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Office of Environmental Management – Grand Junction

MOAB UTAH
UMTRA Project

Remedial Action Plan and Site Design for Stabilization of Moab Title I Uranium Mill Residual Radioactive Material at the Crescent Junction, Utah, Disposal Site

Remedial Action Selection Report

Revision 2

July 2008
Updated December 2012

U.S. Department of Energy

Office of Environmental Management

Prepared by the Remedial Action Contractor under contract number DE-AT30-07CC00014 for the U.S. Department of Energy Office of Environmental Management, Grand Junction, Colorado.

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Moab UMTRA Project
Final Remedial Action Plan and Site Design for Stabilization of Moab Title I Uranium Mill Tailings at the Crescent Junction, Utah, Disposal Site

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Work Performed by Energy*Solutions* Federal Services under DOE Contract No.
DE–AT30–07CC00014 and S.M. Stoller Corporation under DOE Contract No.
DE–AC01–02GJ79491 for the U.S. Department of Energy
Office of Environmental Management, Grand Junction, Colorado

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(Previously Provided in the Draft RAP)

Acronyms

AC	Acres
Act	Floyd D. Spence National Defense Authorization Act
ASTM	American Society for Testing and Materials
Atlas	Atlas Minerals Corporation
BLM	Bureau of Land Management
CAES	Computer Aided Earthmoving System
Cc	Compression Index or Coefficient of Consolidation
CFR	Code of Federal Regulations
cfs	cubic feet per second
cm	centimeter
cm/s	centimeter per second
cm^2/s	centimeter squared per second
CN	curve number
CPT	cone penetrometer test
D_{50}	median particle size
DOE	U.S. Department of Energy
DOT	U.S. Department of Transportation
e_0	Initial Void Ratio
EIS	Environmental Impact Statement
EPA	U.S. Environmental Protection Agency
FE	floating earthquake
FR	Federal Register
ft	foot/feet
ft^3	cubic foot/feet
ft/s	feet per second
FY	fiscal year
g	standard acceleration of gravity
GCAP	Ground Water Compliance Action Plan
GPS	Global Positioning System
HEC-HMS	Hydrologic Engineering Center-Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center-River Analysis System
K_h	pseudostatic coefficient
km	kilometer
LL	Liquid Limit
m^2	square meter
µR/h	microRoentgen per hour
MCE	maximum credible earthquake
mg/L	milligram per liter
mi	mile
mi^2	square mile
MUSLE	Modified Universal Soil Loss
NAS	National Academy of Sciences
NCEER	National Center for Earthquake Engineering Research
NOAA	U.S. National Oceanic and Atmospheric Administration
NOI	Notice of Intent
NP	Not Performed
NRC	U.S. Nuclear Regulatory Commission
NRCS	U.S. National Resources Conservation Service

NSF	National Science Foundation
NUREG-CR	Publications Prepared by NRC Contractors
pcf	pound per cubic foot
pCi/g	picoCurie per gram
pCi/L	picoCurie per liter
pCi/m^2/s	picoCurie per square meter per second
PHA	peak horizontal acceleration
PI	Plasticity Index
PL	Plastic Limit
PMF	probable maximum flood
PMP	probable maximum precipitation
psi	pounds per square inch
RAIP	Remedial Action Inspection Plan
RAP	Remedial Action Plan and Site Design for Stabilization of Moab Title I Uranium Mill Tailings at the Crescent Junction, Utah, Disposal Site
RAS	Remedial Action Selection Report
ROD	Record of Decision
RRM	residual radioactive material
RUSLE	Revised Universal Soil Loss
SCS	U.S. Soil Conservation Service
SITLA	Utah School and Institutional Trust Leads Administration
SOWP	Site Observational Work Plan
SPT	Standard Penetration Test
SRP	Standard Review Plan
TAD	Technical Approach Document
Tc	Time of Concentration
TDS	total dissolved solids
TP	Test pits
UMTRA	Uranium Mill Tailings Remedial Action (Project)
UMTRCA	Uranium Mill Tailings Radiation Control Act
μR/h	microRoentgens per hour
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USGS	U.S. Geological Survey
yd3	cubic yard

Executive Summary

Background

The Moab Uranium Mill Tailings Remedial Action (UMTRA) Project is a remedial action being performed by the U.S. Department of Energy (DOE) to relocate uranium mill tailings and other contaminated materials (residual radioactive material, RRM) from its present location approximately three miles northwest of the city of Moab, Utah, to the Crescent Junction Disposal Site in Utah. The RRM was generated through operations of a uranium processing facility. The Uranium Reduction Company operated the mill from 1956 until 1962 when it was sold to Atlas Minerals Corporation (Atlas). The milling operations ceased in 1984. An interim cover was placed over the RRM as part of decommissioning activities between 1988 and 1995. Atlas declared bankruptcy in 1998, and the property was subsequently designated a Uranium Mill Tailings Radiation Control Act (UMTRCA) Title I site through legislation and DOE was given cleanup responsibility.

Studies were conducted in the early 2000s with an Environmental Impact Statement (EIS) issued in July 2005. The Final EIS established that the preferred alternative for long-term disposal of the uranium mill tailings and associated contaminated materials was relocation to the Crescent Junction Disposal Site. Subsequently, a Record of Decision (ROD) was issued in September of that same year. The remedial action consists of the removal and subsequent relocation of all RRM from the Moab Site to the Crescent Junction Disposal Site. Disposal will consist of constructing an approximately 230-acre engineered cell partially below grade. The cleanup must comply with U.S. Environmental Protection Agency (EPA). Following concurrence by the U.S. Nuclear Regulatory Commission (NRC) of this Final Remedial Action Plan (RAP), construction of the disposal cell is to begin in 2008 (Table 0–1).

Development of the Final RAP

The purpose of the RAP is to document the remedial activities necessary to relocate the contaminated materials from the Moab Processing Site for stabilization at the Crescent Junction Disposal Site. This Final RAP is a compilation of efforts by DOE over the last several years.

The original draft plan was developed under a contract by DOE with S.M. Stoller Corporation. Efforts included development of studies and supporting documents for the selection of the Crescent Junction Site. The work established the geology and seismology of the site, the surface water and ground water hydrology, a conceptual plan for the disposal facility, water resource protection, and the processing site cleanup. The Draft RAP was issued to NRC in 2006. Several meetings to review the plan generated comments and responses. The revised Draft RAP was issued in June 2007 and provided the basis for detailed design and future construction.

This Final RAP was developed under a subsequent DOE contract with Energy*Solutions* Federal Services, Inc. Detailed design has lead to revisions of certain sections of the Draft RAP. The segments establishing the basis of site selection and parameters used in the design have not been revised. The Remedial Action Selection Report (RAS) summarizes the key elements that will ensure compliance with the regulatory requirements at the disposal cell and the former processing site. The RAS sections that were rewritten for the Final RAP include: **4.2** Geotechnical Engineering Evaluation based on the final detailed design of the disposal cell for slope stability, settlement, liquefaction, and cover cracking; **5.0** Radon Attenuation calculations and final design; **6.4** Erosion Protection Design, which now encompasses a "wedge" of protection along the north side of the cell; **6.5** Rock Durability; **6.6** Rock Sources; 6.7 Rock Selection; and **7.0** Disposal Cell Design and Construction Details, which now includes construction details and construction sequencing.

Table 0-1. Moab UMTRA Project Timeline

Uranium Reduction Company Operated Mill	1956 – 1962
Atlas Minerals Corp Operated Mill	1962 – 1984
Decommissioning Activities	1988 – 1995
Atlas Declares Bankruptcy	1998
UMTRCA Title I Site Amended to Include Moab Site	Oct 2000
DOE Assumes Responsibility for Cleanup	2001
National Academy of Sciences Review and Recommendation	2001 – 2002
Notice of Intent in *Federal Register* to Prepare Environmental Impact Statement (EIS)	Dec 2002
EIS Issued	Jul 2005
Record of Decision	Sep 2005
Draft RAP	Aug 2006
Revised Draft RAP Issued	Jun 2007
Final RAP Issued	Feb 2008
Crescent Junction Disposal Cell Construction Begins	2008 –

The following is a brief description of what is found in each section of the RAS:

Section 1.0 provides the background, standards, and collateral documents. Two tables contain a list of the contents of Addendums A through E and Attachments 1 through 5.

Section 2.0 presents the data and analyses that show that DOE has adequately characterized the disposal site regarding the impacts of geologic conditions on the long-term performance of the cell. Geologic, geomorphic, and seismic conditions at the site are analyzed and results are presented.

Section 3.0 consists of characterizing the physical and geochemical properties of the ground water hydrogeology units and documents the water use at the disposal site.

Section 4.0 presents the geotechnical engineering aspects of the remedial action. It includes geotechnical investigations at both the disposal site and at the Moab tailings pile and engineering evaluations of the disposal cell's slope stability, settlement, liquefaction potential, and cover cracking.

Section 5.0 covers radon emanation from the cell.

Section 6.0 presents information on surface water hydrology and erosion protection. Included are the hydrologic description of the area, the probable maximum precipitation and probable maximum flood, infiltration losses, water surface profiles and channel velocities, and the details on the erosion protection for the final design of the cell. Rock durability and potential sources of rock are also in this section.

Section 7.0 provides the disposal cell final design and construction details. Construction of the cell will be performed in stages. The sequencing of the construction activities and testing and inspection are also presented in this section.

Section 8.0 presents the water resources protection strategy for the disposal cell.

Section 9.0 provides information on the radiological cleanup at the Moab Processing Site. Site characterization, standards for cleanup, and verification of cleanup are included. A final decision regarding the process site ground water cleanup approach will be deferred until a later date and documented in a subsequent Ground Water Compliance Action Plan.

Supporting Documents

Documents directly supporting this Final RAP in regards to final design and remediation are contained in **Addendum A through F.** These include DOE responses to NRC comments, final design specifications, drawings, and calculations, the Remedial Action Inspection Plan, and rock durability test data which describes the quality assurance testing during field construction.

Those documents that supported the Revised Draft RAP are included as the original **Attachments 1 through 5.** These documents have been previously submitted to NRC and are referenced in this Final RAP.

End of current text

1.0 Introduction

The Uranium Mill Tailings Radiation Control Act (UMTRCA) (Title 42 *United States Code* Section 7901 et seq.) was passed in 1978 in response to public concern regarding potential health hazards of long-term exposure to radiation from uranium mill tailings. Title I of UMTRCA provides for remediation of abandoned uranium mill tailings sites and associated vicinity properties by the U.S. Department of Energy (DOE). DOE is required to select and perform remedial actions in accordance with standards set by the U.S. Environmental Protection Agency (EPA) (Title 40 *Code of Federal Regulations* Part 192 [40 CFR 192], "Health and Environmental Protection Standards for Uranium and Thorium Mill Tailings") and with the concurrence of the U.S. Nuclear Regulatory Commission (NRC). The selected remedial action is documented by DOE in this *Final Remedial Action Plan and Site Design for Stabilization of Moab Title I Uranium Mill Tailings at the Crescent Junction, Utah, Disposal Site* (RAP), which is submitted to NRC for concurrence with the remedial action. NRC subsequently licenses the completed disposal site.

In October 2000, the Floyd D. Spence National Defense Authorization Act (Act) for fiscal year (FY) 2001 (Public Law 106-398) amended UMTRCA Title I (which expired in 1998 for all other sites except for ground water remediation and long-term radon management), giving DOE responsibility for remediation of the Moab, Utah, Processing Site. That Act also mandated that the Moab Processing Site be remediated in accordance with UMTRCA Title I "subject to the availability of appropriations for this purpose" and required that DOE prepare a remediation plan to evaluate the costs, benefits, and risks associated with various remediation alternatives. The Act further stipulated that the draft plan be presented to the National Academy of Sciences (NAS) for review. NAS was directed to provide "technical advice, assistance, and recommendations" for remediation of the Moab Processing Site. Under the Act, the Secretary of Energy was required to consider NAS comments before making a final recommendation on the selected remedy.

The DOE Preliminary Plan for Remediation (DOE 2001) for the Moab Site was completed in October 2001 and forwarded to NAS. On June 11, 2002, after reviewing the draft plan, NAS provided a list of recommendations for DOE to consider during its assessment of remediation alternatives for the Moab Site. On December 20, 2002, DOE published in the *Federal Register* (FR) a Notice of Intent (NOI) to prepare an Environmental Impact Statement (EIS) for the Moab Site remediation (67 FR 77969). As stated in the NOI, the EIS takes the place of a final plan for remediation for the purpose of supporting decision-making for remediation of the Moab Site. DOE has addressed the NAS recommendations in its internal scoping, in the EIS (DOE 2005), and in supporting documents.

The Record of Decision (ROD) (70 FR 55358, September 21, 2005) detailed the selected alternative for surface remediation as removal of RRM to a disposal cell to be constructed near Crescent Junction, Utah (see further discussion in Section 1.1.3). Rail was selected as the primary mode of transportation for RRM between the Moab Site and Crescent Junction Site.

1.1 Site Background

1.1.1 Location

The Moab Processing Site is located approximately three miles northwest of the city of Moab, in Grand County, Utah, adjacent to the Colorado River (Figure 1–1).

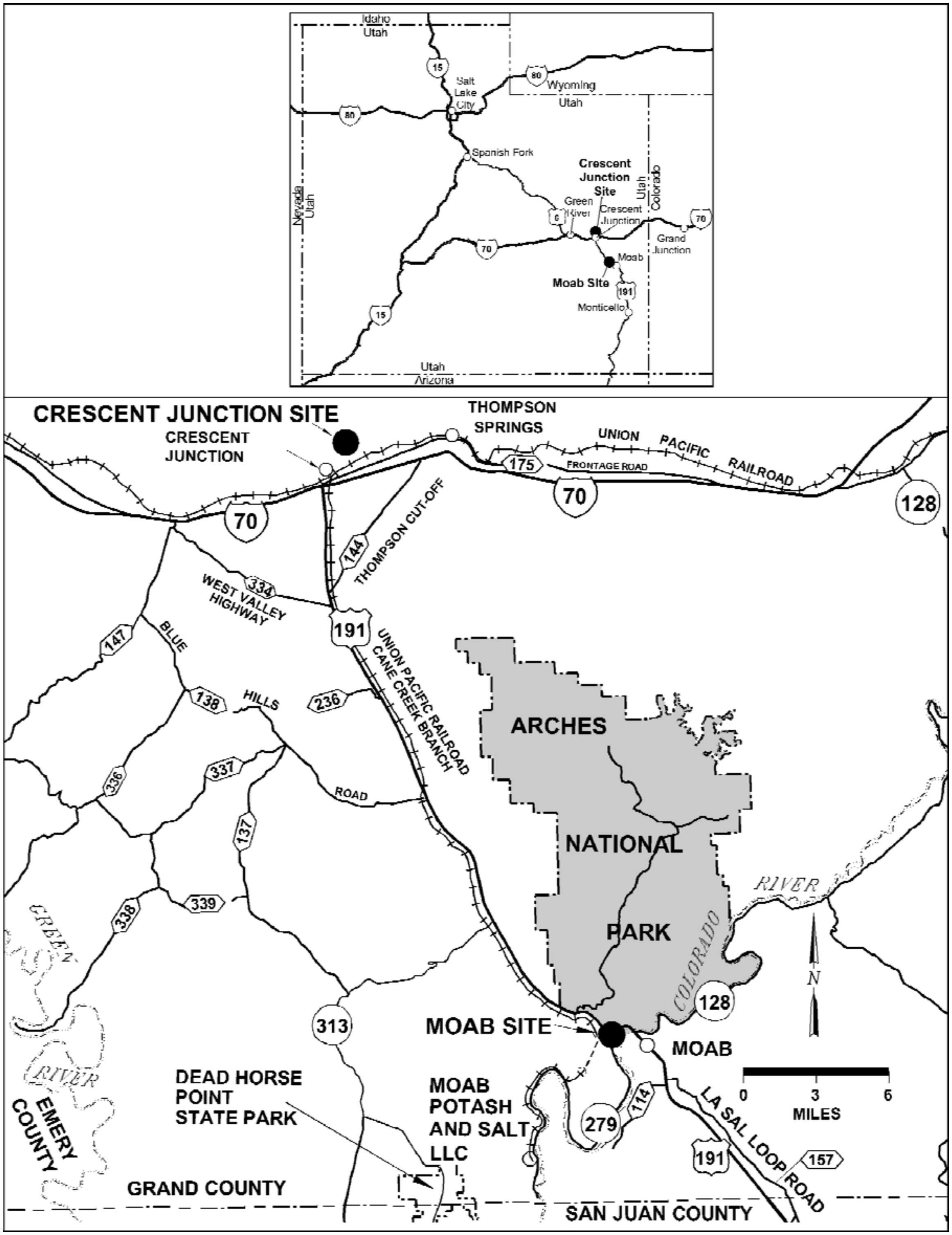

Figure 1-1. Location of the Moab and Crescent Junction Sites in Grand County, Utah

The processing site is on the Moab 7.5-minute topographic quadrangle map in Sections 27 and 28, T25S, R21E, and is shown on the 2005 aerial photograph in Figure 1–2.

The Crescent Junction Disposal Site is located approximately 31 miles north of the Moab Site, and approximately one mile northeast of Crescent Junction, also in Grand County, Utah (Figure 1–1). The disposal site is in a non-populated area just north of Interstate Highway 70 on the Crescent Junction 7.5-minute topographic quadrangle map in Sections 22, 23, 26, and 27, T21S, R19E. The Crescent Junction Disposal Site and surrounding area are shown on the 2005 aerial photograph in Figure 1–3. DOE requested a 5-year temporary withdrawal of approximately 2,300 acres of public domain land near Crescent Junction for the construction of the disposal cell and a buffer zone (“withdrawal area”). The disposal cell footprint occupies only a small portion of the entire temporary withdrawal area (Figure 1–3). An application to transfer 500 acres of the withdrawal area property from U.S. Bureau of Land Management (BLM) to DOE is pending.

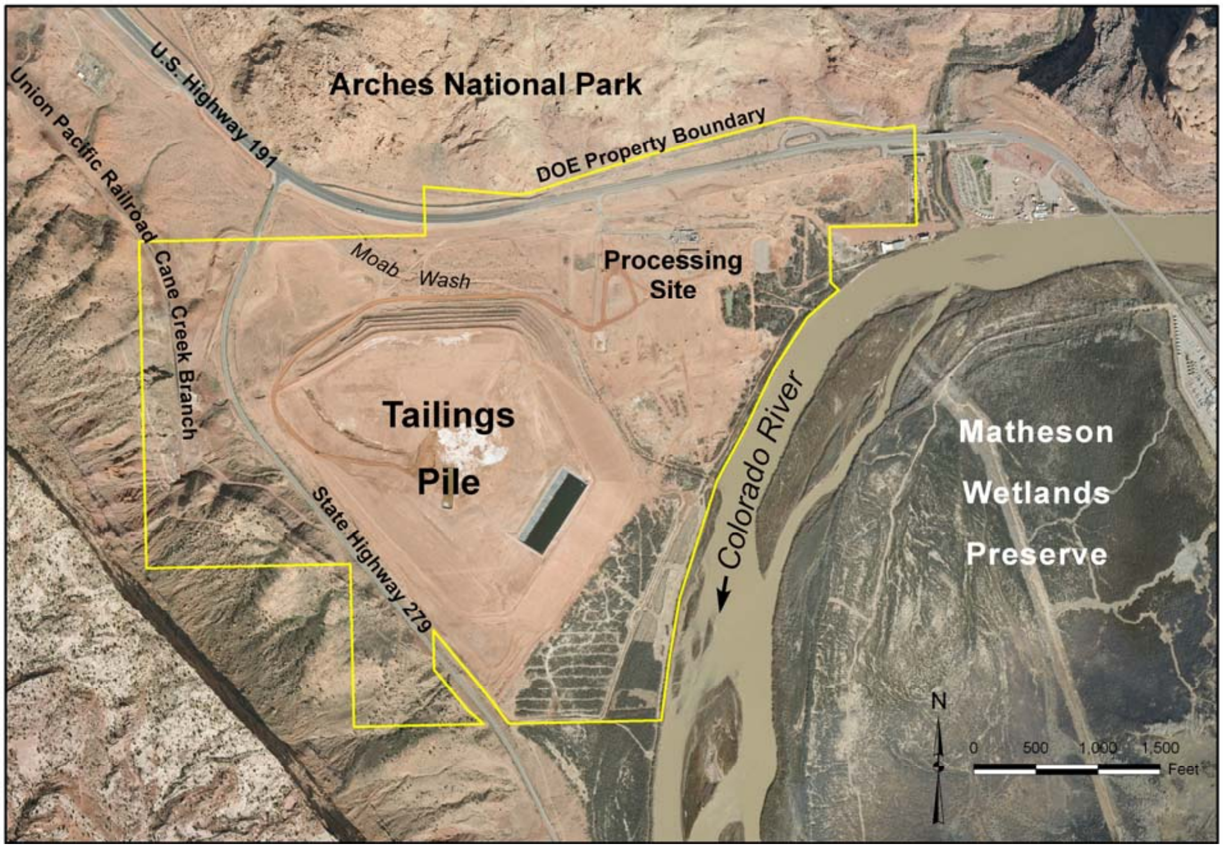

Figure 1-2. Aerial Photograph of the Moab Site

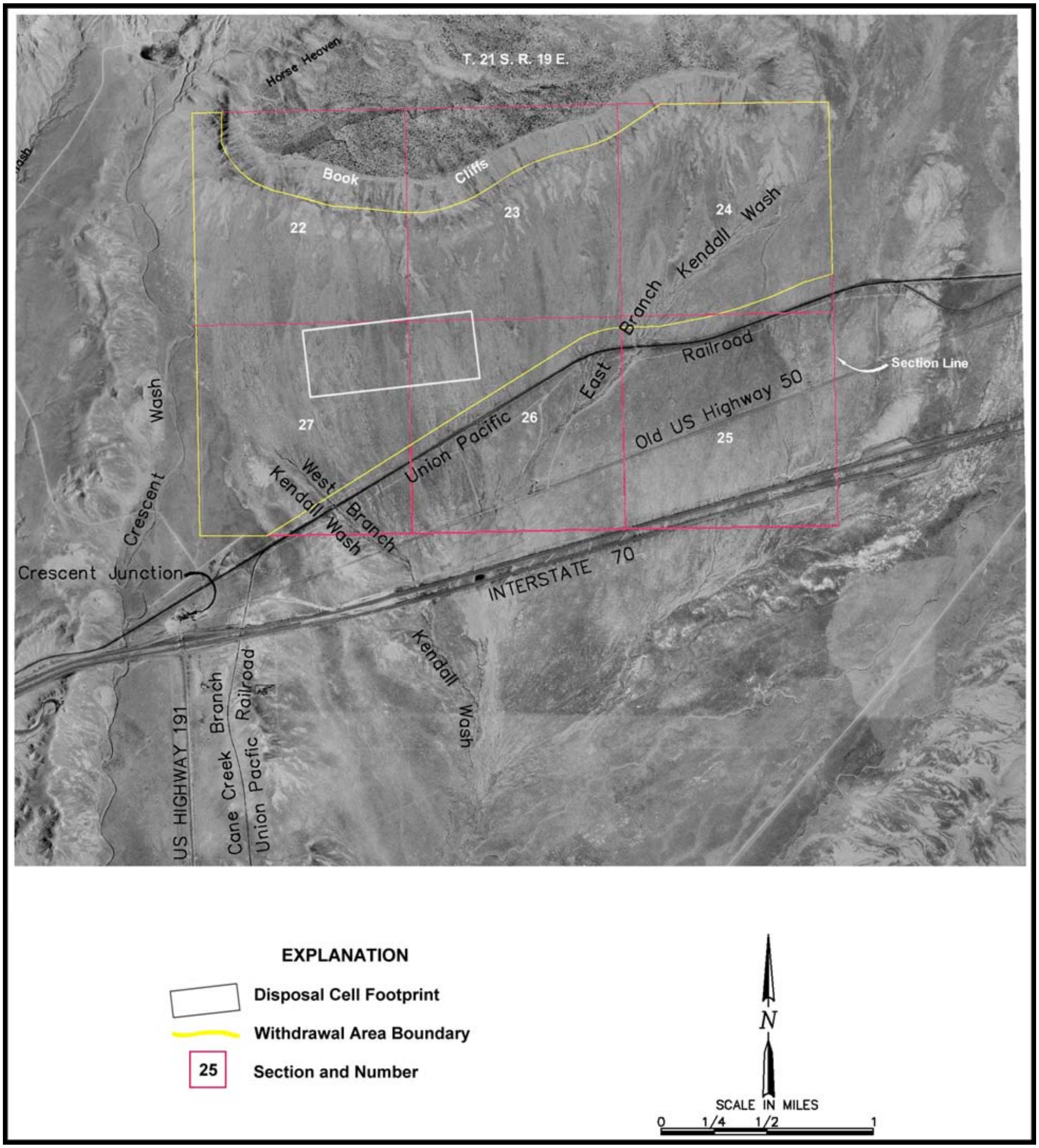

Figure 1-3. Aerial Photograph of the Crescent Junction Disposal Site and Surrounding Area

1.1.2 Site History

The Moab uranium processing facility was constructed in 1956 by the Uranium Reduction Company, which operated the mill until 1962 when the assets were sold to the Atlas Minerals Corporation (Atlas). Uranium processing operations continued under Atlas until 1984. When the processing operations ceased in 1984, the mill had accumulated uranium mill tailings in an unlined impoundment in the floodplain of the Colorado River. The present tailings pile in the west part of the processing site covers approximately 130 acres, is about 0.5 miles in diameter, 94 feet (elevation 4,076 feet) at its highest point above the surrounding ground, and is about 750 feet west of the Colorado River (Figure 1–2). Atlas placed an interim cover over the tailings pile as part of decommissioning activities ongoing between 1988 and 1995.

