions)

Disposal unit grading design (foundation soils contour, lower admixture layer contour, operations layer cover contour)

Grid point listing (grid point number, location, and elevation for all grid points required for construction of the IDF)

Geosynthetic material design (primary geomembrane, secondary geomembrane, geotextile, and geocomposite drainage layer)

Leachate collection and removal system (LCRS) and LDS design (sump design, removal system design–LCRS and LDS, leachate level monitoring system design, transfer pump as required to meet WAC-173-303-665(2)(h)(ii) to ensure that the leachate depth over the liner does not exceed 12 inches).

Leachate temporary storage tank system design (tank volume, tank design, tank materials/ leachate compatibility, tank coating, tank secondary containment system), including electrical and power requirements necessary to support the leachate removal systems.

Pump controls and instrumentation design (control, operations, monitoring, and control building design).

Operational storm water management design

Backfill placement requirements and process (minimize void space, minimize subsidence of waste, placement and material requirements to ensure there are no adverse effects on the waste packages).

Other facility designs identified as necessary to support the project completion.

The Design Report includes design calculations that are prepared in accordance with the requirements of procedure HNF-IP-0842 Vol. 4, Section 3.6 (July 30, 2002). Important calculations that are documented include:

Stability (liner side slope [each liner layer based on interface strength], requirements for verification for critical interface strengths, fill placement ramp, global stability of the overall design, and other relevant stability analysis).

Seismic analysis (side slope and global embankment stability under seismic loading, and seismic design of structures)

Bearing capacity (liner sub-grade soils and other relevant bearing capacity analysis)

Total settlement, differential settlement, and uplift analysis (foundations soils, compacted admixture layers, total settlement, top slope drainage evaluation, subsidence and sinkhole potential, uplift potential, and other relevant settlement analysis).

Admix liner analysis (liner admixture bearing capacity, admix liner specifications, desiccation cracking, and other relevant liner admixture analysis).

Geomembrane liner analysis (liner tension caused by thermal contraction/ expansion, anchor trench pullout analysis, puncture resistance, potential stress cracking, leachate compatibility, chemical and radiation resistance, mechanical degradation from operational traffic, and other relevant geomembrane analysis).

Drainage layers analysis (geotextile analysis and selection, geocomposite selection, drainage gravel selection analysis, and other relevant drainage analysis).

LCRS/LDS analysis (clogging prevention in LCRS, design of leachate collection sumps, design of high capacity and low capacity leachate removal pumping systems, design of leachate storage tank and secondary containment system, leachate depth monitoring system, design of leachate system control building, leachate compatibility of components in the LCRS, and other relevant leachate analysis).

Leachate system earth loading analysis (LCRS and LDS slope riser pipes, LCRS collection pipe, leachate transfer pipes, and other relevant system loading analysis).

Surface stormwater analysis (operations in-cell stormwater management, operations runon/runoff water management, site stormwater collection/evaporation management system, and other relevant storm water analysis).

Leachate production analysis (average annual leachate production, peak daily leachate production, leachate tank storage capacity, leachate transportation truck capacity, and trip frequency)

Action leakage rate (ALR) analysis (the maximum design flow rate that the secondary leachate collection, detection, and removal system can remove without the fluid head on the bottom liner exceeding one foot; calculation and justification of the maximum leachate infiltration rate through the primary liner system; a response action plan in case the maximum ALR is exceeded during operation of the IDF).

Updates to calculations that have occurred through the construction process, either during independent quality reviews of tank systems, in response to contractor's requests for information, or changes implemented during construction have been attached to the original calculations in the appendices.

Compliance matrices have been developed to demonstrate detailed design compliance with the applicable sections of the regulations (WAC 173-303) and with project-specific specifications, criteria, reports, codes, and standards. Updates to the matrices that have resulted from the completion of construction activities and associated documentation are also provided. These matrices are presented in the Design Report in Appendix A.

1.3 Authorization

After careful consideration and evaluation, CH2M HILL elected to self-perform the IDF Phase I Critical Systems design. As such, the design is being performed as an inter-company work assignment by the Affiliate under the direction of CH2M HILL. CH2M HILL was authorized to self-perform the work by the U.S. Department of Energy, Office of River Protection (ORP), in a letter dated December 9, 2002.

CH2M HILL's Prime Contract Number with the ORP is DE-AC06-99RL14047. The inter-company work assignment is Contract 12317, Release 22, dated November 7, 2002.

1.4 General Facility Description

The IDF will consist of an expandable lined landfill located in the 200 East area on the Hanford Facility (HF). The landfill will be divided lengthwise into two distinct cells, one for disposal of low-level waste (LLW) and the other for disposal of mixed waste. The mission of the IDF will include the following functions:

- Provide an approved disposal facility for the permanent, environmentally safe disposition of ILAW packages that meets the environmental requirements and is approved by the DOE and Ecology.
- Receive ILAW from River Protection Project (RPP) tank operations and dispose this waste onsite. Receive waste from the DBVS and dispose this waste onsite.
- A more detailed discussion of waste types and the necessary storage volumes for these wastes is provided in Sections 5 and 6, respectively.
- The IDF will be constructed on 25 hectares of vacant land southwest of the PUREX Plant in the 200 East Area. The IDF will consist of a lined landfill that will be constructed in several phases. The landfill will be segregated into a RCRA permitted cell and a non-RCRA permitted cell. The scope of this permit is limited to the western cell of the landfill where the RCRA waste will be stored and disposed. The landfill is designed to accommodate four layers of vitrified LAW waste containers separated vertically by 0.9-meters of soil.
- This initial construction will start at the northern edge and the size is approximately 223 meters East/West by 233 meters North/South by 14 meters deep. At this initial size, IDF disposal capacity is 82,000 cubic meters of waste. Subsequent construction phase(s) will require a modification to the Part B Permit to be constructed after waste placement has progressed in the landfill to the point that additional disposal capacity is needed. This approach minimizes the open area susceptible to collection of rainwater and subsequent leachate
- The landfill is currently estimated at full build out to be up to 446 meters wide by 555 meters in length by up to 14 meters deep. The RCRA regulated portion of the landfill would be half of that at approximately 223 meters wide by 555 meters long by up to 14 meters deep providing a waste disposal capacity of up to 450,000 cubic meters.
- Both cells will have a RCRA C-compliant liner system that consists of an upper primary liner overlying a lower secondary liner. The upper liner will consist of a composite geomembrane liner and geosynthetic clay liner system on the bottom area, and a single geomembrane on the side slope. The secondary liner will consist of a composite geomembrane, overlying a 3-foot-thick soil admix liner. A LCRS and a LDS will overly the primary and secondary liner system, respectively. A Secondary Leak Detection System (SLDS) will be located below the clay liner, beneath the LDS sump.
- The IDF also will include a less than 90-day accumulation area of leachate for storage in two tanks, one per landfill half. The leachate storage tanks will be located at the north end, in close proximity to the lined landfill. Each tank will be protected by secondary containment (double-lined tanks). Leak detection will be provided by monitoring of the secondary containment. The collected leachate will be stored and sampled before transfer to an onsite Treatment Storage and Disposal (TSD) unit or offsite TSD facility. The less than 90-day storage

leachate collection tank will be operated in accordance with the generator provisions of WAC 173-303-200 and WAC 173-303-640, as referenced by WAC 173-303-200. The overall side development plan is shown in Figure 1-2.

- The landfill will be constructed in several phases. Starting at the northern edge, approximately one-third of the total length of the landfill will be constructed in Phase I. This will include the leachate collection system and 90-day accumulation tanks. The subsequent phases will be constructed after waste has been placed in the landfill and additional disposal capacity is needed. This approach will minimize the amount of open area susceptible to collection of rainwater and subsequent leachate.
- Before disposal, all waste will meet land disposal restriction requirements [Revised Code of Washington 70.105.050(2), WAC 173-303-140, and 40 Code of Federal Regulations (CFR) 268, incorporated by reference in WAC 173-303-140].
- Future landfill development and configuration within the IDF will be subject to change as disposal techniques improve or as waste management needs dictate. Additional IDF landfill development beyond the 62 acres will be subject to an approved permit modification, in accordance with the HF RCRA Permit (Ecology, 2001).
- Public access to the IDF will be restricted. Figure 1-3 depicts the normal transportation routes within the 200 East Area. Trucks typically will be used to transport waste to the IDF and will range in size from heavy-duty pickups to tractor-trailer rigs, depending on the size and weight of the load. In some cases, special equipment (such as transporters) will be used for unusual or unique loads. When special equipment is used, a prior evaluation will ensure that the equipment does not damage the roadways.
- Approximately 60 personnel will traverse this roadway, in personal vehicles in three shifts per 24 hours per week.

Figure 1-1. Integrated Disposal Facility Site Plan

Located in Chapter 1.0, Part A Form

Figure 1-2. Overall Site Development Plan/Transportation Routes

Located in Chapter 4.0, Figure 4.1

2.0 DESIGN REQUIREMENTS

Minimum design requirements for the IDF Phase I Critical Systems Design were provided by CH2M HILL in the SOW for Requisition # 92859, Integrated Disposal Facility Detailed Design Support, Revision 2, February 18, 2003. The IDF Phase I Critical Systems Design has been performed in compliance with all applicable design requirements, defined in Sections 2.1 through 2.7, and these requirements are:

- *Washington State Dangerous Waste Regulations* (WAC 173-303)
- *System Specification for Immobilized Low-Activity Waste Disposal System*, Revision 3 (RPP-7307)
- *ILAW Project Definition Criteria for Integrated Disposal Facility,* Revision 1 (RPP-7898)
- *Hanford Site Environmental Management Specification,* Revision 2 (DOE/RL-97-55)
- *Design Loads for Tank Farm Facilities* (TFC-ENG-STD-06, REV A)
- Technical baseline documents listed in Section 3.1 of the SOW
- Applicable national codes and standards

2.1 Washington State Dangerous Waste Regulations

The *Washington State Dangerous Waste Regulations* (WAC 173-303) implement Subtitle C of Public Law 94-580, the RCRA in the State of Washington. By conforming to the requirements of WAC 173-303, the design of the IDF Phase I Critical Systems also complies with the federal hazardous waste requirements contained in 40 CFR 264, *Standards for Owners and Operators of Hazardous Waste Treatment, Storage, and Disposal Facilities*. Appendix A.1 provides a compliance matrix of where the applicable WAC 173-303 requirements are addressed in the IDF Phase I Critical Systems detailed design documents, or are addressed in documentation developed as a result of facility construction.

2.2 System Specification

The *System Specification for Immobilized Low-Activity Waste Disposal System*, Revision 3 (RPP-7307) contains the Level 1 system requirements for the Immobilized Low-Activity Waste Disposal System, of which the IDF is a part. Appendix A.2 provides a compliance matrix of where the applicable Level 1 system requirements are addressed in the IDF Phase I Critical Systems detailed design documents, or are addressed in documentation developed as a result of facility construction.

2.3 Project Definition Criteria

The *ILAW Project Definition Criteria for Integrated Disposal Facility*, Revision 1 (RPP-7898) contains the design criteria for the IDF, including requirements flow-down from RPP-7303, *System Specification for ILAW Disposal System*, and DOE/RL-97-55, *Hanford Site Environmental Management Specification*. Appendix A.3 provides a compliance matrix of where the applicable design criteria are addressed in the IDF Phase I Critical Systems detailed design documents, or are addressed in documentation developed as a result of facility construction.

2.4 Hanford Site Environmental Management Specification

The *Hanford Site Environmental Management Specification* (site specification), Revision 2 (DOE/RL-97-55) documents the top-level mission technical requirements for work involved in the Richland Operations Office, Hanford Site cleanup and infrastructure activities, under the responsibility of the DOE Office of Environmental Management. It also provides the basis for all contract technical requirements. Section 3.3.2, 200 Area Materials and Waste Management of the site specification contains the requirements for receiving and onsite disposal of ILAW from RPP tank operations. The documents, orders, and laws referenced in the site specification represent only the most salient sources of requirements. As such, the site specification is assumed to have no significant measurable requirements that would directly affect the IDF Phase I Critical Systems design.

2.5 Design Loads for Tank Farm Facilities

The *Design Loads for Tank Farm Facilities* (TFC-ENG-STD-06, REV A) defines the design requirements for systems, structures, and components (SSCs), and provides the minimum criteria for structural design and evaluation of SSCs. The standard establishes structural design loads and acceptance criteria for use in designing new SSCs. Figure 1 of this standard indicates that for new SSCs, structures and anchorage of systems and components are to be designed per DOE-STD-1020-02 and Section 3.0 of this standard. These were used for the design of the IDF Critical Systems facilities. The IDF Critical Systems facilities were defined by CH2M HILL as being Performance Category (PC)-1. The PC-1 requirements in this standard were used in the structural design of the facilities included in IDF Phase I Critical Systems.

2.6 Technical Baseline Documents

The technical baseline documents are listed in Section 3.1 of the SOW. These documents include the System Specification for Immobilized Low-Activity Waste Disposal System, ILAW Project Definition Criteria for Integrated Disposal Facility, Hanford Site Environmental Management Specification, and Design Loads for Tank Farm Facilities, discussed in the preceding sections.

2.7 National Codes and Standards

In addition to WAC 173-303, the system specification, project definition criteria, site specification, and tank farm design loads that are discussed above, the IDF Phase I Critical Systems design was guided by other applicable sections of accepted professional and industry standards. These included the following:

- Air Moving and Conditioning Association
- American Association of State Highway and Transportation Officials (AASHTO)
- American Concrete Institute
- American Galvanizers Association
- American Institute of Steel Construction
- American Iron and Steel Institute
- American National Standards Institute
- American Society for Testing and Materials (ASTM)
- American Society of Civil Engineers (ASCE)
- American Society of Heating, Refrigerating, and Air-Conditioning Engineers
- American Society of Mechanical Engineers
- American Water Works Association (AWWA)
- American Welding Society
- Building Officials and Code Administrators – Basic Building Code
- Code of Federal Regulations (CFR)
- Concrete Reinforcing Steel Institute (CRSI)
- Federal Standards
- Geosynthetic Research Institute (GRI)
- Hydraulic Institute Standards
- Institute of Electrical and Electronic Engineers (IEEE)
- International Conference of Building Officials – Uniform Building Code (UBC)
- Manufacturers Standardization Society
- Metal Building Manufacturers Association
- National Electrical Code (NEC)
- National Electrical Manufacturers Association

- National Fire Protection Association (NFPA)
- National Institute of Standards and Technology
- Occupational Safety and Health Administration
- Sheet Metal and Air Conditioning Contractors National Association
- Steel Door Institute
- Steel Structures Painting Council
- Specialty Steel Institute of North America
- The Aluminum Association, Inc.
- Underwriters Laboratories, Inc.

Washington State Department of Transportation (WSDOT) Standard Specifications for Road, Bridge and Municipal Construction

3.0 SITE CONDITIONS

This section presents information on the Hanford Site and the area on the site where the IDF will be located. This information was obtained primarily from the *ILAW Preliminary Closure Plan for the Disposal Facility* (RPP-6911) and other Hanford Site data sources. It is intended to provide a general characterization of the IDF site conditions that are pertinent to the design of the IDF Phase I Critical Systems.

3.1 Geography

The following paragraphs briefly describe the geography of the IDF site and are prepared from information in the *ILAW Preliminary Closure Plan for the Disposal Facility* (RPP-6911).

3.1.1 Site Location

The location of the IDF is on the Hanford Central Plateau, in the 200 East Area within the Hanford Site boundary. The site identified for the IDF is 68 hectares (168 acres) of vacant and uncontaminated land, located southwest of the PUREX plant in the 200 East Area. It is bounded on the south by 1st Street and on the north by 4th Street.

3.1.2 Site Description

The IDF landfill will occupy approximately 25 hectares (62 acres) of the site identified for the facility. The remainder of the site will be used for soil stockpile, leachate storage tanks, operations support facilities, roads, parking areas, and open space. The IDF in Phase I will be approximately 11 hectares (28 acres). Phase I will be located at the north end of the IDF landfill and will include provisions for expansion to the south for future phases.

3.2 Meteorology and Climatology

The following paragraphs briefly describe the climate of the IDF site and are prepared from information in the *ILAW Preliminary Closure Plan for the Disposal Facility* (RPP-6911), which presented summary data from the Hanford Meteorological Station (HMS). Conditions at the HMS are considered similar to those at the IDF site. Detailed information is available in the *Hanford Site Climatological Data Summary 2001, with Historical Data* (Pacific Northwest National Laboratory, May 2002). The IDF Phase I Critical Systems is designed to operate in the climatic conditions reported in that document.

3.2.1 Precipitation

The site sits within the Pasco Basin, characterized as a semi-arid region because of its low annual precipitation levels. The basin receives 16 cm (6.3 inches) of annual average precipitation, with nearly half occurring in the winter months. Historical records indicate that the annual precipitation has varied from a low of 8 cm (3.1 inches) to a high of 30 cm (11.8 inches). Precipitation of 4 cm (1.56 inches) in 24 hours reportedly can be expected to occur once every 25 years. However, based on the *Hanford Site*

Climatological Data Summary 2001, a value of 1.28 inches was used for the 24-hour, 25-year precipitation in the IDF Phase I Critical Systems stormwater design analysis (see Appendix C.9). Total annual snowfall has varied from 0.8 cm to 110 cm (0.31 to 43.3 inches), with an average annual snowfall of 34 cm (24.4 inches).

3.2.2 Temperature

Temperature conditions for the site range from extremely cold during the winter months to extremely warm during the summer months. Local temperatures can reach -18 degrees C (0 degrees F) during some winter months. January is the coldest month, with an average temperature of -2 degrees C (29 degrees F). The lowest temperature ever recorded was -33 degrees C (-27 degrees F). During some summer months, daytime temperatures can exceed 40 degrees C (104 degrees F). July is the warmest month, with daily high and low temperatures averaging 33 and 25 degrees C (92 and 61 degrees F), respectively. The highest temperature ever recorded was 46 degrees C (115 degrees F).

3.2.3 Wind

Wind conditions can vary considerably throughout the year. The monthly average is about 10 kilometers/hour (6 miles/hour) during the winter and 15 kilometers/hour (9 miles/hour) during the summer. Wind speeds, especially during summer storm activity, can reach many times the average levels. The greatest peak gust was 130 kilometers/hour (81 miles/hour), recorded at 15 meters (50 feet) above the ground at the HMS.

3.2.4 Relative Humidity

The seasonal variation in the relative humidity is considerable, according to records of the HMS. The annual mean relative humidity recorded at HMS is approximately 54 percent, with the highest monthly average relative humidity (80 percent) occurring in December and the lowest monthly average relative humidity (32 percent) occurring in July. Daily relative humidity can change 20 to 30 percent between early morning and late afternoon, except in the winter months when changes are less pronounced.

3.3 Ecology

The following paragraphs briefly describe the ecology of the Hanford Site and are prepared from information in the *ILAW Preliminary Closure Plan for the Disposal Facility* (RPP-6911). The site consists of undeveloped land and is characterized as a shrub-steppe environment. This environment contains numerous plants and animal species, adapted to the regions semi-arid climate. Because of the aridity and low water-holding capacity of the soils, the productivity of both plants and animals is relatively low. The IDF site exhibits many of these same general characteristics, although to varying degrees.

3.3.1 Flora

The dominant plants on the Hanford Site are big sagebrush, rabbitbrush, cheatgrass, Russian thistle, and Sandberg's bluegrass, with cheatgrass providing half of the plant cover. Root penetration to depths of over 3 m has not been demonstrated in the 200 Areas. Rabbitbrush roots have been found only at a depth of 2.4 m (8 feet) near the 200 Areas.

3.3.2 Fauna

A variety of birds and mammals inhabit the Hanford Site. The most abundant nesting birds of the shrub-steppe at the site are the horned lark and western meadowlark. Significant populations of chukar and grey partridge inhabit the Hanford Site. The most abundant mammals at the site are mice, ground squirrels, gophers, voles, and cottontail rabbits. Larger animals include mule deer and elk. The coyote is the principal mammalian predator on the Hanford Site.

3.4 Geology

3.4.1 Regional Geology

The 200 East Area lies on the Cold Creek bar, a geomorphic remnant of the cataclysmic, glacial related floods of the Pleistocene Epoch. As the floodwaters raced across the lowlands of the Pasco Basin and Hanford Site, floodwaters lost energy and began to deposit sand and gravel. The 200 Area Plateau is one of the most prominent deposits. The 200 Area Plateau lies just southwest of one of the major flood channels across the Hanford Site that forms the topographic lowland south of Gable Mountain.

Borehole data provide the principal source of geologic, hydrologic, and groundwater information for the 200 East Area and the IDF site. Numerous boreholes (both vadose zone boreholes and groundwater monitoring wells) have been drilled in the 200 East Area for groundwater monitoring and waste management studies (Figure 3-1 shows the location of groundwater wells near the IDF site). However, data are limited within the IDF site, primarily because no previous construction or waste disposal activities have occurred in this part of the HF. Most boreholes in the 200 East area have been drilled using the cable tool method and either a hard tool or drive barrel to advance the hole. Some boreholes have been drilled by rotary and wire-line coring methods. More recently, boreholes in the area have been drilled, and in five cases cored, by percussion hammer methods. Geologic logs are based on examination of drill core, chips, and cuttings from these boreholes. Chip samples typically are taken at 1.5-meter (4.92 feet) intervals and routinely archived at the Hanford Geotechnical Sample Library.

3.4.2 Site Geology

The IDF site will be located south of the Gable Mountain segment of the Umtanum Ridge anticline and about 3 kilometers (1.86 miles) north of the axis of the Cold Creek syncline, that controls the structural grain of the basalt bedrock and the Ringold Formation. The basalt surface and Ringold Formation trend roughly southeast-northwest parallel to the major geologic structures of the site. As a result, the Ringold Formation and the underlying Columbia River Basalt Group gently dip to the south off the Umtanum Ridge anticline into the Cold Creek syncline.

Geologic mapping on the Hanford Site and examination of drill core and borehole cuttings in the area have not identified any faults in the vicinity of the IDF site (DOE/RW-0164). The closest known faults are along the Umtanum Ridge-Gable Mountain structure, north of the disposal site and the May Junction Fault east of the site (Figure 3-2).

3.4.2.1 Stratigraphy

The basalt and post-basalt stratigraphy for the IDF site is shown in Figure 3-3. Approximately 137 to 167 meters (449 to 548 feet) of suprabasalt sediments overlie the basalt bedrock at the site.

Basalt Bedrock. Previous studies (RHO-BWI-ST-14; Reidel and Fecht, 1994) have shown that the youngest lava flows of the Columbia River Basalt Group at the 200 East Area are those of the 10.5 million-year old Elephant Mountain Member. This member underlies the entire 200 East area and surrounding area, and forms the base of the suprabasalt aquifer. No erosional windows in the basalt are known or suspected to occur in the area of the IDF site.

Ringold Formation. Few boreholes penetrate the entire Ringold Formation at the IDF site, so available data are limited. The Ringold Formation reaches a maximum thickness of 95 meters (312 feet) on the west side of the site and thins eastward. The member of Wooded Island (Figure 3-3) is the only member of the Ringold Formation in the 200 East Area. The deepest Ringold Formation unit encountered is the lower gravel, unit A. Lying above unit A is the lower mud, and overlying the lower mud is an upper gravel, unit E. The sand and silt units of the members of Taylor Flat and Savage Island of the Ringold Formation are not present at the IDF site. Unit A and unit E are equivalent to the Pliocene-Miocene continental conglomerates (Reidel and Fecht, 1994). The lower mud is equivalent to the Pliocene-Miocene continental sand, silt, and clay beds (Reidel and Fecht, 1994).

Only three boreholes have penetrated unit A in the area of the IDF site. Unit A is 19 meters (62 feet) thick on the west side of the site and thins to the northeast. Unit A is partly to well-cemented conglomerate consisting of both felsic and basaltic clasts in a sandy matrix and is interpreted as fluvial gravel facies (Lindsey, 1996). There are minor beds of yellow to white interbedded sand and silt. Green-colored, reduced-iron stain is present on some grains and pebbles. Although the entire unit appears to be cemented, the zone produced abundant high-quality water in borehole 299-E17-21 (PNNL-11957, 1998).

Nineteen meters (62 feet) of the lower mud unit were encountered in one borehole at the IDF site (PNNL-11957, 1998). The uppermost one-meter or so consists of a yellow mud to sandy mud. The yellow mud grades downward into about 10 meters (33 feet) of blue mud. The blue mud, in turn, grades down into seven meters (23 feet) of brown mud with organic rich zones and occasional wood fragments. The lower mud unit is absent in the center of the site (northeast of borehole 299-E24-7 on Figure 3-4).

Unit E is described as a sandy gravel to gravelly sand. Unit E is interpreted to consist of as much as 15 meters (49 feet) of conglomerate, with scattered large pebbles and cobbles up to 25 centimeters (9.84 inches) in size in a sandy matrix. The gravel consists of both felsic and basaltic rocks that are well rounded, with a sand matrix supporting the cobbles and pebbles. Cementation of this unit ranges from slight to moderate. The upper contact of unit E is not identified easily at the IDF site. In the western part of the study area, unconsolidated gravels of the Hanford formation directly overly the Ringold Formation unit E gravels, making exact placement of the contact difficult. The dominance of basalt and the absence of cementation in the Hanford formation are the key criteria used to distinguishing these (PNNL-11957, 1998). In the central and northeast part of the area, unit E has been eroded completely. Unconsolidated gravels and sands typical of the Hanford formation replace unit E.

Unconformity at the Top of the Ringold Formation. The surface of the Ringold Formation is irregular in the area of the IDF site. A northwest-southeast trending erosional channel or trough is centered through the northeast portion of the site. The trough is deepest near borehole 299-E24-21 in the northern part of the site (PNNL-13652, 2001). This trough is interpreted as part of a larger trough under the 200 East Area, resulting from scouring by the Missoula floods.

Hanford formation. The Hanford formation is as much as 116 meters (381 feet) thick in and around the IDF site. The Hanford formation thickens in the erosional channel cut into the Ringold Formation and thins to the southwest along the margin of the channel.

At the IDF site, the Hanford formation consists mainly of sand dominated facies and less amounts of silt dominated and gravel dominated facies. The Hanford formation has been described as poorly sorted pebble to boulder gravel and fine- to coarse-grained sand, with lesser amounts of interstitial and interbedded silt and clay. In previous studies of the site (WHC-MR-0391, 1991), the Hanford formation was described as consisting of three units: an upper and lower gravel facies and a sand facies between the two gravelly units. The upper gravel dominated facies appears to be thin or absent in the immediate area of the IDF site (PNNL-12257, 1999; PNNL-13652, 2001; PNNL-14029, 2002).

The lowermost part of the Hanford formation encountered in boreholes at the IDF site consists of the gravel-dominated facies. Drill core and cuttings from boreholes 299-E17-21, 299-E17-22, 299-E17-23, 299-E17-25, and 299-E24-21 indicate that the unit is a clast-supported pebble- to cobble-gravel with minor amounts of sand in the matrix. The cobbles and pebbles almost are exclusively basalt, with no cementation. This unit pinches out west of the IDF site and thickens to the east and northeast (Figure 3-4). The water table beneath the IDF site is located in the lower gravel unit. The lower gravel unit is interpreted to be Missoula flood gravels, deposited in the erosional channel carved into the underlying Ringold Formation.

The upper portion of the Hanford formation consists of at least 73 meters (240 feet) of fine-to coarse-grained sand, with minor amounts of silt and clay and some gravelly sands.

Holocene Deposits. Holocene, eolian deposits cover the southern part of the IDF site. Caliche coatings on the bottom of pebbles and cobbles in drill cores through this unit are typical of Holocene caliche development in the Columbia Basin. The southern part of the IDF site is capped by a stabilized sand dune. The eolian unit is composed of fine- to coarse-grained sands with abundant silt, as layers and as material mixed with the sand.

Clastic Dikes. A **c**lastic dike was encountered in borehole C3828, adjacent to well 299-E17-25 at the IDF site. Clastic dikes also have been observed in excavations surrounding the site (e.g., U.S. Ecology, the former Grout area, the 216-BC cribs, the Central Landfill, and the Environmental Restoration Disposal Facility [PNNL, BHI-01103]). In undisturbed areas such as the IDF site, clastic dikes typically are not observed because these are covered by wind blown sediments. The occurrence of a clastic dike in borehole C3828 suggests that these probably are present elsewhere in the subsurface at the disposal site.

3.4.3 Seismology

The IDF will be located in Zone 2B, as identified in the UBC (DOE/RL-91-28). The analyses in Sections 5.1 and 5.12 provide additional seismic detail for design of liner and structural systems.

No active faults, or evidence of a fault that has had a displacement during Holocene times, have been found on the Hanford Site (DOE/RL-91-28). The youngest faults recognized on the Hanford Site occur on Gable Mountain, over 4.5 kilometers (2.78 miles) north of the 200 East Area. These faults are Quaternary of age and are considered 'capable' by the Nuclear Regulatory Commission (DOE/RL-91-28).

3.5 Hydrology

The following paragraphs briefly describe the known hydrology conditions of the Hanford Site and most specifically the 200 Area Plateau where the IDF site is located. These are prepared from information in the *ILAW Preliminary Closure Plan for the Disposal Facility* (RPP-6911).

3.5.1 Surface Water

The IDF site is within the 200 East area, which is on a plateau above the Columbia River. The Columbia River runs generally to the east and swings around the site, lying about 8 miles northwest and northeast of the 200 East area. The project area is significantly higher than the Columbia River and is not in the river’s floodplain.

The soils in the project area are sandy with high rates of infiltration. Most of the precipitation falling on the site infiltrates into the ground, and there are no significant long-term surface water features in the project area.

3.5.2 Groundwater

The geologic structure of the 200 East area is composed of multiple layers of sediments that range from sand, silt, volcanic ash, and clay to coarse gravels, cobbles, and conglomerates that overlay thick layers of basaltic lava. An unconfined aquifer exists in the lower part of the sedimentary sequence, overlaying the uppermost basalt layer. This relatively thin aquifer intercepts infiltration from the unsaturated zone above it. The aquifer under the IDF site is approximately 90 to 100 meters (300 to 330 feet) below the ground surface. Therefore, the groundwater table is well below the proposed bottom of the excavation for the IDF and is not expected to influence the facility. The recharge of water into the ground at the IDF site is expected to be small. This condition results primarily from the low levels of annual precipitation that occur in the region of the IDF as well as the rest of the Hanford Site. A more detailed description of groundwater beneath the IDF, developed from various site explorations performed in the site area, is presented below.

The unconfined aquifer under the IDF site occurs in the fluvial gravels of the Ringold Formation and flood deposits of the Hanford formation. The thickness of the aquifer ranges from about 70 meters (230 feet) at the southwest corner of the site to about 30 meters (98 feet) under the northeast corner of the IDF site. The Elephant Mountain Member of the Columbia River Basalt Group forms the base of the unconfined aquifer (Figure 3-3).

The unsaturated zone beneath the land surface at the IDF site is approximately 100 meters (328 feet) thick and consists of the Hanford formation. The water level in boreholes in and around the site indicates that the water table is in the lower gravel sequence of the Hanford formation and at an elevation of approximately 123 meters (404 feet) above sea level. The water table is nearly flat beneath the IDF site. Table 3-1 gives water level information from wells near the site. The locations of the wells are shown on Figure 3-1. The latest water table map shows less than about 0.1 meter (3.94 inches) of hydraulic head across the IDF site (PNNL-13404, 2001).

The Ringold Formation lower mud unit occurs within the aquifer at the southwest corner of the IDF site (299-E17-21) but is absent in the central and northern parts of the site (299-E24-7 and 299-E24-21). The lower mud unit is known to be a confining or partly confining layer at places under the Hanford Site (PNNL-12261, 2000), and this might be the case under the southwest corner of the IDF site. Groundwater samples were collected and analyzed from above and below the lower mud unit during drilling of well 299-E17-21. Chemical parameters (pH, electrical conductivity, and Eh) were different in the two samples, suggesting that the lower mud is at least partly confining in the area. No contamination was found above or below the lower mud. An interpretation of the distribution and thickness of this stratum is shown in Figure 3-4. The surface of the lower mud unit is interpreted to dip gently to the southwest (PNNL-13652, 2001).

Hydrographs for selected wells near the IDF site are shown in Figures 3-5 and 3-6. Hydrographs for the older wells (299-E23-1, 299-E23-2, and 299-E24-7) show two maxima in the water level. These coincide with the operation of the PUREX Plant that operated between 1956 and 1972 and between 1983 and 1988. All the hydrographs show a decline in the water table during recent years. The rate of decline is between 0.18 and 0.22 meters (7.08 and 8.66 inches)/year and will take between 10 and 30 years to stabilize. The reason for the decline is the cessation of effluent discharge to the PUREX Plant and to the 216-B Pond System, centered northeast of 200 East area. Based on hindcast water table maps (PNNL, BNWL-B-360), the water table is expected to decline another 2 to 7 meters (7 to 23 feet) before reaching pre-Hanford Site elevations. The cessations of effluent discharge also are responsible for changing the direction of groundwater flow across much of the 200 East area.

Groundwater flow beneath the IDF site recently was modeled to be southeasterly (PNNL-13400, 2000). This direction differs from the easterly direction, predicted by the analysis of WHC-SD-WM-RPT-241 and other earlier reports. The southeasterly flow direction primarily is attributable to inclusion of the highly permeable Hanford formation sediments in the ancestral Columbia River/Missoula flood channel in the analysis. A southeasterly flow direction is reflected in the geographic distribution of the regional nitrate and tritium plumes in the south-central 200 East area (Figure 3-7) (PNNL-13788, 2002.). As stated in PNNL-13404 (2001), the water table gradient is too low to be used for determining flow direction or flow rate at the PUREX Plant cribs, immediately east of the IDF site.

Hydraulic conductivity directly beneath the IDF site was estimated from data collected during four slug tests at well 299-E17-21 and five slug tests of 299-E24-21. The interval tested at 299-E17-21 was the upper 7.8 meters (26 feet) of the unconfined aquifer from 101.3 to 109.1 meters (332 to 358 feet) depth. That portion of the aquifer is Hanford formation gravel, from 101.3 to 102.1 meters (332 to 335 feet) depth, and Ringold Formation unit E gravels, from 102.1 to 109.1 meters (335 to 358 feet) depth (PNNL-12257, 1999). The interval tested at well 299-E24-21 was entirely in the Hanford formation gravel sequence between 95.2 and 101.3 meters (312 and 332 feet) depth. The best-fit value to the data from 299-E17-21 indicated a hydraulic conductivity of about 68.6 meters (225 feet) per day (PNNL-12257, 1999), and that from 299-E24-21 suggested a hydraulic conductivity of 75 meters (246 feet) per day (PNNL-13652, 2001).

Table 3-1. Water Levels in Groundwater Wells in the Vicinity of the IDF Site

Well	Measure date	DTW [a]	WT elev m[b]	Ref elev m[c]
299-E13-10	3/14/02	101.7	122.5	226.31
299-E17-12	3/14/02	100.0	121.1	221.09
299-E17-13	4/12/01	97.7	122.6	220.34
299-E17-17	4/12/99	97.8	122.8	220.54
299-E17-18	10/3/02	98.5	122.3	220.76
299-E17-20	4/9/97	97.1	123.2	220.33
299-E17-21	4/23/98	100.4	122.7	224.26
299-E17-22	5/20/02	98.1	122.5	220.59
299-E17-23	5/20/02	101.6	122.2	223.84
299-E17-25	5/21/02	98.3	126.7	225.03
299-E18-1	3/14/02	98.2	122.4	220.65
299-E18-3	6/27/96	97.8	123.4	221.20
299-E18-4	6/27/96	97.7	123.4	221.05
299-E19-1	3/22/88	100.4	124.9	225.26
299-E23-1	3/14/02	96.0	122.4	218.39
299-E23-2	12/20/94	97.2	123.5	220.77
299-E24-4	8/10/98	90.6	122.9	213.47
299-E24-7	6/11/97	96.2	123.2	219.34
299-E24-16	10/4/02	97.7	122.3	220.02
299-E24-17	4/7/97	97.36	122.9	220.16
299-E24-18	10/2/02	98.0	122.3	220.35
299-E24-21	3/22/01	95.4	122.6	217.85

a DTW = depth to water
b WT elev = elevation of water table (meters above mean sea level)
c Ref elev = reference elevation (meters above mean sea level, North American Vertical Datam 88 reference), generally top of well casing.

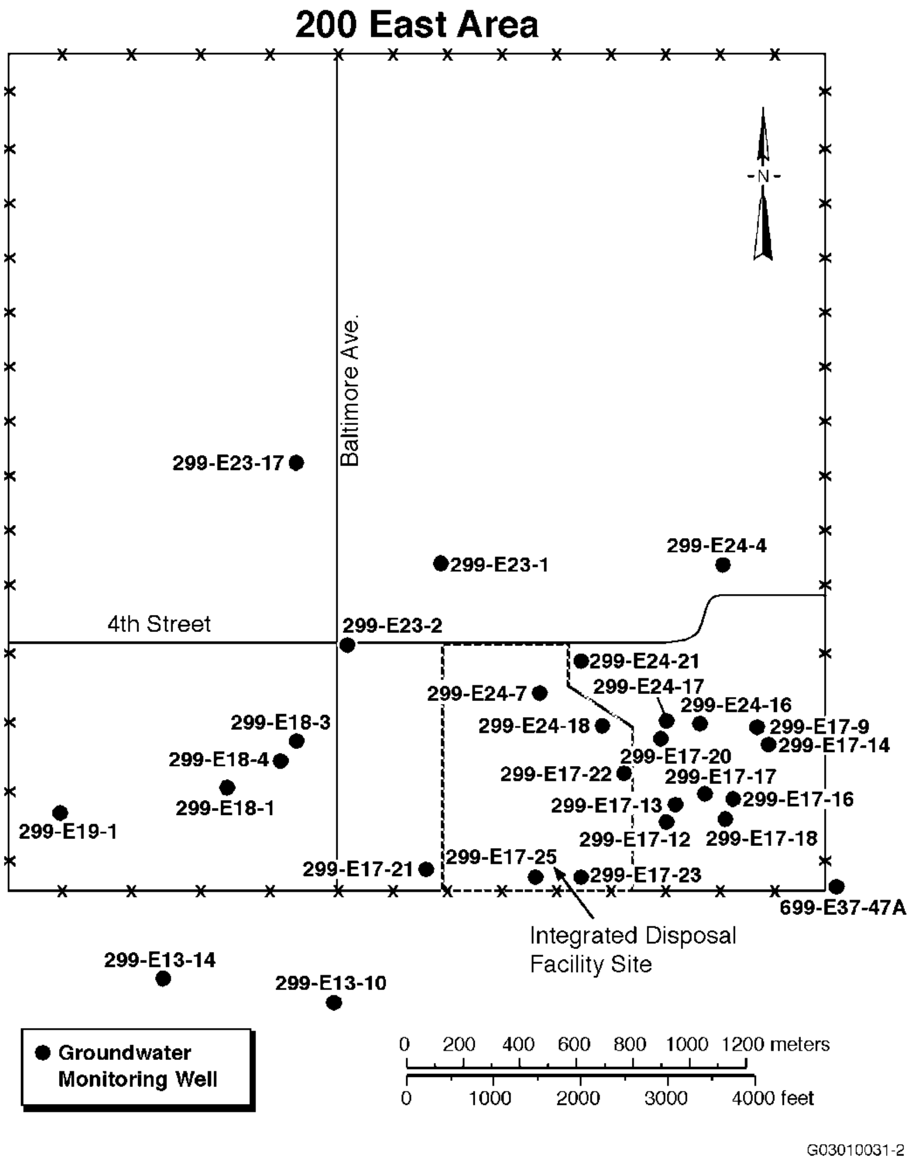
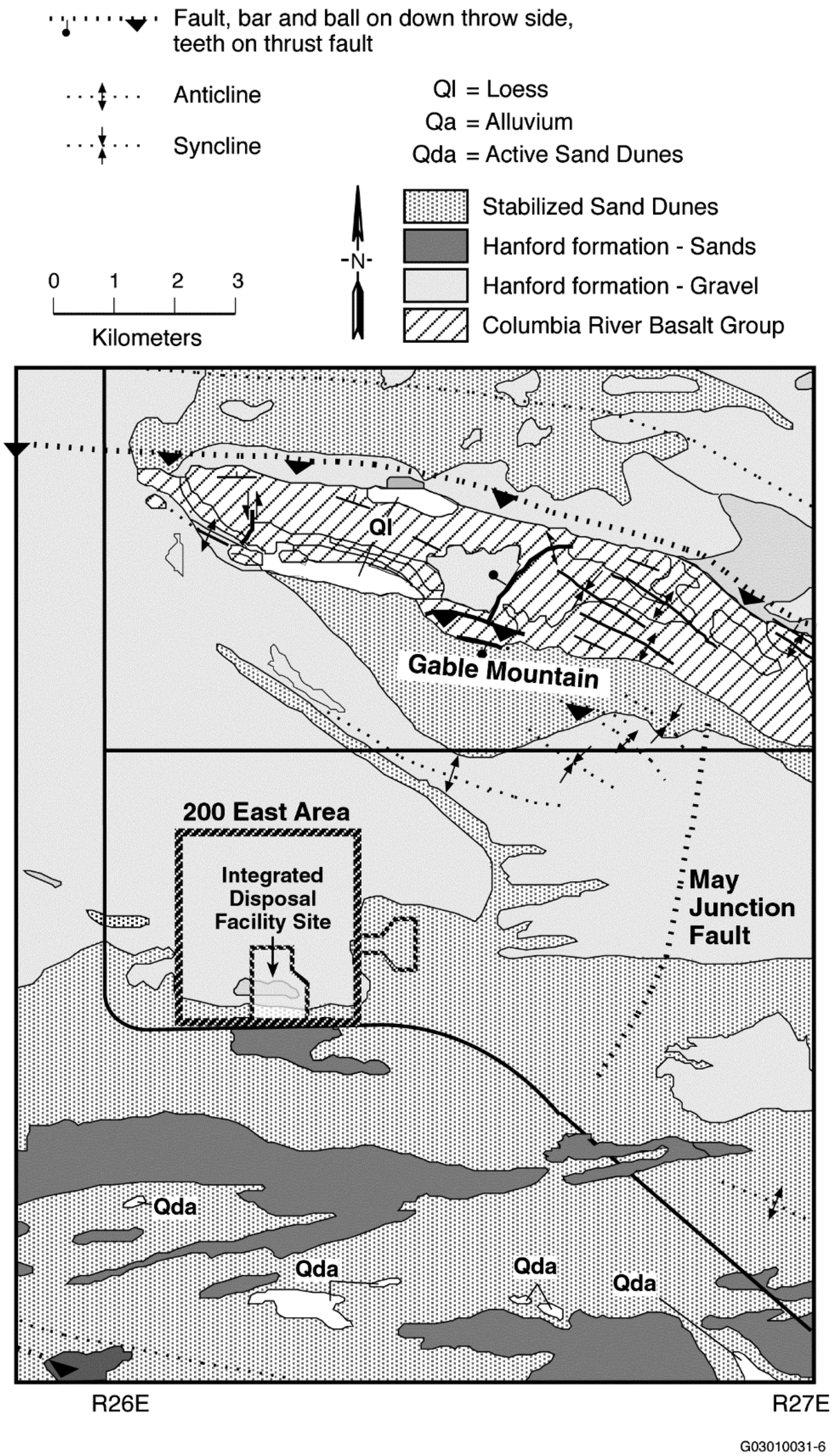


Figure 3-1. Location of the IDF and Nearby Boreholes



1 **Figure 3-2. Geologic Map of the 200 East and 200 West Areas and Vicinity**

2

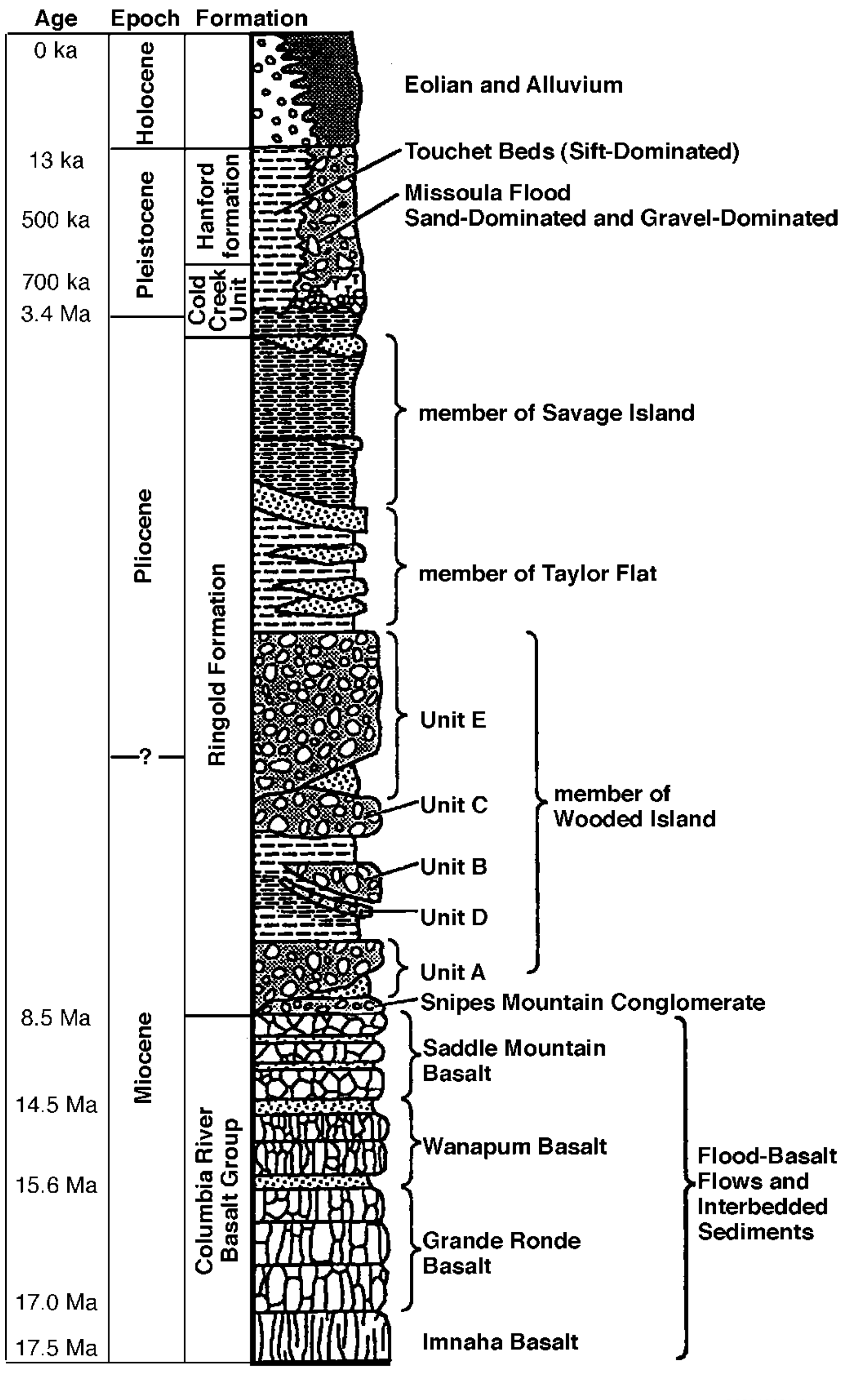
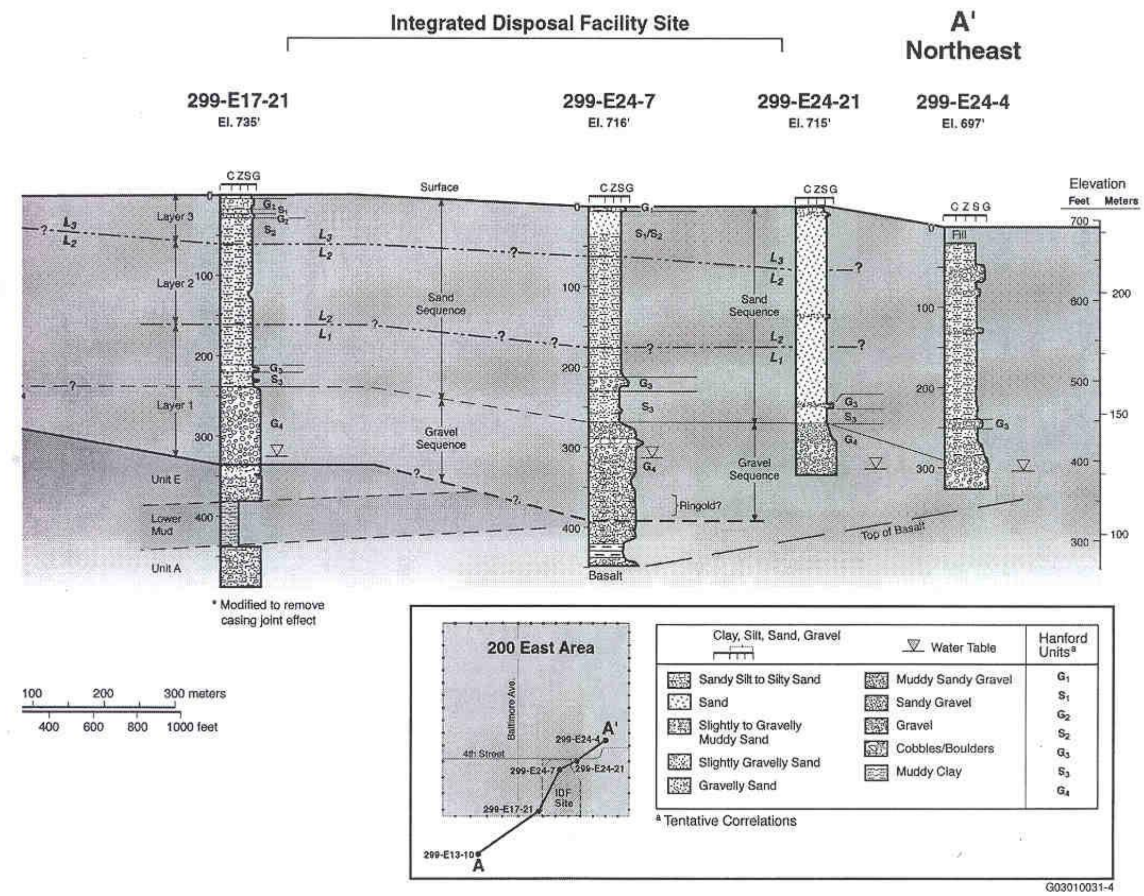


Figure 3-3. Stratigraphy of the Hanford Site



1 **Figure 3-4. Cross-section through the IDF Site (refer to Figure 3-1 for boring**
2 **exploration locations)**

3
4

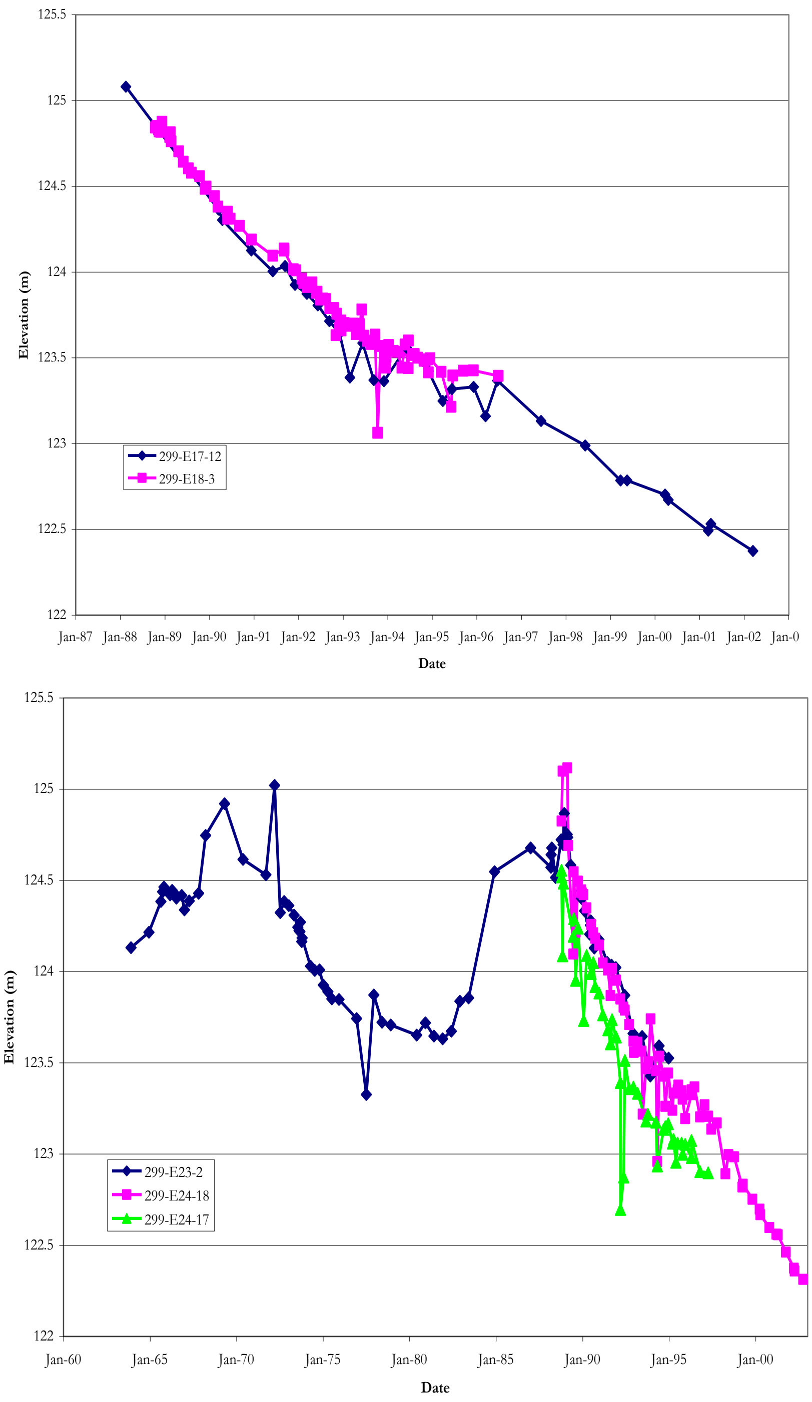


Figure 3-5. Hydrographs for Wells Near the IDF Site (1 and 2 of 3)

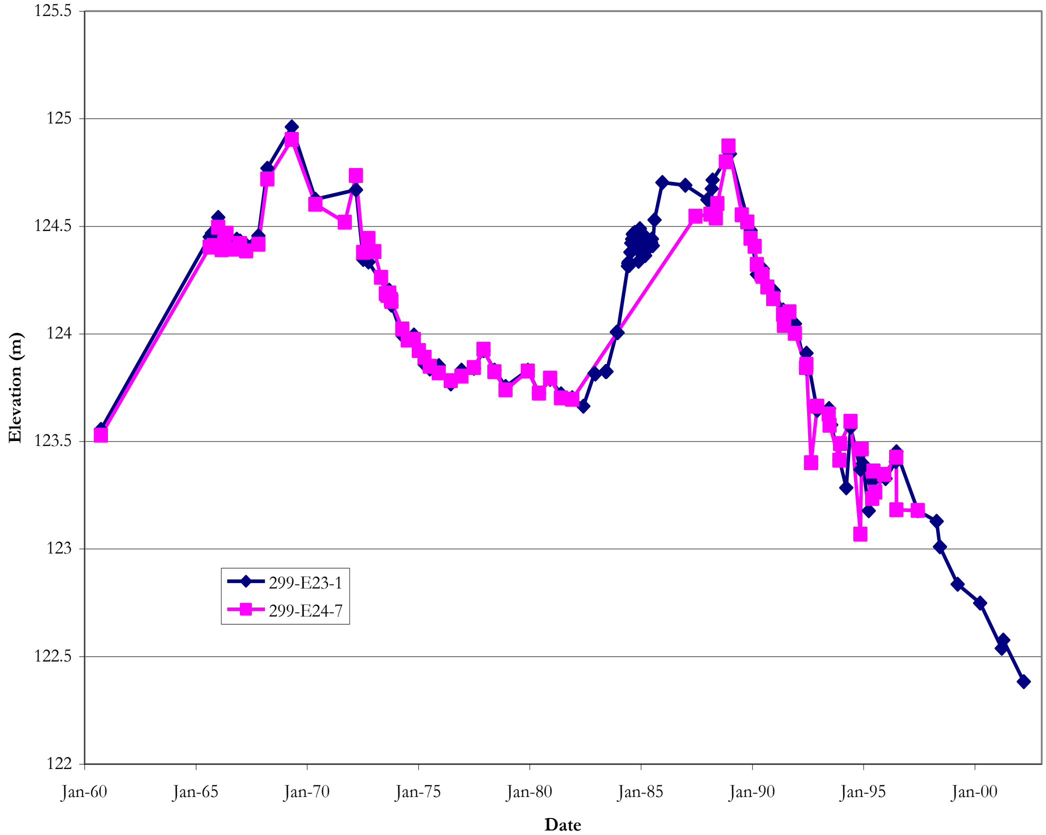


Figure 3-6. Hydrographs for Wells Near the IDF Site (3 of 3)

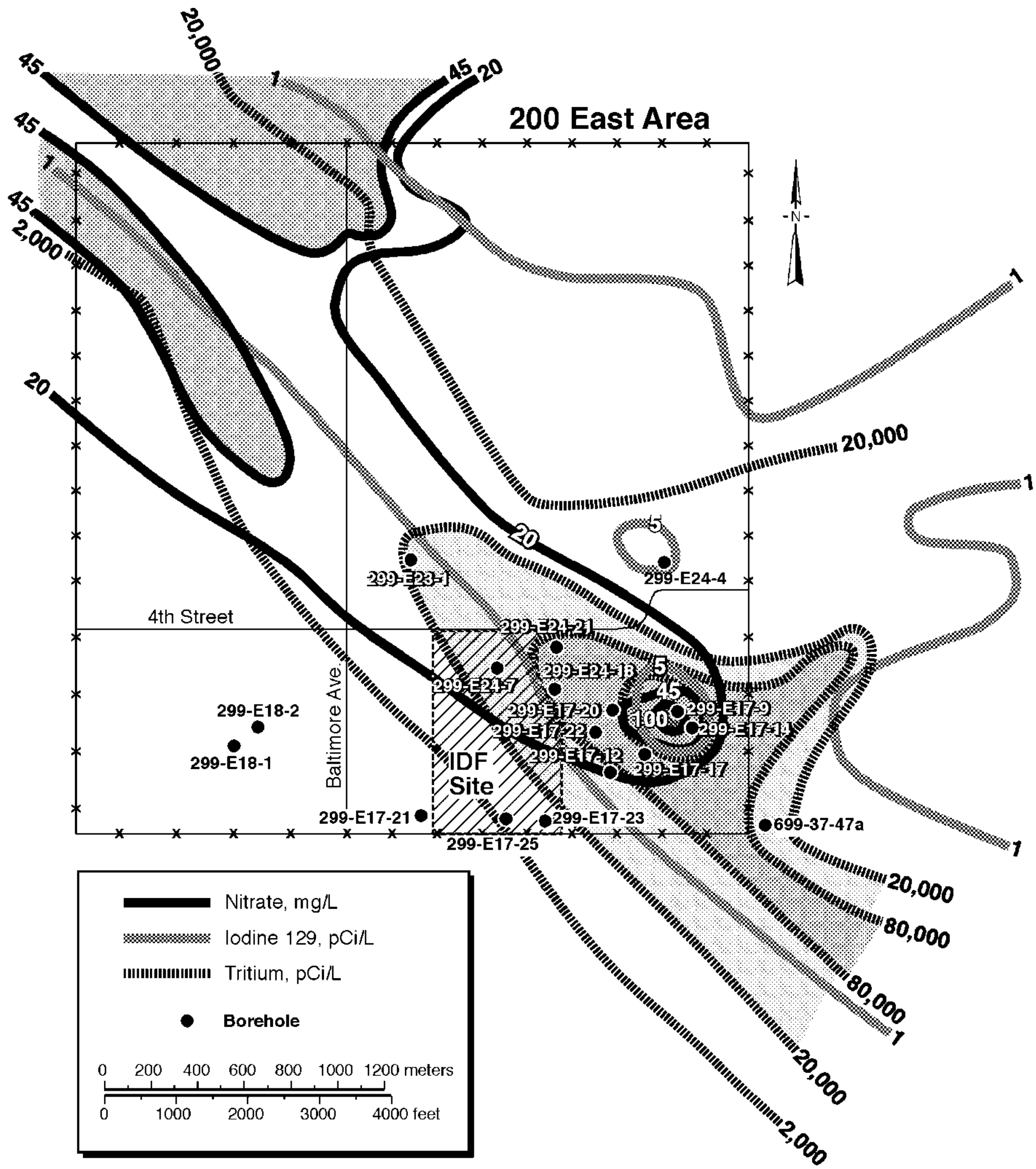


Figure 3-7. Contaminant Plume Map for the 200 East Area

4.0 SITE INVESTIGATION AND LABORATORY TEST PROGRAM

This section presents a summary of the existing, current, and planned explorations for the IDF, along with the laboratory test results for tests conducted during this design effort.

4.1 Field Explorations

This section discusses the existing and current soil explorations for the IDF. The generalized soil profile used in the analysis and design is presented; the engineering soil properties used for design are presented and discussed in Section 5 and related appendices.

4.1.1 Existing Explorations

Several field explorations have been conducted in the general area of the IDF, as discussed in Section 3.4.1 and presented in Figure 3-1. Figure 1-2 shows the current IDF footprint and the closest borings to the planned facility. As shown in Figure 1-2, with the exception of one boring, the existing explorations are all outside of the footprint of the IDF.

The existing field explorations at the IDF site have been conducted primarily for geologic and hydrogeologic characterization on a “big picture” scale. The existing explorations provide detailed information for the purposes for which they were conducted; however, from a geotechnical engineering perspective, the existing borings at or near the IDF site provides only general information, as discussed below.

Depth of Interest–In many cases, the explorations focused on providing detailed information for the entire soil column above the bedrock at the IDF site (300 or more feet below ground surface [bgs]). The primary depth of interest for detailed engineering and design purposes is the depth of the planned cell excavation (roughly 50 feet below the existing ground surface); for a few analyses, information about the material 25 to 50 feet below the base of the excavation is also important.

Type of Information–As intended, the existing explorations was generally focused on providing information for geologic characterization purposes. This focus differs from the key items generally required for geotechnical design, including Standard Penetration Testing (SPT) per ASTM requirements and classification by the Unified Soil Classification System (USCS) in both the field and the laboratory. For coarse-grained soils (sands and gravels), that make up the bulk of the native soil profile, *in situ* SPT in conjunction with grain-size data is the primary basis for determining geotechnical engineering parameters of the soil, such as shear strength. In all cases the SPT values were either not readily available or were conducted with non-standard equipment. Also, the existing grain size data and soil classifications, both for field and laboratory results were based on the Wentworth scale, which differs from the USCS scale at the gravel and fines divisions. These are the key division points for classifying coarse-grained soils. In particular, the break point for fines contents is important in determining the suitability of the excavated soils for use in the admix liner as well as for other on-site filling purposes. Many of the soils within the depth of interest for the IDF are near this classification break point.

Proximity to the IDF–As shown in Figure 1-2, in nearly all cases the explorations were located outside of the IDF footprint. The standard of practice for geotechnical engineering is to place explorations within or very close to the footprint of the proposed structure, if possible.

There have been several geotechnically focused explorations conducted for various projects at Hanford. The projects closest and/or most applicable to the IDF site are:

The Grout Vault project, located approximately one-half mile east of the IDF site (Dames and Moore, 1988).

The W-025 Project, a radioactive mixed-waste land disposal facility designed in accordance with RCRA Subtitle C design criteria, located several miles west of the IDF site (in Area 200W, Golder Associates, 1995, 1994a, 1994b, and 1988)

The RPP-WTP, location approximately 1 mile east of the IDF site (Shannon and Wilson, 2000 and 2001)

These projects all provide geotechnical engineering information; however, the closest site is one-half-mile from the IDF. The standard of care for geotechnical engineering is to either use existing geotechnically based information that is at the site and/or conduct site and project specific explorations. This is to verify that the soil conditions at the site are either still valid (no changes since the time of the existing explorations) or are consistent with existing data.

4.1.2 Current Explorations

Due to the limits of the geotechnical specific data, a subsurface exploration plan specific to the Phase I portion of the IDF was proposed. The suggested locations for the exploration are shown in Figure 1-2. This exploration is currently in planning.

During this design effort, a limited surface sampling plan was conducted at the locations shown in Figure 1-2. The surface samples were taken from the upper 2 to 3 feet of soil, primarily to provide samples for admix testing (to determine if the soils were suitable as a base soil), as well as to help fill in for the absence of a full exploration program at the time of this design effort. As shown in Figure 1-2, samples were taken from primarily from the dune sand borrow area within the IDF footprint (SD-1 through SD-4) and the active sand borrow area (SD-5) to the east of the IDF footprint. One surface sample (SD-6) was obtained from within the IDF Phase I limits.

4.1.3 Site Stratigraphy

In the absence of a comprehensive site and project specific geotechnical engineering data, the existing and current data discussed above was reviewed to determine appropriate soil profile and geotechnical parameters for use in engineering analysis and design. The stratigraphy and soil properties were generally selected conservatively to account for the uncertainty in the subsurface information. The general soil stratigraphy beneath the Phase I section of the IDF was assumed to be:

- 10 feet of Dune (Eolian) sand, overlying
- 50 feet of Upper Hanford sand, overlying

Lower Hanford sand to depth of interest

It is expected that a greater depth of Dune sand exists in the southern portion of the IDF footprint (note topographic change in the southern one-third of the IDF footprint in Figure 1-2).

The engineering properties and parameters assumed for these soil units were based on the information provided in the geotechnical reports listed in the previous section. The individual values are discussed in Section 5 and related appendices.

4.1.4 Future Explorations

It is recommended that a comprehensive, geotechnically focused exploration program be completed, prior to construction, to verify that the assumptions made for soil stratigraphy and engineering properties are valid. A more comprehensive set of explorations is currently being planned. The planned locations for the additional explorations are shown in Figure 1-2, and include three explorations within the Phase I footprint and one exploration in the proposed sand borrow area.

4.2 Laboratory Testing

A limited laboratory testing program was conducted, using the soils collected during the surface sampling program discussed in Section 4.1.3. These samples were used to perform the index testing, admix testing, and geosynthetics interface shear testing.

4.2.1 Index Testing

Index testing was performed to evaluate the basic index and classification properties of the soil obtained from surface sampling program. This testing was conducted to provide data for comparison with both the soils used for the W025 admix liner and also for other soils that are considered for use as the base soil for the IDF project, as the final design and construction proceeds.

The laboratory testing was conducted by Soil Technology, Inc., (STI) of Bainbridge Island, Washington, under subcontract to the Affiliate. Test assignment and coordination was provided by the Affiliate. Index testing included the following ASTM tests:

- ASTM D422 – Test Method for Particle-Size Analysis of Soils (grain size and hydrometer analyses)
- ASTM D698 – Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort
- ASTM D1140 – Test Method for Amount of Material in Soils Finer then the No. 200 Sieve (P200 Wash)
- ASTM D1557 – Test Method for Laboratory Compaction Characteristics of Soil Using Modified Efforts
- ASTM D2216 – Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock
- Compaction characteristics were also determined for a composite of the surface soils, as described in the next section.

4.2.2 Admix Testing Program

The admix testing program was developed to determine two key items:

- Percentage of sodium bentonite required to meet hydraulic conductivity requirements
- Appropriate moisture and density parameters to achieve the required hydraulic conductivity

Index testing of the admix soils was also conducted, as well as a consolidation test. The laboratory testing was conducted by STI. Tests were run in general accordance with the following:

- ASTM D422 – Test Method for Particle-Size Analysis of Soils (grain size and hydrometer analyses)
- ASTM D698B – Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort
- ASTM D1557 – Test Method for Laboratory Compaction Characteristics of Soil Using Modified Efforts
- ASTM D2216 – Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock
- ASTM D2435 – Test Method for One-dimensional Consolidation Properties of Soils
- ASTM D4318 – Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils (Atterberg Limits)
- ASTM D5084 – Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter

The base soil for the admix testing was created by compositing SD-1 through SD-4 from the surface sampling program. This composite did not include SD-5, taken at the base of the existing sand borrow area (lower elevation than the other samples) that has slightly different properties than the remainder of the surface samples. SD-6 was not included at the time of the admix testing because it is not within the footprint of the planned borrow area. The base composite sample was labeled as COMP-1. This composite was then used to create the two other soils for admix testing:

- COMP-2: COMP-1 base soil mixed with 8 percent bentonite
- COMP-3: COMP-1 base soil mixed with 12 percent bentonite
- Moisture and density testing was conducted on all of the composite samples.

The initial hydraulic conductivity testing was conducted using eight and 12 percent bentonite (by weight), based on the results of the admix testing program conducted by Golder for the W025 Project (Golder, 1991b). The target laboratory hydraulic conductivity was less than 10-8 cm/sec when permeated with water. Testing was not conducted with leachate, as no actual leachate exists for the planned waste at this time. Golder Associates used a synthetic leachate to perform compatibility testing on the admix liner. Based on these results, they increased the bentonite percentage from 8 to 12 percent, hence the use of these values in these tests. Because the base soils are expected to be similar to that used by Golder for the W025 landfill, and until a more refined characterization of the IDF leachate is developed, the compatibility testing performed for the W025 project was considered applicable to the IDF project. Hydraulic conductivity testing was performed on all samples in flexible wall triaxial cells with backpressure saturation, in general accordance with ASTM D5084. An effective confining stress of 5 pounds per square in (psi) was applied to each test cell. Appendix B.1 includes the details for the test, including the inflow and outflow data used to confirm that each test had obtained a steady-state hydraulic conductivity value.

After the initial hydraulic conductivity testing was completed, additional samples were set up to determine the range of moisture and density parameters that are expected to produce the required hydraulic conductivity in the field.

As noted above, the samples used for the testing were gathered from the surface sampling program. Once a more comprehensive exploration program is conducted within the IDF footprint, the suitability of the soils within the excavation below a depth of 5 feet (upper 2-3 feet) can be examined for use as a base soil for the admix.

4.2.3 Geosynthetics Interface Shear Testing

A limited soil-to-geosynthetic interface shear testing program was conducted to determine the interface shear values between the operations soil and the composite drainage net (CDN), and the admix liner soils and the high-density polyethylene (HDPE). These interfaces are site specific because of the unique nature of the soils, hence their behavior in interface shear. The testing was conducted by Precision Geotechnical Laboratories in Anaheim, California. Soil samples collected during the surface sampling program were used for testing; GSE Lining Technologies, Inc. based in Houston, Texas provided the geosynthetics for testing.

The interface shear tests were conducted in general accordance with ASTM D5321–Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method. The tests were conducted for both low (100 to 500 psf) and high (1000 to 8000 psf) normal stress levels, to account for the variation in normal stresses that will be applied across the lining system in the final landfill configuration. Both the peak and residual strength values were determined during testing. Additional details for the tests are presented with the test results in Appendix B.2.

Asperity testing was also conducted on the textured HDPE geomembrane, in general accordance with GRI-GM12 – *Asperity Height of Textured Geomembrane*. The purpose of the asperity testing was to establish a baseline roughness of the texturing of the HDPE geomembrane and for future assessments of the interface shear strength of other textured HDPE geomembrane products (e.g., from other manufacturers).