In 1996, Atlas proposed to reclaim the tailings pile for permanent disposal in its current location. Atlas declared bankruptcy in 1998 and subsequently NRC appointed PricewaterhouseCoopers as the Trustee of the Moab Mill Reclamation Trust and licensee for the site. Subsequently, the Act for FY 2001 mandated that the NRC license for the materials at the Moab Site be terminated and that title and responsibility for cleanup be transferred to DOE by October 31, 2001. DOE assumed full cleanup responsibility for the site during FY 2001.

1.1.3 Remedial Action

Based on the process and evaluation documented in the Final EIS (DOE 2005) for the Moab Site, DOE determined that its preferred alternative for long-term disposal of RRM from the Moab Processing Site was relocation predominantly by rail to the Crescent Junction Disposal Site (Figure 1–1).

The Crescent Junction Site was selected as the preferred off-site disposal location because it has (1) the longest isolation period (time in which contaminants could reach the ground water); (2) the lowest land-use conflict potential (although DOE would need to work with holders of existing oil and gas leases to mitigate any possible impacts); (3) the shortest haul distance from the rail unloading facility into the disposal cell, reducing the size of the radiological control area; and (4) flat terrain, making operations easier and safer.

The tailings pile was constructed with five terraces and consists of an outer compacted embankment of coarse tailings, an inner impoundment of both coarse and fine tailings, and an interim cover of soils taken from the site outside the pile area. Debris from dismantling the mill buildings and associated structures was placed in an area at the south end of the pile and covered with contaminated soils and fill. Radiation surveys indicate that some soils outside the pile also contain radioactive contaminants at concentrations above EPA standards in 40 CFR 192 (see Section 9.1).

Besides tailings, contaminated soils, and debris, other contaminated material requiring cleanup include ponds used during ore-processing activities, disposal trenches, other locations used for waste management during mill operation, and buried septic tanks that are assumed to be contaminated. DOE estimates that total RRM at the Moab Site and vicinity properties has a weight of approximately 16 million tons and a volume of approximately 12 million cubic yards (yd^3). Evidence indicates that historical building materials may contain asbestos.

The remedial action consists of the removal and subsequent relocation of all RRM to the Crescent Junction disposal cell. Essentially all RRM will be placed in containers for transport to the disposal site by either rail or truck. Oversize material will be transported via a secure manner.

Disposal will consist of constructing an approximately 230-acre engineered cell partially below grade. The disposal cell is generally rectangular in shape. The cell is designed for two-thirds of the RRM to be below grade and the remainder above grade. The depth of the cell excavation is based on keying into the weathered Mancos Shale bedrock at least two feet and reusing the shale (after conditioning) to construct the radon barrier. Excavated material will be used as material for construction of the disposal cell's exterior berms, interim cover, and freeze-thaw layer, and will be used to construct the protective wedge to the north of the disposal cell.

1.2 EPA Standards

As required by UMTRCA, remedial action at the site must comply with regulations established by EPA in 40 CFR 192, Subparts A-C. The regulations provide standards for both disposal and cleanup. Disposal and ground water protection standards apply at the disposal site (Crescent Junction); cleanup standards for soil and ground water apply at the processing site (Moab). EPA disposal and ground water protection standards in 40 CFR 192 specify that control of RRM and its listed constituents shall be designed to be effective for up to 1,000 years, to the extent reasonably achievable, and, in any case, for at least 200 years.

Additionally, as described in the Standard Review Plan (SRP) for inactive uranium mill tailings (NRC 1993), DOE must meet the following basic requirements to receive NRC's concurrence on DOE's proposed remedial action:

- There must be reasonable assurance of compliance with the EPA control requirements of 40 CFR 192 for durability of stabilization and control of radon, and protection of ground water resources in the disposal cell area; and
- There must be reasonable assurance of compliance with the EPA requirements in 40 CFR 192 for cleanup of the processing site.

This RAP summarizes the key elements that will ensure compliance with regulatory requirements at the disposal cell and the processing site. More detailed discussion of compliance with ground water requirements at the processing site is found in the Site Observational Work Plan (SOWP) (DOE 2003). It is anticipated that the final remedial action plan for ground water cleanup at the Moab Site will be submitted to NRC in FY 2011.

1.3 Scope, Content, and Organization

The purpose of the RAP is to document the remedial activities necessary to relocate the contaminated materials from the Moab Processing Site for stabilization at the Crescent Junction Disposal Site. This involves assessment of contaminated materials at the Moab Processing Site and design of the transportation system to get materials to the disposal site. It also involves characterization of the Crescent Junction Disposal Site, design and implementation of the disposal system, and protection of ground water resources at the disposal site.

This RAP provides a summary level description of the remedial action and a discussion of technical findings leading to the conclusion that the remedial action is consistent with the EPA standards for stability, radon control, water resources protection, and site cleanup. An extensive amount of data and supporting information have been generated that cannot all be incorporated into this single report. Pertinent information, design details, drawings, calculations and data are included in the RAP addendums and Draft RAP attachments.

Portions of the information in this RAP were presented by DOE to NRC during meetings held in April, June, and December 2006. Additional meetings in September and November 2007 were held with NRC to update them on design details as they were being developed and to discuss outstanding issues.

Comments received as a result of those meetings and from NRC review of the Revised Draft RAP submitted in August 2006, and the Revised Draft RAP submitted in June 2007, have been incorporated into this report. A comment resolution/response is included as Addendum A to this RAP and explains how each comment was resolved.

The RAP consists of the following addendums that contain the details for the final design (drawings, specifications, and calculation sets) and attachments from the Revised Draft RAP:

- Addendum A – DOE Responses to NRC Comments
- Addendum B – Final Design Specifications
- Addendum C – Final Design Drawings
- Addendum D – Final Design Calculations
- Addendum E – Remedial Action Inspection Plan (RAIP)
- Addendum F – Freemont Junction Rock Source Data
- Attachment 1 – Draft RAP Disposal Cell Design Calculations
- Attachment 2 – Geology
- Attachment 3 – Ground Water Hydrology
- Attachment 4 – Water Resources Protection
- Attachment 5 – Field and Laboratory Results (two volumes)

Tables 1–1 and 1–2 list the items contained within each RAP addendum and attachment.

1.4 Collateral Documents

The EIS for the Moab Site (DOE 2005) describes existing conditions at the site, the proposed remedial action, the alternatives to the proposed action, and the environmental impacts of the proposed action. Details are in the EIS and are not reported in this RAP. The SOWP (DOE 2003) assesses ground water conditions at the Moab Processing Site and provides alternatives for ground water cleanup and compliance with the EPA ground water protection standards in 40 CFR 192. Ground water restoration is not a part of this RAP. An interim remedial action has been implemented with a final action to be proposed to NRC in FY 2011.

The Technical Approach Document (TAD) (DOE 1989) is an additional supporting document that describes technical approaches and procedures used on the project. It includes discussions of major technical areas, design considerations, surface water hydrology and erosion control, geotechnical aspects of disposal cell design, radiological issues, and protection of ground water resources.

The Technical Approach to Groundwater Restoration (DOE 1993) provides general technical guidance to implement the ground water restoration phase at the processing site.

Table 1-1. Contents of Final RAP Addendums

Addendum A – DOE Responses to NRC Comments	
April 2006	NRC Comments and DOE Responses, April 2006 Meeting
June 2006	NRC Comments and DOE Responses, June 2006 Meeting
February 2007	NRC Comments and DOE Responses, February 2007 Request for Additional Information
September 2007	NRC Comments and DOE Responses, September 2007 Open Issues Meeting
Addendum B – Final Design Specifications	
Number	**Title**
31-00-00 R1	Earthwork
31-00-20 R1	Placement and Compaction of Tailings and Interim Cover
31-00-30 R1	Placement and Compaction of Final Cap Layers
31-32-11 R1	Surface Water Management and Erosion Control
32-11-23 R1	Aggregate and Riprap
Addendum C – Final Design Drawings	
Number	**Title**
E-02-C-100	Overall Site Plan/Key Plan
E-02-C-101	Overall Cell Layout Plan
E-02-C-102	Overall Cell Grading Plan
E-02-C-103	Overall Cell Top of Waste Plan
E-02-C-104	Overall Cell Cap Plan/Fencing Plan
E-02-C-105	Rock Cover Plan
E-02-C-300	Disposal Cell Cross Sections
E-02-C-301	Disposal Cell Cross Sections
E-02-C-500	Details – 1
E-02-C-501	Details – 2
Addendum D – Final Design Calculations	
Number	**Title**
C-02	Disposal Cell Erosion Protection
C-03	Wedge Longevity
C-04	Area Between Cell and Wedge
C-05	Radon Barrier Evaluation
C-06	Drainage During First Phase of Construction
C-10	Slope Stability of Crescent Junction Disposal Cell
C-11	Settlement Analysis of Uranium Mine Tailings at Crescent Junction, UT
C-12	Liquefaction Analysis of Uranium Mine Tailings Repository at Crescent Junction, UT
C-13	Frost Penetration Depth at Crescent Junction Disposal Site
C-15	Analysis for Cover Cracking of the Crescent Junction Disposal Cell
Addendum E – Remedial Action Inspection Plan (RAIP)	
Document	Remedial Action Inspection Plan (RAIP)
Attachment 1	Computer Aided Earthmoving System (CAES) For Landfills
Addendum F – Freemont Junction Rock Source Data	
F1	Green River Remedial Action Plan Appendix D, Addendum D-4, 1988
F2	Rock Durability Laboratory Results for Samples Collected in 2007 & 2008
F3	Green River, Utah Final Completion Report, Volume 2, Appendix E. Material Summary Report, 1991

Table 1-2. Contents of Draft RAP Attachments

Calculation Cross-Reference Guide		
Location	**Calculation Number**	**Calculation Title**
Attachment 1: Draft RAP Disposal Cell Design Calculations		
Appendix A	MOA-02-08-2006-5-19-01	Freeze/Thaw Layer Design
Appendix B	MOA-02-08-2006-5-13-01	Radon Barrier Design Remedial Action Plan
Appendix C	MOA-02-05-2007-5-17-02	Slope Stability of Crescent Junction Disposal Cell
Appendix D	MOA-02-05-2007-3-16-01	Settlement, Cracking, and Liquefaction Analysis
Appendix E	MOA-02-09-2005-2-08-01	Site Drainage – Hydrology Parameters
Appendix F	MOA-02-06-2006-5-08-00	Crescent Junction Site Hydrology Report
Appendix G	MOA-02-04-2007-5-25-02	Diversion Channel Design, North Side Disposal Cell
Appendix H	MOA-02-08-2006-6-01-00	Erosional Protection of Disposal Cell Cover
Appendix I	MOA-01-06-2006-5-02-01	Volume Calculation for the Moab Tailings Pile
Appendix J	MOA-02-08-2006-5-03-00	Weight/Volume Calculation for the Moab Tailings Pile
Appendix K	MOA-01-08-2006-5-14-00	Average Radium-226 Concentrations for the Moab Tailings Pile
Attachment 2: Geology		
Appendix A	MOA-02-04-2007-1-05-01	Site and Regional Geology – Results of Literature Research
Appendix B	MOA-02-04-2007-1-01-01	Surficial and Bedrock Geology of the Crescent Junction Disposal Site
Appendix C	MOA-02-04-2007-1-06-01	Site and Regional Geomorphology – Results of Literature Research
Appendix D	MOA-02-04-2007-1-07-01	Site and Regional Geomorphology – Results of Site Investigations
Appendix E	MOA-02-04-2007-1-08-01	Site and Regional Seismicity – Results of Literature Research
Appendix F	MOA-02-04-2007-1-09-02	Site and Regional Seismicity – Results of Maximum Credible Earthquake Estimation and Peak Horizontal Acceleration
Appendix G	MOA-02-04-2007-1-02-01	Photogeologic Interpretation
Attachment 3: Ground Water Hydrology		
Appendix A	MOA-02-02-2006-2-07-00	Saturated Hydraulic Conductivity Determination of Weathered Mancos Shale
Appendix B	MOA-02-03-2006-2-10-00	Field Permeability “Bail” Testing
Appendix C	MOA-02-03-2006-2-06-00	Field Permeability “Packer” Testing
Appendix D	MOA-02-04-2006-2-03-00	Hydrologic Characterization – Ground Water Pumping Records
Appendix E	MOA-02-05-2006-2-13-00	Hydrologic Characterization – Vertical Travel Time to Uppermost (Dakota) Aquifer
Appendix F	MOA-02-02-2007-3-01-00	Geochemical Characterization – Radiocarbon Age Determinations for Ground Water Samples Obtained From Wells 0203 and 0208
Appendix G	MOA-02-06-2006-2-15-00	Infiltration Modeling for Alternative and UMTRA Cover Designs
Appendix H	MOA-02-06-2007-2-14-00	Hydrologic Characterization – Lateral Spreading of Leachate
Attachment 4: Water Resources Protection		
Appendix A	MOA-02-06-2006-5-24-00	Material Placement in the Disposal Cell
Appendix B	MOA-02-06-2006-3-05-00	Geochemical Attenuation and Performance Assessment Modeling
Attachment 5: Field and Laboratory Results, Volume I		
Appendix A	MOA-02-03-2006-1-03-00	Corehole Logs for the Crescent Junction Site
Appendix B	MOA-02-03-2006-1-11-00	Borehole Logs for the Crescent Junction Site
Appendix C	MOA-02-03-2006-1-04-00	Geophysical Logs for the Crescent Junction Site
Appendix D	MOA-02-03-2006-1-10-00	Test Pit Logs for the Crescent Junction Site
Appendix E	MOA-02-03-2006-4-01-00	Geotechnical Properties of Native Materials
Appendix F	MOA-01-06-2006-5-22-00	Cone Penetration Tests for the Moab Processing Site
Appendix G	MOA-02-05-2006-4-07-00	Seismic Rippability Investigation for the Crescent Junction Site
Appendix H	MOA-02-03-2007-3-04-01	Background Ground Water Quality for the Crescent Junction Site
Appendix I	MOA-01-08-2006-4-08-00	Boring and Test Pit Logs for the Moab Processing Site
Appendix J	MOA-01-08-2006-4-09-01	Geotechnical Laboratory Testing Results for the Moab Processing Site
Appendix K	MOA-02-04-2007-4-03-01	Supplemental Geotechnical Properties of Native Materials
Attachment 5: Field and Laboratory Results, Volume II		
Appendix L	MOA-02-08-2006-1-06-00	Compilation of Geologic and Geophysical Logs
Appendix M	N/A	Radiological Assessment for Non-Pile Areas of the Moab Project Site
Appendix N	MOA-02-05-2007-4-04-00	Supplemental Geotechnical Properties of Tailings Materials from the Moab Processing Site

End of current text

2.0 Geology and Seismology

The objective of this section is to present the data and analyses that show that DOE has adequately characterized the Crescent Junction Disposal Site regarding the impacts of geologic conditions on the long-term performance objectives of the remedial action as defined by 40 CFR 192.02.

EPA standards listed in 40 CFR 192 do not include generic or site-specific requirements for characterization of the geologic conditions at Uranium Mill Tailings Remedial Action (UMTRA) Project Sites. Rather, the standards require the stabilization and control of the tailings to be effective for up to 1,000 years, to the extent reasonably achievable, and, in any case, for at least 200 years. For this long-term stability to be achieved, certain geologic performance objectives must be met. An evaluation of the potential geomorphic hazards is required, and DOE will show that potential geomorphic change will not affect the integrity of the disposal cell for its design life. The seismological characterization of the site will provide estimates of earthquake-induced ground accelerations that could occur at the site, as well as the potential for other types of tectonic hazards that could affect disposal cell performance. In addition, geological site characterization must demonstrate that future resource development will not adversely affect the disposal cell stability. Additional criteria that form the basis of the work described in this document and the evaluation of the adequacy of the site and regional geology are in the TAD (DOE 1989).

2.1 Scope of Work

Geologic, geomorphic, and seismic conditions at the site were investigated in detail. Geology was investigated according to procedures and approaches described in the TAD to gather the data specified in the NRC SRP and the Standard Format and Content guide. These investigations included, but were not limited to: (1) the compilation and analysis of published and unpublished geological literature and data; (2) the review and analysis of historical and instrumental seismic data; (3) geological field mapping and observations; (4) review of site-specific subsurface geologic and geotechnical data, including logs and samples from boreholes and coreholes, test pits, and analysis of recent and historical aerial photographs; and (5) studies of previous work. Details of the data gathering, interpretation procedures, and results are in the calculation sets referenced in this section and in Attachment 2.

2.2 Regional Geology

To provide a background for the detailed site geology and subsurface conditions, regional geologic conditions of the Crescent Junction Disposal Site in east-central Utah are described below. Most of this information is from maps and publications referenced in the following sections and in calculation sets in Attachment 2 of the Draft RAP. The site region is considered as the area within a 40-mi radius (based on a relevant seismic attenuation distance) of the disposal site. That area is used in analyzing seismologic stability, but a smaller area is generally discussed with respect to other geologic aspects.

2.2.1 Physiography

The Crescent Junction Site is in the north end of the Canyon Lands section of the Colorado Plateau physiographic province (Figure 2–1). The Canyon Lands section is characterized by deeply incised drainages, isolated mesas, gently dipping bedrock, and anticlines formed by salt intrusion that have been breached in places by erosion to form anticlinal valleys. North of the Canyon Lands section is the Uinta Basin section of the Colorado Plateau; the boundary between the two sections is the Book Cliffs, an erosional escarpment just north of the site. The Uinta Basin section is characterized by a rugged, intricately dissected plateau bounded on the south by sets of cliffs (one of which is the Book Cliffs) that are highly irregular with many salients and canyons (reentrants). Further physiographic subdivisions recognized in the State of Utah place the site in the Mancos Shale Lowland (Figure 2–1). Elevations in the site region range from approximately 3,900 to 12,000 ft.

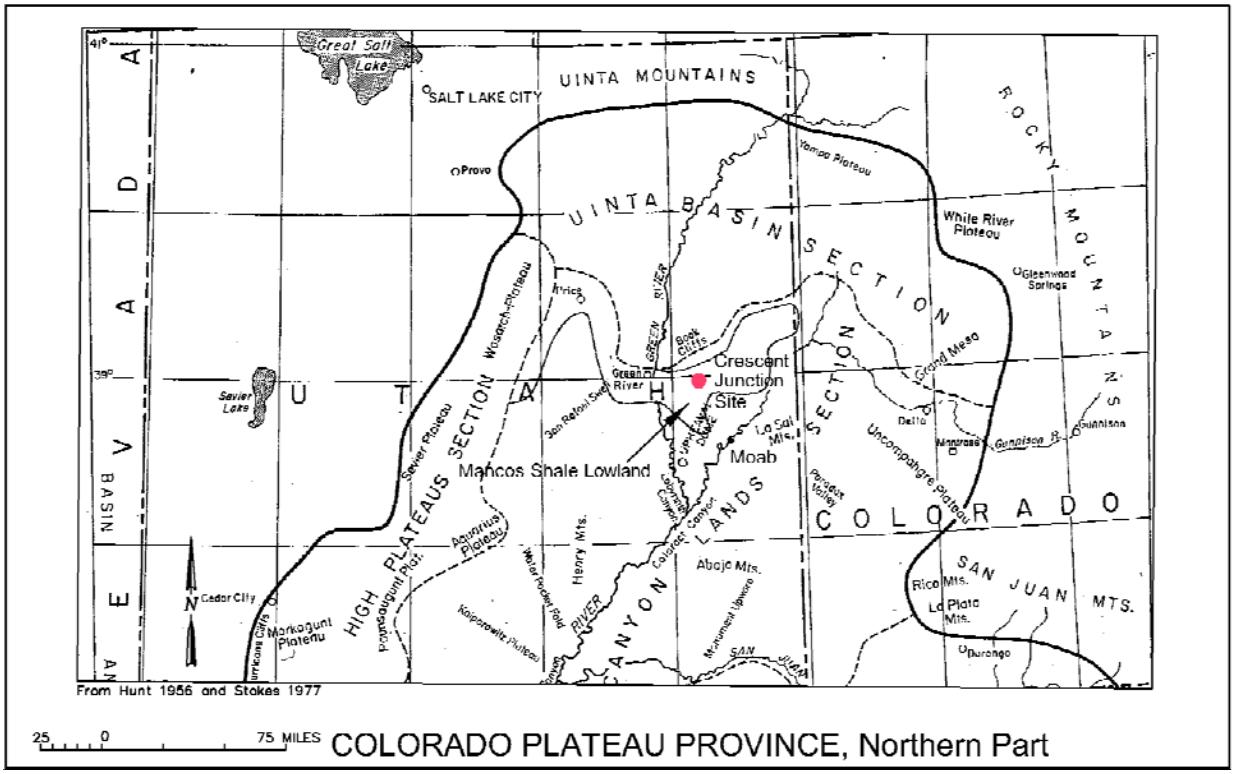

Figure 2-1. Physiographic Setting of the Crescent Junction Site

The main physiographic features of the immediate site area are as follows:

- Type of geomorphic surface that surrounds the site: The surface area of the site is on Crescent Flat—a gently south-sloping area between the base of the Book Cliffs to the north and Interstate Highway 70 to the south.
- General relief and topography of the site: The low-relief surface of Crescent Flat slopes gently southward for approximately two miles, from an elevation of about 5,100 feet to the north to about 4,900 feet to the south. Topography is controlled by the Mancos Shale, which underlies the Mancos Shale Lowland.
- Drainage system: Minor, slightly to moderately incised, ephemeral West and East Branches of Kendall Wash drain the disposal site area. The branches join and drain south into the ephemeral Thompson Wash, which joins ephemeral Tenmile Wash that drains into the Green River about 25 miles southwest of the disposal site area. Bordering Crescent Flat to the west, Crescent Wash is a larger ephemeral system that drains an approximately 22-square-mile (mi^2) area north of the site in the Book and Roan Cliffs.

- Major regional geomorphic processes: Significant processes are the retreat and rock falls associated with the Book Cliffs escarpment, aggradation across Crescent Flat associated with sheet wash from the base of the Book Cliffs, and incision and migration of minor drainage systems.

Additional details of the regional physiographic setting are in Attachment 2, Appendix C.

2.2.2 Stratigraphy

The regional geologic setting of the Crescent Junction Site is shown in the geologic map of east-central Utah in (Figure 2–2). A 5 to 10-mi-wide swath of outcrop of Mancos Shale of Late Cretaceous age corresponds to the Mancos Shale Lowland and the Crescent Junction Site. Rocks in the Lowland area of the site dip generally northward at low angles of less than 10 degrees toward the Uinta Basin. Regionally, approximately 4,000 feet of continental sedimentary rocks of Mesozoic age underlie the marine Mancos Shale, which also is about 4,000 feet thick. The part of the Mancos Shale underlying the immediate site area is about 2,400 feet thick. Above and north of the Mancos Shale in the Book Cliffs area are continental sedimentary rocks of the Mesaverde Group of Late Cretaceous age. Quaternary material consisting of alluvial and colluvial mud, stream alluvium, pediment-mantle deposits, talus, and colluvium mostly cover the Mancos Shale at the site area.

Descriptions and a stratigraphic column of the geologic formations of Mesozoic age that underlie the site and of the Mancos Shale and overlying Mesaverde Group of Late Cretaceous age are in Attachment 2, Appendix A. Also in this calculation set is a description of the unconsolidated Quaternary deposits.

2.2.3 Structural Setting

The Colorado Plateau, an intercontinental subplate with a greater crustal thickness than the adjoining provinces, provides a stable setting for the site. The plateau has been gradually uplifting since the Tertiary Period. Within the plateau, principal structural elements in the site region include the Uinta Basin, Paradox Basin, and Uncompahgre Uplift. The site is near the south edge of the Uinta Basin and in the northwest part of the ancestral Paradox Basin, where salt was deposited in Pennsylvanian time. Northwest-striking anticlines and synclines that formed as a result of movement of the deeply buried salt are in the north part of the Paradox Basin in what is called the Paradox Fold and Fault Belt. Additional description of the structural setting of the site and a map showing the regional structural elements are in Attachment 2, Appendix A.

2.2.4 Seismotectonics

Literature and database searches were the basis for a site-specific evaluation of the seismotectonic stability of the Crescent Junction Site. Results of the evaluation (included in Section 2.4.2) serve as input to the disposal cell design. Data, analyses, and references summarized in this section are included in Attachment 2, Appendixes E and F.

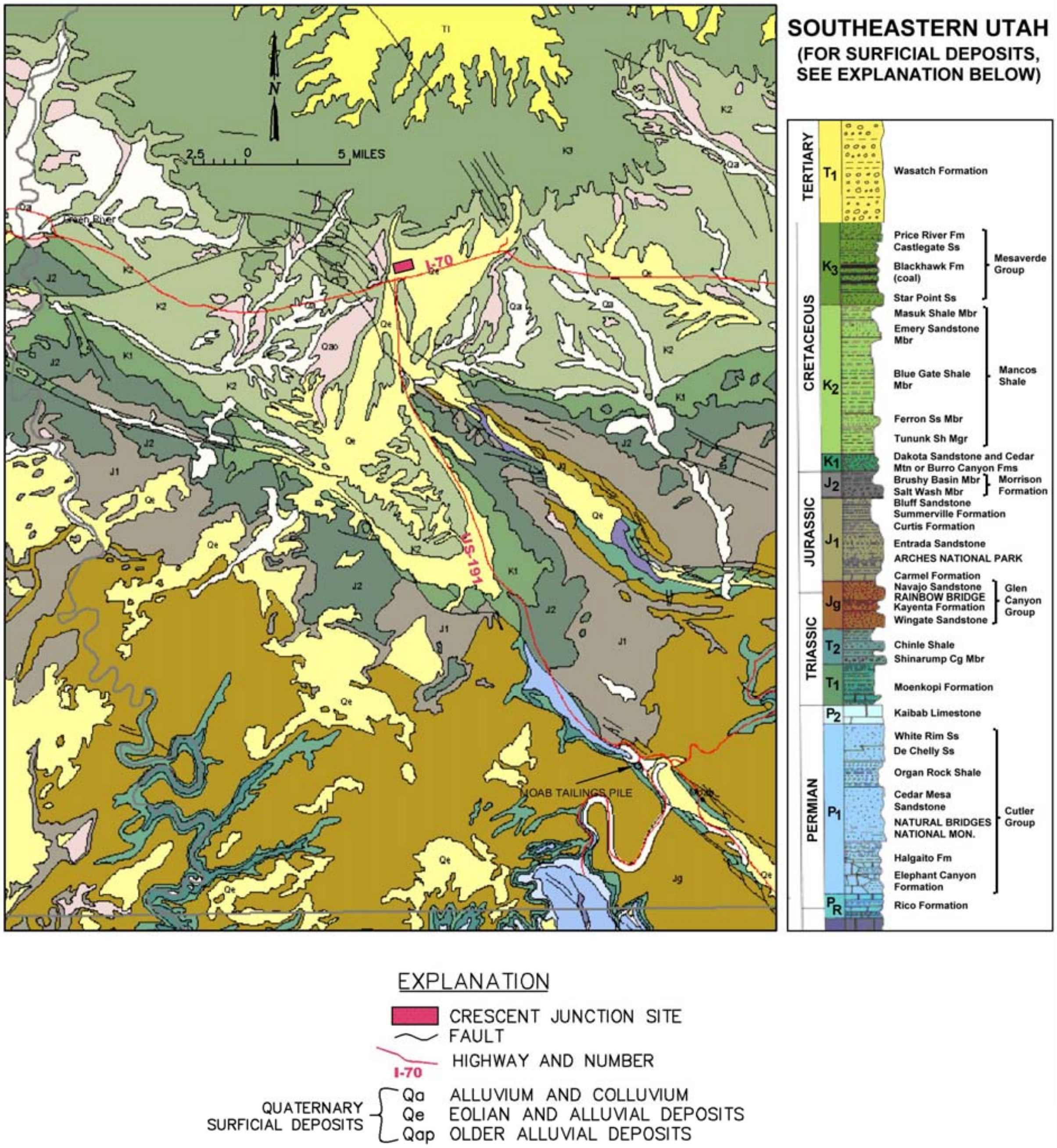

Figure 2-2. Regional Geology of the Crescent Junction Site

The Crescent Junction Site is in the Paradox Fold and Fault Belt of the Colorado Plateau tectonic province, which is relatively stable according to historical earthquake data and is considered to be inactive under the current tectonic regime. Surrounding tectonic provinces are more active and have higher-magnitude earthquakes. Historical earthquake data were compiled for all surrounding provinces, and literature estimates for maximum earthquakes were obtained for each province.

Data regarding known faults in the expanded study area were assembled. Fifteen faults (or fault zones) were identified as having potential to impact the site. Most of the faults and structural features in the study area are associated with salt deformation, dissolution, and collapse. Some of these structures may have had movement in the Quaternary, but the movement is very slow and unlikely to generate large earthquakes. Of the 15 faults, five were either determined active in the Quaternary or of unknown age. The remaining 10 faults were determined inactive in the Quaternary.

No evidence of faulting in the Crescent Junction area was observed during the photogeologic evaluation and follow-up field investigations. The only faults noted were outside the withdrawal area, which encompasses the Crescent Junction Disposal Site.

Peak horizontal ground acceleration (PHA) maps were obtained for both the United States and the State of Utah. These maps showed a range of estimated PHAs for the Crescent Junction area. Recent U.S. Geological Survey (USGS) maps (Frankel et al. 2002) show the peak acceleration to be 0.045 standard acceleration of gravity (g) with a 10 percent probability of exceedance in 50 years, and 0.12 g with a 2 percent probability of exceedance in 50 years. In contrast, Halling et al. (2002) estimated the PHA for the Crescent Junction Site to be approximately 0.5 g. However, this estimate is based on the assumption that the Tenmile Graben is an active structure, which is contrary to evidence presented by Woodward Clyde Consultants (1996). The seismotectonic study conducted for the nearby Green River, Utah, UMTRA Project Site recommended a design acceleration of 0.21 g based on a magnitude 6.2 floating earthquake (FE) occurring 15 kilometer (km) (9.3 mi) from the site. These literature estimates were considered further in the site-specific evaluation of the site (Section 2.4.2).

2.2.5 Resource Development

Historical geologic resource development and the potential for future development at the Crescent Junction Site and nearby region are evaluated and documented in Attachment 2, Appendix A. Geologic resources evaluated were those that, if exploited, could result in disturbance of the disposal site.

Geologic resources and their development potential identified in the site and nearby region are oil and gas, potash and salt, coal, uranium and vanadium, copper and silver, gold, and sand and gravel. These resources and their development potential are documented in both the Mineral Potential Report for the Moab Planning Area (north part of the Moab District, U.S. Bureau of Land Management) (Tabet 2005) and the Mineral Report on the DOE Proposed Disposal Site (Bain 2005). From those reports and the recent oil and gas leasing and drilling activity near the site, it is likely that the only geologic resources at the site that have moderate to high potential for economic development would be oil and gas.

The construction and presence of an approximately 230-acre disposal cell at the site would not preclude the exploration and development of oil and gas resources. Exploration by directional drilling could evaluate the presence of oil and gas directly beneath the disposal cell. Possible oil and gas production from beneath the disposal site at depths between 4,000 and 11,000 feet would not result in subsidence.

2.3 Site Geology

Bedrock geologic conditions at the site are characterized primarily to provide the basic information required for geotechnical stability evaluations (Section 4.0) and for ground water performance assessments (Sections 3.0 and 8.0). Surficial geologic conditions are characterized to establish the geomorphic history and processes at the site that determine if the long-term stability requirements will be met.

Geologic field investigations at the Crescent Junction Disposal Site included drilling of coreholes and geotechnical boreholes, and excavation of test pits. Ten coreholes were drilled to depths of approximately 300 feet into the Mancos Shale. Core samples were logged in the field using visual soil- and rock-classification procedures, and the coreholes were geophysically logged. One hundred geotechnical boreholes were drilled to depths of as much as 26 feet through the surficial unconsolidated material into the shallow weathered Mancos Shale, with samples logged in the field. Five test pits were dug with a trackhoe to investigate subsurface conditions to depths ranging from 15 to 23 ft. Logs for all subsurface investigations are in Attachment 5, Appendixes A, B, C, and D.

Aerial photographs (including high-altitude vertical and low sun-angle) of the area were produced to analyze structural and geomorphic conditions that may affect the site. Historic aerial photographs dating back to 1944 were also used in the analysis of site conditions. The Photogeologic Interpretation calculation is presented in Attachment 2, Appendix G.

The procedures used to characterize site geology and the details of that site characterization are in Attachment 2, Appendix B. Geomorphologic information is in Attachment 2, Appendixes C, D, and G. Brief descriptions of the salient geologic and geomorphic features are in the following sections.

2.3.1 Bedrock Geology

The site area is underlain by the Mancos Shale of Late Cretaceous age that dips gently (approximately 5 to 6 degrees) northward. The shale forms a broad, east-trending belt immediately south of the Book Cliffs. Topographically, the shale forms badlands that are the lower or buttressing part of the Book Cliffs and the wide expanse of lowlands, or “flats”, extend several miles to the south. Total thickness of the Mancos Shale, which generally represents the open-marine mudstones deposited in the Cretaceous Western Interior Seaway, is approximately 3,500 feet in the immediate site area as measured from the top of the Book Cliffs.

Most of the Mancos Shale is a monotonously uniform drab or bluish-gray shale; however, in the site area, which is in the upper third of the formation, an anomalously sandy interval, named the Prairie Canyon Member of the Mancos Shale, represents a period of near-shore deposition. From the sandy (generally very fine-grained) nature of this member, as exposed in a few outcrops, seen in several coreholes and test pits, and expressed as a marked reduction in the gamma ray geophysical log response from coreholes, the thickness of the Prairie Canyon Member in the mapped area is approximately 150 to 200 ft. As much as 150 feet of the Prairie Canyon Member is beneath the north edge of the proposed disposal cell. Underlying and overlying the sandy interval of the Prairie Canyon Member is the Blue Gate Member of the Mancos Shale. The Blue

Gate Member consists mainly of open-marine mudstone and shale, with a few thin siltstone layers. In the site area, the Blue Gate Member is divided into lower and upper parts to accommodate the Prairie Canyon Member. Outcrops of both lower and upper parts of the Blue Gate Member are rare—only one of each was found in the mapped area. A thickness of approximately 2,000 feet of lower Blue Gate Member is in the site area. Below the Blue Gate Member are the lowermost members of the Mancos Shale, the Ferron Sandstone Member underlain by the Tununk Shale Member, that combine for an approximate 300 feet to 400 feet thickness. It is therefore estimated that approximately 2,400 feet of Mancos Shale underlies the center of the proposed disposal cell.

Natural fractures are mostly in the top 50 feet of the weathered Mancos Shale bedrock. Below that, only a few fractures are in the competent bedrock, and no natural fractures were seen deeper than 100 feet into the bedrock. Characteristics of the weathered and unweathered zones of both the Prairie Canyon and Blue Gate Members of the Mancos Shale bedrock have been compiled from corehole lithologic logs and rock quality designation data; details are in Attachment 2, Appendix B. Hydrologic and transport properties of the Mancos Shale are discussed in Section 3.3.

No faults or evidence of faults (slickensides on fracture surfaces) were found in the deep coreholes. Additional evidence for lack of faulting in the site area is the continuity of the stratigraphic horizon composed of dolomitic siltstone concretion masses that mark the top of the Prairie Canyon Member. No evidence for displacement is seen along the line where the scattered dolomitic siltstone concretions crop out. Field investigation and aerial photograph interpretation have further ruled out faulting in the area that could impact the disposal site.

2.3.2 Surficial Geology

Nearly all of the disposal cell withdrawal area is covered by unconsolidated Quaternary material. These deposits cover Mancos Shale (Blue Gate or Prairie Canyon Members) bedrock and are typically about 10 feet to 12 feet thick, but can be as much as 25 feet. Most significant of the Quaternary deposits is gray alluvial mud, which consists mostly of silt and clayey silt that represents successive sheet wash deposits from erosion of Mancos Shale along the lower slopes of the Book Cliffs. A small amount of brown, sandy silt of eolian origin is in discontinuous layers in the alluvial mud. Also, sand to gravel to small boulder-sized material is at the base of the alluvial mud in a few swales and washes that were cut into the Mancos Shale bedrock. One such swale, slightly more than 20 feet deep, was found just southeast of the disposal cell footprint. No evidence of ground water was observed in any of the bedrock swales or surface washes.

Surficial deposits have been emplaced in a stable geologic environment mainly by a slow accumulation of material transported during infrequent heavy rainfall episodes from the base and sides of the Book Cliffs along active sheet wash paths. No evidence of faulting or displacement of Quaternary material is seen in the vicinity of the site.

2.3.3 Geomorphology

Results of literature research on the geomorphology of the site indicated that the site appeared to be suitable for disposal of the Moab uranium mill tailings (Attachment 2, Appendix C). Further site-specific field investigations supported this conclusion and showed that the landscape at

Crescent Flat is dominated by depositional (or aggradational), rather than erosional (or degradational), processes (Attachment 2, Appendix D).

Geomorphic processes in this area that may affect disposal cell performance include fluvial, mass movement, and eolian. Fluvial processes, related to the drainage system of the withdrawal area and the nearby surrounding area, will have the most significant effect on the site area that includes the proposed disposal cell. The other geomorphic processes investigated—mass movement and eolian—will likely have negligible effects on the disposal cell and nearby area. Mass movement processes of rock fall, landslides, and scarp retreat are confined to the Book Cliffs, which are far enough away (approximately 2,000 feet at the closest point) to not affect the disposal cell. Eolian processes, active in drier times earlier in the Holocene Epoch, are not expressed at the site and apparently will not affect the site unless the climate becomes drier. Fluvial processes are discussed below and the potential for rock falls is considered in the next section on geologic hazards.

Long-term incision advance of the tributaries of the West Branch of Kendall Wash has the greatest potential of fluvial erosion processes to affect the disposal cell. Headward incision northward at a rate measured from historical aerial photographs of an eastern tributary to the West Branch could reach the southwest corner of the disposal cell in about 500 years. Increased flows in the drainage created by channeling of several drainages around the west side of the disposal cell will accelerate headcutting and shorten the time for erosion to reach the disposal cell corner. This drainage path was included in the engineering design of the disposal cell to mitigate this headward erosion.

The tendency of Crescent Wash to migrate eastward toward the disposal cell is low because the wash channel will likely soon follow an incipient cutoff channel, resulting in a straightening of the wash course. Long-term incision advance of a tributary of the West Branch of Kendall Wash could capture the Crescent Wash drainage after approximately 1,600 years. At that time, the high-energy Crescent Wash channel could then be about 1,000 feet west of the disposal cell—probably far enough away not to pose an erosion threat to the cell.

Erosional incision advance of the present East Branch of Kendall Wash resulted in capture of an earlier drainage thousands of years ago. Incision advance of this wash and its tributaries will continue, but this erosion is 0.5 to 1.0 miles or more east of the disposal cell and will not affect the site.

2.3.4 Geologic Hazards

Potential geologic hazards in the vicinity of the disposal site include mass movement processes, such as rock fall, landslides, and scarp retreat. These processes are confined to the Book Cliffs, which are far enough away (approximately 900 feet at the closest point) to not affect the disposal cell. Swelling clay in the Mancos Shale also poses a potential but manageable and acceptable risk, as does the presence of radon in the Mancos Shale. These potential hazards are summarized below and discussed in more detail in Attachment 2, Appendix A.

The Mancos Shale formation can exhibit characteristics of moderate swelling, due to the possible presence within the shale of expansive clays and thin gypsum lenses, which expand when hydrated. Though possible, expansion of the shale is not considered to be problematic for the following reasons:

a) The shale formation has extremely low hydraulic conductivity, and though the top surface of the shale will be wetted during the time when tailings are being placed and later as excess capillary water migrates to and along the cell floor, the water will not migrate very far into the shale formation. The thickness of the shale being wetted is not likely more than 1 to 2 feet and the volume of expansive clay or gypsum in that thin layer of shale cannot expand enough to be of consequence. For example, if two feet of shale is hydrated, and 25 percent of the two feet thickness is expansive material, and the expansive material expands 50 percent (typical for some types of gypsum), the total expansion would be three inches.

b) Minor expansion, if it occurs, will take place when the Mancos shale is initially wetted. At that point, the cell is being excavated and the first layers of tailings are being placed. There will not be anything in place at that point that could be damaged by minor soil movement. Damage from soil expansion and contraction tends to occur when a sensitive structure such as a building or highway undergoes differential movement. The disposal cell is not a sensitive structure, especially in the early stages of cell excavation and tailings placement.

c) Expansion and/or contraction of expansive soils takes place when significant changes in moisture content occur. When moisture content is relatively constant, expansion and/or contraction does not occur. A relatively thin layer of Mancos shale may expand when initially hydrated, but once several feet of tailings have been placed over the shale, the moisture content at the cell floor should remain relatively constant. Whether the cell eventually dries out or has some residual moisture at the cell floor long-term, it should not be subject to moisture fluctuations that would result in significant cycles of expansion and contraction.

Rock-fall debris covers some of the badlands slope as talus along the south side of the 800 foot high Book Cliffs. The dislodged rock is sandstone from the Blackhawk Formation and Castlegate Sandstone, both of the Mesaverde Group, which cap the Book Cliffs. An empirical investigation was conducted to evaluate how far this rock-fall material could run out along the base of the Book Cliffs and if it could affect the disposal cell. Based on two profiles near the northeast part of the disposal cell (closest to the Book Cliffs), and with the source of rock fall starting near the base of the Blackhawk Formation at an elevation of approximately 5,700 ft, the distance from the empirical rock-fall runout limits to the edge of the disposal cell footprint is approximately 2,000 ft. This is far enough north away from the disposal cell and any infrastructure or access roads to not pose a rock-fall hazard. Slow scarp retreat (estimated at five feet per 1,000 years) northward of the Book Cliffs over time will continue to reduce this hazard to the disposal cell.

Landslides, mainly on northerly-facing slopes below the Blackhawk Formation/Castlegate Sandstone cap of the Book Cliffs, are just north of the withdrawal area. In general, these landslides are very old, are no longer active, and apparently formed in much wetter climatic conditions during the Pleistocene. During these wetter conditions, small landslides formed even on the south-facing slopes of the Book Cliffs, where several remnants of inactive landslides remain.

Literature review and site test data indicate that swelling clays if present will not impact performance of temporary structures (such as access roads) and permanent structures (cell embankments and cover). Concerns are described in Attachment 2 Geology and in response to comments Addendum A Response to NRC Comments.

The site area has a moderate to high radon-hazard potential for occurrence of indoor radon based on the geologic factors of elevated uranium concentration in the Mancos Shale, soil permeability, and ground water depth. No permanent structures are planned for the disposal site; therefore, high indoor radon concentration will not be a problem.

2.4 Geologic Stability

This section identifies local geologic and seismic conditions that could affect the geologic stability of the disposal cell and the long-term stability of the landscape environment. This section demonstrates that geomorphic processes will not impact the long-term stability of the disposal cell. Potential geologic events, including seismic shaking, liquefaction, and on-site rupture, are ruled out as disturbing forces on the disposal cell either because they will not occur or because the cell is designed to withstand such geologic occurrences.

2.4.1 Geomorphic Stability

DOE provides evidence of the long-term geomorphic stability of the site in Attachment 2, Appendixes C, D, and G. The landscape is dominated by slow depositional processes. The fluvial-geomorphologic features identified at the site pose little risk to the disposal cell. However, sheet wash coming onto the site from the north will have to be redirected to the west around the disposal cell, and the northward advance of headward incision of the West Branch of Kendall Wash will have to be monitored.

Based on these evaluations, DOE concludes that the site is geomorphically stable and will continue to be so for the performance period of the remedial action.

2.4.2 Seismotectonic Stability

A site-specific analysis determined a maximum credible earthquake (MCE) and a corresponding design acceleration. The MCE for the design earthquake was determined according to the steps in the SRP (NRC 1993). That process is described below with a summary of results. Data and specific methods, calculations, and references used in the analysis are in Attachment 2, Appendix F.

Step 1. Floating Earthquake (FE)

An FE magnitude of 6.2 was considered in the seismotectonic analysis of both the Green River, Utah, and Grand Junction, Colorado, UMTRA Project Disposal Sites. Based on a statistical evaluation using historical earthquake data for the Colorado Plateau, a recurrence rate of having a 6.2-magnitude event within 15 km (9.3 mi) of the site was estimated at 77,000 years. The probability of this magnitude being exceeded within the 1,000-year design life for the disposal cell is one percent. A 6.2 magnitude FE for the site was therefore chosen as a conservative estimate for an MCE. Assuming that an FE of magnitude 6.2 occurs within 15 km (9.3 mi) of the site, the PHA for the site was calculated at 0.22 g. This was used as the point of comparison for the rest of the analysis.

Step 2. MCE Associated with Outlying Tectonic Provinces

Following the methodology in the SRP (NRC 1993), literature MCEs for each of the tectonic provinces surrounding the Colorado Plateau were obtained. An MCE was assumed to occur at a point closest to the site in each province; corresponding PHAs for the site were determined. All of these PHA values for surrounding tectonic provinces were less than that for the Colorado Plateau. Therefore, the FE for the Colorado Plateau of magnitude 6.2 is retained as the design earthquake.