Site-specific interface shear testing was not conducted for geosynthetic-to-geosynthetic (such as CDN to geosynthetic clay liner [GCL]) interfaces in this phase of design, as these values are primarily a function of the manufactured product properties. A database of values for geosynthetic-to-geosynthetic interface testing was used to determine the appropriate interface shear values for design. During construction, the actual materials used on the site will be tested as part of the construction QC/QA, to ensure that the installed materials used onsite meet or exceed the interface shear strength values used in the design.

4.3 Laboratory Test Results

The results of the laboratory testing programs are summarized below and presented in Appendix B.1 and Appendix B.2.

4.3.1 Index Testing

The results of the index testing for the surface samples are presented in Table 4-1. The test results are included with the admix liner soils test results in Appendix B.1. Results of the index testing indicate that the grain size analyses for near-surface soil samples from locations SD-1 through SD-6 correlate well with data from the W025 base soil material. The W025 base soil was a dune sand (Eolian deposits) obtained from the upper 15 feet of site excavations. As discussed in Section 5.4, based on the results shown below and due to the limited nature of the near surface soil samples, the base soil is limited to the upper 5 feet of material excavated from the dune sand borrow area or the Phase I site excavation.

Table 4-1. Results of the Base Soil Index Testing

Test	Sample #	% Gravel	% Sand	% Fines	OMC, w_{opt} (%)	MDD, $\square_{dmax}$ (pcf)
Grain Size Testing	SD#1			22.5	--	--
	SD#2		72.2	27.8	--	--
	SD#3			17.5	--	--
	SD#4		78.1	21.9	--	--
	SD#5	2.4	58.5	39.1	--	--
	SD#6		79.5	20.5	--	--
Standard Compaction	SD#6	--	--	--	14	106.6

OMC = optimum moisture content
MDD = maximum dry density

4.3.2 Admix Liner Soils Test Results

The results of the testing program conducted on the admix liner soils are summarized in Tables 4-2 and 4-3 and presented in detail in Appendix B.1. The associated placement and testing requirements during construction are also discussed in detail in Section 5.4.

Table 4-2. Results of the Admix Hydraulic Conductivity Testing

Sample ID	OMC (%)	MDD (pcf)	Remolded MC (%)	Remolded Wet Density (pcf)	Relative Compaction (%)	Saturated Hydraulic Conductivity (cm/sec)[a]	Gradient
COMP2-1	12.8[b]	117.2[b]	13.5	127	95	2×10^{-8}	11
COMP2-2	12.8[b]	117.2[b]	17.7	123	89	4×10^{-8}	10
COMP3-1	13.0[b]	115.5[b]	13.2	124	95	$<1 \times 10^{-8}$	10
COMP3-2	13.0[b]	115.5[b]	17.4	122	90	$<1 \times 10^{-8}$	10
COMP3-3	10.0[c]	126.3[c]	10.3	136	98	$<1 \times 10^{-8}$	12
COMP3-4	10.0[c]	126.3[c]	14.2	139	96	$<1 \times 10^{-8}$	10
COMP3-5	10.0[c]	126.3[c]	8	130	95	$<1 \times 10^{-8}$	18

Table 4-2. Results of the Admix Hydraulic Conductivity Testing

Sample ID	OMC (%)	MDD (pcf)	Remolded MC (%)	Remolded Wet Density (pcf)	Relative Compaction (%)	Saturated Hydraulic Conductivity (cm/sec)[a]	Gradient
COMP3-6	13.0[b]	115.5[b]	10	115	91	$1x10^{-8}$	21
COMP3-7	10.0[c]	126.3[c]	10	123	89	$<1x10^{-8}$	20
COMP3-8	13.0[b]	115.5[b]	11	119	93	$<1x10^{-8}$	16

Abbreviations: OMC = optimum moisture content MDD = maximum dry density pcf = pounds per cubic foot
MC = moisture content
COMP 2 samples had 8 percent bentonite.
COMP-3 samples had 12 percent bentonite.
Average saturated hydraulic conductivity using tap water
Based on standard Proctor compaction curve (D698).
Based on modified Proctor compaction curve (D1557)

Table 4-3. Results of Admix Liner Soils Index Testing

Test	Sample #	% Gravel	% Sand	% Fines	LL (%)	PI (%)
Grain Size Testing	COMP-1	--	77.5	22.5	--	--
	COMP-2	--	70.6	29.4	--	--
	COMP-3	--	68.7	31.3	--	--
Atterberg Limits	COMP-2	--	--	--	40	17
	COMP-3	--	--	--	54	32

LL = Liquid Limit
PI = Plasticity Index

Consolidation testing conducted on the admix liner soils is presented with the rest of the results in Appendix B.1. This results of this test were used for the settlement analysis discussed in Section 5.3.1.

4.3.3 Geosynthetics Interface Shear Tests

The results of the geosynthetic testing program are presented in Table 4-4; the results of the asperity are shown in Appendix B.2. The results are discussed in detail in Sections 5.1.1 and 5.1.3, and their related appendices (Appendix C.1.a and C.1.c, respectively).

Table 4-4. Summary of Geosynthetic Testing

	Test	Peak Friction Angle (°)	Peak Cohesion (psf)	Residual Friction Angle (°)	Residual Cohesion (psf)	Asperity	Comments
Low	Operations Soil-CDN Interface	29.6	205.9	24.6	205.4	--	– Test #1 – dry density = 92 pcf – w_c = 8.7%

Table 4-4. Summary of Geosynthetic Testing

	Test	Peak Friction Angle (°)	Peak Cohesion (psf)	Residual Friction Angle (°)	Residual Cohesion (psf)	Asperity	Comments
	Admix Soil-HDPE Interface	33.3	94.4	33.5	56.8	--	– Test #3 – dry density = 110 pcf – w_c = 14%
High Normal	Operations Soil-CDN Interface	28.3	283.9	28	240.8	--	– Test #2 – dry density = 92 pcf – w_c = 8.7%
High Normal	Admix Soil-HDPE Interface	25.4	400.7	20.3	525.3	--	– Test #4 – dry density = 110 pcf – w_c = 14%
	Textured HDPE Asperity	--	--	--	--	23.5	Average value of two test results of 22 and 25.

As the final design progress and additional information is gathered for the admix soils and the operations soils, these results should be verified with additional testing. Testing during full scale construction is also planned to verify that the materials used in construction, both soils and geosynthetics, produce interface shear values at or greater than those used for design.

5.0 ENGINEERING ANALYSIS

This detailed Design Report finalizes the design for the landfill liner system, the leachate removal system, and the LDS. Engineering analysis components for each of these critical systems is presented in this section. A general description of system components is located in Section 5.6.1, that presents the primary and secondary liner systems that make up the major layers of the landfill (detailed system descriptions are presented in Section 6).

In preparation of the IDF design, a number of design requirements and criteria as presented in Section 2 have been considered. Compliance with these design requirements is provided in Appendix A. The specific criteria evaluated for the IDF design included:

- Slope stability
- Landfill bearing capacity
- Settlement and uplift analyses
- Admix liner
- Geosynthetic liner design
- Liner systems/leachate compatibility
- Drainage layer
- Leachate production
- Leachate collection system
- Surface stormwater
- Action leakage rate
- Building systems analyses
- Civil grading

5.1 Slope Stability

Slope stability for the IDF landfill was examined for liner veneer (side slope) stability, earthwork stability, waste/fill global stability. The analyses for each of these cases are summarized in the sections below; Appendices C.1.a through C.1.c presents the analyses and results in detail.

5.1.1 Liner Veneer (Side Slope) Stability

The veneer stability of the liner system on the side slopes was evaluated for the period prior to waste filling. The analysis examined the potential for sliding of the drainage and operations layers on the liner system before waste is placed.

The analyses were conducted using the weakest of the interface strengths of the various lining system components. The interface strengths were determined from regression analyses of data gathered from various sources, including site-specific test data completed to date. Based on the data (presented in Appendix C.1.a), the critical interface is the textured HDPE/CDN interface. Properties of the cover soil (operations layer) were determined from laboratory testing to date on the materials expected to be used for the operations layer.

Four loading conditions were examined:

- Dead load: self-weight of the lining system (including the first operations layer)
- Dead load + Equipment: self-weight of the lining system with an equipment load
- Dead load + Seepage: self-weight of the lining system with a seepage load (to account for fluid head in the leachate collection system); seepage loads were based on results from the leachate system hydraulic analyses
- Seismic Loading: self-weight of the lining system with seismic loading

The results of the analyses show that the lining system is stable for the conditions analyzed and no anchorage forces are required to meet the minimum factors of safety (1.5 for dead load only; 1.3 for equipment and seepage loading). A minimum interface friction of 25 degrees and cohesion of 0 psf is required to meet the minimum acceptable factors of safety. The slopes are also considered to be stable under seismic loading, based on comparing the calculated yield acceleration and with the design acceleration values provided in the design criteria by CH2M HILL (September, 2002), using the hazard classification assigned to the overall facility.

The critical interface friction values will be verified during construction to ensure that the system will be stable. The analyses and results are presented in full detail in Appendix C.1.a.

5.1.2 Earthwork Stability

The earthwork stability analysis covered the following three cases:

Excavation Case: This case covers the stability of the landfill slopes immediately after excavation and before placement of the lining system. Only static loading was considered since this is an interim configuration that will only exist for the construction period.

Ramp Case: This case covers the stability of the landfill slopes and access ramp at the south end of the cell, including equipment loading on the ramps. Both static and seismic loading were examined, as the access ramps are expected to be in use for a period of at least 10 years.

Dike Case: This case covers the stability of the perimeter dike (shine berm and access road) after construction of the dike and before final closure of the landfill. Both static and seismic loading were examined, since the perimeter dike may be in place until the final cover system is completed (greater than 10 years).

Properties for the native soils are based on existing information, as a site-specific geotechnical engineering investigation program has not yet been completed for the IDF facility. When this investigation is completed, the results of this analysis (and any others that rely on the properties of the native soils) will be verified. Geometry used in the analyses is based on the civil plans (generally 3H:1V slopes with a few short 2H:1V slopes).

The results of the analyses show that the planned configurations of the landfill are stable under static loading (factor of safety [FS] greater than 1.3 and 1.5, depending on the case analyzed); the configurations are also considered seismically stable based on the criteria for the Hanford site. Full details on the analysis method, the input data, and the results are presented in Appendix C.1.b.

5.1.3 Waste/Fill Global Stability

This analysis examined the following conditions:

Phase I Full Build-Out: This case examined the stability of the waste mass in full build-out of the Phase I waste cell. The critical stability examined was the waste sliding on the lining system. Both static and seismic loading conditions were examined.

Final Configuration: This case examined the stability of the waste mass at the final configuration (entire IDF landfill completed) along the edge of the cover system. Only static loading conditions were examined, since this system is not being designed as part of the current effort.

Interim filling conditions and the internal stability of the waste mass were not examined. The internal waste mass stability will primarily be a function of the filling methodology. Possible filling plans for the waste are currently being developed.

For the analysis of the full build-out of Phase I, the critical interface strengths in the lining system were determined in the same way as for the veneer stability (regression analyses of existing and site specific testing data). A combination of peak and residual strengths were used, based on methodology currently being employed in the state of the practice. A final check was also made to confirm that the use of residual strengths in all locations resulted in a factor of safety greater than 1.0.

The results show that the system is stable for the configurations analyzed and for the interface friction values available at the time of the analyses (FS greater than 1.5 in static loading and yield acceleration greater than the 10,000-year event). The system also has a FS greater than 1.0 for the case of residual strengths in all locations. The critical interfaces are the HDPE-CDN on the side slopes (using residual strengths) and the HDPE-GCL on the base liner (using peak strengths) and the internal GCL strength (using residual strengths). These results should be verified when additional site-specific test data becomes available prior to and during construction.

Also, it should be noted that for the full Phase I build-out configuration, the most critical case appears to be a failure surface that is allowed to propagate through the waste mass. As noted previously, the waste mass was considered internally stable for this design effort. During final operations planning, the internal stability of the waste will be examined in conjunction with the proposed waste filling plan.

For the final configuration with the cover in place, the preliminary geometry and assumed cover system properties show that the configuration is stable under static loading (FS greater than 1.5) and the critical failure does not intersect the waste mass. Stability of the final configuration under both static and seismic loading should be examined in more detail as the final design develops for the final closure of the entire IDF facility.

A full discussion of the methodology, input data, and the results is presented in Appendix C.1.c.

5.2 Landfill Bearing Capacity

5.2.1 Subgrade Soil

Based on the available geotechnical data from other projects (as discussed in Section 4), the strength of the native subgrade soils beneath the landfill is expected to be greater than that for the operations layer or any of the liner system components. Greater strengths equate to higher bearing capacities, and hence, the bearing capacity of the subgrade soils within the landfill cell was not determined directly as they are not the controlling factor.

The bearing capacity of the subgrade soils beneath the supporting structures adjacent to the landfill cell was determined for the structural analyses, discussed under Section 5.12.1–Geotechnical Design Parameters, and the results of the analyses are presented in Appendix C.11.a.

5.2.2 Liner Soils

The soil layers in the lining system include the operations layer, drain gravel, and the admix liner soils. The admix liner soils will be placed beneath the geosynthetic lining system, and as such, loading on the admix liner soils is limited to the allowable loads for the GCL. The allowable loads for the GCL are much less than what the bearing capacity of the admix liner soils would be (the admix soils have much higher strengths, particularly for bearing pressures). The drain gravel will be placed just above the lining system; the shear strength and associated bearing capacity are also much greater than the GCL allowable values.

At the time of these calculations, structures that would cause bearing pressure were not yet determined. Hence, the bearing capacity for the operations soils was calculated for foundation widths from 1 to 10 feet and for 2 different shapes (square and strip). Properties for the operations soils were based on laboratory testing conducted to date; these properties will be verified during construction to ensure that the analyses results are valid.

For a factor of safety of 3, the allowable bearing capacities for the operations layer are presented in Table 5-1.

Table 5-1. Operations Soil Bearing Capacities

B, Foundation Width (feet)	q_{all}, square foundation (tsf)	q_{all}, strip foundation (tsf)
1	0.20	0.33
5	1.0	1.6
10	2.0	3.3

As the operations plans are further developed, these values can be updated for the planned structures (such as barrier walls). Details of the analyses are presented in Appendix C.2.

5.3 Settlement and Uplift Analyses

5.3.1 Settlement Analysis of Liner Foundation

The long term settlement of the soils supporting the geosynthetic liner system was estimated based on the maximum loading expected in the landfill at the final IDF completion. The two soil units examined were the admix liner soils and the native subgrade soils. For the admix soils, data from laboratory consolidation testing performed on samples available at the time of the analysis were used to determine the estimated settlements. Elastic methods were used to estimate the settlements of the subgrade soils.

As detailed in Appendix C.3, the estimated long term settlement over the lifetime of the landfill is 2.7 feet under the maximum loading.

5.3.2 Subsidence and Sinkhole Potential

Subsidence of undisturbed foundation materials is generally the result of dissolution, fluid extraction (water or petroleum), or mining. Subsidence is not expected to occur based on the following:

- The soils underlying the IDF are generally dense, coarse-grained, and well-graded sands and gravels that will not be subject to piping effects that could transport soil and result in subsidence. Also, sands and gravels are generally not susceptible to dissolution.
- The groundwater level is deep and will not affect bearing soils.
- The bedrock is basalt (volcanic), which is not generally susceptible to dissolution.
- No mining or tunneling has been reported in the areas beneath or surrounding the site for the IDF.

- Borings in and around the IDF have not identified any soluble materials in the foundation soils or underlying sediments. Consequently, the potential for any sinkhole development will be negligible.

5.3.3 Uplift Potential

The potential for uplift of the composite liner system is very low. The seasonal high-water level is over 200 feet below the base of the base of the landfill cell, so no external hydrostatic pressure is expected from this source. Perched groundwater is not expected to occur due to the absence of continuous aquitards (such as a clay layer) within the coarse-grained native soils at the IDF site. Any infiltration that does occur is expected to percolate rapidly to deeper soil layers.

Gas pressures are also expected to be negligible, as no gas-generating material (i.e., organic material) is expected in the foundation soils. Also, the subgrade soils are coarse grained and unsaturated, so any gas that might occur is expected to be rapidly dissipated.

5.4 Admix Liner

5.4.1 Mix Design

WAC 173-303-665(2)(h)(i)(B) requires that the lower component of a composite bottom liner be constructed of compacted soil material with an in-situ hydraulic conductivity no greater than 10-7 cm/sec. Because of the lack of naturally occurring soils on-site that could achieve this requirement, a test program was developed to determine the admixture requirements for a mixed soil design using on-site base soil from either the Phase I excavation or dune sand borrow area (see Drawing H-2-830826 for location) and sodium bentonite. Details of the base soil field exploration and admix testing program are provided in Section 4.

The results of the limited field exploration for base soil samples and subsequent admix testing program discussed in Section 4 show that a nominal bentonite content of 12 percent will meet the laboratory target hydraulic conductivity of less than 10-8 cm/sec when permeated with water. The laboratory target was established based on results of the soil liner/leachate compatibility study (Golder Associates, 1991b) for the W025 landfill. Details of Golder's study are discussed in Section 5.6. The W025 study concluded that the bentonite content of the admix should be increased from 8 percent (the minimum bentonite percent needed to achieve the required hydraulic conductivity) to 12 percent, to provide adequate resistance against high inorganic concentrations in the synthetic leachate for the W025 project. Index laboratory testing on the limited field exploration at the IDF site (surface sampling) established that the base soil for the IDF was similar to the W-025 project, as discussed in Section 4. Thus, until a more refined characterization of the IDF leachate is developed, the compatibility testing from the W025 testing is applicable to the IDF mix design.

Once initial hydraulic testing confirmed that an admix with 12 percent bentonite content could achieve the laboratory target value, additional samples were set up to evaluate a range of moisture and density parameters and their effect on hydraulic conductivity. The additional hydraulic conductivity tests were performed to define moisture content-density requirements for a range of compactive energy, as outlined by Daniel and Benson (1990). This data was being used to develop an "acceptable" zone of moisture and density for use by QC personnel during construction. The acceptable zone for the 12 percent admix is presented along with the admix design laboratory test results in Appendix B.1.

The acceptable zone was developed based on samples that achieved a hydraulic conductivity of less than 10-8 cm/sec. A lower bound of 95 percent relative compaction, based on Standard Proctor (ASTM D698) compactive effort, was established to ensure adequate shear strength levels. As indicated in the technical specifications (see Section 02666), the moisture-density range of the compacted admixes shall lie within a trapezoidal-shaped field with the following corners:

Moisture Content (%)	Dry Density (pcf)
8	126
14	126
12	110
19	110

Note that the minimum dry density of 110 listed above corresponds to approximately 95 percent of the maximum dry density for admix, as measured by ASTM D698.

5.4.2 Placement and Testing

The moisture-density requirements developed as part of the admix testing program will be included in the specifications for the admix liner (see discussion in Section 5.4.1, and technical specifications, Section 02666). The intent of the placement technical specifications is to help ensure that the admix liner will meet an in-place performance specification for hydraulic conductivity of less than 1x10-7 cm/sec. The contractor is responsible for developing and implementing compaction means and methods that will produce the required relative compaction.

The recommended nominal bentonite percentage (12 percent) and moisture-density parameters for the admix liner have been developed with a one order of magnitude factor of safety between laboratory and field values for hydraulic conductivity. The factor of safety is expected to account for two issues: (1) variations in the hydraulic conductivity between the laboratory soil amendment study and full-scale production, and (2) the laboratory samples were permeated with water rather than leachate, which could lead to a difference in the field hydraulic conductivity. However, factors such as base soil variability at the borrow source and field placement and construction are difficult to quantify until full-scale production begins for the admix liner. A test pad will be constructed as part the IDF construction to model the full-scale production. The purpose of the test pad is to determine acceptable processing, placement, and compaction methods that will produce a low-hydraulic conductivity admix liner with an *in situ* hydraulic conductivity of 10-7 cm/sec or less. The bentonite percentage and moisture content/density range may be modified if the preconstruction testing performed on the test pad indicates an *in situ* hydraulic conductivity greater than 10-7 cm/sec. Construction QA sampling and testing for the test pad is described in the Detailed Design Cell 1 Construction QA Plan (CH2M HILL, March 2004).

5.4.3 Freeze/Thaw

Compacted soil liners, such as the IDF admix liner, are known to be vulnerable to large increases in hydraulic conductivity due to freeze/thaw cycling; current data suggests that compacted soil bentonite admixtures may not be as vulnerable to damage as true clay liners (Kim and Daniel, 1992; Benson and Othman, 1993; Kraus et al., 1997). Existing laboratory data indicate that GCLs are less susceptible to damage from freeze/thaw conditions and therefore, do not undergo increases in hydraulic conductivity (Hewitt and Daniel, 1997; Kraus et al., 1997).

In order to provide adequate freeze/thaw protection for the admix liner and avoid potential damage to the GCL a protective soil cover can be used. The thickness of the protective soil cover should exceed the predicted freeze depth. For the IDF, protective soil cover is provided by the operations layer on the side slope (3 feet) and the drain gravel and operation layer (4 feet total) on the bottom liner.

The analysis was performed on the IDF lining system operations layer to determine the freeze depth or frost penetration for a probable freezing season during the 10-year expected period of waste filling. Both a 10-year return period (90 percent probability on non-exceedance) and 20-year return period (95 percent probability on non-exceedance) air freeze index (AFI) were used to estimate maximum frost penetration depth in the operations layer. If the maximum frost penetration depth were less than the 3-foot minimum thickness operations layer over the lining system, the proposed operations layer thickness would be considered as adequate protection for exposure of the lining system to freeze-thaw cycles.

For the 10-year return AFI, the maximum freeze depth is estimated at 17 inches. For the 20-year return AFI, the maximum freeze depth is estimated at 21 inches. The maximum estimated freeze depths for both the 10-year and 20-year return period freezing seasons indicate that the proposed cover soil thicknesses provide more than adequate protection for the underlying admix liner and GCL from potential damage when subject to freeze-thaw cycles. Details of the freeze depth calculations are included in Appendix C.4.

5.5 Geosynthetic Liner Design

5.5.1 Geomembrane Liner Tension Caused By Thermal Contraction

The HDPE geomembrane for IDF lining system will be subject to temperature-induced tensile strain from expansion/contraction as the geomembrane is exposed to temperature fluctuation.

Strain on the liner was calculated using published values for the coefficient of linear thermal expansion for HDPE geomembrane (Koerner, 1998) and applying this to the maximum slope length. The maximum length is measured from the top of the slope, where liner is anchored, to the toe of the 3H:1V side slope. This is a conservative approach, as using the maximum slope length results in the maximum amount of expansion and strain on the liner. Additionally, a conservative temperature change of 40 degrees C (104 degrees F) was used in the analysis.

The maximum liner strain was estimated to be less than 0.5 percent, based on a maximum temperature change of 40 degrees C (104 degrees F). The estimated maximum of slack in the liner on the side slope is 8.6 inches. The corresponding amount of temperature induced stress is 566 psi. See Appendix C.5.a for supporting calculations.

As shown in the technical specifications, Section 02661 (Table 1), the elongation at yield for the geomembrane that will be used in the liner system is at least 12 percent, with a minimum tensile strength at yield of 2,000 psi. Therefore, the maximum anticipated strains are well below the yield tensile strain and stress for the HDPE geomembrane, and temperature-induced strain will have no adverse impact on lining system function.

It should be noted that temperature-induced strain is only applicable during the construction period when the HDPE geomembrane is exposed to temperature fluctuation. Once covered with 3 to 4 feet of cover soils (drain gravel and operations layer), the ambient temperature at the surface of the geomembrane will be more controlled and not subject to fluctuation.

During installation, care must be taken to allow for expansion/contraction of the HDPE geomembrane to minimize the development of wrinkles that could become future stress points under soil and waste loading. The technical specifications (see Section 02661) provide requirements for control of wrinkle development during liner deployment, including the limitation of working when the temperature is below 0 degrees C (32 degrees F) or above 40 degrees C (104 degrees F) without implementing installation procedures that address the environmental conditions.

5.5.2 Liner System Strain Due To Settlement

The barrier components (geomembrane and GCL) for the IDF lining system will be subject to settlement-induced tensile strains as the underlying soils, primarily the admix soil liner and the subgrade soil, settle over time. Strain within the lining system was calculated based on the results of the liner foundation settlement calculations (see Section 5.3 for settlement of foundation soil [subgrade] and admix liner). The strain calculation assumed that all vertical settlement was translated into strain along the liner rather than just the vector component parallel to the liner. This is a conservative assumption that establishes an upper bound for liner strain.

The maximum liner strain was estimated to be less than 0.6 percent, based on a maximum estimate of 2.7 feet of settlement at the base of the lining system. See Appendix C.5.b for supporting calculations.

As shown the technical specification (Section 02661, Table 1), the elongation at yield for the geomembrane that will be used in the liner system is at least 12 percent. Based on studies of effect of differential settlement on GCLs (LaGatta et al., 1997), the limiting strain was defined as the strain in which an increase in hydraulic conductivity of the GCL was observed, which was taken as 5 percent.

Therefore, the maximum anticipated strains are well below the yield or limiting tensile strain for the barrier components of the lining system (geomembrane and GCL). Settlement-induced strain from foundation and admix soil settlement under maximum landfill content pressure will have no adverse impact on lining system function.

5.5.3 Anchor Trench Pullout Resistance

During construction, the geomembrane could experience pullout forces caused by thermal expansion/contraction or wind uplift. However, tension from thermal expansion and contraction is expected to be small (see Section 5.5.1), and the geosynthetics installer can use sand bags or other approved method to control wind uplift during installation.

After construction and placement of operation layer, the pullout forces on the geomembrane are expected to be negligible, as there is no tension force on the liner. As indicated in the veneer (side slope) stability analyses (see Section 5.1.1), the lining system interface strength exceeds the slope angle on the 3H:1V side slope. Thus, the pullout resistance requirements for the anchor trench are to support the self-weight of the geomembrane and other lining system components. Analyses for liner self-weight support requirements determined that the frictional resistance between geosynthetics exceeds the liner self-weight. Thus, no additional pullout resistance is needed at the anchor trench to support lining system self-weight.

Supporting calculations for the anchor trench design, as shown on Drawing H-2-830838, Detail 3, are included in Appendix C.5.c. Based on the calculations for the configuration shown in the drawing, a pullout resistance ranging from 1840 pound/foot (lb/ft) to 2440 lb/ft is estimated (depending on actual mobilized interface shear strength). The required minimum tensile yield strength for 60-mil HDPE geomembrane in the technical specifications (see Section 02661) is 1440 lb/ft (120 lb/in), which results in the estimated pullout resistance exceeding the geomembrane tensile yield strength. This situation is due primarily to the configuration of the shine berm, which helps to anchor the system. While it is generally not desired for the pullout resistance to exceed the yield strength, this is not expected to be a problem at the IDF, since, as discussed above, the potential causes for geomembrane tension have been addressed and there is not a scenario for mobilizing tensile or pullout forces on the lining system.

5.5.4 Puncture Resistance

The primary geomembrane in the IDF will be overlain by the LCRS. For the side slope lining system, the LCRS consists of a CDN (see Detail 2 on Drawing H-2-830838) that provides protection for the primary geomembrane from the overlying operations layer. A separate discussion of the CDN geotextile puncture resistance is provided in Section 5.7.2. For the bottom lining (floor) system, the LCRS consists of drain gravel overlying the geomembrane (see Detail 1 on Drawing H-2-830838). A geotextile cushion will be required between the drainage gravel and the geomembrane to prevent the gravel from puncturing the geomembrane. An analysis was performed to determine the weight of the geotextile fabric required to prevent geomembrane puncture either from operating equipment loads or from the combined static weight of the waste and final cover.

Koerner (1998) developed a method for estimating required geotextile thickness that considers the size and shape of the rock, as well as other factors that could decrease the long-term strength of the geomembrane. The equation used to determine puncture resistance is based on the mass per unit area of the geotextile and the protrusion height of the puncturing material.

Operating loads were estimated based on a melter transport trailer operating directly on the surface of the first operations layer. Static loads were estimated for the post-closure condition by using the weight of four layers of ILAW packages with cover soil and a 15-foot-thick closure cover, with a 2 percent grade to the center of the landfill. The static load was more than two times greater than the operating load, and therefore was used as the basis for the puncture analysis. Detail calculations for geomembrane puncture resistance and corresponding cushion geotextile requirements are included in Appendix C.5.d.

The proposed design specifies that the LCRS drainage gravel will have a gradation corresponding to WSDOT Standard Specification 9-03.12(4). This gradation has a maximum stone size of 1 inch. From the curves shown in the detailed calculations, the FS for a 12 oz/yd2 geotextile loaded by 1-inch angular rock is 4.5. For subrounded rock or gravel, this is more representative of the specified drain gravel, the FS increases to 8.9. The specified cushion geotextile (see technical specifications, Section 02371) has a nominal weight of 12 oz/sq yd, and therefore should be adequate to prevent geomembrane puncture. Koerner (1998) recommends a FS greater than 3.0 for the condition of packed stones on a geomembrane, such as would be the case for drain gravel over the geomembrane at the IDF.

5.5.5 Operational/Equipment Loading

The effects of loading on the GCL from construction and operational equipment and activities were examined. The maximum loads from the landfill waste itself were found to produce the highest loading on the geomembrane and the CDN; these materials were selected based on this maximum loading, as discussed in the previous sections.

The cases for construction equipment loading and operational loading on the GCL were examined, including the extreme loading case of the crane placing the heaviest waste loads at its maximum reach, a situation which produces very high pad loads. The expected loads were compared to the calculated allowable GCL bearing capacity to determine if the loads would have an effect on the GCL. The allowable GCL bearing capacity was determined from classical geotechnical theory and based on manufacturer’s strength data.

The results of the analyses are presented in detail in Appendix C.5.e. For the construction loading, the analyses show that the specification requirements that limit construction loading are adequate to protect the GCL, based on the standard construction equipment anticipated to be used at the IDF and as examined in the calculations.

For the operational loading cases examined, the critical condition is the crane operating under an extreme condition. The minimum dunnage requirement for the crane pads is 60 square feet, or if square, a 7.7-foot by 7.7-foot dunnage pad. Lower loads will require less dunnage and can be calculated as detailed in Appendix C.5.e. As discussed in the appendix, dunnage requirements calculated in this way are appropriate as long as the lining system is functioning as intended (i.e., no moisture in the LDS). If moisture enters the LDS and the GCL becomes hydrated, the dunnage requirements will be increased by a factor of approximately 2.5.

It should also be noted that the primary purpose of the GCL in the IDF is not as a required lining system component (such as the geomembrane or the admix liner), but to “deflect” leachate from defects or pinholes in the primary geomembrane over the bottom area and longer-term storage areas (such as leachate sump trough), where the leachate head potential is greatest. The primary purpose of the primary GCL is to reduce the actual leakage rate into the LDS in the event of leak in the primary geomembrane. Given these considerations, the GCL should perform as intended under anticipated equipment and operational loading.

As the operations plans for the landfill are developed, loading values can be compared to the results shown in Appendix C.5.e to determine if the loads will affect the GCL.

5.6 Liner Systems/Leachate Compatibility

The purpose of this analysis is to demonstrate that the liner materials proposed for the IDF landfill are chemically compatible with the leachate. Certain materials deteriorate over time when exposed to chemicals that may be contained in hazardous leachate. It is important to anticipate the type and quality of the leachate that the landfill will generate and select compatible liner materials. Data collected from other similar low-level radioactive mixed waste and hazardous waste sites were used in conjunction with the anticipated IDF leachate concentrations to evaluate the allowable concentration of leachate constituents that could be in contact with the IDF landfill liner components.

5.6.1 Lining System Description

Detailed discussion of the lining system design elements is provided in Section 6. A summary is provided in this section to facilitate discussion with respect to the chemical and radiation resistance of the lining system components.

Drawing H-2-830838 (Detail 1) shows the bottom liner section consisting of the following components, from top to bottom:

- A 3-foot-thick operations layer
- A separation geotextile (polypropylene)
- A 1-foot-thick leachate gravel layer
- A minimum 12 oz/square yard cushion geotextile (polypropylene)
- A 60-mil (nominal thickness—see Section 6.3.2.1) textured primary HDPE geomembrane
- An internally-reinforced GCL
- A CDN drainage layer for primary leak detection/collection
- A 60-mil textured secondary HDPE geomembrane
- A 3-foot-thick low-hydraulic conductivity compacted admix (soil-bentonite) liner

For the bottom lining system, both the primary and secondary liners are a composite (geomembrane over admix liner or GCL) system. The addition of a GCL in the primary liner layer provides an extra measure of protection, exceeding the requirements of WAC 173-303-665(2)(h)(i), which stipulates a single geomembrane for the primary liner and composite for the secondary only. This will provide an extra measure of protection on the bottom flatter slopes of the IDF, where higher leachate head levels are more likely.

Drawing H-2-830838 (Detail 2) shows the side slope liner section consisting of the following components, from top to bottom:

- A 3-foot-thick operations layer
- A CDN drainage layer for primary leachate collection
- A 60-mil textured primary HDPE geomembrane
- A CDN drainage layer for primary leak detection/collection
- A 60-mil textured secondary HDPE geomembrane
- A 3-ft-thick low-hydraulic conductivity admix liner

The side slope lining system is a single geomembrane liner over a composite liner, meeting the requirements of WAC 173-303-665(2)(h)(i). The 3H:1V side slopes for the IDF will result in little or no leachate head build-up on the side slope lining system, thus eliminating the need for a lining system design that exceeds the WAC requirements.

In general, the liner system consists of two types of materials, geosynthetics and soil/bentonite mixtures (admix). The geomembranes, geotextiles, and CDN are manufactured from polymeric materials, such as HDPE, and polypropylene, made from synthetic polymers. The GCL consists of a bentonite layer sandwiched between two polypropylene geotextiles to assist in placement and construction. The admix liner is comprised mainly of silt to clay-sized particles, mixed with a silty sand base soil.

5.6.2 Leachate Characterization Assumptions

Several assumptions were made regarding the composition of the leachate concentrations and the applicability of previously conducted studies for this evaluation. Specifically, the studies considered directly applicable to this evaluation were:

- Geosynthetic and Soil Liner/Leachate Compatibility Studies for the W-025 Radioactive Mixed Waste Landfill in Hanford 200 West (Golder Associates, 1991a and 1991b; TRI, 1995; and WHC, 1995)
- Liner/Leachate Compatibility Study for the U.S. Department of Energy's Idaho National Engineering and Environmental Laboratory (INEEL) Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) Disposal Facility (ICDF) (DOE-ID, 2002).

Using these studies is considered appropriate for the following reasons:

- The leachate for the IDF is expected to have similar or lower concentrations of radionuclides than that used in the W025 facility study (since similar waste streams [other than ILAW] may be accepted).
- The leachate chemistry may be of similar composition to the W025 facility study (since similar waste streams [other than ILAW] may be accepted).
- Soils used in the W025 facility admix design are similar to those that will be used in the IDF admix design and will therefore be compatible.
- Similar technical specifications for the geosynthetics and admix liner used in the W025 facility design will be used in the IDF landfill design.
- A similar technical specification for a GCL used in the ICDF facility will be used in the IDF liner design.

5.6.2.1 Synthetic Leachate Concentrations for W-025 Landfill

The leachate generated for the W025 evaluation reflects both the waste materials and the stabilization agents used during waste preparation. Because the landfill will comply with waste acceptance criteria for WAC dangerous waste and RCRA facilities (as does the IDF), organic materials are not expected to be present in the waste after processing. The proposed geosynthetic materials are susceptible to damage from certain organic compounds but generally are not susceptible to damage from inorganic compounds, even with extreme pH values. As a result, the lack of organic materials results in a relatively benign leachate.

The source leachate generated for the W025 studies, was primarily based on the waste treatment and packaging approaches for W025. An aqueous solution of inorganic, with some organic compounds for conservative evaluation, was generated, resulting in a viscous, slurry-like mixture. This mixture was placed in a leaching column, and deionized water was introduced to simulate the effects of leachate generation. Although no organic components were anticipated in the waste, small quantities of benzene, methanol, and light machine oil were included to simulate the presence of organic compounds in the waste material.

The source leachate generated through the leachate column process was chemically analyzed with the following results:

- Concentrations of organics benzene and machine oil were below detection limits. Concentrations of methanol were detected, but at concentrations not considered aggressive for polyester or HDPE.
- Metals added to the waste were below the detection limits in the source leachate.
- Primary constituents of the source leachate were sodium cations and common inorganic anions, with a pH of 9.2.
- Based on these results, a synthetic leachate was generated for testing purposes. The source leachate formula resulted in a solution with total inorganics and dissolved salts of approximately 204,000 mg/L and pH of 9.2 using NaOH or HNO3, as required.

5.6.2.2 Simulated Irradiation Exposure for W-025 Landfill

Samples used to evaluate the effects of radiation were subjected to a 50,000-rad total dose of gamma radiation. This dose is expected to exceed the maximum level of radiation experienced by geosynthetic materials in the landfill under unfavorable conditions. Use of a total dose, rather than radiation type, is considered the primary factor causing damage to polymeric materials and is considered to adequately simulate actual IDF leachate conditions. Samples and leachate were irradiated together so that any synergistic effects would be seen. The following samples were included in the irradiation testing:

- Geomembrane
- Geotextile
- Geonet
- Admix (soil/bentonite mixture)

The synthetic leachate and radiation exposure developed from the W-025 studies were used as the basis of evaluation for the IDF lining system materials. Table 5-2 provides a comparison of the leachate concentrations for the W-025 project with other studies for which the U.S. Environmental Protection Agency (EPA) Test Method 9090 were performed on the lining system.

The ICDF project did not include EPA 9090 tests, however, a model for estimating leachate concentration based on the waste acceptance criteria for the project was developed. The maximum leachate concentrations and radiation exposure developed for the ICDF (DOE-ID, 2002) based on the anticipated waste design inventory were as follows:

- Organics–70 mg/l
- Inorganics–18,400 mg/l
- Radiation Exposure–12,000 rads

Table 5-2. EPA Test Method 9090 Compatibility Studies Comparison

Compatibility Study[a]	Type of Material Tested	General Composition of Leachate	9090[b] Test Concentrations or Radiation Exposure that Demonstrated Compatibility in Each Study
Hanford Liquid Effluent Retention Facility (LERF)	60-mil smooth HDPE from four manufacturers	Organics	16.25 mg/L
Hanford W-025 Landfill	**60-mil smooth HDPE**	**Inorganics Organic Leachate and Radiation Exposure pH**	**204,210 mg/L 50,000 rads 9.2**
Hanford Grout Facility	60-mil smooth HDPE	Inorganics Organic Leachate and Radiation Exposure pH	368,336 mg/L 37,000,000 rads >14
Kettleman Hills Landfills	60-mil smooth HDPE	Organics Inorganics pH	93,040 mg/L 250,000 mg/L >12

a. Detailed compatibility test information is provided in Evaluation of Liner/Leachate Chemical Compatibility for the Environmental Restoration Disposal Facility report (USACE, 1995).
b. EPA Test Method 9090 “Compatibility Test for Wastes and Membrane Liners” (EPA, 1992c).

A review of the studies presented in Table 5-2 leads to the conclusion that the inorganic concentration developed for the W025 is somewhat conservative as it significantly higher than inorganic concentrations developed for the ICDF facilities. Other than the W-025 landfill, the ICDF is estimated to be most similar to the waste type to be received at the IDF of the studies included in Table 5-2. Nonetheless, the liner/leachate compatibility study for the IDF is based on the W025 synthetic leachate. Further analysis of the applicability of these leachate concentrations is recommended, if the conservative nature of this synthetic leachate requires costly revisions to the lining system to demonstrate compatibility.

5.6.3 Chemical and Radiation Resistance

Leachate will be generated from precipitation events and from water added to the waste for dust control and compaction purposes during operations. In reality, as the landfill nears the end of its operational life, concentrations of contaminants will decrease with time as the leachable waste mass is reduced. During the post-closure period, a robust landfill cover will significantly reduce infiltration, and the corresponding volume of leachate. Soluble contaminants leached from the waste will come in contact with the landfill bottom liner system during the operation period (approximately 10 years for each of the four planned phases) and minimum post closure period (30 years). The geosynthetics and admix lining system components may be in contact with soluble contaminants as long as contaminants are present in the landfill.

The expected chemical make up of the leachate for the IDF landfill was determined based on previously conducted compatibility studies (as discussed above) applicable to the same waste stream (the W025 studies), summarized as follows.

5.6.3.1 Geomembrane

HDPE geomembranes can deteriorate from contact with certain leachates, resulting in a decrease of elongation at failure, an increase in modulus of elasticity, a decrease in the stress at failure, and a loss of ductility.

Studies performed on polymer materials like HDPE show that their properties begin to change after absorbing ionizing radiation between 1,000,000 to 10,000,000 rads (Koerner et al., 1990). The HDPE geomembrane lining the bottom of the landfill will absorb ionizing radiation energy from the leachate generated in the landfill. Energy will be absorbed during the operational life of the landfill, as long as there are liquids with ionizing radionuclides in contact with the geomembranes.

Relevant compatibility studies on HDPE geomembranes have been performed for the W-025 Landfill (Golder, 1991a; TRI, 1995; WHC, 1995). The results of these studies indicate that a HDPE geomembrane will function well as a liner beneath the landfill waste. EPA Method 9090 tests performed on HDPE geomembrane for the W-025 landfill, using the synthetic leachate solution (assumed representative of IDF leachate concentrations) resulted in no evidence of geomembrane deterioration. A comparison between the anticipated IDF landfill leachate (W-025 Landfill) and that used in compatibility tests for other facilities is summarized in Table 5-2.

Geomembrane samples tested for the W-025 facility did not produce measurable changes in the HDPE liner properties when irradiated for 120 days with a total dose of 50,000 rads. HDPE geomembranes are manufactured with additives, such as carbon black and antioxidants, to improve ductility and durability. The literature also indicates that these additives allow higher doses than standard HDPE material without additives (Kircher and Bowman, 1964). The literature indicates that thin films (i.e., 0.002 inches) of different types of HDPE material alone can become brittle when irradiated at doses between 4,400,000 and 78,000,000 rads. Studies performed using polymer materials, with carbon black and antioxidant additives, show that properties typically begin to change at a total radiation dose of between 1,000,000 and 10,000,000 rads (Koerner et al., 1990).

The manufacturers of the geosynthetic products proposed for the IDF landfill have published maximum allowable concentrations of various chemical compounds that can contact the HDPE geomembrane without adversely affecting its performance. The most recent recommended maximum concentrations of chemicals were obtained from the manufacturers of HDPE geomembrane (meeting the requirements for the IDF technical specifications). A list of the manufacturers' maximum allowable concentrations for specific leachate constituents for HDPE geomembrane and the GCL materials is shown on Table 5-3.

5.6.3.2 Geosynthetic Clay Liner (GCL)

The GCL underlying the geomembrane in the IDF landfill consists of processed sodium bentonite clay, sandwiched between two geotextile fabrics. Sodium bentonite is an ore comprised mainly of the montmorillonite clay mineral with broad, flat, negatively charged platelets that attract water, which hydrates the bentonite. The swelling provides the ability to seal around penetrations, giving the GCL its self-healing properties. A GCL product with Volclay-type sodium bentonite (manufactured by CETCO) is specified for installation at the landfill.

The compatibility of GCL materials is usually demonstrated by permeating the material with leachate and then determining its hydraulic conductivity. Typically, solutions with high concentrations of contaminants or pure products are allowed to permeate a sample under confining pressure and the saturated hydraulic conductivity of the material is determined using ASTM methods such as ASTM D5084. A significant increase in saturated hydraulic conductivity (approximately one order of magnitude) for a sample permeated with leachate, compared with a sample permeated with water, would be an indicator of incompatibility.

Based on review of the published studies (Ruhl and Daniel, 1997; Shackelford, et al., 2000; and EPA, 1995), GCLs perform well unless exposed to high concentrations of divalent cations, very acidic or basic solutions, or solutions with a low dielectric constant (such as gasoline). The leachate expected at the IDF will have a pH of 9.2, which is a mid-range pH. The studies further demonstrate that, when confined under a higher normal load (greater than 2000 psf) or if water is the first wetting liquid (Daniel et al., 1997), GCLs will perform well when exposed to high divalent cation concentrations. The GCL for the IDF lining system is expected to confine under normal loads in excess of 2000 psf as soon as the first lift or waste is placed.

No studies were identified that considered the long-term effects of radiation on the physical properties of GCL materials. Since long-term studies cannot be conducted, conservative radiation limitations have been employed. Low-hydraulic conductivity soils have been used at multiple DOE facilities containing radioactive waste. The only known potential adverse reaction that can occur with a GCL is high heat that could dry out the materials. The amount of radioactivity is expected to be low in the IDF landfill waste and will not generate a significant amount of heat that can desiccate the admix liner. Also, it is assumed that the ILAW packages will be cooled to ambient temperatures prior to placement with the cell. It should be noted that the operations layer and drain gravel will provide a 3-foot buffer on the side slope and a 4-foot buffer between the liner system and waste for additional thermal protection, if needed.

Sodium bentonite is the primary clay mineral in a GCL that produces the low hydraulic conductivity and high swell potential. Exposure of sodium bentonite to liquids containing concentrated salts (such as brines), or divalent cation concentrations (such as Ca^{++} and Mg^{++}), reduces the swelling potential and increases its hydraulic conductivity. Concentrated organic solutions (such as hydrocarbons) and strong acids and bases can break down the soil, which also increases hydraulic conductivity. The physical mechanism that causes these changes is a reduction of the thickness, and related absorption capacity, of the diffuse double layer of water molecules surrounding the clay minerals. This results in an effective decrease in the volume of the clay, since the water molecules are not attracted to the clay particles.

The GCL manufacturer allows the use of GCL with few restrictions on maximum chemical concentrations. Leachate concentrations for the IDF landfill (based on synthetic leachate from W025) have relatively high inorganics and dissolved salts. The W025 dissolved salt concentrations are above the manufacturers recommended concentration of 35,000 mg/L (see Table 5-3) (CETCO, 2001). As a point of reference, this concentration of dissolved salts is typical of seawater (USGS, 1989). However, the dissolved salt concentrations in the IDF leachate have been characterized as primarily sodium, and the synthetic leachate was comprised of entirely sodium salts, not the divalent cations such as Ca^{++} and

Mg++, as assumed by the manufacturers. As such, the impact on GCL hydraulic conductivity should be less as compared to divalent cation solutions. Additionally, any effects of leachate degradation on the GCL would be minimized by hydration of the GCLs' sodium bentonite with relatively "fresh" water, allowing the GCL to swell initially and decrease hydraulic conductivity.

The rationale for use of the GCL in the IDF landfill primary liner is to "deflect" leachate from defects or pinholes in the geomembrane over the bottom area and longer-term storage areas (such as the leachate sump trough), where leachate head potential is greatest. The main purpose of the primary GCL is to reduce the actual leakage rate into the LDS in the event of leak in the primary geomembrane (see Section 5.10 and Appendix C.10). The GCL is expected to contact leachate only in the event of a leak in the primary geomembrane. These leachate collection and storage areas are subject to flushing throughout the active life of the landfill due to phased development and fill sequence, resulting in a more dilute leachate in leakage areas prior to attaining maximum leachate concentrations. Based on these considerations, the GCL and landfill liner system approach should perform as intended under the anticipated conditions.

5.6.3.3 Admix Liner

The admix layer consists of onsite silty sand mixed with processed bentonite amendment, similar to that used in the construction of GCLs. The swelling of sodium bentonite provides the ability to seal around soil particles, giving the admix a low hydraulic conductivity and self-healing properties. The compatibility of the admix layer with anticipated irradiation and leachate concentrations were evaluated previously as part of the W025 landfill design (Golder Associates, 1991b). The following summarizes the results of the compatibility testing for the admix layer that are directly applicable to the IDF landfill admix liner, since similar materials will be used in construction. More detailed discussion of the IDF admix liner design is provided in Section 5.4.

In the W025 study, samples of the admix were irradiated, similar to that conducted for the geomembrane layer, as discussed previously. Differences between irradiated and non-irradiated samples were not considered significant based on the results of testing.

The initial W025 admix design contained approximately 8 percent bentonite clay. Testing indicated an acceptable hydraulic conductivity of this admix after hydration in fresh water. However, when hydrated in leachate, some hydraulic conductivity test values were twice the allowable limit and, therefore, this admix formulation was not considered acceptable. This is the same leachate chemistry assumed for the IDF landfill. It should be noted that there are two factors not considered in the W025 compatibility study (Golder Associates, 1991b) that would mitigate the impact of the synthetic leachate on the 8 percent admix samples, as listed below:

Effective stress for samples–hydraulic conductivity tests were performed with effective stresses of 5-10 psi across sample (equivalent to less than one full lift of ILAW packages). It is well documented that higher effective stresses will lower hydraulic conductivity and mitigate the effects of shrinking/cracking in clay under attack from chemicals. In reality, by the time any leachate contacts the lining system, there will be a substantial stress load on the liner that will mitigate the impacts of chemicals in leachate on the admix liner.

First wetting liquid–W025 tests were performed using both site water and synthetic leachate as the initial wetting fluid. It is well documented that if a clay soil is "attacked" by inorganics prior to saturation, the increase in hydraulic conductivity will be more dramatic than if water is first permeant. This was confirmed by W025 testing–there was an order of magnitude difference between samples with water as first wetting liquid as opposed to leachate. It is reasonable to expect something closer to water than concentrated leachate will be the first wetting liquid for the IDF admix liner.

Due to the results in the W025 testing showing greater than acceptable hydraulic conductivity in the admix when exposed to the W025 synthetic leachate, the bentonite percentage was increased from 8 to 12 percent. An admix containing 12 percent bentonite clay was permeated with synthetic leachate and tested with a resulting hydraulic conductivity that was 3 to 10 times lower than the maximum allowable limit (10-7 cm/sec). This admix formulation was considered acceptable with respect to W025 leachate compatibility and is applicable to the IDF. Thus, the technical specifications (see Section 02666) require a nominal 12 percent (range from 11 to 14 percent is acceptable) bentonite by weight for the admix liner. Consideration should be given to lowering the bentonite percentage upon further characterization of the IDF leachate and applicability of the mitigating factors discussed above.

5.6.3.4 Other Materials

Other materials for which compatibility needs to be addressed are the CDN and geotextiles (cushion, separation, and bonded to geonet of CDN). While these materials do not serve a barrier function, they provide either for removal of leachate or protection of the lining system and must continue to function when exposed to leachate.

During the W025 design, the effect of the synthetic leachate on the geonet core of the CDN and the geotextiles was evaluated (Golder Associates, 1991a). The study concluded that a geonet core comprised of HDPE provided adequate chemical and radiation resistance. For geotextiles, the study concluded that geotextiles made of polyester fabric were susceptible to degradation and recommended that geotextile material be limited to a more chemically resistant material such as polypropylene. The technical specifications for the IDF require that geotextiles be made from polypropylene (see Section 02371); thus, the geotextiles used for the IDF should have adequate chemical and radiation resistance.

Table 5-3. Maximum Allowable Concentrations in Leachate by Chemical Category for Geosynthetic Components

Chemical Category	Compatible Concentration for HDPE	Compatible Concentration for GCL	IDF Concentration Dose or Value
Organics	500,000[a] mg/L	500,000[b] mg/L	N/A
Acids and Bases	750,000[a] mg/L	500,000[b] mg/L	0[d] mg/L
Inorganic	500,000[a] mg/L	500,000[b] mg/L	204,000 mg/L[c]
Dissolved Salts	No Limit	35,000[a] mg/L	204,000 mg/L[c]
Strong Oxidizers	1,000 mg/L	No limit	0 [d] mg/L
Radionuclides	1,000,000[b] rads	No limit	50,000 rads[c]
PH	0.5 - 13.0[a]	0.5 - 13.0	9.2

a. Based on the typical manufacturers' maximum concentration of the list of constituents by the manufacturers.
b. Based on reported literature values.
c. Based on synthetic leachate formula for W-025
d. Strong acids, bases, or oxidizing compounds were not identified in the W-025 compatibility studies.

5.7 Drainage Layer

The drainage layer for the LCRS consists of three components: the separation geotextile, the CDN, and the drainage gravel. Analyses for the drainage layer required evaluation of these components.

5.7.1 Geotextile Analyses (Separation)

Analyses were preformed to verify that a separation geotextile between the operations layer and leachate collection drain gravel is required by evaluating natural graded filter criteria for these materials. Results indicated that natural filter criteria could not be achieved, thus a separation geotextile is required between the operations layer and drain gravel. Supporting natural filter calculations are included in Appendix C.6.a.

Analyses were conducted to determine the proper apparent opening size (AOS) and permittivity of the separation geotextile. Required AOS and permittivity were determined based on filter, fines retention, and clogging potential criteria. Results of these analyses were used to develop the technical specifications for the separation geotextile (see Section 02371). Supporting geotextile filter calculations are also included in Appendix C.6.A.

5.7.2 CDN Selection

The CDN selection was based on analysis of two design issues, CDN geotextile puncture resistance and CDN required transmissivity.

5.7.2.1 CDN Geotextile Puncture Resistance

The LCRS CDN layer at the IDF will be overlain by the operations layer on the 3H:1V side slope. The operations layer is allowed to contain a particle size up to 2 inches in dimension. An analysis was performed to determine if the geotextile bonded to geonet (to form the CDN) would be punctured by particles/rocks of this size.

The method developed by Koerner (1998) was used to calculate the puncture resistance. Koerner's method considers the size and shape of the rock, as well as other factors that could decrease the long-term strength of the geotextile. The two loading conditions examined were initial placement of the operations layer and the final depth of waste and closure cover. The geomembrane puncture resistance analysis (see Section 5.5.4) provides the details for the load analysis for these conditions. Detailed calculations for CDN geotextile puncture resistance and corresponding cushion geotextile requirements are included in Appendix C.6.b1.

Results of the analyses indicate that the required puncture resistance is 11.2 lbs. The minimum specified value for Type 1 geotextile (see technical specifications, Section 02371) is 65 lbs. Applying a partial safety factor of 2 gives a minimum resistance of 32.5 lbs. Therefore, the proposed geotextile bonded to the geonet of the CDN will resist puncture with a global safety factor of 2.9; it is adequate for resistance to puncture from the overlying operations layer under the pressure of maximum landfill contents pressure. Koerner (1998) recommends a minimum global safety factor of 2.0.

It should be noted that the results of this analysis are considered conservative because the analytical method assumes only a uniform particle size and does not take the surrounding soil matrix into consideration. This would effectively reduce the particle size by a considerable degree.

5.7.2.2 CDN Required Transmissivity

An additional selection criteria for the CDN is the required transmissivity (or flow rate) under design loading conditions. For the IDF two cases require analysis:

LDS CDN on bottom and side slope–For this case, the critical condition is to ensure that the transmissivity as required by WAC and EPA regulations (3 x 10-5 m2/sec) under the maximum load from the landfill contents can be achieved.

LCRS CDN on side slope only–There are actually two loading conditions for the LCRS CDN on the side slope. One is the open slope condition with operations layer only over the CDN, which is a low normal load (1,000 psf) condition. The second is in the filled condition, which is a high normal load (15,000 psf) condition. Based on the results of leachate production analyses using the Hydrologic Evaluation of Landfill Performance (HELP) model (see Section 5.8), the required transmissivity for the LCRS CDN is 6.5 x 10-5 m2/sec for the open slope condition and 1 x 10-5 m2/sec for the filled condition.

For each case, the approach was to compare the required transmissivity to typical manufacturer's data with test conditions (i.e., normal load and material boundary), similar to the design conditions. The allowable transmissivity (φ) was determined using guidance provided by GRI standard GC-8 (2001), *Determination of the Allowable Flow Rate of a Drainage Geocomposite*. The GRI-GC8 standard uses the following equation:

$\varphi_{allow} = \varphi_{100\text{ hr test}}$/Reduction Factors for intrusion, creep, chemical clogging and biological clogging

The FS for design was then determined as follows:

$FS = \varphi_{allow}/\varphi_{required}$

Transmissivity data for the 100-hour test data was obtained from the manufacturer for both 200-mil and 250-mil thickness CDN for normal loads of both 1,000 psf and 15,000 psf. Test data was provided for a number of boundary conditions including flow tests between a geomembrane and a soil, as would be the case for the LCRS or LDS CDN. Test data used as the basis for the analyses are included with the calculations presented in Appendix C.6.b2.

Based on the analyses, a higher flow, thicker (250-mil minimum) CDN is required, due to the reduction of flow under the high normal loads in the final filling configuration. The technical specifications (see Section 02373) provide the required index values for the geonet core of the CDN as well as the CDN itself (with geotextile bonded to both sides of the geonet), based on the results of this analysis. The transmissivity requirements in the technical specifications are index values and not in-service condition values, as determined in this analysis. These index values are representative of testing that manufacturers typically perform in production and are correlated to design conditions using the approach outlined in GRI GC-8.

5.7.3 Drainage Gravel Selection

Section 02315 (Fill and Backfill) in the technical specifications requires that drain gravel meets the requirements of WSDOT 9-03.12(4) for gradation. The technical specifications also require a performance specification for a hydraulic conductivity greater or equal to 10-1 cm/sec.

Hydraulic conductivity of the specified drain gravel was estimated using two different empirical relationships. The most relevant of the two estimates a minimum hydraulic conductivity of 1 cm/sec, based on the specified gradation curve for WSDOT Gravel Backfill for Drains (9-03.12[4]). Supporting calculations are included in Appendix C.6.c.

The minimum estimated hydraulic conductivity for the drain gravel exceeds the required (by WAC and EPA regulations) hydraulic conductivity of 10-2 cm/sec by a factor or 100 to 1,000, and the performance specification hydraulic conductivity of 10-1 cm/sec by a factor of 10 to 100. This exceedance makes an allowance for two items: (1) it allows for the uncertainty in the empirical formulas used to predict hydraulic conductivity, and (2) it also allows for the potential long-term reduction in hydraulic conductivity in the drain gravel as fines from waste filling and the operations layer migrate into the gravel over time.

As part of Construction QA, testing it is recommended that samples of imported drain gravel be tested for conformance with the gradation and hydraulic conductivity requirements in the technical specifications.

5.8 Leachate Production

5.8.1 Leachate Production Analyses

Estimates of the amount of leachate produced during the development and operation of the IDF were needed to design the components of the leachate collection and conveyance system described in Section 5.9, and to provide information necessary when evaluating slope stability of the side slope and bottom liner systems. Leachate is produced when precipitation falls within the lined area and infiltrates vertically through the waste and/or bottom liner system. The amount of infiltration estimated to occur depends on the hydrologic processes and the relative fraction of precipitation that results as leachate and is collected by the leachate collection system.

The water balance components of the hydrologic process were estimated using EPA's *Hydrologic Evaluation of Landfill Performance (HELP) Model* (Schroeder et. al., 1997), a well known standard for water balance modeling. The HELP model has been widely used for evaluating hydrologic conditions and is the standard model used for providing information necessary for the design of landfill systems. Estimates of the water balance components of the hydrologic cycle provided by HELP include precipitation, evapotranspiration, surface water runoff, vertical percolation, soil moisture storage, and lateral drainage in soil layers.

The HELP model requires input of weather data, representing the conditions at the landfill location, soils data representing the various layers of cover soils, waste materials, and soils underlying the waste layers, and other design data used by the model for water balance calculations. A detailed description of the model and modeling inputs are included in Appendix C.7.

The development of the IDF from Phase I through Phase IV was considered to determine the maximum flow condition expected during development and operation of the landfill. That is, various combinations of open and interim closed phases were considered and the combination calculated to produce the maximum amount of leachate was chosen for analysis. The chosen combination was Phase I through III under interim closure condition and Phase IV in the open condition with little or no waste present. The flows from this condition were used to size the LCRS collection piping and pump systems.

Water balance components were taken directly from model output and a spreadsheet was used to calculate the volumes of leachate by multiplying the HELP output parameter by the area of the type of system modeled. For example, the lateral drainage estimated by the HELP model for the uncovered side slope condition in Phase IV development was multiplied by the total side slope area to determine the total volume of leachate from that area. A spreadsheet summarizing the estimated leachate flows is included in Appendix C.7.

The following modeling results were used for various aspects of design of the IDF systems:

LCRS collection system–Modeling results for the peak day event were used to size the leachate collection system piping that conveys flow to the LCRS systems. The peak day event, as predicted by HELP and referenced herein, was a 1.6-inch precipitation event. This event is approximately 25 percent higher than the 25 year, 24 hour peak day storm event of 1.28 inches (Appendix C.9), required by regulations to be used when complying with the maximum 12 inches of head over the liner (WAC 173-303-665, see Section 2). The spacing of the LCRS perforated collection piping and the properties of the drain gravel material that convey lateral drainage flows above the bottom liner geomembrane to the collection piping and LCRS sump area were checked to insure the maximum head buildup above the sump area of the liner system did not exceed the maximum allowed according to regulatory requirements, as outlined in Section 2.

LCRS pump and forcemain systems–Modeling results for the peak day event were used to size the LCRS high flow pump system that conveys flow to the leachate storage tanks and truck loadout facilities. Average monthly flow rates plus one standard deviation (resulting in a conservatively-high expected flow rate) was used to design the LCRS low flow pump system for pumping from the IDF during average monthly conditions.

Leachate Collection Storage–Volumes for the peak day event and assumptions for the operational rate of removal of leachate from the tanks were used to size the storage tanks. Storage tank sizing is described in Section 5.9.2.2.

Liner system material properties and stability analyses–The lateral drainage layers of the side slope and bottom liner systems were checked to insure the transmissivity of the layers was sufficient to convey lateral flows and maintain less than the maximum head buildup over the liner system. The seepage height above the liner was used when checking the liner system for veneer stability.

5.9 Leachate Collection System

5.9.1 Earth Loading Analyses

5.9.1.1 Leachate System Loading Analyses for Piping within Phase I Liner Limits

Loading over the leachate system piping include all layers of soil materials, wastes, and anticipated traffic loading. The maximum loading occurs over the piping in the LCRS and LDS sump area, because of its low elevation and the height of material–both waste and soil layers–overlying the sumps. Loading calculations from the geosynthetic liner puncture resistance calculations described in Section 5.5.4 were modified to represent the maximum loading in the LCRS/LDS sump area. Other pipes in the Phase I area, including piping outside the sump and the side slope riser piping, will be subjected to less than the maximum loading. The maximum loading is listed in Appendix C.8.a, along with the calculations for pipe sizing required to withstand this anticipated pipe loading.

Pipe wall thickness was selected based on the maximum loading anticipated in the sump area such that the pipe will not fail due to excessive deflection, wall buckling, or wall crushing. All other piping in Phase I outside of the sump area was chosen with the same standard dimension ratio (SDR) to withstand the maximum load. Standard analysis methods, as recommended by the manufacturer of HDPE pipe made from PE3408 type resin, were used to evaluate pipe strength under loading. These standard methods are based on flexible pipe design practice as applied to HDPE piping. The manufacturer's recommended design analysis techniques are based on standard analysis techniques, including the Iowa formula (*Waste Containment Systems, Waste Stabilization, and Landfills Design and Evaluation,* Sharma and Lewis, 1994), with conservative factors of safety. The potential loss of strength due to the perforations in the perforated collection piping was assumed non-significant, based on actual test results of perforated pipe under similar load rates. The pipe material assumed is High Density Polyethylene PE3408 pipe with a cell classification of 345434C or better. The flexural modulus and material strength of the pipe was per manufacturer's published literature, based on this classification of pipe.

5.9.1.2 Leachate System Loading Analyses for Piping Outside of Phase I Liner Limits

Piping outside the Phase I liner area includes all underground piping between the crest pad building, combined sump, leachate transfer building, storage tank, and tanker truck load out facility (see Drawing H-2-830846). The civil road layout in these areas is generally configured to allow medium to light duty trucks, such as would be used for operations and maintenance activities. The leachate tanker truck accesses the concrete truck load pad only, and would not normally pass over any piping. However, the piping outside the Phase I Liner area was designed for H-20 semi-trailer type loading to be conservative. The same SDR pipe that used for the high loading within the Phase I liner limits as described in Section 5.9.1.1 was assumed for all piping exposed to earth and traffic loading outside of the Phase I liner limits. The expected pipe loading for H-20 loading plus earth load was compared to the loading used for designing the piping inside the Phase I liner limits and was found to be much lower. Since the pipe SDR is sufficiently strong for the maximum loading inside the Phase I limits, it will have more than sufficient strength for loading expected outside the Phase I limits. Calculations are included in Appendix C.8.a.

5.9.2 Leachate System Hydraulics Analyses

5.9.2.1 Leachate System Hydraulics Analyses

The leachate collection and conveyance system collects leachate that accumulates as a result of precipitation landing within the footprint of the cells, and it conveys the collected leachate from the cells to a storage tank or tanker truck. Perforated collection piping in the LCRS collects and conveys leachate from the bottom liner system and conveys it to a LCRS sump area in both cells. Lateral flow of leachate from the side slope and bottom liner areas also is conveyed directly to the sump area through a high permeability gravel layer and/or geosynthetic drainage net material. Submersible pumps in the LCRS sump and contained within perforated riser pipes convey leachate to the crest pad building and directly to the leachate storage tank or the tanker truck load facility. Hydraulics analysis was conducted to size the gravity flow piping of the LCRS collection piping and the pump and force main system from the sump area to the storage tank and tanker truck load facility. Sizing and design of leachate collection and conveyance systems were based on ultimate buildout of the IDF through Phase IV. That is, the components installed as part of the Phase I design are sized for the ultimate configuration and flows estimated through Phase IV.

5.9.2.2 LCRS Gravity Flow Analyses

The LCRS perforated collection piping was sized using standard gravity flow analysis techniques. The pipe size (nominal 12-inch diameter) was chosen as double the minimum size required for cleanout of the pipe to insure any accumulation of fines would not significantly restrict the flow in the pipe, even though the drain gravel surrounding the pipe will have minimal fines present and geotextiles are present in the lining system to further restrict the migration of any fines. The maximum flow used for sizing was the maximum from the HELP predicted maximum day flow rate or the pump flow rate, based on the pump chosen to convey flow out of the cell.

Perforations in the pipe were sized to allow flow rates much higher than the required maximum flow rate out of the cell, with minimal head loss. This assumption was more conservative by virtue of the fact that the main LCRS collection pipe will only collect and convey a portion of the lateral drainage flow from the cell; the drain gravel and CDN will also convey a portion of the flow. Calculations are included in Appendix C.8.b.

5.9.2.3 Leachate System Pumps and Force Mains Analyses

The pump and forcemain systems for conveying leachate out of the cells and into the leachate storage tanks and to the tanker truck load out facility, and the design considerations for each are described below. Calculations are included in Appendix C.8.b.

LCRS pumps and forcemains–The LCRS pumps and forcemains convey leachate out of the cells to storage tanks or the tanker truck load areas. The criteria for pumping capacity is that the maximum head over the sump area of the cell will not be allowed to exceed 12 inches during the peak day event and during normal operations. To meet the requirement for not exceeding the 12-inch criteria for the peak day event, a LCRS high flow pump was sized to handle the expected peak day flow rate, as estimated and described in Section 5.8, Leachate Production. Hydraulic analyses were conducted to size the pump and forcemain piping according to standard practice to convey the maximum flow rate.

A LCRS low flow pump was sized to convey flow out of the cells under normal, monthly operations. The criteria established for the low flow pump was to convey the average monthly flow plus one standard deviation from the cells, assuming the pump could remove that amount of flow with less than continuous operation. The highest value of the average month plus one standard deviation was used for the maximum flow required of the pump. Under lower flow required conditions, the pump would operate near this rate, depending on the system curve head loss characteristics, but would run for a shorter length of time to remove the volume of leachate from the cell.

LDS pump and forcemain–The LDS pump and forcemain conveys flows from leakage through the LCRS sump area, if in the unlikely event any leakage occurs, to the storage tank or tanker truck load out facility. The LDS system is sized to convey the flow equal to the ALR (described in Section 5.11); however, this rate is so small that the pump capacity is much higher than necessary.

Leachate transfer pump to truckload and forcemain–Under normal operations, leachate conveyed out of the IDF will be routed to the leachate storage tank. Periodically the leachate will need to be conveyed to tanker trucks for transport to an offsite water treatment facility. A transfer pump is required to move water from the storage tank to the tanker truck loadout facility. The pump and forcemain were sized to convey approximately 250 gallons per minute (gpm), a rate commensurate with timely loading of the tanker trucks that have capacities equal to approximately 7,000 gallons. At 250 gpm, the tankers can be loaded quickly, depending on the operational requirements for moving leachate and making storage tank capacity available under high precipitation conditions and/or the condition when the storage tanks are at or near capacity. Storage and operations considerations are described in Section 5.9.2.4.

Combined sump pump and forcemain–The combined sump pump and forcemain must convey flow from the sump to the leachate storage tank. The flow criteria for this pump was set at approximately the same flow as the leachate transfer pump. This is based on the worst case scenario of the leachate transfer pump accidentally being left on when the tanker truck is filled, causing the full 250 gpm flow to overflow the truck, collect on the pad, and drain into the combined sump. Under less than maximum flow conditions, the pump would cycle when any leakage from other systems connected to the sump pump reached the level on control setting for the pump. In this case, the pump would cycle quickly to pump the small volume of the inner sump into the storage tank.

Crest pad building sump pump–A small sump pump is provided in the crest pad building to remove minor amounts of water in the sump from sampling activities or piping leaks. The nominal flow rate was chosen as a minimum of four gpm. The pump discharges into the main forcemain line to the storage tank or tanker truck load out facility.

The pump and forcemain piping systems were modeled using standard hydraulic analysis techniques. Actual pump curves for preliminary pump selections were input and the analyses conducted to determine the estimated run condition for the various operational conditions. For example, a pump was chosen for the LCRS high flow pump and forcemain system, and the analysis was run for the conditions of the pump conveying flow to the leachate storage tank and directly to the tanker truck load out facility. Different flow rates and system pressures resulted, based on the differences in the system curve for each flow path versus the pump curve characteristics. Pump cycle times were considered for the flow requirements and total removed volume. The manufacturer's recommendations for cycle times and other operating requirements, where applicable, were checked.

5.9.2.4 Leachate Collection Storage Analyses

The results of the leachate production analysis indicate a total of approximately 269,000 gallons of leachate must be removed from the IDF landfill within 24 hours after a peak storm event. A temporary storage tank for each cell was sized to store leachate generated by the associated cell. The leachate storage tank capacity is dependent on the flow rate of leachate into and out of the tank as well as a factor of safety.

The leachate production analysis indicates the worst case flow rate out of each cell into the associated tank would be 157 gpm (sum of the required flow rates of the high and low flow leachate pumps). The leachate transfer pump for each cell can fill a tanker truck at a maximum of 250 gpm; however, the limiting factor is how often a truck can be filled.

The calculation in Appendix C.8.c presents the method of determining the appropriate storage capacity of each leachate storage tank. The following leachate tanker truck loading activities were assumed:

- Tanker Capacity 7,000 gallons
- Number of tankers per cycle 1
- Hours per cycle (roundtrip) 2.4
- Hours per shift 8
- Shifts per day 1
- Leachate tank level prior to event 2 feet

The calculation indicates that each tank requires a maximum operational capacity of 375,000 gallons to maintain a safety factor of 1.5. The assumptions made in the calculation must be adhered to during operational activities to maintain the calculated safety factor.

5.10 Surface Stormwater

The surface stormwater analysis was done to determine the sizes of the surface stormwater facilities necessary for the IDF Phase I Critical Systems Design. The surface stormwater analysis is documented in detail in Appendix C.9.