Step 3. Identification and Analysis of Capable Faults

Faults known to be active during the Quaternary Period (Quaternary faults) within the expanded study area (and known faults of indeterminate age) were screened based on lengths and distance from the site to identify actual faults with the potential to generate a PHA of >0.1 g as the result of an MCE. Fifteen faults were further analyzed to determine likelihood of movement and the potential effects at the site. Six faults had PHAs exceeding the FE PHA of 0.22 g. All of these faults were determined to be not active in the Quaternary; and five were determined to be related to salt-dissolution subsidence. None of the six are considered potential design faults. Of the faults considered active in the Quaternary, the highest calculated PHA is 0.13 g. Therefore, the FE for the Colorado Plateau of magnitude 6.2 is retained as the design earthquake.

Step 4. Designation of MCE

The seismotectonic analysis concluded that the greatest impacts at the site would likely come from an FE as opposed to an earthquake generated by a known fault. Therefore an earthquake of magnitude 6.2 occurring at a distance of 15 km (9.3 mi) from the site was recommended as appropriate for the site with a corresponding PHA of 0.22 g.

Specific seismic parameters were used in conjunction with appropriate soil strength parameters, disposal cell geometry, and ground water information to assess slope stability and liquefaction potential.

- Long-term slope stability seismic coefficient is 0.15 (2/3 of PHA).
- Short-term slope stability seismic coefficient is 0.11 (1/2 of PHA).
- Liquefaction analysis: ground surface horizontal acceleration is 0.22 g.

2.5 Geologic Suitability

Based on the site characterization summarized in this section and included in Attachment 2, the details of the Final RAP, and the provisions for stability included in the design of the disposal cell, DOE concludes that there is reasonable assurance that the regional and site geologic conditions have been characterized adequately to meet the requirements in 40 CFR 192.

Results of literature research on geologic and geomorphologic characteristics indicate that the Crescent Junction Disposal Site is apparently suitable for the Moab RRM (Attachment 2, Appendixes A and C). The approximately 2,400 foot thickness of Mancos Shale beneath the disposal cell effectively isolates it from deeper strata that contain ground water (Dakota aquifer). Although faults are present within several miles of the site, they represent adjustments by slow subsidence to the process of dissolution of deeply buried, thick salt deposits. None of the faults appear to have displaced Quaternary surficial deposits, suggesting that significant offset occurred prior to the Quaternary Period.

Geologic investigations in and immediately surrounding the disposal cell footprint found no potential deficiencies in geologic conditions that would adversely affect the geologic suitability of the site. No evidence for faults was seen on the surface or in the subsurface from boreholes.

The two mile long unbroken segment of the Book Cliffs escarpment just north of the site is supportive evidence for lack of faulting in the immediate site area. Core from all the deep boreholes were dry when broken open, indicating lack of saturation in the Mancos Shale bedrock. No natural fractures were noted below a depth of 100 feet into bedrock; most fractures were in the top 50 feet of bedrock, representing the weathered Mancos Shale.

Use of the area as a disposal cell would not preclude the recovery of the only resource that has moderate to high potential for development—oil and gas, which could be explored and recovered (if present) by directional drilling.

The landscape at the disposal site is dominated by depositional (aggradational), rather than erosional (degradational), processes. The fluvial-geomorphological features at the site pose little risk for a disposal cell. Sheet wash from the north will be redirected westward and eastward around the disposal cell by the construction of the wedge. The northward advance of headward incision of the West Branch of Kendall Wash will have to be monitored. The incised channel of Crescent Wash shows little historic or future tendency to migrate eastward toward the disposal cell footprint.

3.0 Ground Water Hydrology

3.1 Hydrogeologic Investigation

The hydrogeologic investigation consisted of characterizing the physical and geochemical properties of the hydrogeologic units and documenting water use at the Crescent Junction Disposal Site. Major points are summarized below. Detailed commentary on the hydrogeologic characterization is provided in Attachment 3.

3.2 Identification of Hydrogeologic Units

The Crescent Junction Disposal Site is underlain by alluvial and colluvial material whose thickness is variable, ranging from a trace to nearly 25 feet in places. This material was deposited in shallow swales and washes that were carved into the weathered Mancos Shale. Under current climatic conditions, none of the shallow swales or washes contain ground water.

The alluvial and colluvial materials are underlain by the Mancos Shale, which is approximately 2,400 feet thick below the site and forms an important regional confining unit. The Mancos Shale is composed of calcareous shale, mudstone, and claystone that contains thin sandstone lenses, interbedded siltstone, and zones of limestone concretions and dolomite or limestone beds. These fine-grained rocks have very low permeabilities and inhibit infiltration of precipitation (Hood 1976). The Mancos Shale forms a massive barrier to horizontal and vertical ground water movement (Freethey and Cordy 1991).

Minor quantities of ground water are present in the Mancos Shale at depths that exceed 100 feet. The ground water is very saline to briny with total dissolved solids (TDS) concentrations ranging from 23,000 milligrams per liter (mg/L) at well 0208 to 42,000 mg/L at wells 0201 and 0204. At these TDS concentrations, the State of Utah designates the ground water in the Mancos Shale to be *Class IV-Saline Ground Water* (Utah State Code, R317-6-4, Ground Water Class Protection Levels). Primarily on the basis of its salinity, this ground water is believed to be connate (Freethey 2006, personal communication) and therefore, very old and unconnected to deeper, regional aquifer systems. It also appears to be disconnected from sources of freshwater recharge and acts as a confining layer. The zone of connate water at the Crescent Junction Disposal Site is not considered an aquifer.

The uppermost aquifer beneath the Crescent Junction Site is the Dakota aquifer, which underlies the Mancos Shale confining unit, approximately 2,400 feet below the ground surface. A schematic diagram of the hydrogeologic units that underlie the Crescent Junction Site is presented in Figure 3–1. The Dakota aquifer is composed of the Dakota Sandstone and the Cedar Mountain Formation. Published accounts of drill holes advanced to the Dakota aquifer within a radius of approximately 20 miles of the Crescent Junction Disposal Site indicate that the ground water is mostly salty (Sumsion 1979). Ground water samples from the Dakota aquifer were not obtained as part of this project because of the great depth at which the aquifer occurs.

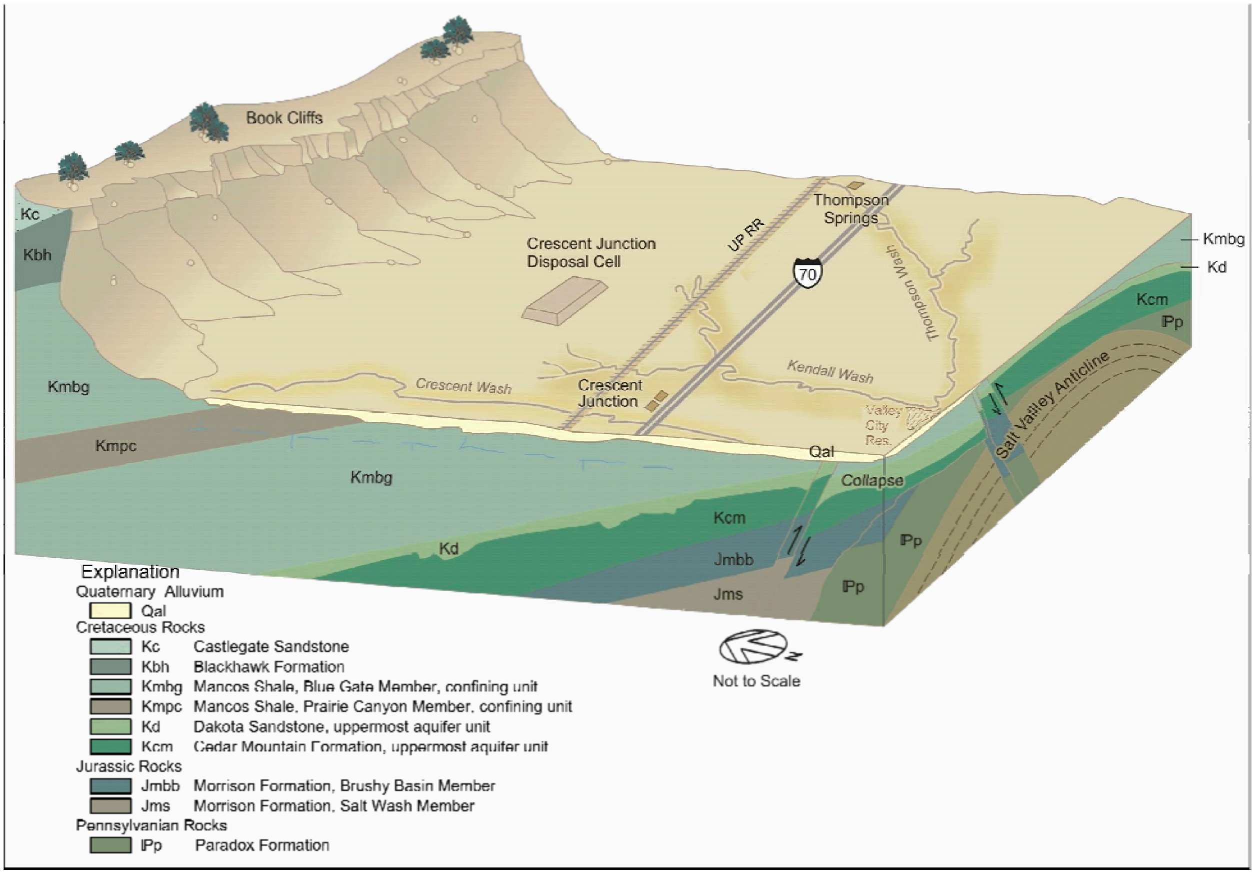

Figure 3-1. Schematic Block Diagram Depicting the Major Hydrogeologic and Topographic Features at the Crescent Junction, Utah, Disposal Site

3.3 Hydraulic and Transport Properties

The Dakota aquifer is recharged by infiltration of runoff and precipitation along the southern flank of the Uinta Mountains, where the aquifer units are exposed. As presented in Figure 3–2, these exposures occur near the town of Vernal, Utah, approximately 100 miles north of the Crescent Junction Disposal Site. From there the ground water in the Dakota aquifer flows in a southerly direction beneath younger hydrogeologic units that comprise the Uinta Basin. The Crescent Junction Disposal Site is located south of the Uinta Basin, where the Cretaceous-age aquifer beds emerge after being buried deeply beneath the Uinta Basin. Sedimentary beds belonging to the Dakota aquifer are exposed at the land surface approximately six miles south of the Crescent Junction Disposal Site, where they are brought to the surface by upwarping caused by the Salt Valley Anticline (Figure 3–2). Ground water discharge from the Dakota aquifer, which could occur as springs or zones of enhanced evapotranspiration along the flanks of the Salt Valley Anticline, was not observed during the field investigation except for one area in Sections 29 and 32, T22S, R21E, approximately 13 miles southeast of the site.

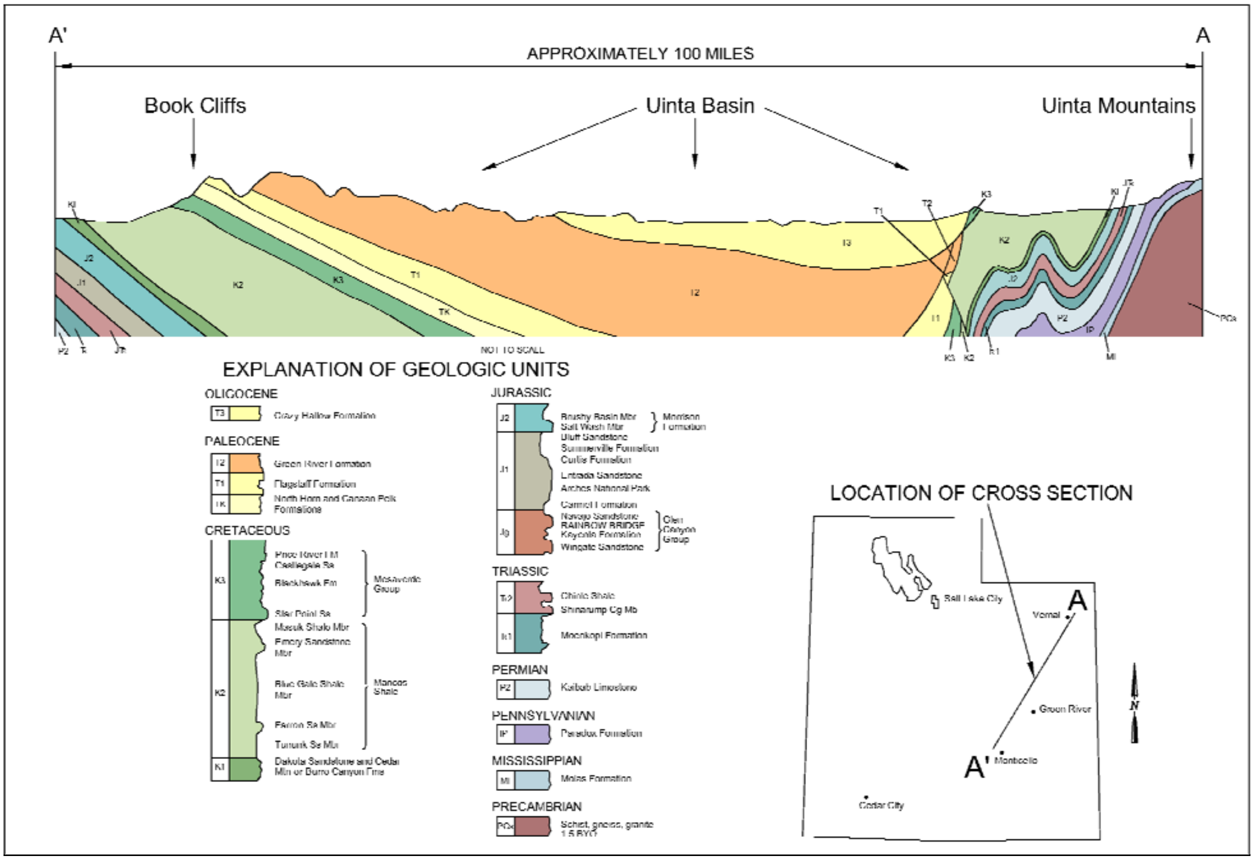

Figure 3-2. Regional Scale Cross Section Depicting Hydrogeologic Elements, Crescent Junction Site, Utah (modified from Hintze et al. 2000)

Hydrologic tests have shown that hydraulic conductivities decrease with increasing depth in the Mancos Shale. Within the weathered zone of the Mancos Shale the horizontal and vertical hydraulic conductivities were found to be approximately 2×10^{-3} centimeter per second (cm/s) and 1×10^{-4} cm/s, respectively. Within the more competent, unweathered Mancos Shale the geometric mean of all measured hydraulic conductivities was approximately 3.5×10^{-8} cm/s. The vertical travel time for ground water to migrate through the Mancos Shale to the Dakota aquifer is conservatively estimated to range from 3,330 to 33,300 years (Attachment 3, Appendix E).

3.4 Geochemical Conditions

The Crescent Junction Disposal Site is located in an area where geochemical processes are likely to attenuate the concentrations of ammonia and uranium (the main constituents of concern in the tailings pile fluids), which might leach from the disposal cell. The chemical retardation of ammonia is anticipated to occur primarily through ion exchange with sodium, potassium, calcium, and magnesium. Most of the ion exchange is projected to involve sodium, which dominates the cation population in the briny connate ground water underlying the site. Uranium is expected to precipitate from solution as it migrates slowly into the deeper recesses of the Mancos Shale. Geochemically reducing conditions are very likely to exist at increasing depth below the surface because of the anoxic conditions imparted by gaseous hydrocarbons and carbonaceous shale. Pockets of natural gas were encountered during the drilling conducted as part of this project. Commercial exploration for oil and gas has been, and continues to be, common in the Crescent Flat area. Based on these conditions, the Mancos Shale beneath the Crescent Junction Site is expected to naturally attenuate any dissolved chemical species in tailings leachate that would be harmful to human health and the environment. Details of the geochemical attenuation modeling and the background ground water quality are presented in Attachment 4, Appendix B, and Attachment 5, Appendix H of the RAP, respectively.

Coupling the results of the geochemical attenuation modeling with the vertical travel time calculations presented Attachment 3, Appendix E yields a more useful evaluation of the impact of chemical attenuation at the Crescent Junction Site. Based on this evaluation, the geochemical attenuation would retard the downward migration of these constituents resulting increasing the vertical travel times to the Dakota aquifer by a factor of 1 to 3.

3.5 Water Use

There are no private or municipal wells within two miles of the Crescent Junction Disposal Site. Figure 3–3 illustrates the occurrence of water resources in the Crescent Junction area.

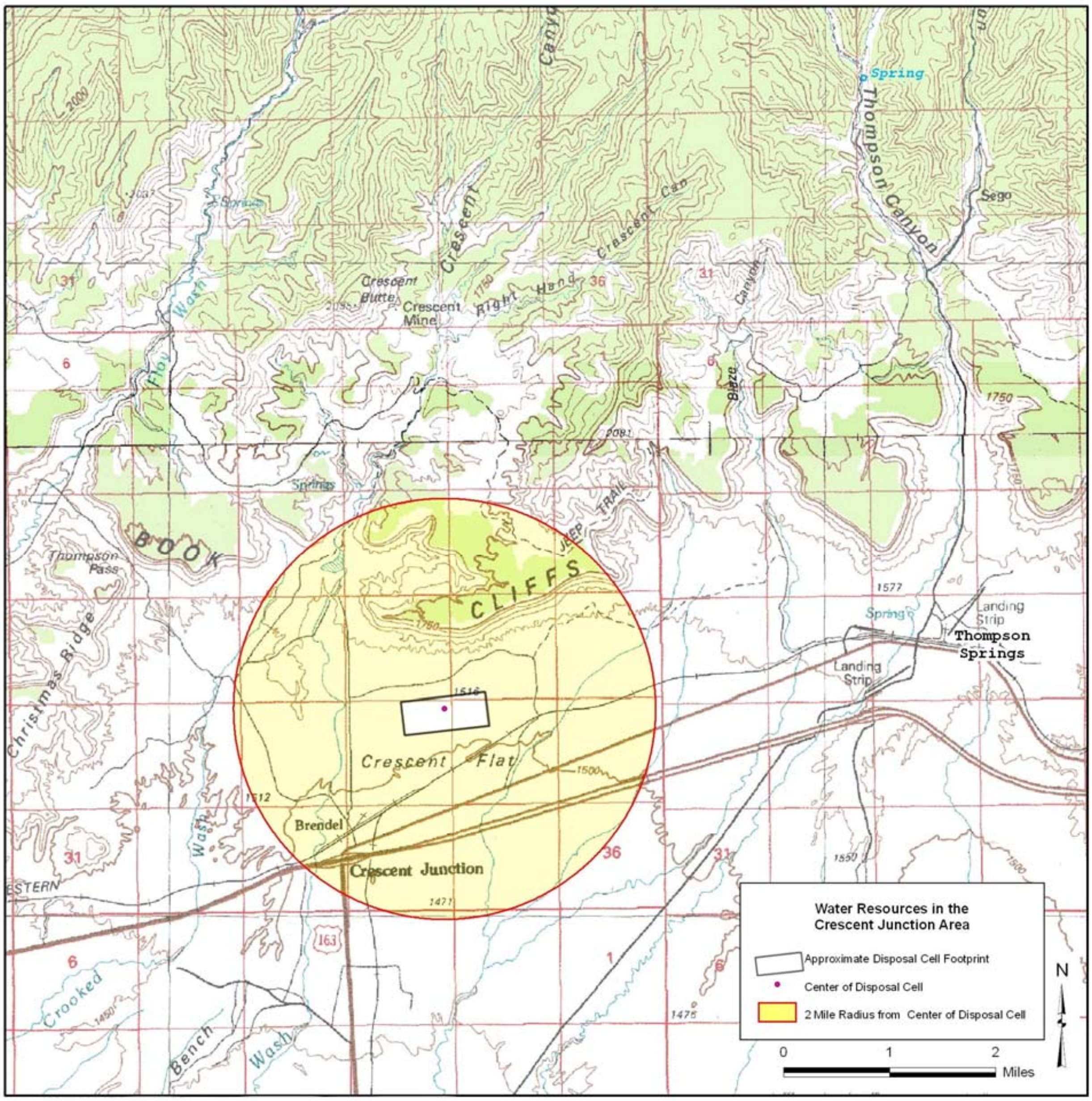

Figure 3-3. Water Resources in the Vicinity of Crescent Junction, Utah

The nearest municipal water supply to the Crescent Junction Disposal Site is in Thompson Canyon, located approximately seven miles north of Thompson Springs, Utah. The springs in this area yield approximately 20 gallons per minute (Sumsion 1979) from a carbonaceous shale layer near the top of the Neslen Formation (Willis

1986), which is a part of the Cretaceous Mesaverde Group. The springs constitute the sole source of potable water in the immediate area. In 2006, DOE installed a new three inch water line, which extends from Thompson Springs and serves residential and commercial customers in the vicinity of the Crescent Junction Disposal Site. A water pipe-line to provide construction water for the disposal cell will be installed prior to cell construction. This non-potable supply is planned to be abandoned following the completion of construction.

End of current text

4.0 Geotechnical Stability

This section and associated reference documents describe the geotechnical engineering aspects of the remedial action. The following aspects of the remedial action are described: the geotechnical information and design details related to the disposal site, the disposal cell and cover, and the properties of the soil materials. Materials described include the foundation and excavation materials and the RRM. Related geological aspects such as geology, geomorphology, and seismic characterization are presented in Section 2.0 of this document.

4.1 Site and Material Characterization

4.1.1 Geotechnical Investigations

Geotechnical investigations were performed at both the Crescent Junction Disposal Site and the Moab Processing Site to define the occurrence and engineering properties of the subsurface materials. Data obtained from these investigations are presented in Attachment 5. Subsurface information was obtained from test pits, boreholes, coreholes, surface geophysical investigations (seismic refraction), and laboratory testing. Each of the test-pit and test-hole locations were continuously observed or logged by a field engineer or geologist.

The subsurface investigation program at the Crescent Junction Disposal Site began in August 2005 with the excavation of two test pits (0151 and 0153) that were advanced through the Quaternary overburden material into the first several feet of the weathered Mancos Shale. The initial test pits were backfilled immediately after they were logged and sampled. Remaining test pits (0152, 0154, and 0156) were excavated and sampled in October and November 2005 and were left open for future inspection by interested stakeholder groups. Logs of the test pits are presented in Attachment 5, Appendix D. Bulk samples collected from the test pits were used to determine material classification, compaction characteristics, hydraulic properties, and strength properties. Results of the geotechnical testing are presented in Attachment 5, Appendixes E and K.

During September through November 2005, the geotechnical investigation of the Crescent Junction Disposal Site continued with the drilling of 100 soil borings within and immediately beyond the conceptual footprint of the disposal cell. These borings were advanced to the depth of practical refusal, which was in the first several feet of weathered Mancos Shale. Drive samples were collected using a Modified California Sampler and a 140-pound hammer falling 30 inches. A registered geologist recorded the blow-count data and made provisional classifications of the soils at the time of drilling. Logs of the geotechnical boreholes are presented in Attachment 5 Appendix B. The soil samples were temporarily stored on site and transported at regular intervals to the geotechnical testing laboratory. Temperature monitoring at the temporary storage area revealed that the samples were not exposed to freezing conditions prior to being transported offsite. Results of the geotechnical testing are presented in Attachment 5, Appendix E.