The governing regulation is WAC 173-303-665(2) (c) and (d). This requires that the stormwater system be designed to prevent flow onto the active portion of the landfill during peak discharge from at least a 25-year storm. It also requires that the runoff management system be designed to collect and control at least the water volume resulting from a 24-hour, 25-year storm.

The primary purpose of the proposed stormwater facilities is to prevent stormwater runoff from areas adjacent to the two Phase I cells from entering the cells during Phase I operation. This will be done by collecting, conveying, and safely discharging stormwater from areas outside of the two Phase I cells that would otherwise run into these cells.

The Department of Ecology has issued State Waste Discharge Permit Number ST 4510 for industrial stormwater discharges to the ground through engineered land disposal structures on the Hanford site (ST 4510, Ecology, 1999; DOE/RL97-67 Revision 3, January 2000). Since the design for this project does include facilities for collecting stormwater runoff and discharging it to the ground, the permit was reviewed to determine whether it applied to these stormwater discharges. To be covered by this permit, the stormwater must be considered an industrial discharge that is collected in an engineered structure and is then discharged to the ground through an engineered structure. A stormwater discharge is an industrial discharge if the stormwater has the potential to come into contact with an industrial activity or is collected within an area of industrial activity. The purpose of the stormwater facilities that have been designed for this project is to prevent the stormwater from areas outside of the Phase I landfill from entering the landfill area. Therefore, the stormwater collected by these facilities would probably not be considered industrial stormwater. To be an engineered structure for the collection of stormwater, the structure has to be an impervious surface that is directly associated with industrial activities. The stormwater collection facilities designed for this project do not have impervious surfaces. Therefore, permit ST 4510 does not apply to the stormwater system designed for this project.

Stormwater facilities were designed only for the operation stage of Phase I and not for interim or final closure conditions. Therefore, no stormwater facilities have been designed for stormwater runoff from the Phase I cells after construction of their interim closure or final closure. Stormwater needs for the construction, operation, and closure of future phases were not considered.

No stormwater collection and conveyance facilities were analyzed and/or designed for any of the roads and support facilities that will be constructed as part of this project. The roads will be gravel surfaced, and stormwater that does run off the roads into adjacent areas will infiltrate. The stormwater from the roofs of the buildings will be caught in gutters and discharged to the ground surface via down spouts. The stormwater that falls on the leachate tanks will evaporate off the floating covers.

5.10.1 Existing Conditions

Under existing conditions, the area around the Phase I site slopes down gently from south to north at an average grade of approximately 0.5 percent. The only area that may generate stormwater that can run into the Phase I excavation is the area that extends south from the excavation area to the crest of the sand dunes, located north of 1st Street (see drainage areas figure in Appendix C.9). This drainage area is moderately vegetated, primarily with large sage brush and grasses. The soils are generally sandy, with relatively high rates of infiltration. This area typically receives little precipitation. There is little to no runoff, and stormwater normally either infiltrates or is used by the vegetation. No existing drainage channels are apparent. The groundwater table is approximately 300 feet below the ground surface.

5.10.2 Proposed Stormwater Facilities

To prevent stormwater from the area south of the Phase I excavation from running overland into the excavation, a combination stormwater berm/ditch will be constructed south of the top of the south slope of the excavation. The south end of the excavation will be approximately 1,400 feet long, and the ground will be essentially flat. The berm/ditch will have a center high point and then slope down to the east and to the west (two discharge points). A berm will be constructed immediately south of the ditch. At the centerline of the excavation, the invert of the ditch will be at the existing ground surface, and the berm will form the south slope of the ditch. The ditch will be excavated, with a longitudinal slope of 0.5 percent to both the east and the west. This will be done in order to minimize the depth of the ditch at its east and west ends. Culverts will be installed at the east and west ditch ends to convey the flow under the access roads. The culverts will discharge into the east and west infiltration areas.

The base map does not show any areas where stormwater runoff from offsite areas may flow into the east or west boundaries of the Phase I excavation. However, if any offsite stormwater should flow toward these boundaries, the fill for the berm access road and the shine berm will prevent the stormwater from flowing into the excavation (see drainage areas figure in Appendix C.9). The intercepted stormwater will flow south along the toe of the fill and either infiltrate or flow overland to the north, away from the site at the north end of the berm access road.

The ground slopes away from the north end of the Phase I site, so there will be no offsite stormwater running toward the north Phase I boundary.

The Phase I liner will end north of the toe of the south slope of the Phase I excavation. In order to reduce potential leachate flows, a stormwater berm/ditch will be constructed just south of the south end of the liner. This berm/ditch will intercept and convey stormwater runoff from the unlined south slope and the unlined southern ends of the east and west slopes. The berm/ditch will be sloped to drain to the east. A stormwater pipe will convey the stormwater under the landing for the access ramp and will discharge to the excavation infiltration area. If this pipe ran straight from the ditch to the infiltration area, it would not have adequate cover. Therefore, a catch basin with a solid cover will be installed near the west end of the stormwater pipe. The invert of the pipe out of the catch basin will be lower than that of the pipe running into this catch basin. The stormwater pipe that will run from the catch basin to the excavation infiltration area will then have adequate cover. The excavation infiltration area will be excavated in the southeast corner of the excavation.

The south edge of the access ramp into the Phase I excavation and the south edge of the “flat” area at the bottom of the access ramp will serve as ditches. The access ramp will have a cross-slope of 2 percent down to the south. The “flat” area at the bottom of the access ramp will have a slope down to the south that varies between 1 and 3 percent. Adjacent to each of these will be the south slope of the excavation. Construction of a full V-shaped ditch along the south side of the access ramp and the “flat” area was considered. This idea was rejected because it would result in a larger excavation with the top of the Phase I south slope moved further south.

The stormwater facilities are shown on the Phase I Grading and Drainage Plan drawing (Drawing H-2-830830).

Stormwater runoff from the north, east, and west lined slopes of Phase I will run into the bottom lined area and will become leachate. There are no provisions in the design of the Phase I critical systems to divert clean runoff from these side slopes and discharge it to the surface water system instead of the leachate system at this time. However, a rain curtain or other approach to reduce the amount of clean runoff from the lined area that enters the leachate system may be considered in the future.

5.10.3 Analysis

The surface stormwater analysis is documented in Appendix C.9 and is summarized below. Stormwater runoff flows were estimated for a 24-hour, 25-year design event, using the Soil Conservation Service curve number methodology as documented in *Urban Hydrology for Small Watersheds* (U.S. Department of Agriculture, June 1986) and the Hydraulic Engineering Cirular-1 (HEC-1) computer program (*Flood Hydrograph Package (HEC-1)*, U.S. Army Corps of Engineers, Hydrologic Engineering Center, revised June 1988). The precipitation data used was based on information from the *Hanford Site Climatological Data Summary 2001* (Pacific Northwest National Laboratory, May 2002). The ground at the project site is periodically frozen during the winter months, when the most precipitation falls. Therefore, it was assumed that the ground was frozen for the runoff flow calculations.

The peak flows (calculated using the HEC-1 model) were checked for reasonableness. The tabular and graphical methods in TR 55 were used to estimate peak 25-year flows for each of the drainage areas modeled in HEC-1. The results confirmed the reasonableness of the peak flows calculated by HEC-1.

The berm/ditches were designed to convey the peak 25-year flow with a minimum freeboard of one foot.

The infiltration areas were sized based on containing and infiltrating the runoff from the 24-hour, 25-year design event, without causing the water surface to extend above the upstream end of the culvert or stormwater pipe that will discharge to the infiltration area. No specific infiltration data have been collected at the IDF project site. However, infiltration rates have been determined for use at the Waste

Treatment Plant (*Geotechnical Report Supplement No. 1*, Shannon and Wilson, April 2001). These infiltration rates were used in sizing each of the infiltration areas.

The culverts and stormwater pipes were designed to convey the peak 25-year flow with a maximum headwater to a diameter ratio of 1.25. Both inlet and outlet flow conditions were analyzed. The starting water surface for the outlet flow condition calculations were the maximum water surface elevation estimated for the associated infiltration area for the 24-hour, 25-year design event.

5.11 Action Leakage Rate (ALR)

5.11.1 LDS ALR

The ALR is defined in WAC 173-303-665(8) and the Final Rule (EPA 1992a, 40 CFR Part 264.222) as the "maximum design flow rate that the leak detection system…can remove without the fluid head on the bottom liner exceeding 1 foot". This calculation was performed to determine the ALR for the IDF lining system. The IDF consists of two cells, each with an area of approximately 8.5 acres.

In addition to determining the ALR, an estimate of actual leakage rate through the proposed primary bottom lining system is provided as a comparison to the calculated ALR. HELP modeling for the side slope indicates negligible head build-up on the side slopes (see Section 5.8), thus an estimation of the actual leakage rate was determined for the bottom primary lining system only.

EPA provides a formula (based on Darcy's Law for calculating this flow capacity), assuming that it originates from a single hole in the primary liner (EPA, 1992b). Calculations presented in Appendix C.10 provide details of the method of analysis and input data. The ALR calculations are dependent on the transmissivity value for the CDN. A value of 3 x 10-5 m2/sec was used in the ALR analysis (equivalent to the value required by WAC and EPA regulations for the LDS, Section 5.7.2). Calculations in Appendix C.6.b2 provide justification for the transmissivity used in the ALR analyses.

The results of the analyses indicate the ALR for each IDF cell is 206 gallons per acre per day (gpad) or approximately 1,800 gallons per day per cell. This ALR includes a factor of safety of 2 in accordance with EPA guidelines (EPA, 1992b). It is also much lower than the capacity of the pump that removes liquid from the LDS. The estimated actual leakage rate for the composite primary lining system is 0.06 gpad (small defect) to 0.08 gpad (larger defect) for a composite liner with good intimate contact, and 0.3 gpad (small) to 0.4 gpad (large) for poor contact. Detailed calculations for both rates are presented in Appendix C.10.

The proposed primary composite lining system has a much lower estimated leakage rate than the ALR. This demonstrates the benefit of the GCL that is included in the primary bottom lining system, to provide a composite lining system and minimize actual leakage rate through the bottom primary lining system.

5.12 Building Systems Analyses

5.12.1 Geotechnical Design Parameters

The key geotechnical parameters and analyses for structural design of the supporting facilities for the Hanford IDF included the following:

- Bearing Capacity
- Settlement
- Modulus of Subgrade Reaction
- Earth Pressures
- UBC Seismic Soil Parameters

The methodologies, input data, and results for each of these categories of analysis are presented in detail in Appendix C.11.A.

5.12.2 Structural

5.12.2.1 Crest Pad Building Foundation Analysis, Pipe Bracing and Winch

The crest pad building foundation was analyzed as a concrete slab on an elastic foundation. The foundation was modeled with springs to model the vertical sub-grade reaction. The value of the vertical sub-grade reaction was provided by the geotechnical engineer. The applied loads and load combinations were input into Visual Analysis (version 4.0), a finite element program. The finite element analyses results include elastic settlement, moments, and shears values of the concrete slab. The results were then used to design slab depth and reinforcing.

Load reactions from the pre-engineered metal building were estimated using hand calculations and applied onto the concrete slab at the corners of the slab. It is a reasonable assumption that the frame loads from the pre-engineered metal building will only occur at the corner of the building, since the size of the building will not require any intermediate framing.

Loads and load combinations were used as required by TFC-ENG-STD-06, REV A. Performance category, PC-1 was used as specified and applied as applicable for both wind, seismic, and load combinations requirements.

In summary, the analyses results showed that an 8-inch thick slab sufficed with #5 reinforcing at 12-inch centers. The analyses results also showed that a 1 foot-10 inch edge thickening around the perimeter of the building would be sufficient. More detailed accounting of the analyses is presented in Appendix C.11.b1.

The pipe bracing and support for the small diameter PVC (polyvinly chloride) piping included both gravity as well as lateral load resistance, due to a seismic event. The governing piping support is assumed a 6-foot-tall cantilever support, with the piping load and 50 pounds of lateral load applied to the top of the support. The 50 pound lateral load was used in lieu of the calculated seismic load because the calculated seismic load was only 19 pounds. Using a 50 pound lateral load gives the pipe support system greater rigidity. Detailed calculations of the pipe supports are included in Appendix C.11.b2.

The winch support was analyzed as a vertical cantilever that supports the winch and resists a total lateral load of 400 pounds. A 400 pound lateral load was used since the entire gravity load of the pump and the hoses adds up to this weight. Therefore, using 400 pounds in the horizontal direction is conservative. Detailed calculations of the winch support are given in Appendix C.11.b3.

5.12.2.2 Leachate Transfer Building Foundation Analysis

As the leachate transfer building foundation is considered as a slab–on-grade, only hand calculations were performed. Foundation soil reactions were considered to be distributed linearly, then soil pressure distributions were applied to the concrete to calculated the moment and shear values for design of the concrete slab and reinforcing steel.

Load reactions from the pre-engineered metal building were estimated using hand calculations and applied onto the concrete slab along the perimeter of the slab.

Loads and load combinations were used as required by TFC-ENG-STD-06, REV A. Performance category, PC-1 was used as specified and applied as applicable for both wind, seismic, and load combinations requirements.

In summary, the analyses results showed that the 2-foot-6 inch-thick slab with #6 bars at 12-inch centers will suffice and appears to be overdesigned. The 2-foot-6-inch thickness is not based on concrete strength requirements but more for frost depth cover, simplifying the ground forming, and reinforcing bending requirements. Detailed calculations of the analyses are presented in Appendix C.11.c.

5.12.2.3 Leachate Tank Foundation Analysis

The leachate tank foundation is considered to be a concrete ringwall, per AWWA D103-97. The tank gravity loads, including both water load and tank dead loads, were considered in the design of the ringwall.

AWWA D103-97, Factory-Coated Bolted Steel Tanks for Water Storage is not listed in the TFC-ENG-STD-06, REV A. AWWA D100-96, Welded Steel Tanks for Water Storage, is listed; however, this standard does not apply, since the tank will be a bolted steel tank. Therefore, the tank will be designed per AWWA D103-97, Factory-Coated Bolted Steel Tanks for Water Storage.

The analysis of the concrete ringwall and reinforcing is based on the hoop tension on the ringwall from the surcharge of the liquid weight on the soil within the ringwall. In summary, a 4-foot-6-inch-deep by 1-foot-6-inch width ringwall with #7 at 12-inch-longitudinal reinforcing on each face of the ringwall will suffice. Detailed calculations of the analyses are presented in Appendix C.11.d.

5.12.2.4 Truck Loading Station Foundation Analysis and Leachate Loading

The Truck Loading Station foundation was analyzed as a concrete slab on an elastic foundation. The foundation was modeled with springs to model the vertical subgrade reaction. The value of the vertical subgrade reaction was provided by the geotechnical engineer. The applied loads and load combinations were input into Visual Analysis (version 4.0), a finite element program. The finite element analyses results include elastic settlement, moments, and shears values of the concrete slab. The results were then used to design slab depth and reinforcing.

Loads and load combinations were used as required by TFC-ENG-STD-06, REV A. As required, AASHTO HB-16 loading was used with an HS 20-44 load wheel pattern. For maximum axle load, 40,000 pounds was used instead of 32,000 pounds as required per HS 20-44. An impact factor was also applied as required by AASHTO HB-16.

The wheel pattern loading was arranged in three positions on the slab to yield the maximum moments and shears. Supporting calculations and further discussions are presented in Appendix C.11.e1.

The leachate loading support was analyzed as a post with an horizontal boom attached near the top of the post. The design load included the dead weight of the post, boom, and piping full of water. Wind loads were analyzed per ASCE 7-98. In addition, the lateral load was compared with a 300-pound point load hanging vertically at the end of the boom. The lateral wind load governed for overall overturning at the base of the post; however, the 300-pound point load governed for the boom attachment to the post.

In summary, a 10-inch by 10-inch tube for the post, with a 6-inch by 6-inch tube as the horizontal boom welded to the post will suffice. The geotechnical engineer has verified that a 5-foot-6-inch-deep and 3-foot-diameter concrete encasement around the post will be sufficient for strength and stability. Supporting calculations and further discussions are presented Appendix C.11.e2.

5.12.3 Mechanical/Heating, ventilating, and air conditioning (HVAC)

5.12.3.1 Crest Pad and Leachate Transfer Building

Heating, ventilating, and air conditioning (HVAC) capacities were calculated for the crest pad and leachate transfer buildings. The temperature within the buildings must be controlled within a range to prevent freezing fluids in piping or overheating electronic devices. The HVAC components for the buildings were selected based on the criteria and calculations provided in Appendix C.11.f and C.11.g.

5.12.4 Electrical/I&C

- This section introduces and summarizes the results of detailed electrical engineering calculations included in Appendix C.11.h.
- IDF leachate collection and handling crest pad facilities (two each)
- IDF leachate storage tank and leachate transfer facilities (two each)

- IDF truck loading facilities (two each)

5.12.4.1 Building Power Supply

Open Items

The Phase I Critical Systems 80% IDF design documents do not identify the following open items:

- Exact location of primary 13.8 kV, 3-phase tie-in
- Exact value of available primary short circuit current at primary tie-in location
- Exact length of primary extension
- Exact location, size, and impedance of utility step-down 13.8 kV – 480/277V three, phase, 4-wire pad mounted transformer(s)

These items are scheduled to be addressed during the next IDF Phase I Non-Critical design.

Assumptions

The following assumptions were made in order to complete the 80% engineering analysis.

- Assume electrical service gear inside each Cell 1 and Cell 2 crest pad building to be powered by separate pad mounted utility transformers
- Assume pad mounted utility transformers to be rated 75 kVA and installed within 100 feet of respective Cell 1 and Cell 2 crest pad buildings
- Assume each pad mounted utility transformer to be radial fed from a common 13.8 kV primary feeder
- Assume each Cell 1 and Cell 2 leachate transfer building to be powered from electrical service gear, located inside respective crest pad buildings
- Assume available short circuit at primary side of pad mounted utility transformer(s) to be 100 MVA with an (X/R) ratio equal to 8
- Assume impedance of 75 kVA pad mounted utility transformer to be 3.2%Z, 2.42%IR, and 2.10%IX
- Assume power factor and efficiency for all pump motors to be 85 percent and 82 percent, respectively
- Assume 25 foot candles of lighting levels to be required for interior of each building
- Assumptions will be reviewed and addressed during the next IDF Phase I Non-Critical design.

Method of Analysis

- Branch circuit, feeder and service calculations in accordance with NEC Code (2002)
- Short circuit analysis (per unit) in accordance with IEEE-Red Book, Standard 141 (1993)
- Grounding electrode analysis in accordance with IEEE-Green Book, Standard 142 (1991)
- Computer analysis by SKM PTW 32 (Power Tools for Windows, 2003)
- Building interior lighting zonal cavity method in accordance with Integrated Engineering Software, Inc. (IES) *Lighting Handbook* (2000)

Analysis Performed Includes

- Calculate and size service, feeder, and branch circuits, based upon demand and design loads
- Calculate and size equipment, equipment bus amperage, protective devices, and motor overloads, based upon demand and design loads
- Calculate and size power feeders and branch circuit wiring, based upon demand and design loads
- Calculate short circuit ratings for equipment
- Calculate feeder and branch circuit voltage drop, and power factor
- Calculate building lighting system requirements

Voltage Drop

Load flow steady state voltage drop calculations for all feeders were based upon an equipment 85 percent power factor. Wire size were calculated and selected so that circuits do not exceed total voltage drop from the source bus to the point of utilization, including feeders and branch circuits:

Service and sub feeders	2 percent	Heat trace from panels	1 percent
Lighting from panels	1 percent	Receptacles from panels	1 percent
Motors from motor control center (MCC)	1 percent	Instrumentation from panels	1 percent

Feeder and Equipment Sizing

Service, feeder, branch circuit conductor ampacity, and protection devices ratings are based upon applicable sections of the NEC (2002) including:

- Lighting Loads per Article 220: Lighting
- Receptacle Loads per Article 220.13: Non-dwelling Units
- Continuous Loads per Article 230: Service
- Motor Loads per Article 220:14 and 430: Motors
- Air Condition Load per Article 440.6: Refrigerant Motor Compressor
- Heat Loads per Article 200.15: Fixed Electric Space Heating
- Non-Coincident Loads per Article 220.21: Non-coincidental Loads
- Heat Trace per Article 427: Fixed Electric Heating Equipment for Pipelines and Vessels

Load Factors

The following table summarizes load factors applied for various equipment in accordance with appropriate sections of the NEC (2002), while determining demand and design load analysis:

Table 5-4. Building Power Supply Load Factors

Item	Panel and Service Load Analysis	Comment
Heater Loads*	100 percent full load amperage (FLA)	Branch circuit sized to 125 percent of FLA
Motor Loads	Sum of motor load (FLA) + 25 percent of largest motor (FLA)	Branch circuit sized to 125 percent of FLA
Receptacles	180 VA /outlet	Non-Continuous Load
Lighting	2 watts/sq.-ft or total connected (FLA), whichever is larger	Continuous Load
Cooling Loads*	100 percent FLA	Branch circuit sized to 125 percent of FLA
Demand Factors	**Demand Factor Percent**	
First 10 kVA	Non-Dwelling Receptacles	100 percent
Remainder over 10kVA	Non-Dwelling Receptacles	50 percent
Non-continuous Load		100 percent
Continuous Loads		125 percent

* Note: The largest of the non-coincidental heat and cooling loads are used for service sizing.

Table 5-5. Input Data Typical for Cell 1 and Cell 2

Description	Ratings	Comments
Pump 219(Y)-LH-P-202	1/2 HP @ 480V, 3-phase	Coincidental load
Pump 219(Y)-LH-P-203	7.5 HP @ 480V, 3-phase	Coincidental load
Pump 219(Y)-LH-P-204	1/2 HP @ 480V, 3-phase	Coincidental load
Pump 219(Y)-LH-P-205	1/3 HP @ 480V, 3-phase	Coincidental load
Pump 219(Y)-LH-P-207	3 HP @ 480V, 3-phase	Coincidental load
Pump 219(Y)1-LH-P-302	3 HP @ 480V, 3-phase	Coincidental load
Heater 219(Y)-LH-UH-001	3.3 kW @ 480V, 3-phase	Non-coincidental and continuous load*
Heater 219(Y)1-LH-UH-002	3.3 kW @ 480V, 3-phase	Non-coincidental and continuous load*
Air Condition 219(Y)-LH-AC-001	2.04 kVA @ 208V, 1-phase	Non-coincidental load
Air Condition 219(Y)1-LH-AC-002	.96 kVA @ 208V, 1-phase	Non-coincidental load
Control Panel 219(Y)-LH-CP-001	1.5 kVA @ 120V, L-N	Continuous load
Bldg. 219(Y) Lighting	71 kVA @ 120V, L-N	Continuous load
Bldg. 219(Y)1 Lighting	29 kVA @ 120V, L-N	Continuous load
Heat Trace 219(Y)201-LH-HT-001	77 kW @ 120V, L-N	Continuous load
Heat Trace 219(Y)201-LH-HT-002	77 kW @ 120V, L-N	Continuous load
Heat Trace 219(Y)1-LH-HT-003	77 kW @ 120V, L-N	Continuous load
Bldg. 219(Y) Receptacles	720 kVA @ 120V, L-N	180VA/ outlet
Bldg. 219(Y)1 Receptacles	360 kVA @ 120V, L-N	180VA/ outlet

Note: (Y) = A,E
Cell 1 (A), Cell 2 (E)
Heater Load is greater than AC load.

Table 5-6. Building Power Supply Results/Conclusions

Description	Ratings
Bldg. 219(X) connected load @ 219(X)-LH-MCC-001	23 kVA connected – 26 kVA design for each crest pad building.
Bldg. 219(X) main service breaker size @ 219(X)-LH-MCC-001	100 amps
Bldg. 219(X) main service feeder to 219(x)-LH-MCC-001	3#1 TW, 1#1 TW (N)
Bldg. 219(X) service transformer	75 kVA, 480V, 3-phase, 4-wire
Bldg. 219(X)1 transfer bldg. feeder breaker size	50 amps
Bldg. 219(X)1 transfer bldg. feeder size	3#4 TW, 1#8 G
219(x)-LH-MCC-001 short circuit available	2,484 amps symmetrical
219(x)1-LH-SW-002 short circuit available	1,632 amps symmetrical
219(x)-LH-LP-001 short circuit available	1,177 amps symmetrical
219(x)1-LH-LP-002 short circuit available	1,068 amps symmetrical
219(X) –LH-LP-001 lighting panel rating	60 amps
219(X)1-LH-LP-002 lighting panel rating	60 amps
219(X)-LH-T-001 lighting panel transformer rating	15 kVA
219(X)1-LH-T-002 lighting panel transformer rating	15 kVA
219(X)-LH-P-203 LCRS high flow pump motor feeder size	3#12 TW, 1#12 G
219(X)-LH-P-202 LCRS low flow pump motor feeder size	3#12 TW, 1#12 G
219(X)-LH-P-204 LDS pump motor feeder size	3#12 TW, 1#12 G
219(X)-LH-P-205 sump pump motor feeder size	3#12 TW, 1#12 G
219(X)1-LH-P-302 transfer pump motor feeder size	3#12 TW, 1#12 G
219(X)-LH-P-207 combined sump pump motor feeder size	3#12 TW, 1#12 G
219(X)-LH-UH-001 unit heater feeder size	3#12 TW, 1#12 G
219(X)1-LH-UH-002 unit heater feeder size	3#12 TW, 1#12 G
219(X)-LH-AC-001 air condition feeder size	3#10 TW, 1#10 G
219(X)1-LH-AC-002 air condition feeder size	3#12 TW, 1#12 G
219(X)-LH-MD-001 motor damper feeder size	2#12 TW, 1#12 G
219(X)1-LH-MD-002 motor damper feeder size	2#12 TW, 1#12 G
219(Y)201-LH-HT-001 leachate storage tank heat trace feeder size	2#10 TW, 1#10 G
219(Y)201-LH-HT-002 leachate storage tank heat trace feeder size	2#10 TW, 1#10 G
219(Y)1-LH-HT-003 truck loading station heat trace feeder size	2#10 TW, 1#10 G
219(X)-LH-CP-001 main control panel feeder size	2#10 TW, 1#10 G

Note: (X) = A,E

Recommendations

Building Power Supply

- Provide separate power distribution equipment (pad mount utility transformer, secondary service, and power distribution gear) for Cell 1 and Cell 2 in order to maximize redundancy.

- Install service rated motor control center inside each crest pad building for providing service entrance, branch, and sub-feeder distribution capability, and complete motor control for various process control systems.
- Power lighting, receptacle, and facility loads from 3-phase, 4-wire lighting panel installed in each building.
- Power instrumentation from surge protected distribution center mounted inside facility control panel.
- Ground Electrode System
- Provide and install ground electrode system for service and each separately derived system that incorporates both ground ring, ground rod, and concrete encased building rebar.
- Provide ground bus inside Process Instrumentation and Control Systems (PICS) control panels and bond to common ground electrode system.
- Bond non-current carrying metallic structure to ground electrode system that has the potential of becoming energized by attached electrical devices such as metallic conduit systems, enclosures, storage tank structures, building metal framing and siding, and above grade metallic process equipment.

5.12.4.2 Crest Pad Building Lighting

Building lighting systems were based upon I.E.S Zonal Cavity method in order to maintain an average 25-foot-candle level for process interior of each building.

Note: Interior lighting levels are based upon *IES Lighting Handbook Indoor Industrial Areas Recommended Illuminance Levels* for interior activities inside work spaces where visual tasks of medium to large contrast are to be performed on occasional basis.

Note: Exterior entrance lighting levels are based upon *IES Lighting Handbook Outdoor Site/Area Recommended Illuminance Levels* for building exterior entrances frequently visited locations.

Open Items
None

Assumptions
The following assumptions were made when analyzing building lighting.

Reflectance for unfinished rooms:

Ceilings	50 percent reflectance
Walls	50 percent reflectance
Floors	20 percent reflectance

Maintenance factor (light loss factor), interior lighting:

Incandescent lighting	.80
Fluorescent lighting	.61
HPS lighting	.70

Maintenance factor (light loss factor), exterior lighting:

HPS lighting	.70

Inputs

Crest pad buildings are unfinished industrial buildings with interior dimensions of:

Room name: Cell 1 crest pad building	Ceiling height: 11 feet
Fixture type: fluorescent two-lamp	Mount height: 9 feet

Room size: width 16 feet and length 21 feet	Area: 336 square feet

Recommendations

- Provide fluorescent low temperature starting wrap-around industrial fixtures for interior lighting of buildings
- Use two lamps in six fixtures for 25-foot candles minimum
- Install low pressure sodium fixture at front entrance on north exterior wall

5.12.4.3 Leachate Transfer Building Lighting

Building lighting system was based upon I.E.S Zonal Cavity method in order to maintain an average 25-foot-candle level for process interior of each building.

Note: Interior lighting levels are based upon *IES Lighting Handbook Indoor Industrial Areas Recommended Illuminance Levels* for interior activities inside work spaces where visual tasks of medium to large contrast are to be performed on occasional basis.

Note: Exterior entrance lighting levels are based upon *IES Lighting Handbook Outdoor Site/Area Recommended Illuminance Levels* for building exterior entrances frequently visited locations.

Open Items

None

Assumptions

The following assumptions were made when analyzing building lighting.

Reflectance for unfinished rooms:

Ceilings	50 percent reflectance
Walls	50 percent reflectance
Floors	20 percent reflectance

Maintenance factor (light loss factor), interior lighting:

Incandescent lighting	.80
Fluorescent lighting	.61
HPS lighting	.70

Maintenance factor (light loss factor), exterior lighting

HPS lighting	.70

Inputs

Crest pad buildings are unfinished industrial buildings with interior dimensions of:

Room name: leachate transfer building	Ceiling height: eight feet
Fixture type: fluorescent two-lamp	Mount height: eight feet
Room size: width 10 feet and length 10 feet	Area: 100 square feet

Recommendations

- Provide fluorescent low-temperature starting wrap-around industrial fixtures for interior lighting of buildings
- Use two lamps in two fixtures for 25-foot candles minimum

- Install low pressure sodium fixture at front entrance on north exterior wall and low pressure sodium on south exterior wall

5.12.4.4 Uninterrupted Power Supply (UPS) Sizing

Uninterruptible power is provided and sized to provide 25 minutes minimum of continuous backup power to the PICS programmable logic controller (PLC), operator interface unit (OIU), and local area network communication equipment.

In the event of a power failure, UPS will maintain communication with remote monitoring sites (future) and insure safe shutdown of power sensitive PICS equipment.

Open Items

None

Assumptions

None

Table 5-7. Input Data Typical for Cell 1 and Cell 2 Control Panel Loads

Description	Ratings	Comments
PLC Power Supply	180 VA	Continuous load
OIU Power Supply	60 VA	Continuous load
Ethernet Switch Power Supply	44 VA	Continuous load
Total *1.25	**355 VA**	

Recommendations

Table 5-8. Fortress Runtimes for Typical Applications in Minutes

Load (VA)	50	100	200	300	400	500	600	750	900	1050	1250	1425	1800	2250
0520-1050U	200	125	63	42	31	24	19	14	11	9.5	-	-	-	-
0520-0750U	132	75	38	26	19	14	11	8.5	-	-	-	-	-	-

Provide 1050 VA 120 Volt- 120 Volt UPS to achieve the 25 minutes minimum of continuous backup, power in the event of a power failure. Additional capacity will compensate for battery cycling deprivation.

5.13 Civil Grading

5.13.1 Waste Volume, Cut/Fill and Stockpile Requirement Calculations

The IDF is designed to provide the waste volume requirements identified by CH2M HILL. Those requirements consist of an ultimate landfill capacity for 1,177,110 cubic yards of waste and a Phase I capacity of 213,515 cubic yards of waste.

The IDF is also designed to balance the cut and fill volumes of the project. The ultimate landfill layout on the project site provides this balance. The volume balance includes excavated material, which will be used for the construction of the closure cap. Since the closure cap will be selected and designed in the future, assumptions for the cap layout and construction were made.

With a phased construction approach planned for IDF and the fact that the material balance includes backfill to construct a closure cap for the ultimate landfill, a substantial volume of material will be stored in stockpiles at the completion of construction of Phase I landfill. The Phase I landfill design volumes for subgrade cut, admix liner, drain gravel, and operations layer material were calculated using a 3-D

AutoCAD model of the landfill. These volumes were used to identify the stockpile requirements to store material once Phase I construction is complete.

Potential stockpile locations are identified on the project site plan. Calculations of these volumes are included in Appendix C.12.a. Calculations in Appendix C.12.a also present confirmation of the available waste volume and cut/fill balance.

5.13.2 Phase I Access Road and Ramp Cross Section Design

Two cross sections using granular material for base and top course were designed for the Phase I landfill access roads and the access ramp into the landfill. The design reflects the estimated wheel loads and vehicles to use the facility daily. Calculations presenting the development of these cross sections are included in Appendix C.12.b.

6.0 FACILITY DESIGN AND CONSTRUCTION

6.1 Facility Layout

6.1.1 Location

The IDF will be located approximately 1,400 feet east of Baltimore Avenue and directly north of 1st Street in the 200 East Area of the Hanford Site. Phase I of the IDF landfill will measure approximately 800 feet by 1,500 feet, with its north-south axis being the shorter dimension. Leachate handling facilities will be located immediately north of the Phase I cells. The excavated depth to subgrade (not including sump depressions) will range from approximately 44 to 51 feet. Excavation will be deepest at the landfill's north end, near the sumps and along the centerline of each cell. It will be shallowest at the southwest and southeast corners of Cells 1 and 2, respectively. Stockpile locations for excavated materials will be situated east and southeast of the Phase I landfill excavation. At the completion of Phase I construction, exposed surfaces of the stockpiles and disturbed areas will be covered with a layer of topsoil, then seeded and mulched. A borrow area of soil to supplement admix preparation is located south of the Phase I excavation location.

6.1.2 Access Roads and Ramps

For access to Phase I of the IDF, waste hauler and operations vehicles will follow an access road and travel north from 1st Street. All roads and ramps at the Phase I IDF site will be constructed with crushed surfacing material for the base and top courses. The access road from 1st Street will be aligned with the landfill's west berm access road. The road will also follow the alignment of the west access berm road for the future IDF cells.

The access road will lead north, approximately 1,000 feet from 1st Street to where it widens into an intersection. At this location, a turn to the east will lead down a 5 percent grade, 800-foot-long access ramp into the Phase I landfill. The access ramp slope was selected to allow use by both waste haul trucks and the melter transporter. The grade of the access road from 1st Street was also limited to a maximum of five percent for this same reason. The access ramp into the landfill and the access road from 1st Street to the intersection area will be both 30 feet wide.

At that base of the ramp into the landfill, there will be adequate room for waste haul vehicles to turn and move the waste into the cells. The liner system will be installed to extend approximately 50 feet south beyond the estimated toe of slope of Phase I waste placement. This extension will allow waste haul vehicles to be staged or unloaded over a lined area.

At the access road intersection, continuing north will lead up a short ramp and onto the berm access road. The berm access road will be 20 feet wide on the east and west sides of the landfill. The road will widen to 30 feet at the northwest and northeast corners of the landfill and along the landfill's north side. The wider road in these areas will allow operations vehicles to traverse around road corners and the crest pad buildings.

The access road will continue from the northwest corner of the berm access road to the Cell 1 and Cell 2 leachate storage tank facilities. A cul-de-sac area will be provided just east of the Cell 2 leachate facilities to provide a turnaround area for operations vehicles and leachate tanker trucks. A road will also be provided to allow operation vehicles to travel south between the leachate facilities and onto the berm access road at the centerline of IDF landfill.

Future projects are being planned to upgrade the 1st Street pavement and construct an operation building north of the IDF landfill. It is anticipated that these facilities will connect to access roads designed for the Phase I landfill.

Related to permanent access roads and their use, the construction contractor will be required by the project general requirements to submit a plan, which details their use during construction. This plan will address locations and limits of stripping/grubbing, construction haul roads, stockpile/borrow areas and other construction staging areas.