Between August and December 2005, a total of 10 coreholes (0201 through 0210) were advanced to a depth of 300 feet below the land surface, tapping into the firm, unweathered portions of the Mancos Shale. The coreholes were drilled by advancing conventional soil borings to refusal in the top several feet of weathered bedrock, coring 15 feet beyond the refusal depth and cementing surface casing to that depth, attaching a typical oil-field blow-out preventer to the top of the surface casing, and coring to a depth of 300 feet in the Mancos Shale. Conventional geotechnical soil sampling was performed in the unconsolidated soil zone, and continuous HQ size (3.38 inches) core was obtained from the bedrock. Three additional, shallow coreholes (0211 through 0213) were drilled to a maximum depth of 42 feet into the weathered Mancos Shale for hydrologic testing. Logs of the coreholes are presented in Attachment 5, Appendix A. Under the direction of the site geologist, the rock coring was conducted using an air-water mist to minimize the introduction of foreign fluids into the rock formation. Accumulated fluids, which included formation water in some coreholes, were periodically air lifted out of the advancing hole. Natural gas was detected in several of the coreholes as they were being drilled; however, highly pressurized gas pockets were not encountered at the site. Samples from the coreholes were analyzed for geochemical characteristics (i.e., soluble mineral species, x-ray-diffraction, distribution coefficients, and sequential batch leaching) and these results were developed into a reactive transport model (Attachment 4, Appendix B). Borehole geophysical logs, which included optical and acoustical televiewer, caliper measurements, compensated density, neutron logs, induction resistivity, natural gamma, and rock quality designation, are found in Attachment 5, Appendix C.

In October and November 2005, seismic refraction was used to characterize the rippability of the subsurface materials at the Crescent Junction Site. Orthogonal seismic refraction lines were established at coreholes 0202, 0204, 0206, 0207, and 0208. Each seismic line was 500 feet long and geophones were spaced at approximately 10 foot intervals. Three velocity zones were identified in the subsurface: (1) alluvial overburden with an attendant shear wave velocity of approximately 1,200 to 1,300 ft/s, (2) weathered Mancos Shale with an attendant shear wave velocity of approximately 4,100 to 5,200 ft/s, and (3) competent Mancos Shale with a shear wave velocity of approximately 9,000 to 10,000 ft/s. Based on the seismic shear wave velocity, the weathered Mancos Shale is considered rippable with a dozer with at least 300 horsepower (D8) with 50,000 pounds pry out force on a single point ripper. Details of the seismic refraction analysis are presented in Attachment 5, Appendix G. In October 2007, three test pits were excavated with a tracked excavator to confirm shale rippability. The Mancos Shale was found to be rippable with an excavator cutting in one direction to the depth of the proposed disposal cell floor.

During August 2005 through December 2005, geotechnical borings, test pits, and cone penetrometer test (CPT) soundings were advanced into the tailings pile material at the Moab Processing Site. A total of 24 boreholes (0700 to 0723) were advanced to a maximum depth of 96.5 feet below the surface; twelve test pits (0621 to 0632) were dug to a depth of 20 feet below the surface; and 15 CPT soundings with pore-pressure dissipation tests (0381 through 0395) were advanced to a maximum depth of 81.9 feet below the surface. Logs of the geotechnical borings and test pits are presented in Attachment 5, Appendix I. Results from the cone penetration tests are presented in Attachment 5, Appendix F. Soil samples from the tailings characterization were classified for index properties, hydraulic properties, and strength properties. Results of the geotechnical tests are presented in Attachment 5, Appendixes J and N. These results were used to develop preliminary materials-handling recommendations, and to ascertain the volume and weight of the tailings (Attachment 1, Appendixes I and J).

4.1.2 Disposal Site Stratigraphy

Unconsolidated Quaternary material that reaches a maximum thickness of approximately 23 feet covers most of the disposal site. These deposits cover Mancos Shale bedrock, which has a thickness of approximately 2,400 feet beneath the center of the disposal cell.

The Quaternary deposits are typically 10 feet to 12 feet thick and consist mainly of alluvial mud and lesser amounts of eolian material and coarse deposits in a few swales. Alluvial mud deposited by sheet wash is mostly silt and clayey silt, and highly calcareous. Eolian material is mostly sandy silt that occurs in thin, discontinuous layers in the lower part of the alluvial mud deposits. Coarse material that consists of sand, gravel, and small boulders occurs in a few places at the base of the alluvial mud where channels or swales have been cut as deep as 20 feet into Mancos Shale bedrock.

The Mancos Shale consists of the Blue Gate Member in the south part of the site overlain by the Prairie Canyon Member in the north part of the site. The Blue Gate Member consists mostly of mudstone, and the Prairie Canyon Member contains some layers of very fine-grained sandstone and siltstone in addition to the mudstone. The top 50 feet of Mancos Shale bedrock is weathered; the top 10 to 30 feet is most weathered and contains abundant natural fractures that are typically coated or filled with gypsum (and some calcite). Fractures are rare below a depth of 50 feet into the Mancos Shale and are absent below a depth of 100 feet into bedrock.

Materials that will be used in construction of the disposal cell cover (including the radon barrier) will be obtained from the disposal cell excavation. Modeling using data collected from samples of weathered Mancos Shale indicates that these materials will meet the cover design criteria required by the TAD (DOE 1989).

The disposal cell floor elevation was determined will be excavated a minimum of two feet into the weathered and fractured Mancos Shale. As described in Section 3.3, the weathered and fractured Mancos Shale has hydraulic conductivities of 10^{-4} to 10^{-3} cm/s. The cover system constructed on the disposal cell will have hydraulic conductivities significantly lower than the subsoil values, thereby meeting the requirements of 40 CFR 264.228 to prevent “bathtubbing”.

4.2 Geotechnical Engineering Evaluation

This section and referenced supporting documents present the geotechnical engineering evaluation of the information and analyses that have been undertaken to demonstrate that the remedial action will meet relevant EPA standards for long-term disposal cell stability. Information and analyses that have been performed include slope stability, settlement and cover cracking, and liquefaction analyses. Specific calculation sets that discuss information and present numerical analyses are listed in and included in Addendum D. Analyses are performed for design-basis events such as the design earthquake (Attachment 2, Appendix F), the design flood arising from the Probable Maximum Precipitation (PMP) (Attachment 1, Appendix E), and extreme meteorological conditions.

4.2.1 Slope Stability

The proposed disposal cell will be partially below and partially above the existing ground surface. A clean fill embankment will be constructed around the perimeter of the disposal cell to form the sides of the above ground section of the disposal cell. A multi-layer cover, eight feet thick, will be placed over the RRM and will extend over the perimeter embankments. The stability of the perimeter embankments and the UMTRA cover is important for maintaining long-term containment.

The slope stability analyses are presented in Addendum D, Calculation C-10. These analyses show that for both static and dynamic conditions, the cell foundation, the slopes of the disposal cell, and the cover system will not fail or otherwise adversely affect the disposal cell. The most critical slope section was analyzed for both short-term (end-of-construction) and long-term conditions. The following is a brief description of the work done to support these conclusions.

Material Properties

Material properties (Table 4–1) used in the stability analysis were obtained from borings, laboratory test results, and previous analyses. No ground water table is present at Crescent Junction and none was included in the analysis.

Table 4-1. Material Properties Used in Stability Analysis

Material	Unit Weight			Shear Strength			
				Total		Effective	
	Dry (pcf)	Moisture Content (%)	Moisture (pcf)	Friction Angle (degree)	Cohesion (psf)	Friction Angle (degree)	Cohesion (psf)
UMTRA Cover	111	11.7	124	26	0	26	0
Tailings	98	17.4	115	0	615	32	0
Dike Fill	111	11.7	124	19	0	26	0
In-situ Overburden Material-ML	92	6.7	98	26	0	26	0
Weathered Mancos Shale	104	7.3	112	25	0	25	0

Critical Slope Geometry

A section of the disposal cell south embankment with the greatest height along the perimeter embankment represents the critical slope, and has the following slope geometry:

- Existing ground surface slopes from north to south.
- Ground surface elevation: inside the cell (north) = 4,954 feet.
- Ground surface elevation: outside the cell (south) = 4,944 feet.
- Top of embankment elevation: 4,967 feet.
- Cover material nine feet thick with top elevation: 4,978’ – 4982’ at the analysis location.
- Water surface was not used.
- Embankment exterior slope was configured at 5:1.

Method of Analysis

The analysis was performed with computer program SLIDE, Version 5.0, by Rocscience. The SLIDE program analyzes the slope with multiple methods to determine factor of safety, including Bishop Simplified, Janbu Simplified, Janbu Corrected, Spencer, Morgenstern-Price, and Corps of Engineers Methods. Bishop and Janbu methods employ limit equilibrium analysis method, Spencer and Morgenstern-Price methods use both force equilibrium and moment equilibrium to determine safety factors. In this analysis, Spencer results yielded the lowest factor of safety.

The analysis was performed for the end-of-construction (short-term) and long-term cases. Stability of the disposal cell perimeter embankment and cover system was also assessed for the design seismic event for both the short-term and long-term cases. Seismic conditions were analyzed using guidance provided in the TAD (DOE 1989). The TAD requires the use of pseudo-static approach where peak horizontal ground acceleration (PHA) value of 0.22 g (previously determined) is taken as half of PHA or 0.11 g for end-of-construction case, and two-thirds of PHA or 0.15 g for long-term case.

Results of Analysis

The analysis results, summarized in Table 4–2, indicate that the safety factor of the critical slope exceeds the safety factor required by the TAD for all of the cases. The stability results indicate that the proposed disposal cell site, perimeter embankments, and cover system will be stable when constructed of on-site materials and with the planned embankment geometry.

Table 4-2. Summary of Slope Stability Analysis

Loading Condition	Calculated Factor of Safety	Factor of Safety Required by TAD
End-of-construction:		
Static	2.15	1.3
Pseudostatic (k_h = 0.11 g)	1.31	1.0
Long-term:		
Static	2.78	1.5
Pseudostatic (k_h = 0.15 g)	1.51	1.0

K_h = pseudostatic coefficient

4.2.2 Settlement

Evaluation of tailings settlement in the disposal cell is presented in Addendum D, Calculation C-11. The evaluation was based on geotechnical test results on tailings sampled by Shaw E&I Inc., 2006, and Golder Associates, Inc., 2006. Consolidation characteristics were determined for remolded samples of sand tailings, transition tailings, and slimes tailings. Primary and secondary settlements are estimated based on compression of the tailings under their own weight and by each subsequent tailings layer and by the cover material.

The magnitude of primary and secondary settlement was calculated based on the consolidation tests and on the following inputs and assumptions:

- All natural overburden will be removed, and the excavation for the disposal cell will extend to a minimum of two feet into the Mancos Shale. Settlement of the foundation soil will therefore be negligible.
- RRM will be placed, spread, and compacted in layers or until all RRM has been moved. Settlement of the RRM will be due largely to settlement of material under its own weight and ultimately due to the additional weight of the protective cover.
- RRM will be mixed and dried to near optimum moisture content prior to transport to the Crescent Junction Site. Once there, RRM will be placed in layers per specifications and

compacted to 90 percent of the maximum density per American Society for Testing and Materials (ASTM) D698.

- RRM thickness assumed to be 46.7 feet and cover thickness assumed to be 9 feet.
- Consolidation properties of newly placed RRM (Cc – Compression Index or Coefficient of Consolidation, e_0 – initial void ratio) will be similar to the ones obtained for this analysis by averaging values for the sand tailings, transition tailings, and slimes.

Table 4–3 contains test results from consolidation testing of tailings material from the Moab Site. Table 4–4 contains a complete summary of geotechnical properties for the tailings material tested.

Table 4-3. Consolidation Test Data

Sample No.	Soil Type	Coefficient of Consolidation (Cc)	Initial Void Ratio (e_0)
GABT -04	Sand tailings	0.15	0.880
GABT -06	Sand tailings	0.07	0.638
GABT -09	Transition tailings	0.20	0.808
GABT -10	Transition tailings	0.17	0.703
GABT -11	Slime tailings	0.38	1.157
GABT -13	Slime tailings	0.34	1.052

For the settlement calculations, e_0 of 0.87 and a Cc of 0.16 were used. The e_0 was determined by averaging the void ratios of the different types of tailings material – sand, transition, and slimes. The compression index was selected based on the anticipated behavior of the combined sand, transition, and slimes material dried to near optimum moisture content for compaction. The combined material will behave more like the sands and transition material than a straight numerical average would indicate.

The results of the settlement analyses indicate that primary settlement of the tailings will be 11 inches and secondary settlement will be approximately eight inches. For the total height of the tailings and cover, the magnitude of total settlement is insignificant. Also, because of the granular composition of the tailings, most of the primary settlement will take place rapidly.

For monitoring settlement during long term surveillance and maintenance, an initial (base) survey would be performed at the end of cell construction. Following the baseline, a survey would be taken after two years and then every five years until it is determined that it is no longer necessary.

A potential monitoring procedure is as follows:
A grid would be established along 10 transects evenly spaced along the length of the cell (west to east) with pre-selected grid points. Total number of grid point surveys would be roughly 10 locations for each of the 10 transects. Grid spacing would be approximately 500 ft by 200 ft. For the entire cell, there would be 100 survey points. The points would be taken along each transect line starting at the base of the south side slope, proceeding up to the top of the side slope, then several points would be taken along the south-facing top slope and at the ridge line.

Table 4-4. Geotechnical Properties of Moab Tailings Pile Material

Bench Test Sample No.	**Soil Type**	**Atterberg Limits (LL/PL/PI) ASTM D4318**	**Sieve/Hydrometer Analysis**				**Sample Prep Dry Unit Weight (pcf) / Water Content (%) / Confining Pressure (psi)**	**Hydraulic Conduc-tivity (1) (c/s)**	**Triaxial Shear Strength (2)**		**Coefficient of Consoli-dation (Cc)**	**Volume Moisture Content at 15-bar (3)**	**Maximum Dry Density (pcf) (4)**	**Optimum Moisture Content (%) (4)**	**Settled Compaction (%) (4)**
			% Gravel	% Sand	% Silt	% Clay			C psf	Effective Friction Angle (degrees)					
GABT-01	Cover Soil	NP	4	73	18	5	106.3/7.0/2.5 106.3/7.0/2.5 106.3/7.0/2.5	4.7E-06 7.6E-06 1.1E-06					117.7	11.9	82.0
GABT-02	Cover Soil	NP	3	80	14	3							109.2	13.8	85.8
GABT-03	Sand Tailings	NP	1	83	15	2	90.5/14.4/2.25 90.5/14.4/2.25 90.5/14.4/2.25	2.7E-04 3.8E-04 7.9E-06	0	34.5			106.3	12.7	79.3
GABT-04	Sand Tailings	NP	0	76	21	3	88.2/17.5/2.25 88.2/17.5/2.25 88.2/17.5/2.25	1.7E-04 1.3E-05 1.8E-05	0	36.5	0.15	6.1	103.9	15.6	82.2
GABT-05	Sand Tailings	NP	3	76	17	5	101.7/15.3/2.25 101.7/15.3/2.25 101.7/15.3/2.25	3.1E-04 2.2E-04 2.1E-04	0	38.3			113.3	13.1	90.9
GABT-06	Sand Tailings	NP	1	83	13	4					0.07	24.4	107.3	14.6	82.6
GABT-07	Transition Tailings	31/22/9	1	49	42	8	96.3/20.5/2.5 96.3/20.5/2.5 96.3/20.5/2.5	1.2E-05 1.4E-05 1.3E-05	0	47.2			107.3	18.4	78.8
GABT-08	Sand Tailings	NP	7	72	19	9	101.4/17.9/2.25 101.4/17.9/2.25 101.4/17.9/2.25	3.2E-05 2.1E-05 7.4E-05	0	37.1			112.8	16.0	83.3
GABT-09	Transition Tailings	23/20/3	0	42	50	8	91.8/23.0/2.5 91.8/23.0/2.5 91.8/23.0/2.5	6.4E-05 6.9E-05 7.1E-05	0	36.3	0.20	24.4	102.0	21.1	87.9
GABT-10	Transition Tailings	19/17/2	0	70	24	6					0.17	>50.5	107.8	18.7	94.6
GABT-11	Slimes Tailings	56/27/29	0	22	53	25					0.38	27.6	96.0	27.8	68.5
GABT-12	Slimes Tailings	35/19/16	0	41	47	12	83.6/20.9/2.5 83.6/20.9/2.5 83.6/20.9/2.5	8.4E-05 2.1E-05 1.9E-05	0	50.8			101.6	22.5	39.3
GABT-13	Slimes Tailings	49/23/23	0	12	63	25					0.34	25.1	95.0	28.7	84.9
GABT-14	Slimes Tailings	43/22/21	0	16	62	22	81.2/22.8/3.0 81.2/22.8/3.0 81.2/22.8/3.0	2.7E-06 1.8E-06 1.8E-06	0	37.6			101.5	20.9	76.6

LL/PL/PI – liquid limit, plastic limit, plasticity index.

(1) Hydraulic conductivity tests performed at low confining pressures.
(2) Triaxial shear strength tests performed at low confining pressures.
(3) Capillary-moisture relationships analyzed with WP4 potentiometer.
(4) Test results from GAI 2006.

One or two additional survey grid points would be taken between the ridge line and the top of the north side slope. The final grid point would be at the base of the north side slope. Any visual surface depressions or bulges could dictate additional survey points.

The final monitoring procedure will be established in the Long Term Surveillance Plan and administered by the DOE Legacy Management.

4.2.3 Liquefaction Potential

Evaluation of tailings liquefaction potential in the disposal cell is presented in Addendum D, Calculation C-12. For liquefaction to occur, the tailings material in the disposal cell would have to be relatively loose under saturated conditions. The evaluation was performed in the unlikely event that the tailings become saturated.

Although the tailings will be placed in the disposal cell in a compacted and unsaturated condition, downward migration of water may create saturated zones within the tailings. The potential liquefaction of saturated zones of the tailings was checked with standard procedures outlined in Day (1999). The Mancos Shale underlying the tailings disposal site is not considered to be liquefiable.

The evaluation of tailings liquefaction potential is performed using Seed-Idriss Simpified Procedure based on Standard Penetration Test and modified per the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/National Science Foundation (NSF) Workshops on Evaluation of Liquefaction Resistance of Soils. Calculation of liquefaction potential involves comparison of the seismic cyclic stress ratio that would cause liquefaction with the cyclic resistance ratio for tailings at a specific depth of analysis. The factor of safety against liquefaction in a tailings layer is calculated by dividing the shear stress required to cause liquefaction in the layer by the shear stress generated in that layer by the design earthquake.

For the calculation, the following assumptions and inputs were made.

Assumptions:

- Existing tailings at the Moab Site will be dried to optimum moisture content prior to transport to the Crescent Junction Disposal Site. Once there, tailings will be placed in layers per specification and compacted to a minimum of 90 percent relative compaction. In general, tailings material should not be saturated.
- For analysis purposes only, tailing were assumed saturated at full height (worst case).
- Seismic design input as given in the TAD and RAP Attachment 1, Appendix D, for estimated peak acceleration at the ground surface for Crescent Junction.
- Liquefaction potential will be analyzed using earthquake moment magnitude of 6.5 (Addendum D, Calculation C-12).
- Standard Penetration Test (SPT) blow counts can be reasonably estimated for the placed and compacted materials based on assumed relative density of the compacted tailings layers.

Inputs:

- Assumed tailings thickness: 46.7 feet (saturated soil thickness: 46.7 feet)

- Assumed cover thickness: 9 feet
- Peak acceleration at ground surface: 0.22 g (TAD allows 0.11 g at end of construction and 0.15 g for long-term conditions)
- Earthquake moment magnitude: 6.5

The evaluation of liquefaction potential was performed for two cases, tailings with 17 percent fines and tailings with 46 percent fines. The results of the analyses indicate that liquefaction of the tailings will not occur under the assumed soil and seismic conditions. Furthermore, it is considered likely that field SPT N-counts in 90 percent relative density material may result in higher blow counts than assumed in this liquefaction analysis. The TAD indicates the minimum factor of safety considered acceptable for UMTRA sites is 1.5. The calculated factors of safety ranged from 1.37 to 2.38 for the tailings containing 17 percent fines, and from 1.74 to 3.04 for the tailings with 46 percent fines. Due to the extreme (and unlikely) assumption made for saturated conditions to be present at full height of the tailings, it is concluded that the tailings when placed, compacted, and covered in the disposal cell will not be liquefiable.

4.2.4 Cover Cracking

RRM that will be placed at the Crescent Junction Disposal Site is to be stabilized and contained by placement in an encapsulated disposal cell. The cover of the disposal cell serves to prevent the escape of radon from the RRM, as well as to inhibit infiltration of precipitation. Cracking of the disposal cell cover can adversely impact the ability of the cover to achieve those two purposes. Cover cracking was evaluated by comparing the allowable strain of the cover materials with the maximum calculated strain due to differential settlement in the cover (Addendum D, Calculation C-15).

Settlement analyses determined that the settlement of the RRM will be 19 inches or 1.58 feet, occurring at a location with a tailings depth of 46.7 feet. The settlement of the tailings at the top of the perimeter embankment will be 0 inches where the thickness of tailings tapers to 0 feet. The horizontal distance between the location of maximum settlement and zero settlement is 114 feet. Therefore, the total distortion equals 1.58 feet / 114 feet = 0.014. Maximum covered strain calculated for a distortion of 0.014 is less than the allowable strain. For clayey soils, the maximum tensile strain at failure equals 0.065 percent (Gilbert and Murphy 1987). The actual strain, 0.014 percent is much less than the allowable strain of 0.065 percent, thus the cover is not anticipated to crack.

End of current text

5.0 Radon Attenuation

5.1 Cover Design

The remedial action at the Moab Processing Site and the placement of RRM at the Crescent Junction Disposal Site is summarized in Section 1.1.3. The cover design is shown in Figure 5-1. This is the typical UMTRA Project cover using a compacted clay radon barrier to control the rate of radon emission from the cell. The design includes a minimum one-foot-thick interim cover placed directly on the RRM surface as a best management practice to control wind transport of fine material and to provide for a relatively clean, uniform work surface upon which to construct the radon barrier.

The UMTRA Project cover design consists of an interim cover constructed of clean native alluvial materials to a minimum thickness of one foot, a compacted clay radon barrier constructed from conditioned on-site weathered Mancos Shale, a 0.5-foot-thick infiltration and biointrusion barrier consisting of sandy gravel, and a 3.5-foot-thick frost protection layer that includes the 0.5-foot-thick rock mulch erosion protection layer. The thickness of the radon barrier depends on the thickness of the interim cover, since both layers reduce the rate of radon emission. The thickness of the required radon barrier for the Crescent Junction disposal cell is four feet for a one-foot-thick interim cover.

The radon barrier layer thickness was selected for reduction of radon gas flux to rates below 20 picoCuries per square meter per second (pCi/m^2/s). The erosion protection, frost protection, and drain layers were not considered in the calculation of the radon barrier thickness, due to the high permeability of these materials. The side slopes will be constructed of clean fill materials and will be much thicker than the required cover and, therefore, will be adequate to meet the EPA standard for radon flux. Consequently, the side slopes have been evaluated solely for erosion protection. The covers for the side slopes are described in Section 6.4.1.

5.2 Radon/Infiltration Barrier Parameters

The radon barrier design parameters and supporting calculations were used in conjunction with the RADON model (NRC 1989b) to determine the cover thickness necessary to meet the EPA radon flux standard of 20 pCi/m^2/s. Guidance provided in the TAD (DOE 1989) was considered in developing the cover design. As with previous UMTRA Project Title I cover designs, the attenuation of radon by the frost protection, drainage, biointrusion, or erosion protection layers is not considered in the baseline analyses, though these layers will further reduce the radon flux rate at the disposal cell surface.

Specific design parameters include long-term moisture content, radon diffusion, radon emanation, density, porosity, layer thickness, average radium-226 activity, and ambient radon concentration. Addendum D, Calculation C-05 presents the input parameters used for each model run as well as the model run results.

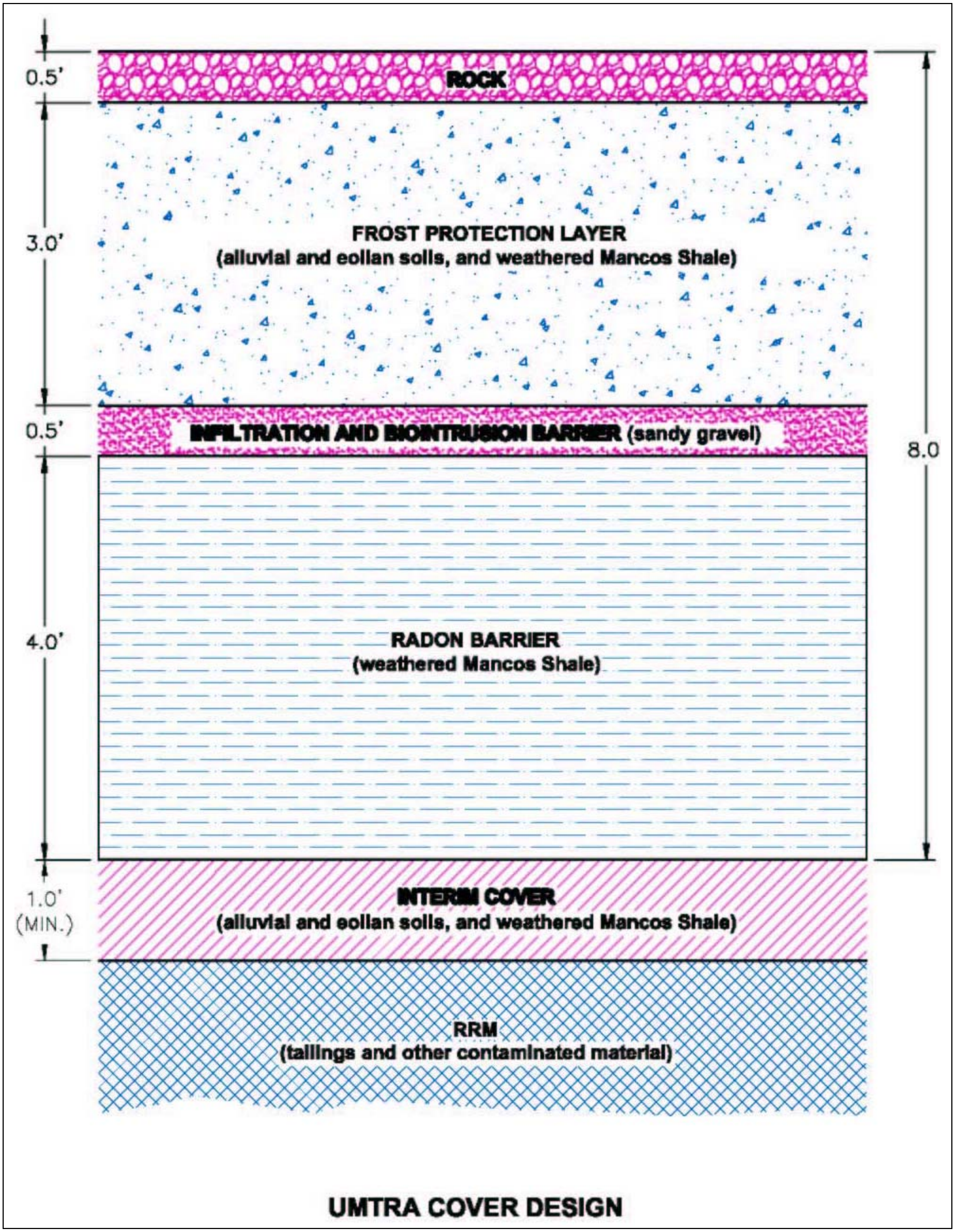

Figure 5-1. Disposal Cell Cover Design

5.2.1 Long-Term Moisture Content

The mean long-term moisture content of the tailings has been modeled as 15 percent. The draft RAP stated “The mean weight percent moisture of the tailings has been modeled as 15 percent, which is in the typical range for tailings and is below that value used for the modeling of the Grand Junction UMTRA Site (18 percent). Sensitivity analyses for the influence of long-term tailings moisture content were used to evaluate the influence of this parameter on predicted radon barrier thicknesses. Values of 10 percent moisture content and 20 percent moisture content were modeled.”

The results of the sensitivity analysis performed for the draft RAP are summarized below.

Table 5-1. Draft RAP Sensitivity Analysis of Tailings % Moisture Content

% Moisture Content in the Tailings	Required Radon Barrier Thickness (cm)	Required Radon Barrier Thickness (feet)
10	119.1	3.91
15	119.8	3.93
20	111.7	3.66

No laboratory test were performed to determine the correct moisture content and the % clay and % organic material of the tailings were not measured so the Rawls and Brakensiek equation could not be evaluated.

A sensitivity analysis performed on the current radon barrier design yielded the results shown in the graph on Figure 5-2. As can be seen the required radon barrier thickness is relatively insensitive to the % moisture content of the tailings below approximately 14 to 15%. Lacking data, the NRC publication *Regulatory Guide 3.64; Calculation of Radon Flux Attenuatlon by Earthen Uranium Mill Tailings Covers* states that:

“If acceptable documented alternative information is not furnished by the applicant, the staff will use a reference value of wt = 6% for the tailings moisture content because 6% is a lower bound for moisture in western soils.”

Comparing the calculated required radon barrier thickness for a disposal cell with 6% moisture content in the tailings with the current modeling results with 15% moisture content indicates a radon barrier thickness only marginally greater than the 4 feet in the current design. (Table 5-2).

Table 5-2 Comparison of 6% and 15% Moisture Content for Current Radon Barrier Design

% Moisture Content in the Tailings	Required Radon Barrier Thickness (cm)	Required Radon Barrier Thickness (feet)
6	123.40	4.05
15	121.74	3.99

The calculated radon barrier thickness with a tailings moisture content of 6% deviates from the current design by only 0.6 inches which is considerably less than the precision with which the barrier can be constructed.

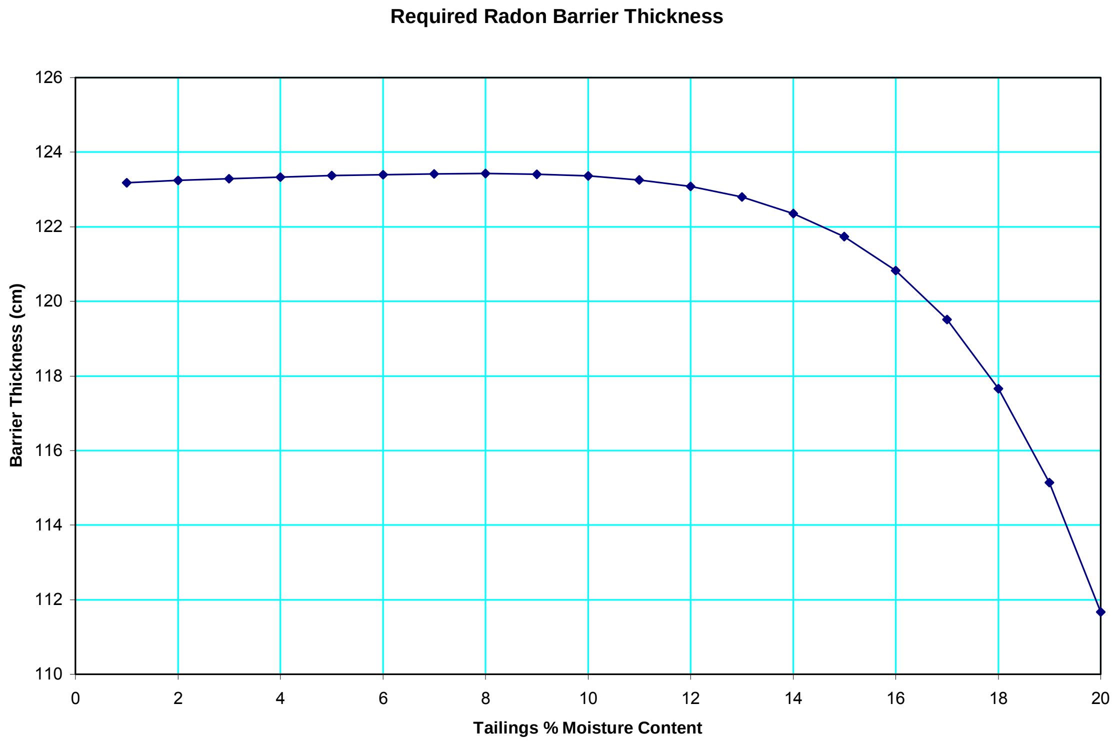

Figure 5-2 Required Radon Barrier Thickness

The mean long-term moisture content of the interim cover is modeled as nine percent. This value is based on the mean of 20 measured 15-bar moisture content analyses as determined by ASTM Method D3152 and presented in Addendum D, Calculation C-05. This mean measured value was evaluated for reasonableness using the Rawls and Brakenseik equation as presented in the NRC Regulatory Guide 3.64 (NRC 1989b) and described in the TAD (DOE 1989). The calculated value using the Rawls and Brakenseik equation is 7.5 percent (DOE 2007b), which agrees well with the measured value of site-specific soils of nine percent.

The mean long-term moisture content of the compacted clay derived from the on-site weathered Mancos Shale is modeled as 12 percent. This value is based on the mean of 12 measured 15-bar moisture content analyses of remolded Mancos Shale as determined using ASTM Method D3152 and presented in Attachment 5, Appendix E and Addendum D, Calculation C-05. Measured in-situ moisture content for weathered Mancos Shale was not included in the calculation of the mean because in-situ moisture content is not representative of compacted, weathered Mancos Shale. Long-term moisture content of the compacted, weathered Mancos Shale in the radon barrier is better represented by the measured 15-bar moisture content test results from remolded weathered Mancos Shale, due to the difference in material fabric between as-placed cover and the in place native material. This mean measured value of remolded Mancos Shale was also evaluated for reasonableness using the Rawls and Brakenseik equation as presented in the NRC Regulatory Guide 3.64 (NRC 1989b) and described in the TAD (DOE 1989). The calculated value is 12.4 percent, which agrees well with the mean of the measured values of 12 percent.

The Rawls and Brakenseik equation was used only as a check for reasonableness of the measured values, as the mean of the measured values using ASTM Method D3152 was used in the RADON model to evaluate cover designs.

5.2.2 Radon Diffusion

The radon diffusion coefficient used in the RADON model (NRC 1989b) can either be calculated within the model (based on an empirical relationship with degree of saturation and porosity) or input directly into the model using values measured from laboratory testing. The radon diffusion equations in the 1989 version of RADON are not consistent with a later equation based on a much larger set of data correlating radon diffusion with soil cover materials. Therefore, the RADON code was modified to implement layer-specific radon diffusion coefficients based on the most current relationship using equation 9 from Rogers and Nielson (1991). The code was also modified to direct output to a user specified file instead of a printer.

For the tailings, the calculated radon diffusion coefficient was 0.01037 centimeter squared per second (cm^2/s), for a moisture content of 15 percent by weight and a porosity of 0.44. For the interim cover, the calculated radon diffusion coefficient of 0.01636 cm^2/s was applied based on a moisture content of nine percent and a porosity of 0.38. The radon diffusion coefficient for the UMTRA Project cover compacted clay radon barrier was calculated to be 0.004636 cm^2/s based on the long-term moisture content of 12 percent and a porosity of 0.33.

5.2.3 Radon Emanation

A radon-emanation coefficient of 0.35 was used for all of the tailings, random fill, and cover materials. This is the conservative default value used in the RADON model (NRC 1989b). This value agrees well with the value used for other UMTRA Project sites (e.g., the Grand Junction Disposal Site in Colorado used a radon-emanation coefficient of 0.36).

5.2.4 Dry Densities and Porosities

The dry densities, specific gravities, and porosities were determined from standard compaction tests. The as-placed tailings density was based on compaction to 90 percent of average standard Proctor density. Interim cover and freeze/thaw protection layer materials are all the same material and were based on compaction to 90 percent of the average standard Proctor density. The UMTRA Project cover compacted clay barrier (remolded Mancos Shale) was based on compaction to 95 percent of standard Proctor density.

The porosities of these materials as placed were calculated based on the dry density and the specific gravity of the actual materials. A tailings average specific gravity of 2.8 (based on five samples) was used to calculate an average tailings porosity of 0.44. An average specific gravity of 2.67 (based on seven samples) for site alluvial materials was used to calculate an average porosity of 0.38 for the interim cover. An average specific gravity of 2.65 (based on two samples of on-site weathered Mancos Shale) was used to calculate an average porosity of 0.33 for the compacted clay radon barrier of the UMTRA Project cover.

5.2.5 Layer Thickness

The layers and material sequences are illustrated in Figure 5–1 and represent the geometries of the tailings and of each cover-layer component. Clean fill embankments made of native materials will be used around the perimeter of the disposal cell constructed with 5:1 (horizontal:vertical) exterior side slopes and a minimum 30-foot-wide crest. Because the tailings side slope thicknesses will be far in excess of the cover requirements and with properties comparable to the interim cover material, radon flux through the side slopes was not modeled. A model run with only a RRM layer and a layer of interim cover material did, however, indicate that a side slope thickness of 11 feet would be sufficient to limit the radon flux rate to less than 20 $pCi/m^2/s$. Information on layer thicknesses is in Attachment 1, Appendix B, and Addendum D, Calculation C-05.

For all model runs, RRM thickness of 1310.7 centimeters (cm) (43 ft) is used. This is the maximum thickness of the RRM in the design of the disposal cell. The tailings consist of two layers, a lower layer that is 1,097.3 cm (36 ft) thick and an upper layer that is 213.4 cm (seven feet) thick. This configuration was chosen to allow higher activity waste to be placed in the lower layer providing that the radium activity of the RRM in the upper layer is 707 picoCuries per gram (pCi/g) or less.

The UMTRA Project cover design evaluated for radon flux consists of a one-foot-thick interim cover constructed of uncontaminated native alluvial materials and a compacted clay radon barrier constructed from conditioned on-site weathered Mancos Shale. The drainage and biointrusion layer, frost protection layer, and rock mulch erosion protection layer are not considered in the modeling.

5.2.6 Radium-226 Activity

Radium-226 activities for the tailings pile materials were assessed (by gamma spectroscopy) on 104 samples of tailings sands, slimes, transitional tailings, and other contaminated materials. The estimated volumes of tailings material are provided in Attachment 1, Appendix K, and Addendum D, Calculation C-05. The average radium-226 activity of these 104 samples is 707 pCi/g. The number of samples per unit volume of slimes was greater than for the other materials to be placed in the disposal cell. Because the average radium activity of the samples collected from the slimes (1,349.3 pCi/g) is greater than for any of the other materials, this simple average overestimates the radium activity of RRM that will be well mixed before being placed in the cell. Accounting for the volumes and the radium activities of the different materials, the radium activity of completely mixed contaminated material from the Moab Site would be 565 pCi/g.

As the RRM is placed in the lower layer of the cell, the radium activity will be monitored only occasionally. As the RRM is placed in the upper layer (seven feet) the radium activity will be carefully monitored to ensure that the average radium activity in the upper seven feet does not exceed 707 pCi/g. In modeling the rate of radon emission from the top of the radon barrier, the radium activity of the lower layer has been set equal to the average of the slimes (1,349.3 pCi/g) and the upper layer to the average of all samples (707 pCi/g). This is a conservative approach as the overall volume-weighted average radium activity is 565 pCi/g and the modeled volume-weighted average is 1,245 pCi/g.

The radium-226 activity of the alluvial materials to be used for the interim cover and the clean fill perimeter dikes is based on five samples of native materials collected from the Crescent Junction Site. The radium-226 activity of the alluvial material ranged from 1.4 to 2.3 pCi/g, with a mean value of 1.9 pCi/g.

The radium-226 activity value for the compacted clay layer is based on two samples of Mancos Shale collected from the Crescent Junction Site that will be used to construct the compacted clay radon barrier and clean-fill perimeter dikes. The radium-226 activity of the weathered Mancos Shale ranged from 1.6 to 3.0 pCi/g, with a mean value of 2.3 pCi/g.

5.2.7 Ambient Radon Concentration

The RADON default ambient radon activity in air of 0 picoCuries per liter (pCi/L) was used for the RADON model (NRC 1989a) because it has little influence on the model. Activities of air samples collected at background locations have a range of 0.5 to 1.2 pCi/L.

5.3 Evaluation of the Radon Barrier

This section summarizes the manner in which the input parameters presented above were evaluated to develop a radon barrier design that will comply with the EPA radon flux standard of 20 pCi/m^2/s using parameters as discussed in Section 5.1 as input for the RADON model (NRC 1989a). Several runs of the RADON model were performed to determine the minimum required radon barrier for radium activities corresponding to the raw average and the volume-weighted average of the RRM. The RADON model runs are summarized in Addendum D, Calculation C-05.

Three model runs for the UMTRA Project cover design were performed to assess model sensitivity to certain variables as described below.

- Model run UMTRA 1a uses mean input values for the UMTRA Project style cover with a one-foot-thick interim cover. The RRM is placed in a single layer 43 feet thick with a radium activity of 707 pCi/g. The thickness of the radon barrier layer is optimized to limit the radon flux rate to 20 pCi/m^2/s.
- Model run UMTRA 1b is identical to UMTRA 1a except that the radium activity of the RRM is set equal to the volume-weighted average of 565 pCi/g.
- Model run UMTRA 1c the RRM is divided into two layers. The lower layer is 36 feet thick with a radium activity of 1,349.3 pCi/g and the upper layer is seven feet thick with a radium activity of 707 pCi/g. The radon barrier layer is four feet thick and the model is run to predict the rate of radon flux through the barrier layer.

Modeling results indicate that for UMTRA 1a, a radon barrier thickness of 3.9 feet is required, for UMTRA 1b, the optimized barrier thickness is 3.6 feet and for UMTRA 1c, the radon flux through the radon barrier layer would be 19.9 pCi/m^2/sec.

The final cover design will be based on actual measurements of the as-placed contaminated materials and will demonstrate compliance with the radon flux standard.

5.4 Summary and Conclusions

The disposal cell and radon barrier design (four feet thick) will control radon flux to levels below EPA standards stated in 40 CFR 192.02(b). DOE has committed to stabilizing the RRM for long-term control in accordance with EPA standards, NRC guidelines, and UMTRA Project health and safety requirements.

The radium activity of the upper seven feet of waste will be closely monitored as it is placed to ensure that the final radon barrier thickness will limit long-term radon flux through the barrier layer to 20 $pCi/m^2/sec$. If the results of this monitoring indicate the need, the higher activity material will either be mixed with lower activity material or placed in a lower segment of the cell still under construction. In a worst-case scenario, the thickness of the radon barrier layer will be adjusted to ensure compliance with 40 CFR 192.02(b).

6.0 Surface Water Hydrology and Erosion Protection

6.1 Hydrologic Description of Current Conditions

The Crescent Junction Disposal Site is located on a low-gradient, south-facing slope known as Crescent Flat. The Book Cliffs lie to the north of the disposal site. The average grade of Crescent Flat is approximately 1.4 percent, sloping southward down from the base of the Book Cliffs. There are four major drainage basins in and adjacent to the disposal site that are defined based on four ephemeral streams in the area: East and West Branches of Kendall Wash, which join immediately upstream of I-70; Crescent Wash, located west of the disposal cell site; and Blaze Wash, located east of the cell site. All four washes ultimately drain into the Green River 25 miles south to southwest of the disposal cell site. The major basins associated with these washes are shown on Figure 6–1.

The disposal site lies within the West Kendall Wash drainage area, designated as Basin 1. This is a small drainage of 2.6 mi^2, beginning at the top of the Book Cliffs and running south to the railroad crossing south of the cell. Drainage in this basin tends to run off as sheet flow until concentrated at the railroad crossing. The overland sheet flows tend to produce localized rill erosion, whereas concentrated flows at the railroad crossing tend to produce more notable scour.

The East Branch of Kendall Wash combines with Blaze Wash north of the railroad to form Basin 2. Flows in this basin also go overland until converging at the same railroad crossing, east of the disposal cell site. Runoff from Basins 1 and 2 combines between the railroad and I-70, designated as Basin 3, and forms a small ephemeral stream. Several culverts three feet to four feet in diameter provide drainage for flows west of Blaze Wash to pass under I-70. A pair of six foot diameter culverts allow Blaze Wash to pass under I-70. Together these culverts provide discharge for flows from Basin 3 southward under I-70. At the low point of the Kendall Wash basin a 20 foot diameter culvert allows discharge of Basin 3 to the south under I-70. Given the small capacity of the two foot to three foot culverts, when compared to the 100-year and PMP flood events and the potential for sediment plugging, this analysis is conservatively based on routing all of Basin 3 to the 20 foot culvert crossing.

Crescent Wash is a well-defined ephemeral stream with a basin area of 22.5 mi^2. Crescent Wash is located approximately 2,000 feet west of the disposal cell.

Peak runoff flow rates and flood evaluations for all three basins are determined at specific locations in the vicinity of the Crescent Junction Site for the 100-year, 24-hour storm, and the PMP local storm. Although there are culverts beneath I-70, the capacity of those culverts is small relative to the runoff from the storm events, such that the entire storm runoff was conservatively routed to the west along I-70 in Basin 3.