6.1.3 Survey Grids

Survey grids for this project use the Washington State Plane coordinate system (South Zone–feet, NAD83 Lambert Projection). Contours are based on 200 Area topographic mapping database, provided by Hanford HGIS Department and dated 1991. A 1-foot contour interval was used on the design drawings.

As part of the Phase I landfill design, construction control points were developed for landfill and sump subgrades as well as for the anchor trenches, stormwater facilities, and the finished grades for all roads and ramps. North and east coordinates and elevations for these points are included in a survey control table on Drawing H-2-830829, Sheet 2 of 2. The control points and lines between them will provide a location grid that will allow construction of the subgrade, liner system, operations layer, and the finished grades for the IDF.

6.2 Landfill Geometry

6.2.1 Waste Volumes and Types

6.2.1.1 Volume

Two key design criteria were provided by CH2M HILL concerning waste volumes:

- Phase I of the IDF should be designed to receive a waste volume of 213,515 cubic yards, which is equal to163,250 cubic meters. CH2M HILL identified the waste volume for placement in all phases of IDF (ultimate landfill size) as 1,177,110 cubic yards, or 900,000 cubic meters.
- Both the Phase I landfill and the ultimate landfill volumes should be sized for an air space, which includes 1.5 cubic yards of clean fill for every cubic yard of waste.
- Using these criteria, Phase I was designed to provide air space for placement of 533,620 cubic yards of waste and clean fill.

6.2.1.2 Waste Types (Note: The disposal of MLLW other than ILAW, DBVS waste, and IDF generated waste is not permitted at this time by this permit.)

The IDF will receive waste types including ILAW, DBVS Waste, and LLW. These wastes include both contact and remote-handled wastes. As identified in the project kickoff meetings by CH2M HILL, the waste volumes (in cubic yards) are estimated to include the following:

Waste Type	Phase I	All Phases
ILAW	50,025	753,350
MLLW	57.550	146,485
LLW	105,940	277,275
Total	**213,515**	**1,177,110**

These volumes are based on waste forecast information provided by Fluor Hanford, Inc. (FH). The waste volume forecasts are updated by Hanford Site contractors on a regular basis. The volumes above represent an average between the FH 2002 Forecast and the FH 1999 (with EIS) Forecast. Short descriptions of the waste types are given below:

Immobilized Low-Activity Waste (ILAW)–The ILAW packages are stainless steel cylinders that have been filled with vitrified low-activity waste (physically similar to glass), sealed, and cooled. The source of these waste cylinders is the Waste Treatment and Immobilization Plant. The packages are 7.5 feet in height and 4 feet in diameter, and could weigh up to 22,050 pounds each.

Contact-Handled Mixed Low-Level Waste (CH MLLW)–This waste has a dose rate equal to or less than 200 mrem/h and contains radioactivity not classified as high-level waste, spent nuclear fuel or transuranic (TRU) waste (TRU is defined as concentrations of transuranic radionuclides greater than or equal to 100nCi/g of the waste matrix). The waste is also defined as dangerous (hazardous) waste in WAC 173-303.

Remote-Handled MLLW – This waste has a dose rate greater than 200 mrem/h and contains radioactivity not classified as high-level waste, spent nuclear fuel, or TRU waste. The waste is also defined as dangerous (hazardous) waste in WAC 173-303.

Low-Level Waste Category I (LLW I)–This waste contains radioactivity not classified as high-level waste, spent nuclear fuel, or TRU waste. The waste also meets the radionuclide limits for category I waste, defined in the *Hanford Site Solid Waste Acceptance Criteria* (RH, 1998). This waste may be comprised of either contact- or remote-handled waste considered low-activity waste with very low concentrations of long-lived radionuclides. This waste is not a dangerous (hazardous) waste as defined in WAC 173-303.

Low-Level Waste Category III (LLW III)–This waste contains radioactivity not classified as high-level waste, spent nuclear fuel, or TRU waste. The waste also exceeds the radionuclide limits for category I waste and meets the category III limits, defined in the *Hanford Site Solid Waste Acceptance Criteria* (FH, 1998). This waste may be comprised of either contact- or remote-handled waste considered moderate- to high-activity waste with low to moderate concentrations of long-lived radionuclides, in stabilized form that minimizes subsidence for a period of 1,000 years. This waste is not a dangerous (hazardous) waste as defined in WAC 173-303.

Remote-Handled LLW – This waste has a dose rate greater than 200 mrem/h and contains radioactivity not classified as high-level waste, spent nuclear fuel, or TRU waste. This waste is not a dangerous (hazardous) waste as defined in WAC 173-303.

6.2.2 Landfill Phases and Dimensions

The IDF will be a single, expandable RCRA Subtitle C disposal facility that provides ultimate capacity for 1,177,110 cubic yards (900,000 cubic meters) of waste. The facility is currently anticipated to be constructed in four phases. Phase I will have two cells. Only Phase I is being permitted at this time. Each cell has a floor width of approximately 543 feet and a lined floor length of 360 feet. The total floor width of the IDF will be 1,085 feet. Side slopes of the landfill will be 3:1 (horizontal: vertical). At the south end of the Phase I cells, there will be a stormwater berm/ditch system with an infiltration area. The south side of IDF will be unlined for Phase I.

IDF will be expanded by relocation of the landfill's unlined south slope from earlier phases and installation of liner system and operations layer. When expanded to its final configuration, the floor of IDF will be 1,385 feet long, measured along its north-south axis.

6.2.2.1 Depth and Length

The landfill depth for all phases of the IDF is set to accommodate four layers of ILAW waste packages, placed on end, and each layer will be covered with 3.3 feet of clean soil. In some cases, the waste packages received for placement in the mixed and low-level waste side of IDF will have heights that vary from the ILAW package dimensions. In these cases, waste heights will vary from the four layers of ILAW waste described. The total depth, measured from the top of the operations layer to the top of the cover layer over the fourth waste layer, will be 43.4 feet. This is sized for the 7.5-foot tall ILAW package dimension. However, other waste package types can be accommodated. The waste/clean fill depth (43.2 feet) will be uniform over the entire landfill floor, due to the operations layer and the top of the landfill both sloping up 1 percent from north to south. The operations layer will be flat in the east-west direction.

6.2.3 Materials Balance

The IDF was designed to achieve near soil balance. This will minimize excess soil stockpile at the end of the life of the IDF facility and minimize the cost of hauling offsite borrow material for construction. It is important to note that the soil balance was calculated for completing IDF through all its phases and the balance included soil required for construction of the final closure cap. The closure cap design was not part of the critical systems design, completed for this project.

Having a soil balance at the completion of all phases means that at the end of Phase I, a substantial amount (approximately 991,000 cubic yards) of material will be stockpiled onsite. The project design identified potential stockpile sites that were adequate in size for the material to be stockpiled. A portion of the stockpiled material will be used as clean fill during the waste placement in the Phase I cells. However, the stockpile will be replenished during the construction of cells for each subsequent IDF phase.

A description of the resulting soil cut and fill volumes can be found in Appendix C.12.a of this Design Report.

6.2.4 Erosion Control Measures

Permanent erosion control measures (for both wind and water caused erosion) will be provided for areas disturbed by Phase I construction.

Areas that are disturbed by the construction that are outside of the Phase I excavation will be stabilized with a 6-inch-thick layer of topsoil that will be seeded with grass. The south stormwater berm/ditch, the east and west infiltration areas, and the soil stockpiles will also be stabilized with topsoil and grass.

Geotextile and quarry spalls will be placed around each end of the culverts and the stormwater pipe to provide erosion protection.

Stormwater runoff will be conveyed along the south side of the access ramp and the south side of the flat area at the bottom of the access ramp, and will be discharged to the southwest corner of the excavation infiltration area. Road surfacing will reduce the erosion potential on the ramp and flat area. To prevent erosion of the south side slope adjacent to the ramp and flat area, a strip of erosion control matting will be installed on the south side slope, immediately adjacent to the ramp and flat area. Geotextile and quarry spalls will be placed in the southwest corner of the excavation pond in order to minimize the potential of erosion due to the stormwater that will be discharged from the south edge of the flat area to the top of the infiltration area.

Erosion control matting will also be placed on the shine berm to minimize the potential for wind erosion. The erosion control matting will be a plastic matting with an estimated service life at least equal to the 10-year period that the Phase I cells are expected to operate.

To reduce wind erosion, all of the side slopes of the Phase I excavation will be stabilized with a spray-on application of a soil stabilization material. Additional applications of the soil stabilization material may have to be done annually on the areas that remain exposed.

The contractor will also be required to prepare and implement a dust control plan for the construction.

6.3 Lining System Materials

6.3.1 Liner Selection Basis

WAC 173-303-665(2)(a)(i) requires submittal of an engineering report with the permit application under WAC 173-303-806(4) stating the basis for selecting the liner(s). The report must be certified by a licensed professional engineer. The intent of Section 6.3 of the Design Report is to satisfy this requirement of the WAC 173-303, Dangerous Waste Regulations.

Specific requirements to addressed as the basis for liner selection include:

- The liner must be constructed of materials that have appropriate chemical properties and sufficient strength and thickness to prevent failure due to pressure gradients (including static head and external hydrogeologic forces), physical contact with the waste or leachate to which they are exposed, climatic conditions, the stress of installation, and the stress of daily operation.
- The liner must be placed on a foundation or base that is capable of providing support to the liner and is able to resist pressure gradients above and below the liner to prevent failure of the liner due to settlement, compression, or uplift.
- The liner must be installed to cover all surrounding earth likely to be in contact with waste or leachate.
- The lining system must include a LCRS immediately above the liner that is designed, constructed, maintained, and operated to collect and remove leachate from the landfill. Design and operating conditions will ensure that the leachate depth over the liner does not exceed 1 foot. The LCRS shall be:
- Constructed of materials that are chemically resistant to the waste managed in the landfill and the leachate expected to be generated, and of sufficient strength and thickness to prevent failure under the pressures exerted by overlying wastes, waste cover materials, and any equipment used at the landfill.
- Designed and operated to function without clogging through the scheduled closure of the landfill.
- Engineering analyses were presented in Section 5 that address the above requirements for basis of lining selection. Of particular note is Section 5.6 that addresses lining system/leachate compatibility for all components of the lining system. Compatibility of the lining system components with the chemical and radiological constituents of the expected leachate is a critical aspect of the liner selection basis.
- Based on results of the engineering analyses presented in Section 5, the following liner sections are proposed for the IDF bottom (floor) and side slope lining systems. Section 6.3.2 provides a detailed discussion of the liner materials for the barrier components of the lining system, and Section 6.3.3 provides a detailed discussion of the liner materials for the drainage and protection components of the lining system.

Drawing H-2-830838 (Detail 1) shows the bottom liner section, consisting of the following components, from top to bottom:

- A 3-foot-thick operations layer
- A separation geotextile (polypropylene)
- A 1-foot-thick leachate collection drain gravel layer
- A minimum 12 oz/square yard cushion geotextile (polypropylene)
- A 60-mil textured primary HDPE geomembrane
- An internally-reinforced GCL
- A CDN drainage layer for the LDS
- A 60-mil textured secondary HDPE geomembrane
- A 3-foot-thick low-permeability compacted admix (soil-bentonite) liner

Drawing H-2-830838 (Detail 2) shows the side slope liner section, consisting of the following components, from top to bottom:

- A 3-foot-thick operations layer
- A CDN drainage layer for the LCRS
- A 60-mil textured primary HDPE geomembrane
- A CDN drainage layer for the LDS
- A 60-mil textured secondary HDPE geomembrane
- A 3-ft-thick low-permeability admix liner

6.3.2 Liner Materials – Barrier Components

6.3.2.1 Geomembranes

WAC 173-303-665(2)(h)(i) requires that the IDF lining system have both a primary and secondary geomembrane. The geomembrane for the IDF will serve as leachate barrier and as a flow surface routing leachate to the LCRS sump (for the primary geomembrane) or LDS sump (for the secondary geomembrane).

HDPE has been selected as the geomembrane liner material because it is generally acknowledged to have the highest chemical resistance of commercially-available liner materials, has been widely used at similar facilities, and has a high level of acceptance by regulatory agencies. Details of HDPE geomembrane compatibility with expected leachate is discussed in Section 5.6

A nominal thickness of 60-mil has been selected for the HDPE geomembrane. A nominal thickness of 60-mil results in a minimal allowable thickness of 54-mil, as indicated in the technical specifications. Thus, 60-mil nominal thickness is the minimum required to achieve the 50-mil minimum thickness specified by Ecology guidance. Textured (roughened) geomembrane will be used to maximize shear strength along adjacent interfaces and to reduce the potential for sliding of the liner system. Analyses of the various stresses that the geomembrane is designed to withstand under construction and operational loads are presented in Section 5.5. Required material properties as a result of these analyses are included in the technical specifications.

Details of required HDPE geomembrane properties are provided in the technical specifications (see Section 02661).

6.3.2.2 GCL

A GCL will only be included in the primary bottom lining system. For the bottom lining system, both the primary and secondary liners will be a composite (geomembrane over admix liner or GCL) system. The addition of a GCL in the primary lining system will provide an extra measure of protection, exceeding the requirements of WAC 173-303-665(2)(h)(i) for a single geomembrane for the primary liner and composite for the secondary only. This will provide an extra measure of protection on the bottom flatter slopes of the IDF, where higher leachate head levels are more likely. A GCL will not be included on the side slope lining system. The 3H:1V side slopes for the IDF will result in little or no leachate head expected on the side slope lining system, thus eliminating the need for a lining system design that exceeds the WAC requirements.

Commercially-available reinforced GCL products consist of bentonite sandwiched between a woven and non-woven geotextile that are then needle-punched together. Other combinations of upper and lower, woven and non-woven geotextiles can also be manufactured and specified.

For the IDF lining system, a needle-punched, reinforced GCL with non-woven geotextiles on both sides was selected. This type GCL product was selected primarily because of the tensile strength requirements required for landfill global stability (Section 5.1.3). The tighter weave non-woven geotextile minimizes the amount of bentonite that migrates to the interface with the geomembrane, thus minimizing the potential to create a slip surface.

Details of required GCL properties are provided in the technical specifications (see Section 02667).

6.3.2.3 Admix Liner

Details of the admix design test program are provided in Sections 4.2.2 and 5.4.1. Placement and testing requirements are described in Section 5.4.2.

The admix liner will have a minimum 3-foot thick compacted soil/bentonite admixture and will be located immediately beneath the secondary HDPE geomembrane, as required by WAC 173-303-665(2)(h)(i)(B). The admix liner typically will consist of base soil mixed with a nominal 12 percent sodium bentonite, by dry weight. Mixing and processing of the base soil/bentonite admixture is required to be performed under carefully controlled conditions, using a pugmill operation.

The base soil for the admix liner will consist of natural soil, derived from the dune sand borrow area to the south of the Phase I cell (as shown on Drawing H-2-830828) or from within Phase I cell excavations. Based on the results of the limited field exploration for near surface base soil samples (discussed in Sections 4.1 and 4.2), base soil from either source will not be excavated below a depth of 5 feet bgs (after stripping) without further evaluation of the material suitability.

Base soils excavated from the dune sand borrow area or site excavation will meet the following requirements:

- The base soil will be free of roots, woody vegetation, frozen material, rubbish, and other deleterious material.
- Rocks greater than 1 inch in dimension will not comprise more than 2 percent by weight of the base soil.
- Base soil will have 20 percent minimum passing a No. 200 U.S. sieve.
- The in-place hydraulic conductivity of the admix liner will be 10-7 centimeters per second or less, consistent with WAC requirements for secondary soil liners. The upper surface of the admix liner will be trimmed to the design grades and tolerances. The surface will be rolled with a smooth steel-drum roller to remove all ridges and irregularities. The result will be a smooth, uniform surface on which to place the overlying geomembrane liner.
- Before production installation of the admix liner, a full-scale test pad of the admix liner will be conducted for both the bottom floor (horizontal) and side slope areas of the IDF. Details of the test pads are provided in the technical specifications (see Section 02666) and the IDF Construction QA Plan. The primary purpose of the test pad(s) will be to verify that the specified soil density, moisture content, and hydraulic conductivity values will be achieved consistently, using proposed compaction equipment and procedures. In-place density will be measured using both the nuclear gauge (ASTM D2922) and rubber balloon (ASTM D2167) or sand cone (ASTM D1556) methods. In-place hydraulic conductivity will be determined from a two-stage borehole permeameter (ASTM D6391). Admix liner hydraulic conductivity will be estimated from thin-wall tube samples (ASTM D1587) obtained from the test fill and tested in the laboratory (ASTM D5084). During construction, field density (e.g., ASTM D2922, D2167, and/or D1556) and moisture content (ASTM D2216) will be measured periodically. Thin-wall tube samples (ASTM D1587) will be taken at regular intervals and will be tested for hydraulic conductivity (ASTM D5084). Additional details of Construction QA testing and acceptance during admix liner test pad and production installation is provided in the IDF Construction QA Plan.

Details of required admix liner properties and placement requirements are provided in the technical specifications (see Section 02666).

6.3.3 Liner Materials–Drainage and Protection Components

6.3.3.1 Geotextiles

Two types and weights of geotextiles will be used in the IDF project. The separation (Type 1) geotextile has a nominal weight of 6 ounce/square yard and was selected based on the ability of the geotextile to retain the soil and to prevent the soil from entering the LCRS drain gravel. Required AOS and permittivity were determined based on filter, fines retention, and clogging potential criteria. The waste disposed in the IDF is expected to contain a minimal amount of organic material, and consequently, biologic clogging is not expected to be a problem.

The cushion (Type 2) geotextile has a nominal weight of 12 ounce/square yard and was selected based on providing the required cushion protection for geomembrane on the landfill bottom (floor). The drain gravel will have the potential to produce localized stress on the geomembrane liner during gravel placement with construction equipment and under the maximum static pressure from landfill contents at full waste height with final cover. A puncture analysis was performed to select a sufficiently thick geotextile to protect the liner. This analysis included the maximum load from landfill contents and final cover, expected construction vehicle ground pressures, and maximum drain gravel particle size listed in the technical specifications.

Both types of geotextiles are specified as non-woven needle-punched and made from polypropylene material. This material was selected because of its higher chemical resistance to the expected leachate (Golder Associates, 1991a).

Details of required geotextile properties are provided in the technical specifications (see Section 02371).

6.3.3.2 CDN

The CDN is a drainage geocomposite consisting of a HDPE geonet core with a layer of non-woven polypropylene geotextile thermally bonded to each side. The CDN selected for the IDF lining system has two drainage related functions. On the side slopes, it will function as the LCRS. A CDN is selected for the LCRS on the side slope to avoid construction stability problems associated with placement of clean granular material on slopes, thereby minimizing the potential for damaging the underlying liner system. Localized placement of drain gravel is required on side slopes (as shown on Drawing H-2-830848, Section C), to provide adequate backfill and bedding for leachate collection riser piping. On the side slope and bottom lining system, the CDN will function as the LDS.

Analyses were performed to evaluate the geotextile puncture requirements for the LCRS CDN on the side slope and the transmissivity requirements for both the LCRS and LDS CDN. These analyses and discussion are presented in Section 5.7.

The analyses for CDN geotextile puncture resistance determined that the specified geotextile is adequate for resistance to puncture from overlying operations layer, under the maximum static pressure from landfill contents.

The analyses for allowable transmissivity with applied reduction factors for intrusion, creep, and chemical and biological clogging determined that a higher flow, thicker (250 mil minimum) CDN is required, due to the reduction of flow under the high normal loads in the final filling configuration.

Details of required CDN properties are provided in the technical specifications (see Section 02373).

6.3.3.3 Drain Gravel

The LCRS for the bottom liner will be located below the operations layer and will provide a flow path for the leachate flowing into the LCRS sump and sump trough. Between the operations layer and the underlying drain gravel, a geotextile layer will function as a filter separation geotextile (as discussed in Section 6.3.3.1). The separation geotextile will prevent migration of fine soil and clogging of the drain gravel. The gravel will be a minimum 1-foot thick layer of washed, rounded to subrounded stone, with a hydraulic conductivity of at least 10-2 cm/sec, as required by WAC 173-303-665(2)(h)(iii)(B). In addition, a slotted HDPE leachate collection piping will be placed within the drain gravel to accelerate leachate transport into the LCRS sump during high precipitation events. Slots on the leachate collection piping are sized to be compatible with the drain gravel gradation and particle sizes. Details of the leachate collection piping design are provided in Section 6.4.1.

Based on review of expected subsurface conditions for the IDF, it is not likely that material meeting drain gravel is available on or near the site. Thus, drain gravel will have to be an imported material. The technical specifications require that drain gravel meet the requirements of WSDOT Standard Specification 9-03.12(4) for gradation. The technical specifications also require a performance specification for a hydraulic conductivity greater or equal to 10-1 cm/sec.

As discussed in Section 5.7.3, the minimum estimated hydraulic conductivity for the drain gravel exceeds the required (by WAC regulations) hydraulic conductivity of 10-2 cm/sec by a factor or 100 to 1,000, and the performance specification hydraulic conductivity of 10-1 cm/sec by a factor of 10 to 100. This allows for uncertainty in the empirical formulas used to predict hydraulic conductivity, and the potential for long-term reduction in hydraulic conductivity in the drain gravel, if fines from waste filling and the operations layer migrate into this layer over time.

Details of required drain gravel material properties are provided in the technical specifications (see Section 02315).

6.3.3.4 Operations Layer

The purpose of the operations layer will be to protect the underlying lining system components from damage by equipment and waste canisters during IDF construction and operation. This layer also will protect the admix liner from freeze/thaw damage and desiccation cracking. This is especially the case on the side slopes, expected to be exposed (prior to waste placement) for longer duration than the bottom (floor) of the IDF cell.

The operations layer material typically will consist of onsite granular soil from the IDF Phase I excavation. The excavated material is expected to be a fine-grained sand to silty sand with traces of gravel. The technical specifications require the material to have a maximum particle size limit of two inches or less, and fines will be limited to maximum 25 percent fines (percent passing the U.S. No. 200 sieve). Based on review of expected subsurface conditions for the IDF excavation, the majority of soil excavated from the IDF Phase I excavation is expected to be suitable for use as operations layer without processing. As discussed in Section 4, additional geotechnical exploration within the IDF Phase I limits are recommended prior to construction to verify these findings.

Details of required operations layer material properties are provided in the technical specifications (see Section 02315).

6.4 Leachate Collection System

The leachate collection system for each cell in Phase I will consist of lateral flow media built into the cell's bottom and side slope liner system, a leachate collection pipe in the center of the cell, a sump at the north end of the cell where all leachate drains, pumps and leachate transfer piping to convey leachate out of the cell, and a network of piping and storage tanks for storing the leachate for later transfer to tanker trucks for offsite disposal. Below the bottom liner and under the LCRS sump area will be an LDS sump, pump, and associated piping. All components for Phase I of the leachate collection system are designed and configured for eventual full development of the IDF through Phase IV.

The type and configuration of the leachate collection system described below has been used successfully at other disposal facilities, and a very similar facility was recently (2002) implemented at the INEEL site near Idaho Falls, Idaho. This ICDF will accept waste with radioactive characteristics and is located in a region with dry weather conditions, similar to Hanford.

6.4.1 Leachate Collection Piping

6.4.1.1 Description

Lateral drainage media (drain gravel in the bottom liner section and CDN in the side slope section of each cell) will convey leachate by gravity to the leachate collection piping and to the LCRS sump area. The leachate collection piping system in both cells will have one 12-inch diameter HDPE slotted pipe running the length of the cell centerline from south to north. This main collector pipe will be sloped at 1 percent and will convey leachate from the south edge of the cell to the LCRS sump at the north end, where the bottom liner will intersect the side slope liner. The main collection pipe will change to solid pipe at the bottom of the side slope, continue up the side slope, and terminate at a cleanout, located just south of the crest pad building. Leachate in the sump will be collected through perforated pipes for the LCRS low flow and high flow pumps, which will be 12-inch and 18-inch HDPE slotted pipe, respectively. The riser pipes will protect the pumps and separate them from the surrounding drain gravel, allow removal and re-insertion of the pumps for maintenance, and provide a high inflow-rate screen for leachate to supply the pumps. A small-diameter pipe (housing a transducer to control the on/off levels for the pumps) will run from the LCRS sump up the side slope to the crest pad building.

The slotted portion of the riser pipes will extend from the toe of the side slope to the end of the LCRS sump area. The transducer pipe will also be slotted but for a shorter distance in the LCRS sump, whereupon it will be solid for the remaining distance to the crest pad building. A solid HDPE pipe (of the same diameter as the slotted portion of the pump riser pipes) will extend from the intersection of the side slope and bottom liner to the top of the shine berm where the pipes enter the crest pad building.

Pipe cleanouts will be provided at both ends of the main collection pipe in the center of each cell. The cleanout at the north end of Phase I, near the crest pad building, will be permanently available throughout the life of the IDF to allow access for cleaning and/or video inspection. The cleanout at the south end of the cell will also be available for cleaning and access, but only during the operation of Phase I. It will be removed and the Phase II collection pipe will be butt-fused to the pipe as the Phase II cell is brought online. Ultimately, a permanent cleanout will be installed at the south end of Phase IV, to allow cleaning and inspection of half of the collection pipe, with the other half being accessed by the permanent cleanout located at the crest pad building on the north side of Phase I.

Access to the riser pipes for cleanout or inspection, in the unlikely event this is needed, will be through the access points used for removal and re-insertion of the pumps within the crest pad building.

6.4.1.2 Design Considerations

The material chosen for piping within the Phase I lined area was HDPE , made of resin meeting the requirements of ASTM D3350 for PE 3408 material, with a cell classification of 345434C or higher. Design calculations were based on this material and pipe type, which is routinely used for leachate collection and disposal facilities and other applications. The pipe material is well suited for use in disposal facilities because of its high strength, high resistance to degradation from leachate constituents, and superior characteristics compared to all other readily available pipe materials. HDPE compatibility with leachate and the presence of radioactivity in the waste overlying the pipe were evaluated and discussed previously in Section 5.6.

The diameter of the riser pipes was chosen to provide ample clearance for the pumps to be inserted and removed on a routine basis, and specifically so that the pumps will have sufficient clearance when traveling through the angle points at the intersection of the bottom liner and side slope, and clearance at the radius transition from the side slope to the crest pad building. The pumps (described in Section 6.4.3) are specifically designed for this type of leachate collection system, where the riser pipes allow insertion of pumps down a side slope and into a sump area.

Lateral drainage media in the bottom liner and side slope liner, and the leachate collection piping system were chosen and configured to meet the regulatory requirement of no more than 12 inches of leachate head buildup over the sump area of the bottom liner as a result of a 25-year, 24-hour storm event.

The slots in the slotted pipe were designed to both be compatible with the granular material in the drain gravel and to allow a high rate of flow from the surrounding lateral drainage layers into the pipe. Slots were sized at 0.128 inches wide, with five rows of slots spaced equidistant around the perimeter of the pipe, and eleven slots per foot of pipe.

The thickness of the pipes expressed as the SDR (standard dimension ratio) was chosen to resist the highest estimated load for the IDF in its final configuration, including final cover and equipment loading (internal pressure was not a factor since the pipe will convey flow by gravity, and under the expected flow rates the pipes will only be partially full). A SDR of 17 was chosen for all piping to handle the maximum estimated load. In addition, a blanket of manufactured drain gravel will be placed around and to the sides of all collection piping and compacted to a firm, unyielding condition consistent with the soil modulus values used in the pipe loading calculations.

All piping will be butt-fused for maximum strength, and all fittings, whether available molded from the manufacturer or fabricated, will have the same or higher pressure rating than the pipe. During construction, piping will be butt-fused by certified technicians, using welding equipment approved by the manufacturer. All solid pipe will be pressure tested, even though the collection piping will see little or no internal pressure during gravity conveyance of leachate.

6.4.2 Leachate Transfer Piping

6.4.2.1 Description

At each cell, the leachate transfer piping will begin with the piping from the pumps in the LCRS and LDS sumps to the crest pad building. From the crest pad building, transfer piping will connect the leachate transfer building, storage tank, and tanker truck load facility. All underground transfer piping outside the Phase I liner limits will be double contained, that is the pressure pipe conveying leachate between various facilities will be contained in an outer pipe. The pressure pipe in the center of the double containment piping will be termed carrier pipe, while the outer pipe will be termed containment pipe. In the event of a leak in the carrier pipe, the containment pipe or leak detection pipes draining the containment pipes will convey the leakage to a combined sump facility for detection, sampling, and transfer. Any accumulation of leachate in the combined sump will be pumped through a transfer pipe to the storage tank. Piping within the crest pad building, transfer building, truck load facility, and combined sump, will not be double contained because the buildings or facilities will provide secondary containment and have sumps present to remove any leachate that accumulates as a result of leaking pipes or appurtenances. Leak detection pipes draining containment pipes and the leak detection pipe from the storage tank will be single pipes because they only will convey leakage and will not function as transfer piping (required to have double containment).

The transfer piping system also will include valves, fittings, flow meters, and other appurtenances necessary for operational functions for systems described in Sections 6.4.3, 6.4.4, and 6.4.5.

6.4.2.2 Design Considerations

All transfer piping outside of buildings will meet the same requirements as the HDPE pipe chosen for the leachate collection piping (described in Section 6.4.1). Single pipe and containment pipe exposed to earth and traffic loading will be SDR 17, while the carrier pipe, that will not be exposed to earth or traffic loading, will be SDR 21, with a pressure rating of 80 psi and a safety factor of 2 for the highest expected operating pressure in the system (SDR 17 piping has a pressure rating of 100 psi). All piping will be butt-fused except for the transfer piping from the LCRS and LDS sump pumps. This pipe will be HDPE, with quick release fittings to allow removal of the pumps from the sumps. Fittings will be pressure rated and re-useable. As the pumps are withdrawn from the sumps and moved up the riser pipes, each joint in the pipe will be unhinged to allow the pipe to be removed in 8-foot sections.

Piping inside buildings will be PVC, schedule 80, with solvent welded fittings. This pipe and classification is rated for higher pressure than required with a factor of safety of 8. PVC was chosen for application inside buildings because of its relative ease of fabrication with the solvent weld joint system.

Flange connections will be used between pumps and piping; valves and other appurtenances and piping; and joints between PVC and HDPE piping. Appurtenances will include air release valves to allow purging of any air trapped in the piping system, magnetic flow meters for measuring flow to the tanker truck load output and to and from the leachate storage tanks, and valves for flow control and diversion of flow between the various facilities. The flow control scheme and control logic for the transfer piping system are described in Section 6.4.5.

6.4.3 Leachate System Pumps

Three submersible leachate pumps will be required for each cell. For convenience and operational versatility, roller-mounted pumps were selected for all leachate removal facilities. The submersible pumps are standard stainless steel well pumps that have been installed within a screened stainless steel cylinder fitted with rollers. The configuration will allow the pumps to be installed from the crest pad building within riser piping that follows the slope of the landfill until the riser piping bends horizontally to terminate within the cell sump at the toe of slope. This type of pump can be lowered into the leachate sump through the riser pipe and removed as needed, using a winch mounted within the crest pad building. Each pump will have its foot valve removed to prevent freezing or retaining of the leachate in the pump discharge piping. Advantages of this type of pump include easy access for maintenance and inspection, no power equipment required to remove/install, and its small size will lend itself to being inserted within a curved riser pipe and evacuating nearly all of the leachate within the cell sump. Each pump will have the capability to pump either to the storage tank or truck loading station.

6.4.3.1 LCRS Pumps

Two of the three submersible pumps will be installed within the LCRS sump area of each cell above the primary liner. These pumps are required to maintain less than 12 inches of hydraulic head above the primary liner, per regulatory requirements. The pumps will be installed in a 6-inch depression within the LCRS, in order to minimize the area of permanent leachate storage at pump shutoff and allow full pump operation through the 12-inch maximum liner head zone over the primary liner. Only in the localized area of the LCRS sump depression will a maximum leachate head of 18 inches cover the primary liner. The leachate head over the primary liner will be maintained at or below 12 inches in the main sump area and throughout the landfill. One low-flow pump is required for typical pumping of leachate; a high-flow pump is necessary in the event that a large storm (24-hour, 25-year storm event) exceeds the capacity of the low-flow pump.

The selection of the low-flow pump was based on the average leachate flow from the landfill, determined in the leachate production analysis (Section 5.8.1). The analysis indicated that the maximum leachate flow, based on monthly data, is approximately 13 gpm. The hydraulics of the low-flow pump was modeled and a pump was selected, based on the hydraulic characteristics of the piping system and the required flow rate, determined in the leachate system hydraulics analysis (Section 5.9.2.1). An EPG Companies, Inc. (EPG) model WSD 3-3 (or equal) with a 0.5-horsepower motor was selected for the LCRS low-flow pump.

The selection of the high-flow pump was based on the 24-hour, 25-year storm event, determined in the leachate production analysis (Section 5.8.1). The analysis indicated that the high-flow pump capacity necessary to remove the leachate per regulatory guidelines is approximately 160 gpm. The hydraulics of the high-flow pump was modeled and a pump was selected, based on the hydraulic characteristics of the piping system and the required flow rate, determined in the leachate system hydraulics analysis (Section 5.9.2.1). An EPG model WSD 30-3 (or equal) with a 7.5-horsepower motor was selected for the LCRS high-flow pump.

6.4.3.2 LDS Pump

The third submersible pump will be installed within each cell in the LDS sump, under the primary liner and above the secondary liner. This pump will detect and recover leachate that has leaked through the primary liner by pumping the leachate to the crest pad building. This pump was sized for low leachate generation flows.

The hydraulics of the LDS pump were modeled and a pump was selected that can produce 4 gpm, based on the hydraulic characteristics of the piping system and the required flow rate, identified in the leachate system hydraulics analysis (Section 5.9.2.1). An EPG model 1.5-3 (or equal) with a 0.5-horsepower motor was selected for the LDS pump.

6.4.3.3 Crest Pad Building Sump Pump

The sump pump within the crest pad building will be a submersible floor sump, activated by float switches within the floor sump. The function of the sump pump is to remove leachate that accumulates in the crest pad building as a result of unexpected spills or pipe leaks. The pump discharges water to the leachate storage tank via the crest pad building discharge piping.

The hydraulics of the sump pump was modeled and a pump was specified, based on the hydraulic characteristics of the piping system and the required flow rate identified in the leachate system hydraulics analysis (Section 5.9.2.1).

6.4.3.4 Leachate Transfer Pump

The leachate storage tank will be drained by using the leachate transfer pump, located in the leachate transfer building. The pump was sized to deliver a capacity of 250 gpm to the truck loading station, where it will discharge into a tanker truck. The typical volume allowed in a tanker truck is 7,000 gallons, corresponding to a loading time of approximately 30 minutes.

The hydraulics of the leachate transfer pump was modeled and a pump was selected, based on the hydraulic characteristics of the piping system and the required flow rate, identified in the leachate system hydraulics analysis (Section 5.9.2.1). A standard horizontal centrifugal pump, Paco model 30707 (or equal) with a 3-horsepower motor was selected for the leachate transfer pump.

6.4.3.5 Combined Sump Pump

The combined sump will be a 76-inch-diameter HDPE manhole with a 42 inch diameter HDPE manhole placed inside. The outer manhole will have a height of approximately 8 feet, and the inner manhole height will be approximately 6 feet. The secondary containment portion of all the buried HDPE pipelines, leachate tank, and leachate transfer building floor sump will drain to the annular space (leak detection chamber) between the two manholes. The leak detection chamber will include instrumentation to detect leachate and alarm accordingly. The sumps installed within the truck loading slab typically will collect precipitation that drains off the slab. The precipitation will be conveyed directly to the inner manhole of the combined sump, where the combined sump pump will be located. The combined sump pump then will pump the precipitation to the leachate storage tank.