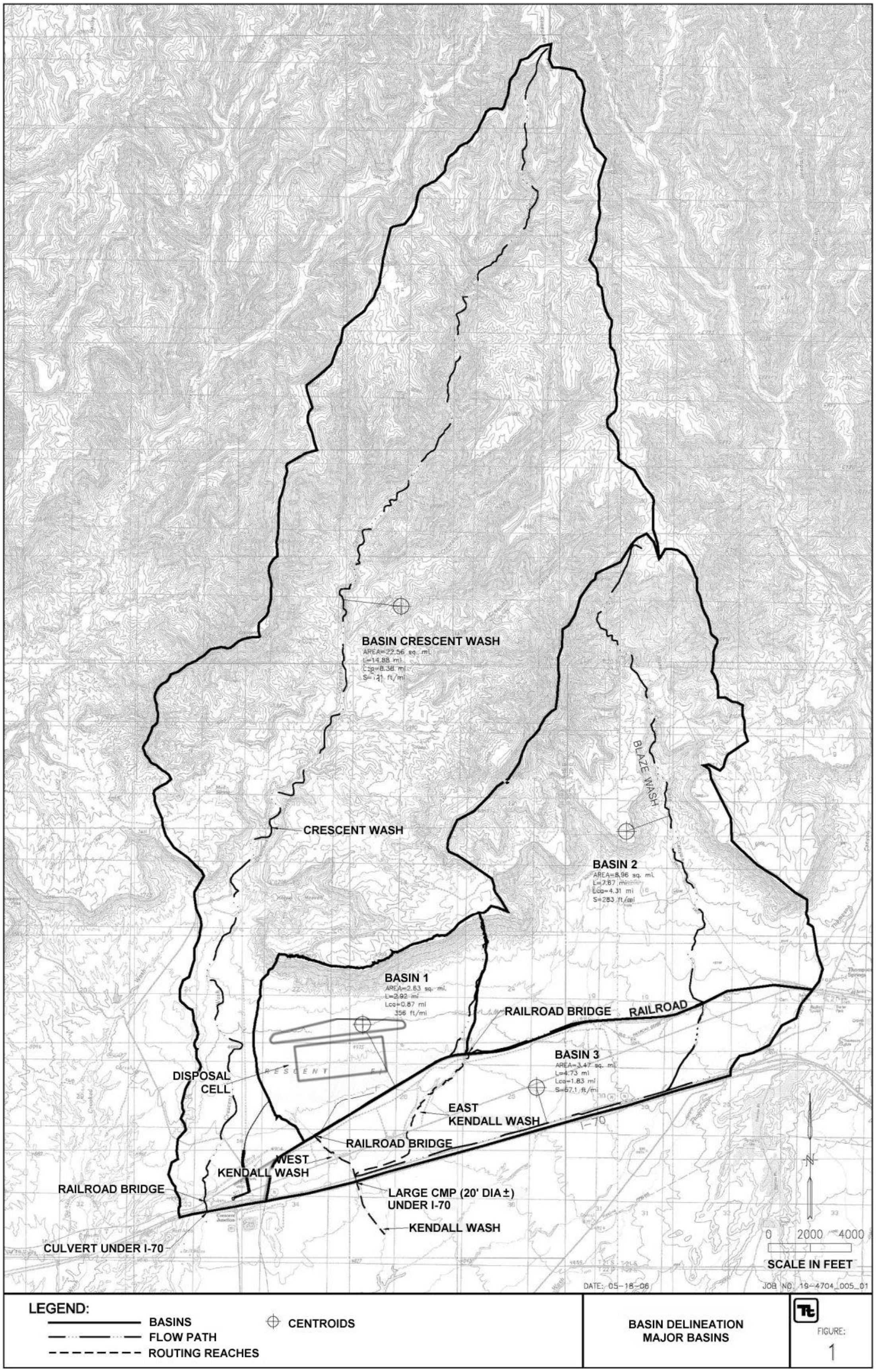

Figure 6-1. Basin Delineations in and Adjacent to the Crescent Junction Disposal Site

6.2 Flooding Determinations

6.2.1 PMP and Distribution

Design storm information is provided in Attachment 1, Appendix E, which calculates the local storm PMP for storms of less than 1 mi^2 to 22 mi^2. This analysis also includes determination of storms in basins covering 1.4, 2.7, 3.5, 9, and 15 mi^2. Additional depth-duration models are developed so that the size of the storm is equivalent to the drainage area contributing to the disposal site. The depth-duration relationships for the modeled storms are summarized in Table 6–1.

Table 6-1. Depth-Duration for Modeled Storms

	Precipitation Depth (inches) for Specified Duration							
	5 min	15 min	1 hr	2 hr	3 hr	6 hr	12 hr	24 hr
Storm Event								
100-yr, 24-hr	0.53	0.99	1.65	1.82	1.84	1.95	2.16	2.35
PMP – Local								
<1.0 mi^2	4.5	7.1	8.2	8.8	8.9	9.0		
1.4 mi^2	4.3	6.8	8.0	8.6	8.7	8.9		
2.7 mi^2	4.1	6.5	7.9	8.4	8.5	8.7		
3.5 mi^2	4.0	6.2	7.6	8.3	8.5	8.6		
9.0 mi^2	3.4	5.4	6.9	7.6	7.7	8.0		
15.0 mi^2	3.0	4.8	6.4	7.0	7.2	7.7		
22.0 mi^2	2.7	4.3	6.0	6.7	6.9	7.4		

6.2.2 Infiltration Losses

The U.S. National Resources Conservation Service (NRCS) classifies the well-draining sands and sandy loams (Toddler-Ravola-Glenton soil family association) in the disposal site area as Group B soils, which have a range of final infiltration rates of 4 to 8 millimeters per hour (0.16 to 0.31 inch per hour) (NRCS 2007). A 0.15 to 0.3 inch per hour minimum infiltration rate is recommended by the U.S. Bureau of Reclamation (USBR 1987) for Group B soils. For the purpose of this analysis, a value of 0.3 inch per hour is used for modeling the existing undisturbed watershed, and 0.15 inch per hour is used for the cell site. Other loss parameters are noted as follows:

- A U.S. Soil Conservation Service (SCS) curve number (CN) value of 70 was used for Group B soils with sparse vegetation.
- Manning's **n** value, K_n, representing the hydraulic characteristics of the drainage network, varies with flow; 0.042 was used for the probable maximum flood (PMF), and 0.054 was used for the 100-year flow.
- For the PMF:
  - Loss method in existing watershed: Initial loss of 0.0 inch, constant loss of 0.3 inch per hour.
  - Loss method for the disposal cell: Initial loss of 0.0 inch, constant loss of 0.15 inch per hour.

- Loss method for the disposal cell (erosion protection calculations): 0.0 inch per hour.
- Transform method: User-specified unit hydrograph.
- Baseflow method: None.
- Routing reaches: Kinematic wave.
- Meteorology model: PMP calculations, no evapotranspiration, no snowmelt.

- For the 100-year, 24-hour storm:
  - Loss method in existing watershed: SCS CN method with initial loss of 0.86 inch, based on a CN of 70 and constant loss of 0.3 inch per hour.
  - Loss method for the disposal cell: SCS CN method with initial loss of 0.86 inch, based on a CN of 70 and constant loss of 0.15 inch per hour.
  - Transform method: User-specified unit hydrograph.
  - Baseflow method: None.
  - Routing reaches: Kinematic wave.
  - Meteorology model: Precipitation frequency data from U.S. National Oceanic and Atmospheric Administration (NOAA) Atlas 14, no evapotranspiration, no snowmelt (NOAA 2004).

6.2.3 Computation of PMF Events

The methodology for determining the unit hydrograph is detailed in *Design of Small Dams* (USBR 1987) using the dimensionless unit hydrograph data for the Colorado Plateau regions of Arizona, southern California, western Colorado, Nevada, New Mexico, and Utah. Basins in this arid region are generally typified by sparse vegetation, fairly well-defined drainage networks, and terrain varying from rolling to very rugged in the more mountainous areas. The unit hydrograph lag time is defined as:

$$L_g = C\left(\frac{LL_{ca}}{\sqrt{S}}\right)$$

where:

L_g = unit hydrograph lag time, hours. The unit hydrograph lag time is the time from the midpoint of the unit rainfall excess to the time that 50 percent of the volume of unit runoff from the drainage basin has passed the concentration point (USBR 1987).

C = constant = 26 K_n. K_n = average Manning's **n** value representing the hydraulic characteristics of the drainage basin. K_n is a function of the magnitude of the flows and normally decreases with increasing discharge. K_n values for the PMF are based on recommendations from (USBR 1987), which suggests that the lowest value representative of the region be used. A regional K_n value of 0.042 represents the lower limit of the accepted range for PMF determination and is typical of desert terrain. For other storm events, a higher value is appropriate. K_n ranges from 0.042 to 0.070 in the Colorado Plateau region (USBR 1987). A value of 0.054 is selected for the

25-year and 100-year storm events, representing an area on the White River near Watson, Utah, that is relatively close to the disposal site (Table 3–3 in USBR 1987).

L = the length of the longest watercourse from the point of concentration to the boundary of the drainage basin.

L_{ca} = the length along the longest watercourse from the point of concentration to a point opposite the centroid of the drainage basin.

S = the overall slope of the longest watercourse (along *L*).

Hydrologic parameters and spreadsheets are used to create the basin-specific unit hydrographs for use by the Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) (U.S. Army Corps of Engineers (USACE) 2006) models and are presented in Attachment 1, Appendix F. The peak flow rates at each of the design points are summarized in Table 6–2.

Table 6-2. Peak Flow Rates, Major Storm Events

Design Point	**Area** (mi^2)	**Peak Flow Rate (cubic feet per second [cfs])**	
		100-yr, 24-hr	PMP - Local
Crescent Wash at RR Bridge and I-70	22.6	5,983	45,197
West Branch Kendall Wash Branch at RR Bridge	2.6	2,135	21,288
Blaze and East Branch Kendal Wash at RR Bridge	9.0	3,453	29,869
East Branch Kendall Wash at I-70 culvert	15.1	5,109	40,835

6.3 Water Surface Profiles and Channel Velocities

The following potential flooding sources were evaluated for this effort: East and West Kendall Wash, Blaze Wash, and Crescent Wash. Analysis of each of these washes extends to a distance sufficient to determine the impacts, if any, on the disposal cell. This requires distances of approximately two miles to three miles for each reach. Flood events are evaluated for the 100-year, 24-hour storm, and the PMP local storm.

6.3.1 Method of Analysis

Hydraulic models are developed to calculate the 100-year and PMF water surface elevations using the USACE HEC-River Analysis System (RAS) (USACE 2005) one-dimensional model assuming fixed bed conditions. Required input includes channel cross sections that are derived from two sources. The first source is from topographic cross-section surveys performed by Keogh Land Surveying of Moab, Utah, during the winter and spring of 2006. The second source is from aerial topographic data with two foot contours, used to supplement survey data. The cross-section points were extracted using AutoCAD 2005 Land Development Desktop. All elevations and topographic mapping are based on NAD 83 and NAVD 88 datum.

Other parameters and modeling methods are noted as follows:

- Manning's **n** values: A Manning's **n** value of 0.028 is used for the channel. This selection is supported by comparing these two channels to similar channels in Barns (1967). The overbank **n** value was determined to be 0.045 and was selected on the type and relative density of vegetation using standard references, including Barns (1967) and Chow (1959).

- Starting water surface elevations: Starting water surface elevations for Crescent Wash and the branches of Kendall Wash are based on normal depth and an energy gradient approximately equal to the starting channel slope.

6.3.2 Results of Flood Analysis

Calculations indicate that the disposal cell location lies outside of the floodplains generated from the 100-year flood event and the PMF from Crescent Wash and the East and West Branches of Kendall Wash. Under PMF conditions, overtopping at the railroad bridges will occur at all three drainages. Overflow from the east branch of Kendall Wash splits with some flow passing over the railroad bridge and some flow turning westerly, flowing along the north drainage swale created by the elevated railroad bed. These flows join with the West Branch of Kendall Wash at the railroad bridge, and the West Branch of Kendall Wash again splits and either overtops the railroad bridge or flows westerly. For the purposes of this analysis it is assumed that the existing culverts under the railroad between East and West Kendall Wash are plugged and have little capacity for reducing the diverted flows running along the north side of the railroad. This is the worst-case scenario in terms of potential for floodwater encroachment at the disposal cell site. The PMP and 100-year floodplains are delineated on Figure 6–2. Detailed hydraulic calculations are included in Attachment 1, Appendix F. Because of differences in the level of accuracy of the two foot contour aerial mapping compared to the surveyed cross sections, there may be slight discrepancies between the model results and the mapped results.

6.4 Erosion Protection Design

The parameters used in the hydrologic analyses for erosion protection of the cell are somewhat more conservative than those in the previous sections. The principal differences are in the watersheds between the top of the Book Cliffs and the disposal cell. The analyses in this section employ a somewhat higher curve number for the Toddler-Ravola-Glenton soil family association, 75 instead of 70, because the vegetation in the area near the Book Cliffs is in generally poorer condition than the average in the larger areas. Compared with the areas of Toddler-Ravola-Glenton soil family association in these smaller watersheds, there are also significant areas of Hanksville family-Badland complex comprising the area near and on the steep slopes of the Book Cliffs. These soils have a higher runoff potential and a lower constant infiltration rate than the Toddler-Ravola-Glenton soil family association. The initial abstractions and constant infiltration rates for these watersheds are, therefore, different from the parameters used in the large scale hydrologic analysis. Soil properties were obtained from the web soil survey (NRCS 2007).

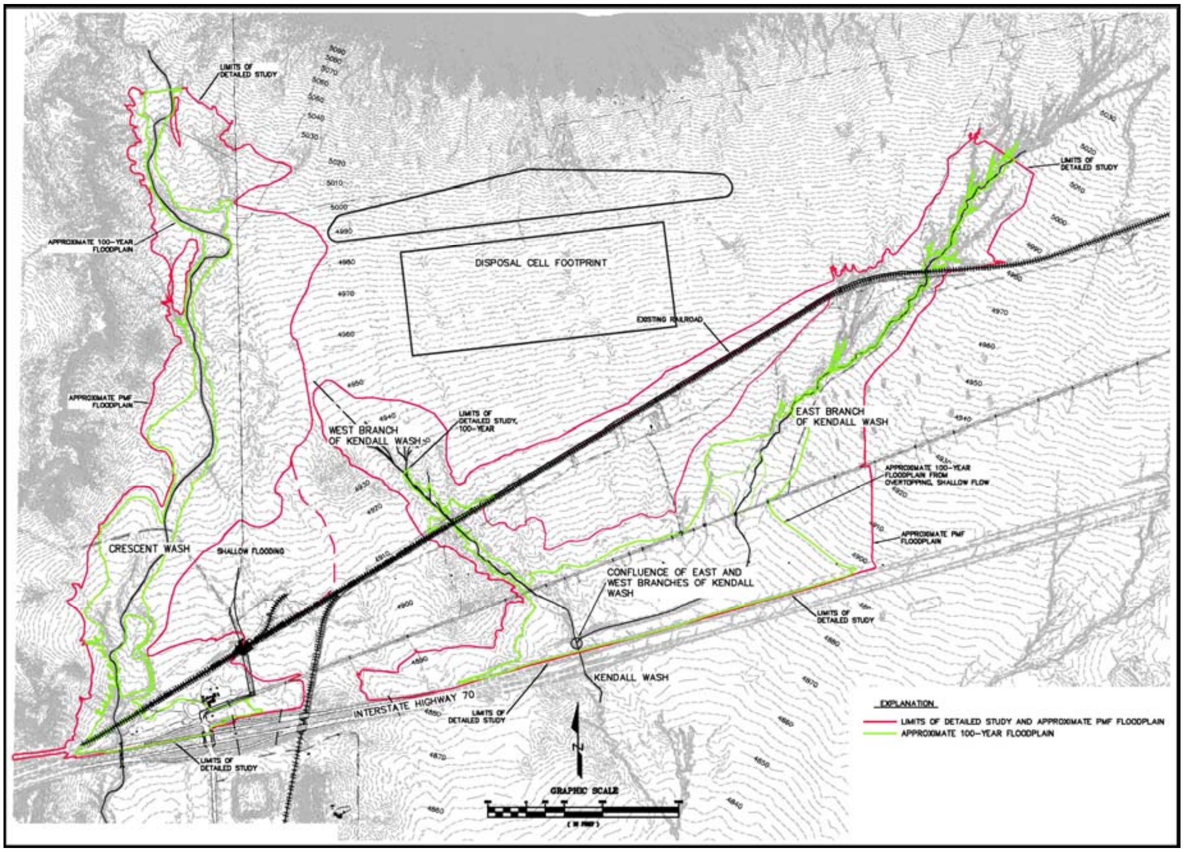

Figure 6-2. PMP and 100-Year Floodplain Delineations for the Crescent Junction Disposal Site

6.4.1 Top Slope and Side Slopes

A plan view of the cell is shown in Figure 6-3. To protect the top surface of the disposal cell against erosion, the surfaces will be covered with rock mulch. The area of the top slope draining to the south will be covered with a six inch layer of rock or rock mulch with a median particle size (D_{50}) of at least 1.8 inches and the area of the top slope draining to the north will be covered with a six inch layer of rock or rock mulch with a D_{50} of at least 1.2 inches. The calculations to determine the D_{50} and thickness of the rock mulch layers on the top and side slopes and on the aprons at the toe of the side slopes are presented in detail in Addendum D, Calculation C-02.

The north side slope, which receives runoff from the north top slope and from rainfall directly on the north side slope, will require a minimum 8.2 inch thick layer of rock mulch with a minimum D_{50} of 4.1 inches while the south side slope will be covered with a minimum 11.6-inches-thick layer of rock with a D_{50} of 5.8 inches. The east and west side slopes will carry substantially equal flows and require rock mulch layers with a minimum D_{50} of 2.3 inches and a minimum thickness of 4.6 inches.

The rock protection placed on the north and south side slopes will overlay a 6-inch-thick sand bedding layer. Rock sizing was estimated using the Safety Factor Method (Nelson et al. 1986) for the top slope, and the Abt and Johnson (1991) Method for the side slopes. Unit flows were calculated based on the PMP event, assuming no infiltration, and a concentration factor of three to account for potential flow channelization. Conservative values were used for input parameters, including a specific gravity for rock of 2.65 and an angle of internal friction of the rock mulch of

37 degrees. In addition, a coefficient of movement of 1.35 was used in the Abt and Johnson (1991) Method to design against rock movement as well as failure. The calculated required rock sizes are based on angular rock that meets NRC durability requirements without oversizing. A summary of the required riprap sizes for erosion protection of the disposal cell slopes is provided in Table 6–3.

Table 6-3. Summary of Erosion Protection Materials

Drainage Area	Unit PMP Discharge (cfs/ft)	Concen-tration Factor	Stone Movement Ratio	D_{50} (in)	Minimum Layer Thickness (in)	Apron Width (10 ft Minimum)	Scour Depth (ft)
South top slope	0.98	3		1.8	3.6		
North top slope	0.54	3		1.2	2.4		
South side slope	1.02	3	1.35	5.8	11.6		
North side slope	0.55	3	1.35	4.1	8.2		
East side slope	0.20	3	1.35	2.3	4.6		
West side slope	0.20	3	1.35	2.3	4.6		
South apron	1.02	3	1.35	11.6	34.7	15	1.66
North apron	0.55	3	1.35	8.2	24.5	10	1.18
East apron	0.20	3	1.35	4.7	14.0	10	0.67
West apron	0.20	3	1.35	4.7	14.0	10	0.67

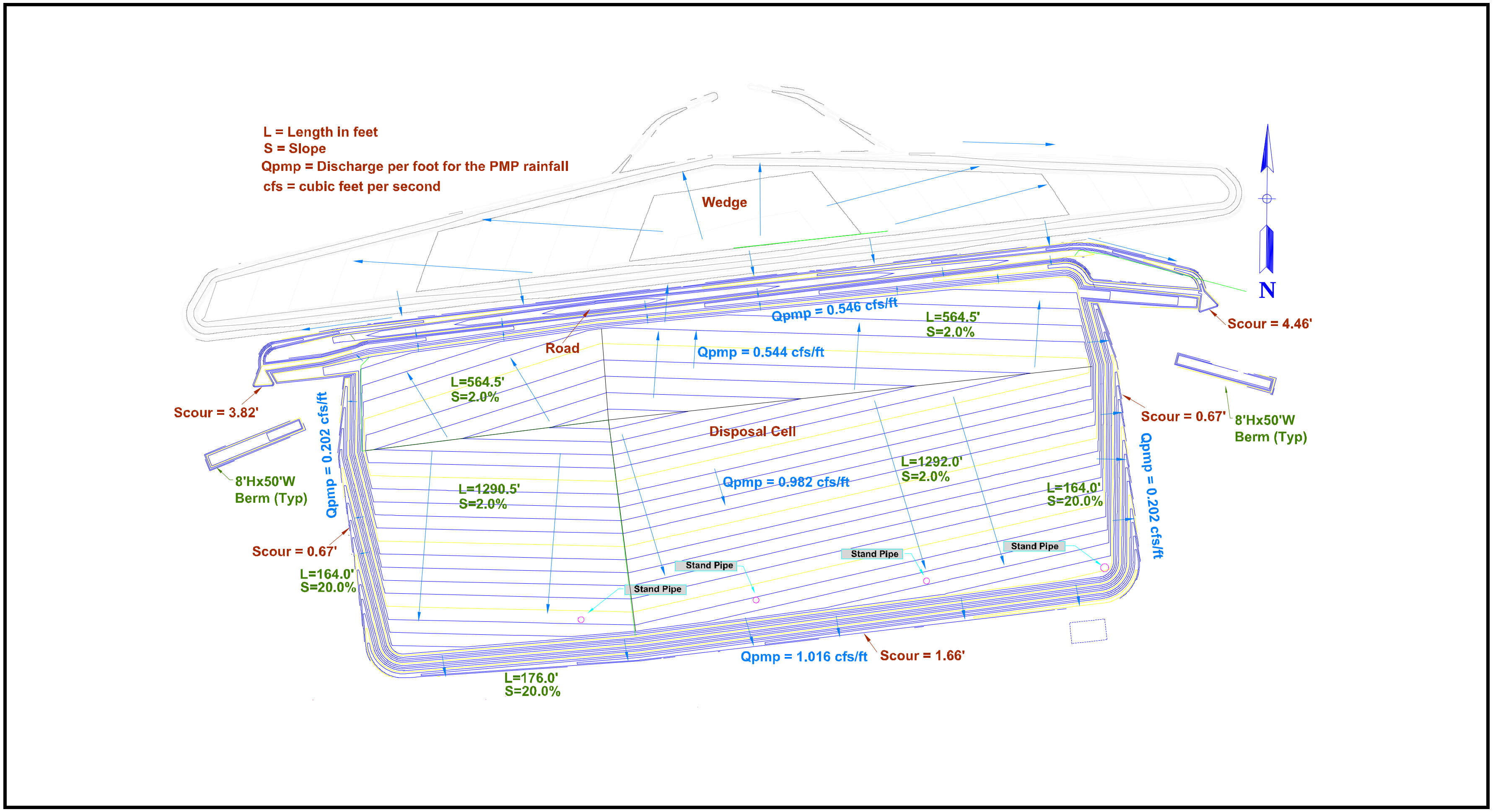

Figure 6-3. Layout of the Cell Showing Dimensions, Flows, and Scour Depths

6.4.2 Toe of Slopes

To protect the toe of the disposal cell, a toe apron will be constructed. The toe area at the base of the south side slope will be protected with 11.6-inch rock (D_{50} minimum). The base of the north slope will require a minimum D_{50} of 8.2 inches. The toe areas at the base of the west and east slopes, which have shorter slope lengths and contributing flow areas, will be protected with a minimum D_{50} of 4.7-inch rock. The thickness of these rock aprons will be a minimum three times the D_{50} and the minimum width will be the greater of 15 times the D_{50} or 10 feet. These rock aprons serve to dissipate flow energy as flow transitions to native ground and provide protection against scour. A summary of the required riprap sizes for erosion protection of the disposal cell toe aprons is provided in Table 6–3.

6.4.3 North Side of Cell

The north side of the disposal cell will experience runoff from the area between the Book Cliffs and the cell (Basins A and B, Figure 6–4). Protection from this runoff is provided by placing the excess material excavated during the construction of the disposal cell between the Book Cliffs and the disposal cell to divert flow around the cell. This material (the wedge) will be placed as shown in Figure 6-3 and compacted to a density of 90 percent of the laboratory determined maximum density in accordance with ASTM D 698. An access road between the cell and the wedge will be left in place after construction of the disposal cell is complete. Runoff from the south side of the wedge will flow to the east and west in a ditch along the north side of this road and runoff from the north side of the cell will flow east and west along the south side of the road.