The combined sump pump was conservatively sized for a capacity of 250 gpm. This large capacity was chosen based on an off-normal event that assumed the tanker truck was overtopped during leachate transfer activities, resulting in 250 gpm flowing into the inner sump. Another off-normal event considered was the remote possibility that the leachate tank primary liner failed catastrophically. This flow of leachate could eventually inundate the leak detection chamber and overflow into the inner manhole.

The hydraulics of the combined sump pump was modeled and a pump was selected based on the hydraulic characteristics of the piping system and the required flow rate, identified in the leachate system hydraulics analysis (Section 5.9.2.1). A Hydromatic model SB3S (or equal) with a 3-horsepower motor was selected for the combined sump pump.

6.4.4 Leachate Temporary Storage Tank

6.4.4.1 Tank Volume

A leachate temporary storage tank is required for each cell. The working capacity of each tank is 375,000 gallons that include a 1.5 safety factor. This volume is based on the results of the leachate production analysis (Section 5.8.1) and the leachate collection storage analyses (Section 5.9.2.4). The storage tank capacity is dependent on the net volume of leachate accumulation in the tank from flow into and out of the tank. The flow out of the tank via the leachate transfer pump is based on several assumptions, described in Section 5.9.2.4. Actual leachate transfer operations will affect the tank volume safety factor.

6.4.4.2 Tank Design

A bolted, corrugated steel tank, approximately 100 feet in diameter with a side wall height of 8 feet 2 inches, was selected for use as the leachate temporary storage tank. The tank will include a dual containment liner system that will act as the floor of the tank and will be bolted to the top of the tank side wall. The tank will be open-topped with a floating geomembrane cover to keep precipitation, debris, and wildlife from contacting the leachate.

The tank side wall will be bolted to a 1.5-foot thick, 4-foot-deep concrete ringwall to resist hydrostatic pressure of the leachate water. In addition, the top edge of the tank ringwall will include angle bracing, bolted around the tank perimeter to provide rigidity in the side wall to resist wind loads on the exterior of the tank. The maximum operating level of the tank is approximately 6 feet 2 inches; however, the tank is designed for a maximum water level of 8 feet 2 inches.

The inlet piping for the tank will be through the side wall of the tank. The inlets will all be located near the top of the tank, above the maximum leachate water operating level. This is to ensure that a siphon cannot develop in the inlet piping. Check valves will be installed throughout the system; however, if piping between the check valve and the tank leaked into the secondary containment system, there would not be an easy method of stopping the flow if the pipe was below the water surface of the tank.

The outlet pipe for the tank will be through the side wall, near the bottom of the tank. This method was chosen to provide a flooded suction for the leachate transfer pump that will provide added protection against pump damage.

6.4.4.3 Tank Liners

The tank liners will be constructed with an XR-5 geomembrane. XR-5 is a proprietary geomembrane manufactured by Seaman Corporation. XR-5 is the preferred liner of several tank manufacturers due to its higher strength properties and lower thermal expansion coefficient, as compared to HDPE geomembrane. As such, it is more readily constructible in the tank configuration, and it does not expand and contract as much as HDPE, so it’s operating performance over the temperature range at Hanford should be improved. For the exposed condition at the IDF tanks, this is an important consideration. HDPE was considered for use as the tank liner system, but its high coefficient of expansion will not lend itself to the temperature extremes that the liner system will be subjected to and also it is not reinforced like the XR-5. The expansion and contraction of an HDPE liner exposed to the environment could put undue strain at the inlet and outlet connections as well as at the leak detection connection that could result in liner leakage.

Chemical compatibility of leachate with the liner system is also a consideration for liner material selection for the leachate storage tanks. As discussed in Section 5.6.3.1, compatibility testing on HDPE geomembrane was performed with synthetic leachate for the W-025 landfill with no evidence of geomembrane deterioration. With regard to leachate compatibility, XR-5 is comparable to HDPE in terms of compatibility with typical leachate constituents. The geomembrane manufacturer requires immersion testing for conclusive compatibility determination. Testing of this type has not been performed, but the manufacturer is confident that immersion testing results will be acceptable since XR-5 is generally comparable to HDPE. To address the issue of chemical and radiation resistance for XR-5 with anticipated leachate constituents, an immersion test program is included in the technical specifications for the tank liner. Details are provided in Section 13205 of the technical specifications. This immersion testing program requires the construction general contractor to submit tank liner sample coupons to the design engineer for immersion testing, as part of the construction submittal process and certification of the tank liner.

In addition, it should be noted that leachate compatibility is not as critical an issue for the tank system as compared to the landfill liner system. The leachate tank liner system will be subject to continuous monitoring through the tanks' LDS, as is the landfill liner system. The difference is that the tank liners will be subject to routine maintenance and inspection that will be developed around liner warranty, performance observation, and manufacturer's requirements. Operation and maintenance procedures for the tank will be established that require that the tanks be drained, sediment removed, and the liner inspected for holes and seam integrity. Since liner performance guarantees are required in the technical specifications for the tank manufacturer for three years following installation, it is likely that the inspection program would be initially set up around this time frame and gradually be increased over the life cycle of the tank. Replacement of the leachate tank liner system is anticipated periodically throughout the life cycle of the landfill.

The tank lining system is a double-lined system. The primary and secondary tank liners will include a LDS beneath the primary tank liner. The LDS consists of a HDPE drainage net with a geotextile material, laminated to the drainage net that cushions the XR-5 liner. A geotextile material will also be used between the secondary liner and the inside face of the tank shell to create a cushion for the XR-5 against the tank shell and tank shell bolt heads. The bolt heads are also recessed for further liner protection.

6.4.4.4 Tank Leak Containment System

The HDPE drainage net between the primary and secondary liner will allow leachate that leaks through the primary liner to drain to the center of the tank. At the center of the tank under the secondary liner will be a depression in the underlying granular backfill that will form a shallow sump. The leak detection pipe will connect to the secondary liner at this sump location and convey leaking leachate to the leak detection chamber of the combined sump.

The tank inlet and outlet penetrations will be areas susceptible to leaks as a result of penetrations through the primary liner. Additional robust methods for sealing these locations were added over and above the typical manufacturer recommendations in an effort to make sure that these will not be points of leakage.

6.4.5 Pump Controls and System Instrumentation

The process and instrumentation diagrams for Cell 1 and Cell 2 are shown on Drawing H-2-830854, sheets 1 through 4. Detailed information regarding the instrumentation and control system, equipment listing, instrument listing, and loop descriptions can be found in the technical specifications, Section 13401 (Process Instrumentation and Control System).

6.4.5.1 Crest Pad Building

The leachate pumps within the landfill will be automatically controlled, based on leachate level set points within the cell sump. The level transducer that controls the LCRS pumps will be inserted into the sump via a slope riser pipe. The level transducer that controls the LDS pump is integral to the LDS pump. Leachate pumped by the leachate pumps will be monitored by a flow-indicating totalizer within the crest pad building. Controls will be in place to stop automatically the leachate pumps operation if alarm conditions are present for the leachate storage tank high-high level, leak alarm in the crest pad building sump, or a leak alarm in the combined sump.

The crest pad building sump pump will be automatically controlled by float switches within the building floor sump. In addition, a leak detection switch will be installed in the floor sump that will be capable of detecting small quantities of water in the sump before the float switches. This feature will add an extra level of conservatism to make sure unexpected spills are identified and controlled immediately. Controls will be in place to stop automatically the crest pad building sump pump operation if alarm conditions are present for the leachate storage tank high-high level or for a leak alarm in the combined sump.

6.4.5.2 Leachate Transfer Building

The leachate transfer pump will be manually controlled except for automatic shut-off during specific alarm events. Controls will be in place to stop automatically the transfer pump operation if alarm conditions are present for the leachate storage tank low-low level or for leak alarm in the combined sump. Additional instrumentation (associated with the leachate transfer pump) will include a flow meter (measuring rate and total volume) and transmitter on the discharge of the leachate transfer pump. In addition, a local totalizer will be in the leachate transfer building to know exactly how much water is being transferred to the tanker truck. This totalizer will include a reset function to allow the total to be reset to zero, prior to every truck loading event.

6.4.5.3 Leachate Storage Tank

Instrumentation within the leachate storage tank will be contained within two vertical stilling wells that will penetrate through openings in the floating cover. The stilling wells will be small diameter pipe with perforations near the bottom that will allow the leachate within the stilling well to rise and fall with the level of the leachate in the tank. Analog instrumentation within one stilling well will provide a signal to the control system for alarm interlocks and constant monitoring of tank level. The second stilling well will contain discrete instrumentation for high-high and low-low alarm set point trips. The discrete instrumentation will provide conservatism in the off chance that the analog signal malfunctions, allowing the leachate level to reach extreme high or low levels.

6.4.5.4 Combined Sump

The combined sump pump will be automatically controlled by float switches within the inner manhole of the combined sump. Controls will also be in place to stop automatically the combined sump pump operation if alarm conditions are present for the leachate storage tank high-high level. A leak detection switch also will be installed within the leak detection chamber that will be capable of detecting a small quantity of water. The leak detection switch will provide a signal to the control system that automatically will shut down all the cell pumps except the combined sump pump. The pumps will be shut down because any one of the pipelines associated with the pumps could be leaking into the leak detection chamber. Operations will then need to determine which secondary containment pipeline supplied the water that drained into the leak detection chamber.

6.4.6 Process Instrument Control System (PICS)

6.4.6.1 Introduction

This section provides a summary of the PICS design and construction elements of the project, providing introduction and reference to the project layout and key design components for the following IDF facilities:

- IDF leachate collection and handling crest pad facilities (two each)
- IDF leachate storage tank and leachate transfer facilities (two each)
- IDF truck loading facilities (two each)

The PICS design identifies, specifies and integrates PICS components to automatically monitor and control IDF process control equipment and facilities including:

- LCRS
- LDS
- Crest pad and leachate transfer building environmental controls
- Leachate storage tank system
- Leachate transfer and truck loading system
- Combined sump system
- Secondary containment LDS

6.4.6.2 Key Design Components (Elements)

PICS design and construction elements of the project incorporate the following key PICS design components for each IDF facility:

- Instrumentation for continuous analog process monitoring
- Instrumentation for discrete process monitoring
- Instruments and programmed safety interlocks and alarming
- Programmable logic controller (PLC) system
- Operator Interface Unit (OIU)
- Communication Local Area Network (LAN)
- PICS application software
- Main and local control panels
- Uninterruptible power supply

6.4.6.3 Open Items

The IDF Phase I Critical Systems design documents do not identify the following items:

- Identification of communication LAN from IDF control panels to central supervisory control and data acquisition (SCADA)
- Extension of communication LAN from IDF control panels to central SCADA

These items are scheduled to be addressed during the IDF Phase I Non-Critical design of the project. As such, the following assumptions were made in order to complete IDF Phase I Critical Systems design:

- Assume 10/100 megabits per second (MBPS) Ethernet communication LAN from IDF control panels to central SCADA
- Assume fiber-optic multi-mode extension of communication LAN from IDF control panels to central SCADA

6.4.6.4 PICS Architectures

The PICS design identifies various architectures, designed to enable operators to locally and remotely interface and change program settings by the use of an Ethernet LAN. This document does not identify components and architectures to be provided and configured under the IDF Phase I Non-Critical design in order for personnel remote monitor and control processes over the LAN.

6.4.6.5 PICS Instrumentation Architecture

The PICS design identifies instrumentation architecture that consists of single variable level (submersible pressure), flow, and temperature elements and transmitters that provide continuous process data to PICS PLC and OIU architectures. Process signals from each instrument are monitored for the purpose of controlling, displaying, recording, and alarming all process data. PICS instrumentation will be wired directly into PLC input modules (i.e., Allen-Bradley 1746 I/O modules).

6.4.6.6 Instrumentation

The PICS design identifies all set-point adjustments as being programmed into the PLC via the OIU architecture. Field instruments incorporate the following signal types:

- Analog signals, current type: 4-20 mA dc signals conforming to ISA S50.1.1
- Transmitters type: 2-wire and 4-wire
- Transmitter load resistance capacity: Class L
- Fully isolated transmitters and receivers
- Discrete signals, voltage type: 24 VDC

6.4.6.7 Analog Instrumentation

The PICS design identifies flow analog instrumentation, consisting of electromagnetic flow elements and integral transmitters that will enable operators to monitor pump discharge flow for the following processes:

- Landfill LCRS pump discharge flow
- Landfill LDS pump discharge flow
- Leachate transfer truck loading station discharge flow

The PICS design identifies level analog instrumentation, consisting of submersible pressure transmitters that will enable operators to monitor liquid levels for the following:

- Landfill LCRS
- Landfill LDS
- Leachate storage tank system

The PICS design identifies temperature analog instrumentation, consisting of an element and transmitter that will enable operators to monitor temperature levels inside the following:

- Crest pad buildings
- Leachate transfer buildings

6.4.6.8 Discrete Instrumentation

The PICS design identifies level instrumentation, consisting of radio frequency (RF) admittance probes and transmitters that will enable operators to monitor discrete liquid levels inside the leachate storage tank system. The PICS design identifies level discrete instrumentation, consisting of magnetic float switches that will enable operators to monitor discrete liquid levels inside the following:

- Crest pad building sump
- Combine sump
- Combine sump interstitial

The PICS design identifies operator instrumentation, consisting of switches, indicating lights, and control relays that will enable operators to monitor the following discrete status:

- Crest pad building and control power status
- Landfill LCRS pumps ON/OFF, AUTO and FAIL status
- Landfill LDS pumps (on/off, auto, and fail) status
- Combined process sump pump (on/off, auto, and fail) status
- Leachate transfer pump (on/off, auto, and fail) status

6.4.6.9 PICS Programmable Logic Controller (PLC) Architecture

The PICS design identifies PLC architecture designed around Allen Bradley Ethernet small logic control technologies. PLC architecture consists of the following:

- PLC processor
- PLC input/output (I/O) modules
- PLC ancillary power supplies, chassis and cabling
- PLC application and development software and hardware
- The PLC processor is the microprocessor-based device that uses programmable ladder logic for implementing process monitoring and control, emulating the functions of conventional panel-mounted equipment such as relays, timers, counters, current switches, calculation modules, Proportional, Integral and Derivative controllers, stepping switches, and drum programmers.

- PLC(s) are programmed to interface with instrumentation and process motor control equipment. PICS PLC(s) are programmed to automatically operate (start/stop) all process control equipment as well as process flow totals, equipment runtime, operation alarms, equipment, and building status.
- Instrument architecture (analog and discrete control devices) interface with PLC via PLC I/O modules, installed in a common chassis with the PLC power supply.
- The type of I/O modules utilized include analog (4-20 mA) input, 24VDC discrete input, and 120VAC/ 24VDC discrete output.
- The PICS design identifies PLC application software that provides functions unique to the project and not provided by PLC system software alone, such as programmable controller ladder logic, math operations on input process variables (scaling, alarming, totalizing, comparisons).
- The PICS design identifies PLC standard system software packages that enable personnel to communicate and program PLC processor and configure I/O modules. PLC development and application software reside on the programming laptop from which the application is downloaded into the PLC processor.

The PICS design identifies communication protocols establishing data exchange between PLC, programming laptop, OIU architecture, and future remote SCADA as follows:

- Allen Bradley RS-232, RS-4585, and DF1
- Ethernet

6.4.6.10 PICS Operator Interface (OIU) Architecture

The PICS design identifies OIU architecture that allows operators to visually monitor process system data and interface with the facility's programmable logic controllers. OIU enables operators to view alarms and change process set points.

PICS OIU architecture is designed around Allen Bradley PanelView, communicating with PLC architecture over a communication local area network. OIU architecture includes:

- OIU assembly
- Local area network copper cabling
- OIU application and standard system software
- The PICS design identifies OIU application software that provides functions unique to the project and not provided by system software alone. These include, but are not limited to, programmable controller ladder logic, databases, reports, control strategies, graphical display screens, and operation scripts.
- The PICS design identifies OIU standard system software packages that enable personnel to communicate and program OIU. OIU application and standard system software reside on the programming laptop from which the application is downloaded into the OIU processor.

6.4.6.11 PICS Communication LAN Architecture

The PICS design identifies communication between PLC processors, OIU, programming laptop, and future IDF SCADA over a local area network consisting of a local 10/100 MBPS Ethernet switch, local PLC, OIU LAN drivers, and a cable system. The PLC processor and OIU are addressable over the LAN, allowing each device to share data and control points between each other and future devices.

6.4.6.12 Back Up Power

The PICS design identifies UPS mounted inside each main control panel. UPS(s) was sized to enable PLC and OIU networks to maintain monitoring of process control systems during a power failure as well as provide for an orderly shutdown. UPS does NOT power process control equipment such as solenoids, instruments, motorized valves, pumps, and motors.

6.4.6.13 Control Panels

The PICS design identifies the main control panel, mounted inside each crest pad building housing PLC processor and associated I/O modules, ancillary power supplies, termination devices, UPS, and control circuit protection devices. OIU and process flow and level indicators are mounted on front doors of control panels.

The PICS design identifies local control panels, integrating discrete level instrumentation, control relays, intrinsic safety relays, and providing interlock signals between PLC architecture and MCC pump controls.

6.5 Stormwater Management

The proposed stormwater system to be constructed just south of the south end of the Phase I excavation will intercept stormwater runoff from the area to the south for the 24-hour, 25-year storm event so that it will not flow into the Phase I excavation and will discharge the intercepted stormwater into the ground via infiltration. This system will consist of the south stormwater berm/ditch, two culverts, and the east and west infiltration areas. The berm will be two feet high above the existing ground surface. The minimum combined depth of the berm and ditch will be two feet. The ditch will be V-shaped with 3:1 side slopes. The culverts will be 18-inch-diameter, corrugated polyethylene pipe with smooth interior. Geotextile and quarry spalls will be placed around each end of the culverts to provide erosion protection. The east and west infiltration areas will have bottom lengths of 220 and 225 feet, respectively. Each of the infiltration areas will have a bottom elevation of 719 feet and a bottom width of 15 feet. In order to allow access for future maintenance into each of these infiltration areas, their north and south ends will be sloped at 15 percent and surfaced with quarry spalls placed on a geotextile.

The proposed stormwater system to be constructed at the south toe of slope within the Phase I excavation will intercept stormwater runoff from the unlined portions of the excavation for the 24-hour, 25-year storm event so that it will not flow into the active cells and will discharge the intercepted stormwater into the ground via infiltration. This system will consist of the excavation stormwater berm/ditch, a stormwater pipe, one catch basin, and the excavation infiltration area. There also will be a flow path along the south side of the access ramp that will continue along the south side of the flat area at the base of the access ramp and into the southwest corner of the excavation infiltration area. The south stormwater berm/ditch will slope to drain to the east. The combined depth of the berm and ditch will be two feet. The stormwater berm will be 2 feet high at its west end, and the corresponding depth of the ditch will be zero. The berm will gradually reduce in height as the depth of the ditch increases. The berm will end when the ditch depth reaches 2 feet. The ditch will be V-shaped with 3:1 side slope on the south and 2:1 side slope on the north. The stormwater pipe will be 18-inch-diameter corrugated polyethylene pipe with smooth interior. Geotextile and quarry spalls will be placed around each exposed end of the stormwater pipe to provide erosion protection. The catch basin will be used to lower the elevation of the stormwater pipe so that there will be adequate cover over the pipe for protection against wheel loads. The infiltration area will have a bottom elevation of 678 feet, a bottom width of 15 feet, and a bottom length of 50 feet. In order to allow access for future maintenance into this infiltration area, the west end will be sloped at 15 percent and surfaced with quarry spalls placed on a geotextile.

If the water builds up in the east or west infiltration area, it will eventually flow out of the north end of the infiltration area. The water would flow overland, north along the toe of the fill for the berm access road, and continue generally northward.

If the water builds up in the excavation infiltration area so that it extends into the ditch, then the operator will have to bring in a portable pump and pump the water into the east infiltration area.

Maintenance for each of the infiltration areas, the ditches, and the ends of each of the culverts and stormwater pipes will be primarily to remove accumulated sediment and debris.

6.6 Building Systems

6.6.1 Crest Pad Buildings

The crest pad building is designed as a pre-engineered, rigid frame metal building on a slab-on grade foundation. The building slab is separated into two portions. The lower portion of the slab is where the piping associated with the leachate pipe will be contained, and the higher slab is where the electrical and control equipment will be located. The slab where the leachate piping will be located is lowered to create a containment area for the leachate. Construction joints within this area have waterstops to ensure that leachate cannot egress through the construction joints. Additionally, a sump has been placed to drain the containment area, if required. The entire floor and sump area also is to be coated to provide even greater resistance to the leachate.

6.6.2 Leachate Transfer Buildings

The leachate transfer building is designed as a pre-engineered, self framing metal building on a slab-on-grade foundation. The metal building is supported on an 8-inch curb that travels continuously around the exterior of the building. The curb is continuous, even through the door threshold, to provide a containment area for the leachate in case of spillage. In order to maintain conformance with building code requirements, a landing is used to eliminate the curb tripping hazard at the door threshold. Construction joints within this area have waterstops to ensure that leachate cannot egress through the construction joints. Additionally, a sump has been placed to drain the containment area, if required. The entire floor and sump area also is to be coated to provide even greater resistance to the leachate.

6.6.3 Truck Loading Station

The truck loading station is designed to receive trucks to load with leachate. The station is essentially a slab-on-grade. The station is designed to contain minor spillage of leachate by sloping the floor slab towards the center and using rounded curbs at the slab entrance and exits. Two sumps will be placed in the center of the station to drain the station as required. The entire floor and sump area also is to be coated to provide even greater resistance to the leachate.

6.7 Electrical Service and Lighting

6.7.1 Introduction

This section provides a summary of the electrical design and construction elements of the project, providing introduction and reference to the project layout and key design components for the following IDF facilities:

- IDF leachate collection and handling crest pad facilities (two each)
- IDF leachate storage tank and leachate transfer facilities (two each)
- IDF truck loading facilities (two each)

The electrical design identifies, specifies, and integrates power distribution systems that incorporate transformers, breaker panels, motor control, safety switches, conductors, and lighting for the safe, reliable, and maintainable operation of IDF process and facility equipment including:

- Process equipment (leachate collection and removal pump motors, leak detection pump motors, transfer pump motors, and instrumentation)
- Building facility equipment (lighting, power outlets, heating units, cooling fans, and building sump pumps)
- Personnel and equipment safety systems (standby egress lighting, process alarm lighting, surge protection, and process piping heat trace)
- Electrical design and installation shall be in accordance with NFPA 70 (NEC, 2002)

6.7.2 Key Design Components (Elements)

Key electrical design components (elements) for each IDF facility include:

- Electrical secondary service and monitoring
- Electrical secondary service and feeder protective device coordination
- Electrical secondary service ground electrode system
- Electrical service, equipment, and associated metal structures grounding
- Electrical low voltage motor control
- Facility maintenance outlets (standard, ground fault circuit interrupter [GFCI], weatherproof)
- Facility interior, exterior, and egress safety lighting
- Facility environmental control (heating and cooling)
- Facility hazardous classification
- Process equipment heat trace, ambient monitoring, and power indication
- Facility electrical system surge and phase protection
- Materials and methods of electrical construction (i.e., conduit, wire, control and safety device, and enclosure selection)

6.7.3 Open Items

IDF Phase I Critical System design documents do not identify the following primary and secondary electrical service items:

- Exact location of primary 13.8 kV, 3-phase tie-in
- Exact value of available primary short circuit current at primary tie-in location
- Exact length of primary extension
- Exact location, size, and impedance of utility step-down 13.8 kV – 480/277V, 3-phase, 4-wire pad mounted transformer(s)

6.7.4 Assumptions to Open Items

These items are scheduled to be addressed during the IDF Phase I Non-Critical design. As such, the following assumptions were made in order to complete the Phase I design:

- Assume electrical service gear inside each Cell 1 and Cell 2 crest pad building are powered by separate pad mounted utility transformers.
- Assume pad mounted utility transformers are rated 75 kVA and are installed within 100 feet of respective Cell 1 and Cell 2 crest pad buildings.
- Assume each pad mounted utility transformer is radial fed from a common 13.8 kV primary feeder.
- Assume each Cell 1 and Cell 2 leachate transfer building is powered from electrical service gear, located inside respective crest pad buildings.
- Assume utility short-circuit contribution to be 100 MVA at 13.8 kV, three-phase.

6.7.5 Crest Pad Building Electrical Secondary Service and Metering

Electrical design identified 480 volt, 3-phase, 4-wire secondary service cables eventually powering a service-rated MCC mounted inside each crest pad building.

Type	Designation	Configuration
Cell 1 Service rated MCC	219A-LH-MCC-001	480V, 3-ϕ, 3-wire, 4-wire
Cell 2 Service rated MCC	219E-LH-MCC-001	480V, 3-ϕ, 3-wire, 4-wire

The service-rated MCC will operate as a main service gear, power distribution center, and motor control assembly. A MCC distributes 480 volt, 3-phase power to the following 3-phase equipment:

- LCRS three-phase pump motors
- LDS three-phase pump motor
- Combine sump three-phase pump motor
- Crest pad building and leachate transfer building unit heaters
- Crest pad and leachate transfer lighting panel transformers

Secondary 3-phase power is monitored by phase loss and phase reversal protection relays mounted inside MCC(s). In the event of a phase loss or phase reversal condition, the protection relay will shunt the MCC main service breaker. With main service breaker shunted (open), a UPS mounted inside each PICS main control panel will continue the operation of voltage sensitive PICS equipment (i.e., PLC, OIU, local area network communication), allowing for future remote alarming (future SCADA) and the safe shutdown of sensitive equipment.

Incoming power is also monitored through the use of analog-style voltage and current meters. Operators will be able to observe operating status of incoming power by manually selecting analog-style voltage and current meters to Phase A, Phase B, and Phase C.

MCC associated gear (frame, bussing, and feeder protective devices) were sized to adequately and safely handle the calculated design and demand operating loads, and to safely withstand calculated short circuit interrupting currents.

6.7.6 Utilization Voltages

The electrical design identified utilization voltages for the following equipment and systems:

Equipment or System	Voltage, Phase
Lighting	120 V, 1-ϕ
Heat trace	120 V, 1-ϕ
Convenience outlets	120 V, 1-ϕ
Instrumentation control circuits	24 V DC
Motor control	120 V, 1-ϕ
Air conditioner	208 V, 1-ϕ
Motors, less than 1/3 hp	120 V, 1-ϕ
Motors, 1/3 hp and larger	480 V, 3-ϕ
Unit heaters	480 V, 3-ϕ
Instrument power	120V, 1-ϕ

6.7.7 Leachate Transfer Building Electrical Service

The electrical design identified three phase motor loads inside leachate transfer buildings as being powered from MCC, located inside each crest pad building. Power will be routed from MCC to service-rated disconnect, wire-way, enclosed breaker, and mini-power center (panel/transformer assembly), located inside each leachate transfer building.

Type	Designation	Configuration
Cell 1 service-rated disconnect	219A1-LH-SW-002	480V, 3-ϕ, 3-wire, 4-wire
Cell 2 service-rated disconnect	219E1-LH-SW-002	480V, 3-ϕ, 3-wire, 4-wire

6.7.8 Crest Pad and Leachate Transfer Building Lighting Panelboards

The electrical design identified lighting panel boards installed in each IDF facility to provide 120/208V 3-ϕ, 4-wire power to non-three-phase motor loads. Lighting panelboards will be fed from 480V- 120/208V 3-ϕ, 4-wire step-down transformers. Lighting panelboards inside crest pad buildings will be mounted along with step-down transformers inside MCC. Lighting panelboards (mini-power centers, along with integral step-down transformers) inside leachate transfer buildings will be wall mounted.

Type	Designation	Configuration
Cell 1 crest pad building lighting panel	219A- LH-LP-001	120/208V, 3-ϕ, 4-wire
Cell 1 crest pad building lighting panel	219E- LH-LP-001	120/208V, 3-ϕ, 4-wire
Cell 1 leachate transfer building lighting panel	219A1- LH-LP-002	120/208V, 3-ϕ, 4-wire
Cell 2 leachate transfer building lighting panel	219E1- LH-LP-002	120/208V, 3-ϕ, 4-wire

Lighting distribution panelboards will provide 120 volt power to all single-phase equipment including:

- Building lighting
- Emergency lighting
- Receptacles
- Main control panel
- Instrumentation will be powered from surge-protected circuit breakers inside each crest pad building main control panel.
- Lighting distribution panelboards will provide 120/208 volt, single and three-phase power to equipment including the building air conditioner, and heat tracing for process piping.

6.7.9 Feeder and Branch Circuits

The electrical design identified feeder and branch circuit breakers and conductor's size, based upon connected and operating loads. Style of feeder and branch circuit breakers will be thermal-magnetic.

6.7.10 Raceways

480V power circuits–Standard rigid galvanized steel (RGS) in exposed locations, PVC conduit systems will be buried, RGS will be coated when conduits transition from below grade to above grade areas

120V power circuits–Standard RGS in exposed locations, PVC conduit systems buried, RGS coated when conduits transition from below grade to above grade areas.

6.7.11 Raceway Sizing, Selection, and Installation Guidelines

The electrical design identified conduit wire fill and size, based upon THW (thermoplastic, vinyl insulated building wire; flame retardant, moisture and heat resistant, 75°C, dry and wet locations) insulated conductors for wiring 600 volts and below. Minimum raceway sizes will be as follows in the designated locations:

Minimum Raceway Size:	Location:
3/4-inch	Exposed on walls and ceiling
3/4-inch	Concealed in frame construction and finished ceilings
1-inch	Underground for circuits below 600 volts, including instrumentation
3-inch	Fiber optic

The electrical design identified underground raceways assemblies as concrete ductbank constructed.

6.7.12 Wire and Cable

The electrical design identified stranded copper conductors that will be used for all wiring, except for lighting and receptacle circuits where solid copper will be used. Minimum conductor size of No. 12 will be used for power and lighting branch circuits. Conductors installed in all branch circuits rated 100 amps or less was sized based upon NEC table for 60°C TW conductors.

- No. 12 AWG copper for lighting and receptacle branch circuits
- No. 10 AWG, minimum, wiring for all outdoor power circuits
- No. 14 AWG, minimum, for all instrumentation 24VDC discrete control and instrument power
- No. 16 AWG, minimum, shielded for all instrumentation 24VDC analog control

6.7.13 Convenience Receptacles

The electrical design identified weatherproof 20 amp duplex receptacles for indoor service, weatherproof GFCI 20 amp duplex receptacles for outdoor service.

6.7.14 Motor Control

The electrical design identified full voltage non-reversing (FVNR) combination motor starter assemblies, to be mounted inside MCC for each constant speed motor. FVNR combination motor starter assemblies will consist of thermal-magnetic, trip-molded case circuit breakers; full voltage combination starters; control power transformers; indicating lights; and control switches. All combination motor starters will be operated in AUTO mode by PICS.

6.7.15 Overload Protection

The electrical design identified each motor as being provided with thermal overload protection in all ungrounded phases. Each controller will be provided with overload heaters and controller-mounted relays with external manual reset.

6.7.16 Grounding

The electrical design identified the grounding electrode system for each IDF facility, integrating ground ring rods, and connection to building rebar. The electrical design identified electrical service neutral, and the neutrals of derived sources, electrical equipment, and PICS control panels that will be bonded to grounding electrode systems.

6.7.17 Equipment Grounding

The electrical design identified noncurrent-carrying parts of all electrical equipment, devices, panelboards, and metallic raceways that will be bonded to grounding system.

The electrical design identified noncurrent-carrying parts of all mechanical equipment, to which electrical components will be attached and may potentially become energized, that also will be bonded to the grounding system, including building metal structures and leachate storage tank.

All conduits that will be provided have an equipment grounding conductor.

6.7.18 Lighting

The electrical design identified lighting fixtures that will be installed at each IDF facility to maintain an average 25-foot candle inside each building, and 5-foot candles at entrance doorways.

Note: Interior lighting levels are based upon *IES Lighting Handbook Indoor Industrial Areas Recommended Illuminance Levels* for interior activities inside work spaces where visual tasks of medium to large contrast are to be performed on occasional basis.

Note: Exterior entrance lighting levels are based upon *IES Lighting Handbook Outdoor Site/Area Recommended Illuminance Levels* for building exterior entrances frequently visited locations.

6.7.19 Emergency Lighting System

The electrical design identified emergency illumination (battery-pack wall-mounted units or luminaries powered by integral battery-powered ballasts) that will be provided in all IDF facilities.

6.7.20 Circuiting and Switching

The electrical design identified interior process area lighting, switched to provide adequate lighting. Exterior building lighting will be controlled by photocells.

6.7.21 Heat Trace

The electrical design identified electrical heat trace for above grade process piping freeze protection. Heat trace cable will be the self-limiting type with the overall system controlled by an ambient control thermostat. Heat trace design incorporates circuit power indication.

6.7.22 Hazardous Classification

The electrical design identified the interior of the combined sump as Class 1, Division 2 group, C hazardous. The electrical design for the combined sump will incorporate materials and intrinsic safety devices compatible for the installation of electrical equipment in Class 1, Division 2, Group C hazardous locations.

6.8 Construction QA Requirements

The Construction QA Plan describes the QA activities for constructing the Phase I IDF. QA activities will be required during construction to ensure the following:

- Firm and stable foundation system for liners
- Stability of dikes or embankments
- Low permeability soil liners that inhibit contaminant migration
- Geosynthetic layers that function as either a hydraulic barrier or a drainage system, depending on intended function
- LCRS and LDS that remove leachate and control head on the lining systems
- The Construction QA Plan has been prepared to describe the activities that will be performed during construction of the lining system, leachate collection system, and operation layer of Cell 1 and Cell 2. The Construction QA Plan satisfies the regulatory requirements and guidance established in 40 CFR 264.19, the EPA technical guidance document, *Quality Assurance and Quality Control for Waste Containment Facilities* (EPA 1993), and WAC 173-303-335.

The specific physical components that the WAC requires the Construction QA Plan to address include:

- Foundations
- Dikes
- Low-permeability soil liners
- Geomembranes
- LCRS and LDS
- Final cover systems
- The WAC requires the Construction QA Plan to include the following:
- Identification of applicable units and how they will be constructed
- Identification of key personnel
- Description of inspection and sampling activities

The Construction QA Plan is intended to be implemented by an independent, qualified Construction QA certifying engineer, familiar with EPA's technical guidance document, *Quality Assurance and Quality Control for Waste Containment Facilities*, as well as the Construction QA Plan. The Construction QA certifying engineer will be supported by other Construction QA representatives, as necessary, to implement the requirements in the Construction QA Plan and document the work.

The Construction QA Plan establishes general administrative and documentation procedures that will be applicable for selected activities of construction. The Construction QA Plan addresses only those activities associated with the soils, geosynthetics, and related liner and leachate collection system piping components for the Phase I IDF landfill. Other aspects of construction, such as transmission piping, utilities, concrete, and storage tanks will require QA testing and oversight. These requirements are not mentioned in the Construction QA Plan, but they will be included in future construction inspection documents, accompanying the bid-ready drawings and specifications.

6.9 Interface with Non-Critical Systems

Critical systems for the Phase I IDF include three primary design components:

- Liner systems
- LCRS
- LDS

In addition, the Phase I IDF detailed design also involves completing all design work required for an operable landfill.

Non-critical systems for the Phase I IDF include the following components:

- Entrance facilities, including entrance area, scales, and staging areas
- Administration and control facilities
- Waste delivery access road improvements to the IDF site from the WTP
- Waste treatment and staging areas
- Gates and fences
- Utilities including fire protection, process water, electrical power, or instrumentation cables

The IDF Phase I Critical Systems design has been prepared to interface with these non-critical systems that are necessary for operational readiness for the IDF. The following discussion details interface elements of the current design with these non-critical systems.

There is the potential for the DOE to procure an independent contractor to provide operation and maintenance services for the IDF. These services could also include the detailed design and construction of a portion or all of the non-critical systems for the facility. If this should be the case, careful consideration will be given to these interface elements in the development of performance criteria that will be included as part of any contract package for these services.

6.9.1 Entrance Facilities

Entrance facilities will control the flow of waste into the IDF. These facilities will provide for waste delivery, inspection, check-in, and final authorization for disposal into the IDF. Typically, the location for the entrance facilities is adjacent to the in-bound access road, prior to reaching the disposal area. Other factors that can influence their location include access to existing utilities and other operational facilities such as waste treatment, soil stockpiles, or staging areas. Based on the current configuration planned for the IDF, there will be room for entrance facilities to the south of the Phase I disposal area, along the western access road. Typically, these facilities require connection to such utilities as fire protection, power, and process water. Utility interfaces are discussed later in this section.

Design criteria and detailed design elements for the IDF entrance facilities have not been developed. The overall mission for the facility has expanded from handling just the ILAW packages to other wastes including Waste fdrom the DBVS and LLW materials. This may require the entrance facilities to have expanded capabilities for waste load staging, inspection, verification, and scaling, prior to release for disposal into the IDF. This could impact the location selected for the entrance facilities, since complete development of the IDF to its full capacity will leave little room to the south of the southern perimeter berm for the facility (refer to Drawing H-2-830827). This could require the entrance facilities to be located along 1st Street, if a permanent initial location is desired. Otherwise, a more mobile entrance area could be developed and relocated along with phased development of the facility.

6.9.2 Administration and Control Facilities

Administration and control facilities will provide the control center for LCRS operations and monitoring, as well as monitoring for LDS and other emergency systems (fire, power interruption, and HVAC controls). The administration building will service facility operations, including waste tracking and record keeping systems as well as provide for staff needs including office facilities, lunch room, lockers, and storage. Other functions that may take place in this facility area include equipment maintenance, an equipment and staff decontamination area, and equipment storage.

The proposed location of the administration building is shown on Drawing H-2-830827, to the north of the leachate storage and handling area (north of the IDF Phase I development area). This location provides quick access to the leachate control buildings and storage tanks, as well as good interface with existing utilities that will come from existing facilities to the east and west of the IDF. Power and control/communications cables will connect the administration building to the leachate control buildings (crest pad buildings, leachate pump buildings, and leachate storage tanks), as well as to other leachate control structures including the combine manholes and truck loading stations for Cell 1 and Cell 2. Additional utilities will service the administration building including fire protection, process water, potable water, communications, and power. Calculations for power supply to future facilities are provided in the *Integrated Disposal Facility (IDF) Detailed Design: Site Utilities Design Report*, (RPP-18515, Revision 1).

Design criteria and detailed design need to be established for the administration building. The expanded mission of the IDF may influence existing criteria that have already been determined for this facility as provided in conceptual design documents for the original ILAW W-520 Project. Modular units may be considered for this facility.

6.9.3 Waste Delivery Access Road

The waste delivery from the WTP will access the IDF from 1st Street and enter the IDF along the western perimeter of the landfill. Waste delivery from other areas will access the facility from one of three gates (810, 812, or 815) to the 200 East Area, as discussed previously in Section 1 (refer to Figure 1-3). The Phase I access road is aligned horizontally with the proposed western berm of the complete IDF landfill. The vertical alignment of Phase I access road coordinates with the existing topography of the site between 1st Street and the Phase I landfill area, to minimize cut and fill requirements for this road construction. As such, the Phase I vertical alignment does not follow the vertical alignment of the future western perimeter berm of the landfill and will need to be modified in future expansion phases. All-weather pavement for the Phase I road as well as for 1st Street will need to be completed as part of non-critical design. It is anticipated that pavement will be asphalt concrete pavement.

Access for waste haul vehicles will require upgrades to 1st Street to be designed as part of non-critical systems. Design criteria for this upgrade will be based on the anticipated haul vehicles and wheel loads for the various wastes to brought to the facility. From the Phase I Critical Systems design, the melter transport vehicle represents the most restrictive design condition for the road in terms of axle load and radius/grade limitations. However, there are also substantial wheel loads and larger volumes for ILAW package transport vehicles and other MLW and LLW wastes.

It should be noted that there will be a significant grade differential between the southern end of the IDF perimeter berm and the existing 1st Street road grade. The western berm climbs at a uniform 1 percent grade to the south. As such, it will have an elevation of approximately 741 feet at the southern perimeter road. The existing grade of 1st Street at the western perimeter of the IDF is approximately 734 feet, and so 1st Street will need to be raised to make this transition and keep vertical road grades at a maximum of 5 percent to accommodate the melter transport vehicles.

6.9.4 Waste Treatment and Staging

Currently, no waste treatment facilities have been planned for the IDF. Consideration of waste treatment may be necessary as part of the IDF's expanded mission to take mixed wastes and low-level wastes from both onsite and offsite sources, depending on the waste acceptance criteria that are established for the facility. Waste staging areas are associated with waste receipt and inspection activities, as mentioned previously. Staging and storage areas may also be needed for waste treatment as well. Design of non-critical facilities may need to consider development of these waste treatment and staging areas.

During Phase I operation, there is adequate area south of the Phase I landfill area for treatment and staging. Some staging also can occur within the landfill itself that offers the advantage of occurring over lined areas with leachate collection systems in place. However, as wastes are placed and cell lifts become full, staging areas may be limited until new lifts are ready for waste placement. Regulatory requirements for waste staging and storage may also impact location and operational requirements for these areas.

6.9.5 Gates and Fences

The IDF is being developed within the 200 East area of the Hanford Site, that has controlled access with a perimeter fence and access control gates (refer to Figure 1-1). As such, it is currently not anticipated that additional fencing and gates will be required for access control to the facility. However, operationally it may be determined that a perimeter fence and additional gates may be warranted for isolation of the IDF from adjacent existing facilities and, if so, these need to be designed during implementation of non-critical design components. Site standards for fences and gates would be followed for this design.

6.9.6 Site Utilities

As mentioned previously, site utilities are included in non-critical systems design. Site utilities will interface with existing utilities that service facilities in the 200 East area. As such, substantial coordination will be required to locate these utilities, determine the best interface tie-in location, and bring these to the IDF site. Key utilities that are needed for the IDF include:

- Power to buildings and operating systems, as well as to area lighting
- Communication between administration building and operating systems, as well as from the IDF to other area networks
- Fire protection water
- Process (non-potable) water for operations and facility construction
- Potable water

Power requirements for leachate control and monitoring systems have been designed during this Phase I Critical Systems design. Access vaults to power and control systems are provided outside of both crest pad buildings (shown on Drawing H-2-830858). It is anticipated that the administration building will connect at these access vaults and will provide power for system operation and an Ethernet connection for controls. Transformer design for bringing power from the site to the administration building (and to leachate control facilities) will be performed during non-critical design, as will design of the Ethernet connection and administration control systems. Calculations for power supply to future facilities are provided in the Integrated Disposal Facility (IDF) Detailed Design: Site Utilities Design Report, (RPP-18515, Revision 1).

Utility corridors need to be developed to bring these utilities to facility areas. It is recommended that these corridors be developed outside of landfill embankment areas and access roads, to allow for uninterrupted waste placement and facility operation, for future landfill phase development, for protection of liner system anchor trenches, and for protection of utilities from heavy wheel loads. In addition, the future final cover of the IDF is located over the perimeter embankments and catches existing ground at the outside toe of the embankment.

7.0 OPERATING PROVISIONS

7.1 Waste Placement

7.1.1 Introduction

To establish a baseline for design, construction, and operation of the IDF, a plan for filling the landfill cells was developed. This plan was developed mainly to ensure that landfill configuration and size as proposed for the IDF Phase I Critical Systems were adequate for safe placement of the ILAW, waste from the DBVS, and LLW, both remote handle and contact handle, that will be placed in the Phase I development. The proposed configuration and size of the IDF Phase I landfill are identified in Section 6 of this report.

The drawings that show the waste placement plan are included in Appendix D.1. This plan was based on the concept of completely filling the first lift in both cells before beginning filling of the succeeding lift. The plan represents one approach to filling the cells within the proposed configuration. It is possible that other approaches, such as proceeding to a subsequent lift before completely filling the previous lift, also are workable, but development of the plan did not consider alternative methodologies to fill the cells. Development of the plan is also based on conformance with the operational procedures identified for the Base Alternative in Appendix K of the *Conceptual Design Report for Immobilized Low-Activity Waste Disposal Facility, Project W-520* (RPP-7908, Revision 0), (CDR).

This waste placement plan is intended to meet the applicable functional criteria identified in the *System Specification for Immobilized Low-Activity Waste Disposal System* (RPP-7303, Revision 3). "As low as reasonably achievable" principals (keeping radiation exposures to as low as reasonably achievable) are embodied in the waste placement plan that was developed. Because of the area available for waste disposal in each cell, the plan provides the capability to relocate filling operations to another area within each cell, if an event occurs that causes operations to halt temporarily, placing waste packages at the current working position. This will allow waste package placement to continue while the situation that caused the operations to cease is resolved.

7.1.2 Phase I Configuration

Under the proposed configuration for the IDF Phase I, there will be two cells, identical in size. One cell will be for disposal of ILAW and waste from the DBVS; the other cell will be for disposal of LLW. This waste placement plan proposes disposal of ILAW and DBVS waste Cell 1 and disposal of LLW in Cell 2. Provisions are included for disposal of both remote handle and contact handle waste in each cell.

The configuration of the IDF Phase I development as it will exist at the completion of construction, prior to beginning filling operations, is shown in Appendix D, Drawing D.1-1. The initial operations layer, placed as part of Phase I construction, will cover the entire bottom liner and LCRS. The top of the operations layer will be level in the east-west direction and slope down at 1 percent from the south to the north. The operations layer will extend up the west, north, and east side slopes. Access to the facility will be from 1st Street along the western site boundary. An access ramp from the southwest corner of Phase I will lead down the south excavation slope from the west side to the bottom of Phase I and connect to the top of the operations layer near the south east corner of Cell 2.

7.1.3 Waste Receipts

As stated in Section 6.2, the IDF will receive ILAW and Waste from the DBVS. The volumes stated in Section 6.2 are based on waste forecast information provided by FH. The waste volume forecasts are updated by Hanford Site contractors on a regular basis. Actual waste receipt rates at the IDF will likely vary from the estimated amounts. Depending on the receipt rate of ILAW and DBVS waste versus the receipt rate of LLW, each lift of Cell 1 and Cell 2 may fill at different rates. The waste placement plan can accommodate differing rates of waste receipt because filling in subsequent lifts in each cell could be begun at different times as soon as the prior lift was complete. The cell that has the higher waste receipt rate will fill faster than the other cell and will determine the time when subsequent phases of development will need to begin so that additional disposal capacity is available when it is needed.

7.1.4 General Waste Placement Procedures

The discussion of waste placement in this plan is based on placement of the uniform height ILAW packages using remote handle. Some adjustments may need to be made for the variable height LLW containers and for contact handle waste, but in general, the waste placement concept will be the same for all types of waste.

The configuration of IDF Phase I provides a height sufficient for four layers of ILAW packages, each covered with one meter of operations layer soil to provide shielding to operations personnel during waste package placement. LLW, which will be in variable height containers, can be accommodated within each of these four lifts. However, in some cases the LLW containers may exceed the lift height and, therefore, will not be completely covered by placement of the operations layer soil. In these cases, it may be necessary to mound cover soil around the individual projecting LLW containers to provide sufficient cover for shielding until they are completely covered by subsequent lifts.

Each lift will contain multiple ILAW package arrays that span the width of each cell. The packages will be placed in close-packet hexagonal arrays, with placement tolerance averaging 10 centimeters (4 inches) center to center. As the packages are placed in the cell, the array will proceed along the width of the cell. The earth cover will proceed shortly behind the advancing package array, the distance behind the front package limited by the repose slope of the fill soil. The array width (number of columns of packages) will be limited according to the amount of radiation generated by the total number of packages that can be exposed. The CDR indicates that even at some distance from the advancing array, the dose rate becomes a concern when the array approaches more than ten or twelve packages in width.

Off-loading of the ILAW packages and other waste containers will take place in the cell. A standard, manually operated, rubber-tired crane will off-load packages, move temporary shielding walls (concrete blocks), and place the interstitial fill between the packages using a hopper. In the CDR, the total weight of the shielding bell, package grapple, load cell, hooks, and other rigging is estimated at 20 metric tons (23 tons). The crane, as identified in the CDR, will be a Grove GMK 5100, a 108 metric ton (120 ton), rough terrain rubber-tired crane with a telescoping boom and a maximum reach of 15 meters (50 feet), with a load of 20 metric tons (23 tons). Pad loads could exceed 55 metric tons (60 tons) when placing an ILAW package at the maximum allowable reach. Dunnage required under each outrigger pad of the crane for lifts of this size has been determined to be 60 square feet, when operating directly on the base operations layer at its point of minimum thickness over the bottom liner system. Dunnage requirements for subsequent lifts would be less, but have not been determined. Refer to Section 5.5.5 and Appendix C.5.e of this Design Report for dunnage requirement calculations.

7.1.5 Moveable Shielding Wall

With off-loading operations in close proximity to the advancing package array, a moveable shielding wall will be set up between the crane and transporter operations and the placed packages (CDR, Drawing No. ES-W520-BASE). With the 15-meter (50-foot) maximum reach of the crane, the shield walls will have to be moved after every five rows of packages are placed. For a ten-package-wide-array, the wall will need to be relocated after fifty packages have been deposited, or about every eight days during Phase I.

To prevent the crane crew from receiving a high exposure rate, a new shield wall will be erected before the first shield wall is removed. A remote grappling system will be required to prevent rigging of the previously placed shield wall from causing high dose rates to operations personnel. Even then, the amount of time it will take to move the wall is estimated in the CDR to be 26 hours, four to five shifts, or a little less than two days when operating a full 24 hours per day.

An alternative to the movable shielding wall is to use contact handle waste to construct the shielding wall and to leave it in place after placement of each ILAW array rather than moving it. This can reduce operations labor and expenses. It can also result in the use of less cover soil because the space between the package arrays will be partly filled with contact handle waste, rather than with all soil. This alternative needs to be considered further when developing the operations plan for operating the disposal facility.

7.1.6 Typical Array Size

The moveable shielding wall set up between the crane and transporter operations and the placed package configuration will limit the proximity of package placement to between 15 meters (50 feet) and 7.5 meters (25 feet) of the crane. The 7.5 meters (25 feet) usable range of the crane reach, working over the shielding wall, and the ten or twelve maximum package width (because of dose rate limitation) determines the nominal array size that can be placed by the crane from a single set point. The 1.22 meters (4 feet) diameter ILAW packages will be staggered in the array to minimize the space between the packages. A column that is five packages deep can fit within the 7.5 meters (25 feet) available range of the crane reach while working over the shield wall. A width of ten packages is within the reach of the crane and is less than the allowable limits for the dose rate. Allowing for a 10 centimeters (4 inches) average tolerance in package placement, the five-row by ten-package-wide array is roughly 6 meters (20 feet) deep by 13.3 meters (44 feet) wide. A typical array is shown in Appendix D, Drawing D.1-1.

7.1.7 Cover Soil

Prior to the shield wall being relocated, the crane will place interstitial soil material between the packages, using a hopper. The filling operation is expected to take about one shift, according to the CDR, using up the balance of the two days needed to move the shield wall. To make up the time spent moving the shield wall and placing the interstitial fill soil, the average rate of package placement will have to be increased to seven packages per day for five days, according to the CDR.

While the shield wall is being relocated, a soil cover will be placed over the packages from on top of the lift of previously placed packages. Dump trucks will drive over the previously covered portion of the array and back up to near the edge of the packages that are still exposed and dump a load of fill soil for spreading by a bulldozer. The soil will be spread over the top of the top and exposed side of the array. The side slope from soil, cascading off the top, will be formed in no less than 1.5 H: 1V for reasons of safety, and will use approximately a 5-meter (16-foot) wide space between lines of arrays. Approximately 300 cubic meters (400 cubic yards) of soil will be required to cover the top and side of the five-row-deep by ten-package-wide array. The cover soil will be held back from the advancing end of the array so that the toe of the cover soil does not extend beyond the outer package in the array. This will allow the next array to be placed in close proximity to the previous array. After the bulldozer spreads the soil to a somewhat uniform 1-meter-plus thickness over the packages, a sheepsfoot-style compactor will make several passes to consolidate the fill soil. The cover soil effort will take approximately 12 hours or two shifts, as estimated in the CDR, and will take place at the same time that the portable shield wall is being relocated.

7.1.8 Failed Melter Disposal Area (Note: Disposal of failed melters is not permitted at this time by this permit.)

Failed melters can be disposed of as MLLW in Cell 1. A failed melter disposal area is provided on the bottom of Cell 1 at the southern toe of the waste lifts. Disposing of the failed melters in this area would eliminate placing them within the lifts along with the ILAW packages and other MLLW.

7.1.9 Access Ramps

Two 30-foot wide access ramps will be built into the south slope of the waste lifts to accommodate the movement of transport vehicles and equipment from one lift to the next. A third access ramp will be built through the north shine berm onto the top of the third lift to accommodate transport vehicles and equipment during the construction of Phase II, when the access ramp leading down the south excavation slope to the bottom of Phase I will be removed. The access ramp into Cell 1 and the access ramp from the north side would have a maximum slope of 5 percent to accommodate failed melter transporters, if it becomes necessary to dispose of the melters in the waste lifts rather than in the designated area at the bottom of Cell 1. The access ramp into Cell 2 would have a maximum slope of 8 percent that would accommodate the ILAW, DBVS containers, and LLW waste transporters. The access ramps at the bottom of Phase I would have minimum outside turning radii of 75 feet, to accommodate the failed melter transporters. The dimensions of the access ramps provide flexibility to accommodate the various waste haul vehicles that could use the ramps.

7.1.10 Filling Lift 1

Filling of remote handle ILAW and DBVS waste in Cell 1 will begin in the northwest corner and proceed to the southeast. Filling of remote handle LLW in Cell 2 will begin in the northeast corner and proceed to the southwest. Filling of contact handle LLW will begin in the northwest corner of cell 2 and proceed southeast (see Appendix D, Drawing D.1-2). This filling approach places the remote handle wastes farthest apart from each other, with contact handle wastes between them, and eliminates the need for additional shielding provisions that would be necessary if the two remote handles wastes were located adjacent to each other. This filling approach will be continued in the three subsequent lifts.

Nearly all of Lift 1 can be filled with the crane and transporters, operating from the top of the first operations layer. A 5-meter (17-foot) wide separation will be maintained between Cell 1 and Cell 2 to separate the ILAW and DBVS waste from the LLW. This separation area will be filled with soil. Using a low permeability soil in this area will maximize separation of leachate between the two cells. Two access lanes (ramps) will be maintained into the cells for transporter access. The transporters can turn around within the cells until the packages are within 7.5 meters (25 feet) of the area needed for the unloading operations.

Before the space for filling Lift 1 from the top of the first operations layer is consumed, the two access ramps will be extended with soil and contact handle waste to the top of Lift 1. The crane and transporters will go to the top of Lift 1 and will finish placing the remainder of the Lift 1 waste packages from the top (see Appendix D, Drawing D.1-3). At this point, it will also be possible to begin using the failed melter disposal area (also shown on Drawing D.1-3).

7.1.11 Filling Lift 2

Lift 2 will be filled similarly to Lift 1 (see Appendix D, Drawing D.1-4). This filling approach will continue the pattern that was established in Lift 1. Nearly the entire lift can be filled with the crane and transporters operating on the top of Lift 1. The 5-meter (17-foot) wide soil-filled separation will be maintained between Cell 1 and Cell 2 to separate the ILAW and DBVS waste from the LLW. The two access ramps will be maintained into both cells for transporter access. The transporters can turn around within the cells until the packages are within 7.5 meters (25 feet) of the area needed for the unloading operations. Before the space for filling Lift 2 from the top of Lift 1 is consumed, the two access ramps will be extended with soil and contact handle waste to the top of Lift 2. The crane and transporters will go to the top of Lift 2 and will finish placing the remainder of the Lift 2 waste packages from the top (see Appendix D, Drawing D.1-5).

7.1.12 Filling Lift 3

Lift 3 will be filled similarly to Lift 2 (see Appendix D, Drawing D.1-6). Nearly the entire lift can be filled with the crane and transporters operating on the top of Lift 2. The 5-meter (17-foot) wide soil-filled separation will be maintained between Cell 1 and Cell 2 to separate the ILAW and DBVS waste from the LLW. Two access ramps will be extended into the cells for transporter access. The transporters can turn around within the cells until the packages are within 7.5 meters (25 feet) of the area needed for the unloading operations. Before the space for filling Lift 3 from the top of Lift 2 is consumed, the two access ramps will be extended with soil and contact handle waste to the top of Lift 3. The crane and transporters will go to the top of Lift 3 and will finish placing the remainder of the Lift 3 waste packages from the top (see Appendix D, Drawing D.1-7).

7.1.13 Filling Lift 4

Lift 4 will be filled similarly to the previous three lifts, but with a few differences (see Appendix D, Drawing D.1-8). Most of the lift can be filled with the crane and transporters operating on the top of Lift

3, using the access ramps from the south. However, only the easterly access ramp from the south is planned to be extended to the top of Lift 4 for transporter access. The westerly access ramp from the south will not be extended because, as shown on Appendix D, Drawing D.1-9, it would reach the top of Lift 4 too close to the west side slope to accommodate an adequate turning radius for the transport vehicles. The access ramp will be blocked by waste placement in Cell 1. However, with some minor adjustment in its location and/or increase in its slope, it will be possible to extend the access ramp into Cell 1, if desired. Also, at some point during the filling Lift 4, construction for Phase II to the south will begin, and the access road from the south will be removed from service.

Prior to the westerly access ramp becoming blocked with waste and the access road from the south removed for construction of Phase II, a third access ramp will be constructed from the north down onto the top of Lift 3 to provide additional access. This access ramp will maintain separation between Cell 1 and Cell 2, to separate the ILAW and DBVS waste from the LLW. The transporters can turn around within the cells until the packages are within 7.5 meters (25 feet) of the area needed for the unloading operations.

Before the space for filling Lift 4 from the top of Lift 3 is consumed, the easterly access ramp will be extended with soil and contact handle waste to the top of Lift 4, and the access ramp from the north will be graded out onto the top of Lift 4. The crane and transporters will go to the top of Lift 4 and will finish placing the remainder of the Lift 4 waste packages from the top (see Appendix D, Drawing D.1-9). Completion of Lift 4 will end the filling operations in Phase I. The configuration at the end of Lift 4, prior to placement of the final cover system, is shown on Appendix D, Drawing D.1-10.

7.1.14 Transitioning between Lifts

As the available operating space in a lift gets smaller, operations efficiency will decrease to a point where it will become necessary to move part of the operations to the next lift before the active lift is completed. This will allow completion of each lift, using selected waste that will be easier to handle in the remaining space available on the lift. An example of this would be to use only contact handle waste to complete the filling of each lift while operating on the top of the lift that is being completed (see Appendix D, Drawings D.1-3, -5, -7, and -9) and sending all remote handle waste into the next lift.

7.1.15 Planning for Phase II and Operations During Phase II Construction

Phase II will need to be constructed and ready for operations sufficiently ahead of completion of filling operations in Lift 4 of Phase I to allow a smooth transition without operational constraints. Planning, design, and construction of Phase II may require several years. Phase II should be planned to be ready for operation at least six months, and preferably one year or more, before Lift 4 in Phase I is anticipated to be completed. This will provide a reasonable margin for changes in the incoming waste quantities and other variables while still having Phase II ready for operation, prior to reaching capacity in Phase I.

While Phase II is under construction, the access road on the west will be out of service for a period of time and the access ramp on the south into Phase I will be removed. During this time, it will be necessary for all waste transport vehicles to enter Phase I, using the access ramp on the north side. As currently designed, some access roads on the west and north sides of Phase I that normally would be used to reach the north access ramp might not accommodate all of the transport vehicles. In particular, the berm access road on the west side of Phase I and the access roads around the leachate storage tanks on the north do not have widths and turning radii as large as required by the waste transport vehicles. These roads would have to be widened and their turning radii increased to meet the requirements for transport vehicles, particularly the failed melter transporters.

7.2 Operational Interfaces

Operations and maintenance procedures will be prepared in the future as a separate project. These procedures will address operations, monitoring, and maintenance activities for the IDF.

This section of the Design Report presents important operational interfaces that have been identified by the design team. These interfaces should be considered during preparation of the operation and maintenance procedures. The interfaces are grouped by three categories–landfill excavation, liner system, and leachate handling system.

7.2.1 IDF Landfill Excavation and Related Subsystems

Operational interfaces for the landfill excavation and related subsystems include the following:

- Due to the containerized nature of the waste, the landfill is designed to be filled in a bottom-up fashion in four or more layers. The number of layers will depend on waste package size. Some waste packages may be larger in dimension than the ILAW packages. Operational procedures should be developed to accommodate various package sizes and their placement.
- Clean fill placement between waste packages must be done to minimize the potential for future consolidation and potential subsidence.
- Operations layer on side slopes of IDF will be monitored for material loss due to wind and water erosion. Lost material should be replaced. Annual application of spray-on type soil stabilization material to exposed areas of Phase I IDF should be considered.
- Shine berms should be monitored for erosion and height and should be repaired as necessary. Erosion control matting on the berm will be maintained and repaired or replaced if damage occurs.
- Stormwater control facilities should be maintained annually. Maintenance would include debris removal from the ditches and application of weed control. Periodically, if capacity of infiltration areas is diminished due to collection of fines, fines removal will be necessary. To maintain infiltration capacity, no other vehicle access should be allowed into these areas.
- Stormwater accumulation in the in-cell excavation infiltration area should be visually monitored. Pumping of the area may be necessary if accumulation becomes significant (near liner levels) in wet weather seasons. Periodically, if capacity of infiltration areas is diminished due to collection of fines, fines removal will be necessary. To maintain infiltration capacity, no other vehicle access should be allowed into these areas.
- Due to the heavy wheel loads on the access roads and ramps, gravel surfacing will be maintained with regular maintenance. Maintenance activities may include addition of more top course material, and grading and compaction of this material.
- Active faces of stockpiles will require periodic application of spray-on soil stabilization material.

7.2.2 IDF Liner System

Operational interfaces for the lining system include the following:

- Only equipment with ground pressures less than 4,400 lb/ft should be used for construction and maintenance on the side slopes, when operating directly on the operations layer. Bulldozers or other equipment may operate on the side slopes until a rain event in excess of 0.15 inches per hour occurs. In that event, equipment should be kept off the side slope (directly on the operations layer) and should not be permitted to operate on slopes until two hours after the end of the rainfall event. The precipitation event applies to both the lined slopes and the unlined slopes at the southern end of the Phase I cell.
- For equipment on ramps, equipment should be kept a minimum of 2 feet away from the edge of ramps, to avoid localized sloughing of the ramp edges.
- When operating equipment or placing waste on the operations layer above the lining system, care should be taken to avoid damaging the liner. Special care will be necessary for equipment operation on the side slopes.
- Any loads placed on the surface of the first operations layer must be examined to verify that they do not create loads on the lining system in excess of the allowable GCL bearing capacity. As an example, different types of waste other than canisters should be examined as the waste plan is

more fully developed. Care should also be taken to avoid impact loading, such as dropping a canister.

- For static loading (such as for a barrier wall), refer to the discussion in Section 5.2 and Appendix C.2.
- For operational/equipment loading, refer to the discussion in Section 5.5.5 and Appendix C.5.e to determine applicable load limits and crane dunnage requirements.

The waste plan, as it is developed, should be followed for placement and density requirements. Any revisions to the proposed waste filling plan (discussed in Section 7.1) should be reviewed by the design engineer, to evaluate impacts on the waste/fill global stability analyses (Section 5.1.3 and Appendix C.1.c).

As part of the waste/fill global stability analyses, the waste mass was considered internally stable for this design effort. Internal waste mass stability is a function of the waste filling approach. There are numerous options available to stabilize the waste through operational methodologies, such as providing a greater soil buttress on the open 3:1 south slope. During subsequent design phases, the internal stability of the waste should be evaluated in conjunction with the waste filling plan.

7.2.3 IDF Leachate Handling System

Operational interfaces for the leachate handling system include the following:

- Coordinate with Liquid Effluent Retention Facility (LERF) for leachate hauling and removal of leachate from tanks to satisfy the 90-day accumulation period (Treatment capacities at LERF and leachate flows for critical periods should also be coordinated. See Section 5.9.2.4 for additional leachate hauling constraints.)
- Use leak detection history for leachate storage tanks, during the operation of IDF, to manage and plan for replacement of tank liner system and temporary storage required during its replacement
- Periodic preventative inspection and maintenance for all rotating equipment should be scheduled.
- For leachate tanks floating covers, rain or snow will need to be pumped off with the manufacturer-included sump pump (mounted on side of tank). Water should not be allowed to accumulate except at the perimeter of the floating cover. Excessive water may prevent vent operation and cause mixing between precipitation water and leachate on top of the cover.
- An adequate store of critical spare electrical and mechanical parts should be maintained.
- All valves should be exercised at least annually.
- A small “contractor-type” trash pump with hose should be kept on hand that can be used to pump from the leak detection chamber within the combined sump to its inner sump.
- Periodically, test operation of the combined sump pump should be done.
- Annual testing of all leachate pumps for proper operation should be scheduled.
- Regular verification of level transducer calibration in cells should be done.
- Prior to winter months, proper operation of all heat tracing system should be checked.
- Periodic testing of all control relays, switches and contacts should be scheduled.
- Additional operational interface items will be developed, based on completion of design of the control system for the leachate handling system. This will be part of the IDF administration building design.
- Maintenance should be provided in accordance with manufacturer’s recommendations.

7.3 Leakage Response Action Plan

WAC 173-303-665(9) regulations require the owner of the operator of a landfill unit to have an approved Response Action Plan (RAP) before receipt of waste. The RAP is a site-specific plan that establishes actions to be taken if leakage through the upper (primary) lining system of a landfill exceeds a certain rate. The intent of the RAP is to assure that any leachate that leaks through the primary lining system will not migrate out of the landfill into the environment.

A key element of the RAP is the ALR, a threshold value which triggers the responses described in the RAP, but below which no special actions are required. Because landfill liner systems have not yet been perfected, a small amount of leakage through the primary liner generally occurs, despite the use of best available materials, construction techniques, and QA procedures. (This leakage is collected by the LDS system and removed from the landfill.) Hence, the ALR is set at some level higher than normally expected leakage rates to serve as an indicator that the primary lining system is not functioning as expected. Exceeding the ALR may reflect serious failure of the primary lining system and indicates the need for investigation and possibly corrective action while the problem is still manageable.

This RAP has been prepared in accordance with requirements of WAC 173-303-665(9). The requirements for determining the ALR are contained in WAC 173-303-665(8) and EPA guidance document, *Action Leakage Rates for Leak Detection Systems* (EPA 1992a).

The following sections establish the ALR and discuss response actions to be taken if the ALR is exceeded.

7.3.1 Action Leakage Rate

Section 5.11 provides a detailed discussion of the analysis to determine the ALR into the LDS for the IDF. Based on this analyses, the ALR for each IDF cell is 206 gallons per acre per day, or approximately 1,800 gallons per day per cell (each cell area is approximately 8.5 acres). This value includes a factor of safety of 2 in accordance with EPA guidelines (EPA 1992b). It is also much lower than the LDS pump capacity. Details of the calculation are presented in Appendix C.10.

In accordance with WAC 173-303-665(8)(b), the flow rate used to determine if the ALR has been exceeded will be calculated as the average daily flow rate into the sump, expressed as gallons per acre per day (unless Ecology approves a different calculation). This calculation will be performed on a weekly basis during the active (operational) life of the landfill, and monthly after the landfill has been closed. Post-closure frequency may be reduced if only minimal amounts of leachate accumulate in the LDS sump. As outlined in WAC 173-303-665(4)(c)(ii), during post-closure monitoring, if the liquid level in the LDS sump stays below the pump operating level for two consecutive months, monitoring of the amount of liquid in the LDS sumps can be reduced to at least quarterly. If the liquid level in the LDS sump stays below the pump operating level for two consecutive quarters, monitoring of the amount of liquid in the LDS sumps can be reduced to at least semiannually. Pump operating level is defined as a liquid level approved by Ecology, based on pump activation level, sump dimensions, and level that minimizes head in the sump.

7.3.2 Response Actions

WAC 173-303-665(9) lists several required actions if the ALR is exceeded. In the event that the ALR is exceeded, DOE will:

- Notify Ecology in writing of the exceedance within 7 days of the determination
- Submit a preliminary written assessment to Ecology within 14 days of the determination, as to the amount of liquids, likely sources of liquids, possible location, size, cause of any leaks, and short-term actions taken and planned
- Determine, to the extent practicable, the location, size , and cause of any leak
- Determine whether waste receipt should cease or be curtailed, whether any waste should be removed from the unit for inspection, repairs, or controls, and whether or not the unit should be closed
- Determine any other short-term and longer-term actions to be taken to mitigate or stop any leaks

Within 30 days after the notification that the ALR has been exceeded, submit to Ecology the results of the analyses specified in bullets 3, 4, and 5 of this section, the results of actions taken, and actions planned.

Monthly thereafter, as long as the flow rate in the LDS exceeds the ALR, the owner or operator must submit to the regional administrator a report summarizing the results of any remedial actions taken and actions planned.

If the ALR is exceeded, the DOE will submit the required notifications to Ecology, as stated above. The EPA will also receive copies of this confirmation.

The leachate will be analyzed for chemical compounds and radionuclides. If the analytical results indicate that these constituents are present, and if the constituents can be traced to a particular type of waste stored in a known area of the landfill, then it may be possible to estimate the location of the leak. However, because the waste will meet land disposal restrictions, it will contain no free liquids and will be stabilized or solidified. In addition, the canister(s) or other type of waste package(s) may not undergo enough deterioration during the active life of the landfill to permit escape of its contents. For these reasons, it is possible that the leachate may be clean or the composition too general to indicate a specific source location.

If the source location cannot be identified, large-scale removal of the waste and operations layer to find and repair the leaking area of the liner would be one option for remediation. However, this procedure risks damaging the liner. In addition, waste would have to be handled, stored, and replaced in the landfill. Backfill would need to be removed from around the waste packages to accomplish this. If the waste packages are damaged during this process, the risk of accidental release may be high. For these reasons, large scale removal of waste and liner system materials is not considered a desirable option and will not be implemented except as a last resort.

The preferred options for remediation include covers and changes in landfill operating procedures. The preferred alternative will depend on factors such as the amount of waste already in the landfill, the rate of waste receipt, the chemistry of the leachate, the availability of other RCRA-compliant disposal facilities, and similar considerations. Hence, at this time no single approach can be selected. If the ALR is exceeded, potential options will be evaluated prior to selecting a remediation process. If necessary, an interim solution will be implemented while the evaluation and permanent remediation is performed. Examples of potential approaches include the following:

- The surface of the intermediate soil cover over the waste could be graded to direct runoff into a shallow pond. The surface would then be covered with a discardable, temporary geomembrane (e.g., 30-mil PVC or reinforced polypropylene). Precipitation water would be pumped or evaporated from the pond and would not infiltrate the waste already in the landfill. Waste packages would be placed only during periods of dry weather and stored temporarily at other times. This type of approach would also be used, if necessary, to reduce leakage during the time immediately after the ALR was exceeded, while other remediation options were being evaluated.
- If the landfill was nearly full, partial construction of the final closure cover might be an option. This would reduce infiltration into the landfill and possibly the leakage rate, if the cover were constructed over the failed area.
- A layer of low-permeability soil could be placed over the existing waste, perhaps in conjunction with a geomembrane, to create a second “primary” liner higher in the landfill. This new liner would intercept precipitation and allow its removal.
- A rigid-frame or air-supported structure could be constructed over the landfill to ensure that no infiltration occurred. Although costly, this approach might be less expensive than constructing a new landfill.

In general, the selected remediation efforts would be those that are easiest to implement, with more difficult or expensive options to be applied only if earlier approaches were not satisfactory.

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