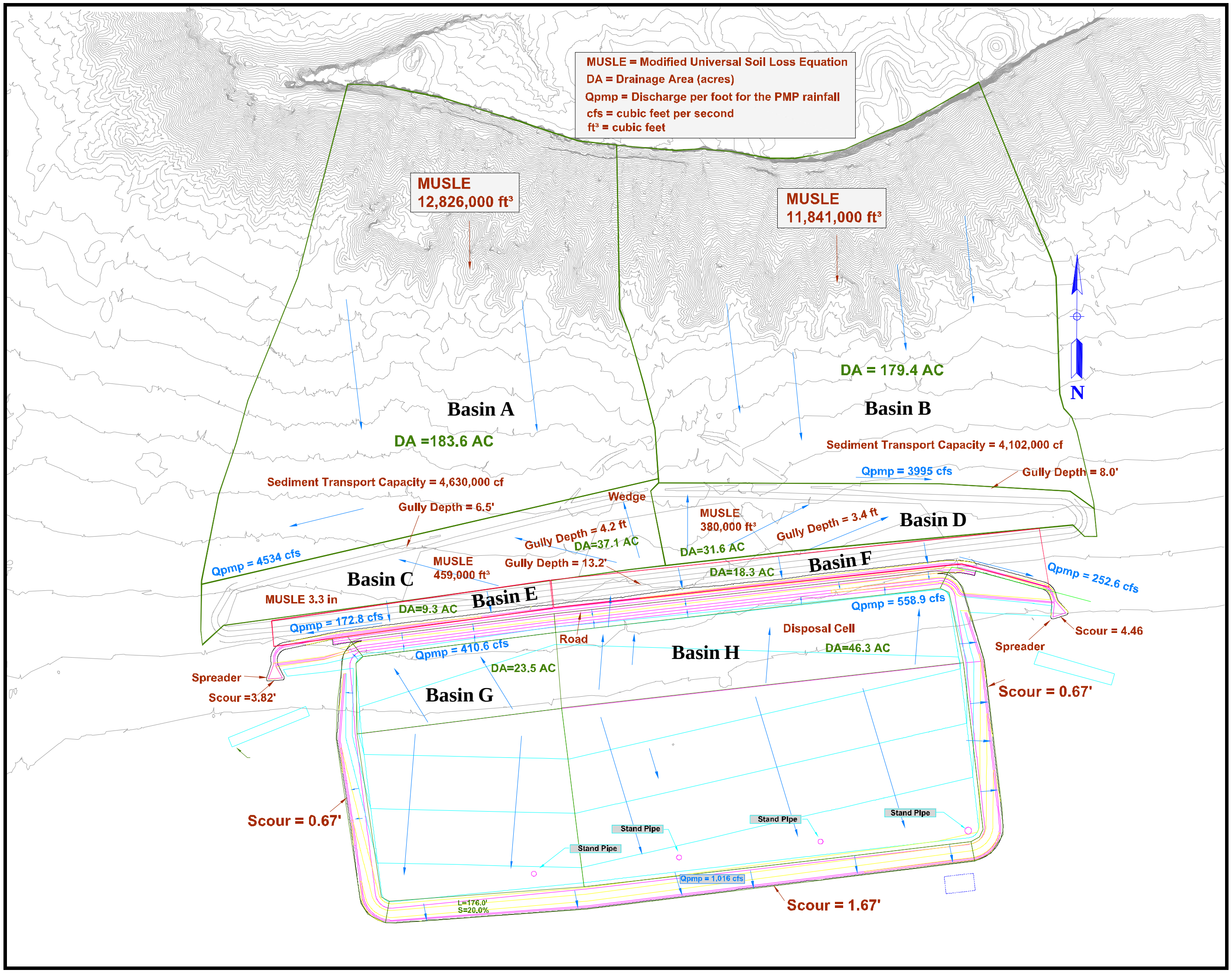

Figure 6-4. Layout of the Cell, Wedge, and Drainage Basins North of the Wedge.

6.4.3.1 Wedge

As shown in Figure 6–3, a wedge consisting of approximately 3 million yd^3 of soil excavated from the disposal cell will be placed between the Book Cliffs and the disposal cell to divert runoff from the Book Cliffs area around the cell. It is critical that the wedge remain intact and perform this function for the 1,000-year design life of the cell. Several possible mechanisms by which the cell could fail have been assessed. These are:

1. Erosion of the wedge by drainage from the watersheds to the north as the water flows to the east and the west along the north side of the wedge. This erosion will be mitigated by sediment supply from the Book Cliffs and the area between the cliffs and the wedge
2. Uniform erosion of sediment from the top of the wedge by precipitation falling directly on top the wedge.
3. Concentration of flow from precipitation forming gullies as it flows across the top of the wedge and down the sides to the northeast and northwest.
4. Uniform erosion of sediment from the south side of the wedge by precipitation falling directly on the south side slope of the wedge.
5. Concentration of flow from precipitation on the south slope of the wedge forming gullies as it flows to the south into the drainage along the north side of the access road. Detailed results are presented in Section 6.4.3.2 with the discussion of the area between the disposal cell and the wedge.

Detailed calculations of the processes analyzed are described in Calculations C-03 and C-04 of Addendum D.

<u>Erosion Along the North Side of the Wedge</u>

The potential for erosion from the north side of the wedge by runoff from the watersheds to the north was evaluated using methods for estimating the sediment transport capacity of flow in open channels described in NRC guidance (Johnson 2002). Estimates of sediment supply from these watersheds were made using the Modified Universal Soil Loss equation (MUSLE) (Nelson et al. 1986). The procedure was to:

- Compute the runoff from the watersheds between the top of the Book Cliffs and the wedge, Basins A and B, and from the top of the wedge, Basins C and D, for a series of design storms with return intervals from one year to the PMP.
- Calculate the potential sediment transport in a hypothetical channel that carries the runoff along the north side of the wedge and around the disposal cell using methods from Johnson 2002. A cross section of the northern edge of the wedge is shown in Figure 6–5.
- Calculate the sediment yield of the areas between the Book Cliffs and the wedge using the MUSLE (Nelson et al. 1986).
- Compute the net potential sediment addition to or subtraction from the wedge.

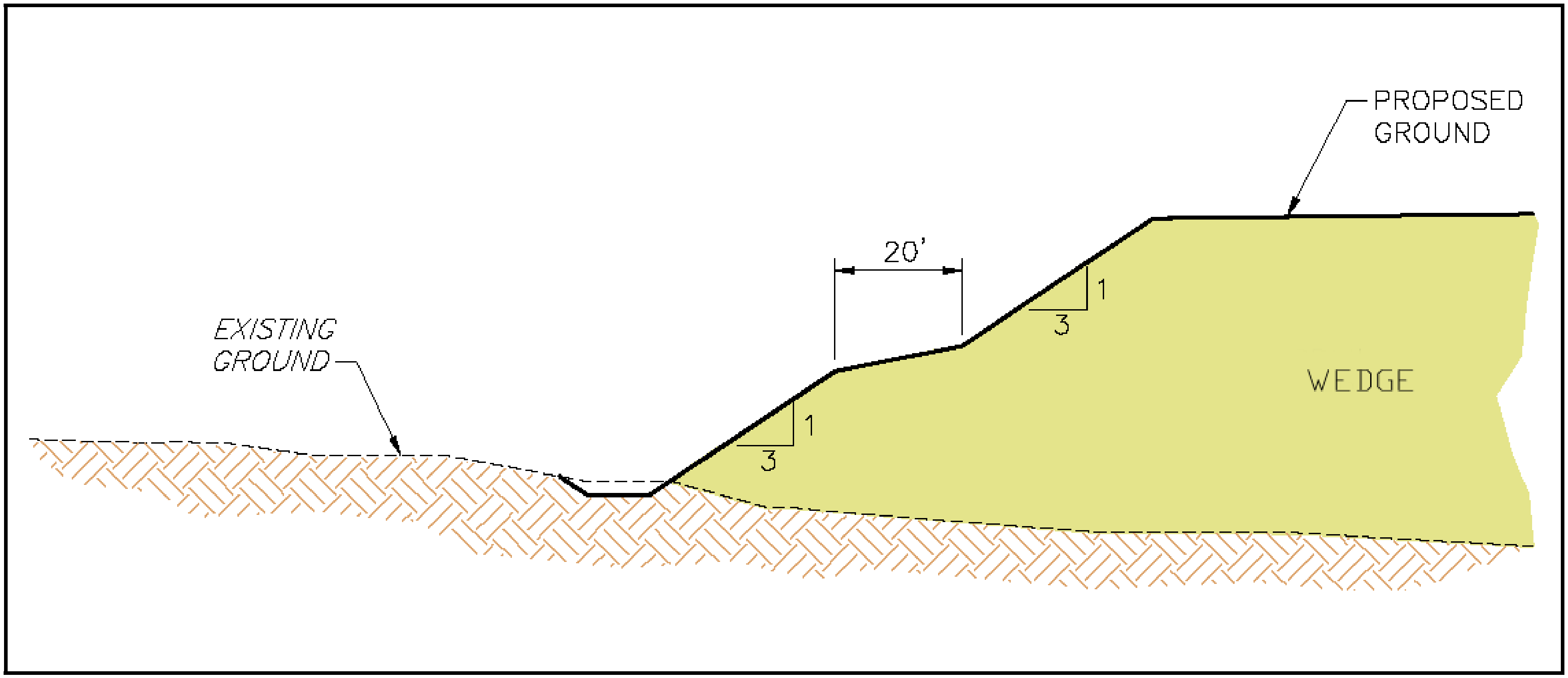

Figure 6-5. Cross Section of the North Edge of the Wedge and Existing Ground

Sediment Transport Potential

The storms selected for the erosion potential analysis are listed in Table 6–4. The series of storms for the runoff calculations was developed from the hydrology data in Attachment 1, Appendix E, and from NOAA Atlas 14 (NOAA 2004). The number of storms of each depth was chosen conservatively as follows:

- A storm with rainfall equal to or greater than the 1,000-year storm occurs on the average once every 1,000 years. Because the amount of rainfall may be anywhere between the 1,000-year storm and the PMP, the PMP was used for this storm.
- A storm with rainfall equal to or greater than the 500-year storm occurs on the average twice every 1,000 years. Because the amount of rainfall may be anywhere between the 500-year storm and the 1,000-year storm, the 1,000-year rainfall was used for this storm. Because the PMP has already accounted for one storm greater than the 500-year storm, only one 1,000-year storm was used.
- A storm with rainfall equal to or greater than the 200-year storm occurs on the average five times every 1,000 years. Because the amount of rainfall may be anywhere between the 200-year storm and the 500-year storm, the 500-year rainfall depth was used for this storm. Because two larger storms have already been applied, three 500-year storms were used.

Following this logic through storms of all return periods included in Atlas 14, the resulting in the distribution of rainfall and number of storms are listed in Table 6–4. All storms represent the 24-hour precipitation except for the PMP, which is a 6-hour depth.

Table 6-4. Distribution of Storms Used in Computing Sediment Transport Capacity Over a 1,000-Year Time Period

Return Interval Represented (yrs)	Return Interval Employed (yr)	Precipitation (in)	Number of Storms Equal or Greater than the Interval Represented	Number of Storms Employed
1000	PMP (6 hour)	9.00	1	1
500	1000	3.73	2	1
200	500	3.15	5	3
100	200	2.58	10	5
50	100	2.35	20	10
25	50	2.12	40	20
10	25	1.91	100	60
5	10	1.63	200	100
2	5	1.42	500	300
1	2	1.16	1000	500
< 1	1	0.93	Unknown	1000

The hydrologic analysis was performed as described in Section 6.2 with some differences in parameters as described below.

1. The watersheds between the Book Cliffs and the wedge are composed of both the Toddler-Ravola-Glenton (USDA 2007) soil family association, which are hydrologic Group B soils and the Hanksville family Badland complex, which are Group C soils. A runoff CN of 75 was assigned to the Group B soils as herbaceous arid rangeland in fair to poor condition and

a CN of 87 to the type C soils for the same use in poor condition (TR-55). The composite CNs are 79.4 and 81.1 for the western and eastern drainages, respectively. The resulting initial abstractions are 0.52 and 0.47 inches. Using a constant infiltration rate of 0.3 in/hr for the Group B soils and 0.03 in/hr for the Group C soils (USDA 2007) results in composite constant infiltration rates of 0.20 in/hr and 0.16 in/hr for the eastern and western drainages, respectively.

2. The wedge is constructed of compacted soil so that natural soil properties and the USBR unit hydrograph transform approach are not appropriate. The SCS unit hydrograph approach was used to transform excess rainfall into runoff. An initial abstraction of 0.2 inches and a constant infiltration rate of 0.1 in/hr were assumed for compacted soil.

Based on field observations of West Kendall Wash, the runoff was assumed to flow along the north side of the wedge in hypothetical trapezoidal channels with bottom widths of three feet and a side slope of 1.5:1. The slopes of the channels are 0.007 to the east and 0.009 to the west as determined from the topography of the site and the configuration of the wedge. A table was constructed of potential sediment transport in a 5-minute period as a function of discharge in each channel. The flow in each 5-minute period of a runoff hydrograph was then used to interpolate the potential sediment transport during each five minute increment. The sediment transport of each hydrograph was then computed as the sum of these 5-minute contributions.

NRC guidance (Johnson 2002) states that a runoff-to-rainfall ratio of 0.127 provides a reasonable estimate for the arid and semi-arid regions of the western United States. Because the total calculated runoff from the storms listed in Table 6–4 is less than 12.7 percent of the average annual rainfall of 9.97 inches at Thompson Springs (NOAA 2004) (approximately five miles east of the disposal cell), additional runoff and erosion were calculated. It was assumed that the runoff unaccounted for by the listed storms had a sediment concentration equal to that of the one-year storm. The additional erosion potential was computed as the product of the additional volume of runoff and the computed concentration of sediment in the runoff from the 1-year storm.

Sediment Supply

The runoff from the area between the top of the Book Cliffs and the wedge will transport sediment toward the wedge. The total sediment loss from the two watersheds delineated over a 1,000-year period can be estimated using the MUSLE (Nelson et al. 1986).

The equation is:

$$A = R \times K \times LS \times VM$$

where:

A = soil loss in tons per acre per year.
R = rainfall factor.
K = soil erodibility factor.
LS = topographic factor.
VM = dimensionless erosion control factor relating to vegetative and mechanical factors.

An average slope of 3.5 percent was used in the calculations. This is a representative slope for the area between the wedge and the base of the Book Cliffs. The soil loss (sediment supply) from the Book Cliffs area is most likely underestimated since the slope from the base to the top of the Book Cliffs is 40 to 50 percent and the erodibility factor of the soil in that area is comparable for the two soil types in the watershed. More sediment than calculated should be eroded from this area, but much of the additional sediment will be deposited as the slope flattens near the wedge.

The relative sediment yield of a more realistic watershed shape has been assessed with the Revised Universal Soil Loss Equation (RUSLE) using the computer program RUSLE2 (NRCS 2001). In this simulation three slopes were used, 1,000 feet at 40 percent to represent the Book Cliffs, 800 feet at 3.5 percent and 800 feet at 2.5 percent to represent the area between the base of the Book Cliffs and the wedge. A RUSLE2 simulation was also performed with the same three segments, but with each having a slope of 3.5 percent to mimic the calculations performed using the MUSLE. These calculations yield more than three times the sediment delivery at the north edge of the wedge with the varying slope than with the single slope of 3.5 percent indicating that the assumption of a single 3.5 percent slope in the MUSLE calculation was conservative.

The MUSLE was also used to calculate the volume of sediment that would be lost from the top of the wedge to the area on the north side of the wedge. The volumes of sediments over a 1,000-year period calculated with the MUSLE, and the sediment transport potential along the north side of the wedge are summarized in Table 6–5.

Table 6-5. Sediment Budget for the North Side of the Wedge

Area	Sediment Transport Capacity (ft^3)	Sediment Yield from MUSLE (ft^3)
Channel along wedge to the west	4,630,000	
Channel along wedge to the east	4,102,000	
Western area between Book Cliffs and the wedge		12,826,000
Eastern area between Book Cliffs and the wedge		11,841,000
Western portion of the top of the wedge		459,000
Eastern portion of the top of the wedge		380,000
Total sediment yield toward the west portion of the wedge		13,285,000
Total sediment yield toward the east portion of the wedge		12,221,000
Ratio of sediment supply from Book Cliffs to channel sediment transport capacity (west)	2.8:1	
Ratio of sediment supply from Book Cliffs to channel sediment transport capacity (east)	2.9:1	

These results indicate that the water flowing along the north side of the wedge to the west and the east does not have sufficient sediment transport capacity to remove the supply of sediment from the areas between the top of the Book Cliffs and the wedge. The northern edge of the wedge is expected to expand northward during the 1,000-year life of the disposal cell and offer increasingly more protection to the cell as time passes. Even discounting the sediment supply from the north, the total sediment transport potential over 1,000 years is only about 12 percent of the volume of the wedge.

Erosion from the Top of the Wedge

Due to the nearly flat slope on top of the wedge, the predicted erosion from the top of the wedge, using the MUSLE, is only 3.3 inches over a 1,000-year period. Since the height of the wedge ranges from 28 to 48 feet, this is an insignificant depth of erosion.

Gully Formation on the Wedge

In addition to potential erosion of the wedge by runoff from the Book Cliffs area and sheet and rill erosion from precipitation directly on top of the wedge, runoff from the top of the wedge is expected to form gullies on the top and on the steep slopes on the north side as the runoff from the top of the wedge flows to the northwest and the northeast. The potential depth of these gullies can be estimated with an approach detailed in NRC guidance (Johnson 2002). The estimated maximum depth of gully incision is

$$D_{\max} = G_f L_{total} S$$

where:

G_f = function of the total volume of runoff and the embankment height.
L_{total} = maximum length of flow contributing to gully formation.
S = the original slope of the embankment.

The results of these calculations are summarized in Table 6–6.

Table 6-6. Summary of Calculations of Gully Depths on the Wedge

Description	Top Slope West	Side Slope West	Top Slope East	Side Slope East
Height of embankment (ft)	10	18	8	22
Horizontal length of embankment (ft)	1339	95	1254	92
Length of embankment along slope (ft)	1339	96.7	1254	94.6
Maximum gully depth (ft)	4.2	6.5	3.4	8
Gully width at maximum depth (ft)	7.7	12.7	5.9	16
Maximum distance from top of slope (ft)	248	4.1	204	4.7

Runoff from precipitation on the south side slope is also expected to form gullies on these steep slopes. The calculation of these gully depths is described in Section 6.4.3.2 and summarized in Table 6–9.

6.4.3.2 Drainage Between the Wedge and the Cell

An access road between the cell and the wedge (Figures 6–6 and 6–7) will be left in place after construction of the disposal cell is complete. Runoff from rainfall on the north slope of the cell (Basins G and H) will flow east and west in a ditch on the south side of the access road. Runoff from rainfall on the south slope of the wedge (Basins E and F) will flow east and west in a ditch on the north side of the road. The sides of the ditch on the south side of the road will be the north slope of the cell and the south slope of the road berm. The bottom of the ditch will be the north toe apron of the cell. The ditch on the north side of the road will be formed by the north side of the road berm, the south slope of the wedge, and a 15-foot wide flat bottom.

The ditches on the north side of the road will continue for several hundred feet to the east and west of the cell boundary where they will turn southerly and discharge via 100 foot wide spreaders. The ditches on the south side of the road will increase in width from 15 to 20 feet at the boundary of the disposal cell to carry the flow to the spreaders along the north side of berms. The berms will ensure that if there is any spillover from the ditches, the flow will still be routed to the spreaders. The configuration of these ditches and berms is shown in Figures 6–6 and 6–7.

Two drainage areas were delineated between the wedge and the access road draining to the southwest and to the southeast (Basins E and F). Two more were delineated between the watershed divide on top of the cell and the access road to the northwest and the northeast (Basins G and H). Pertinent properties of the four drainage areas are presented in Table 6–7.

Table 6-7. Drainage Area Characteristics

Drainage Area	Area (acres)	Max Flow Length (ft)	Time of Concentration (min)	Lag = 0.6 Tc
Southwest wedge side slope (Basin E)	9.3	2062	23.38	14.0
Southeast wedge side slope (Basin F)	18.3	3470	35.53	21.3
Northwest portion of cell (Basin G)	23.5	1471	25.38	15.2
Northeast portion of cell (Basin H)	46.3	2891	41.96	25.2

The calculated depths of the gullies that will form on the south side slope of the wedge over a period of 1,000 years are substantial, ranging from 9.6 to 13.2 feet. Nonetheless, these gullies are not expected to threaten the integrity of the wedge. In each case the height of the wedge is more than three times the calculated gully depth and the minimum north-south dimension of the wedge is about 120 feet, much greater than the expected gully depth.

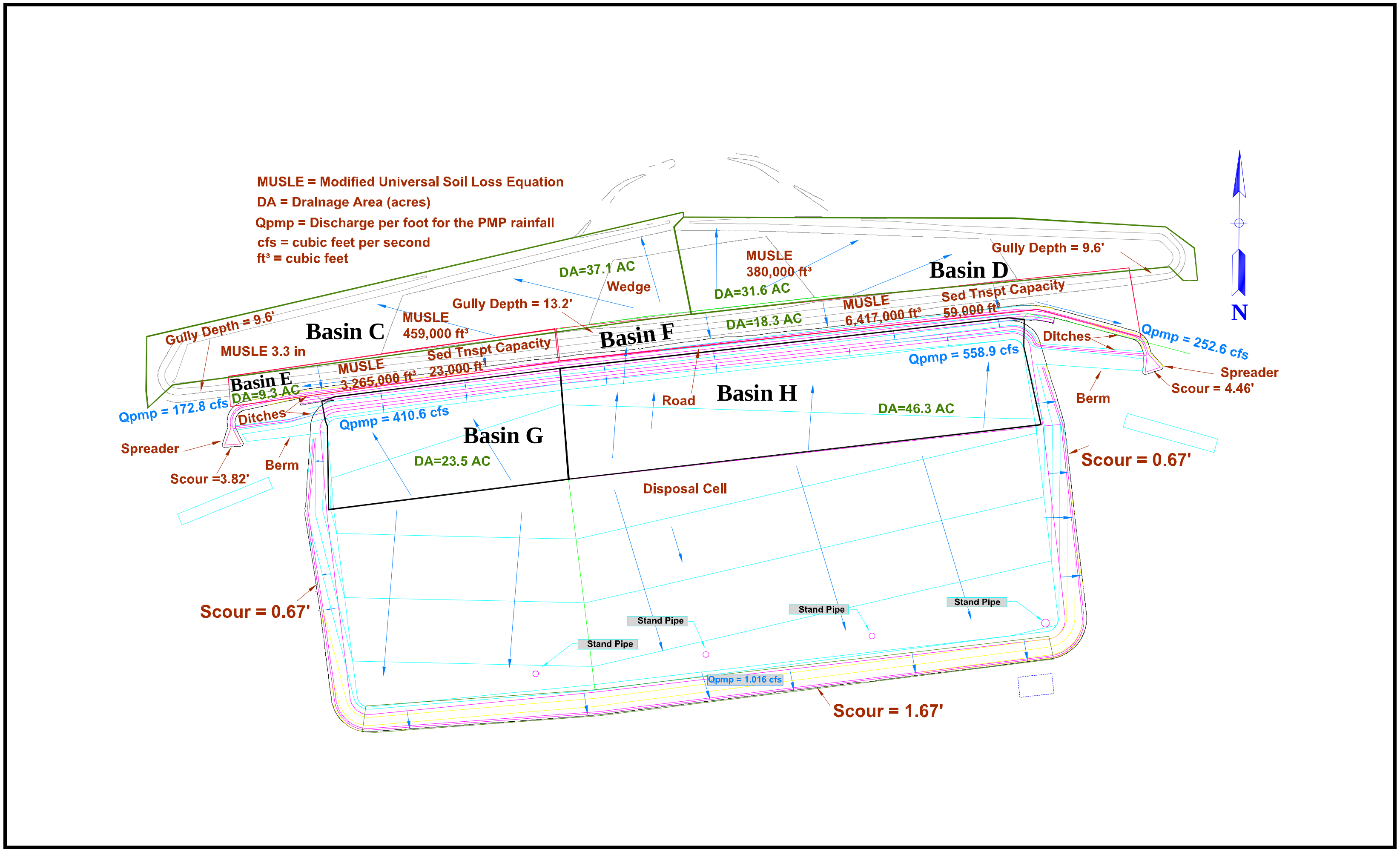

Figure 6-6. Configuration of the Cell, the Wedge, and the Drainage Between the Cell and the Wedge

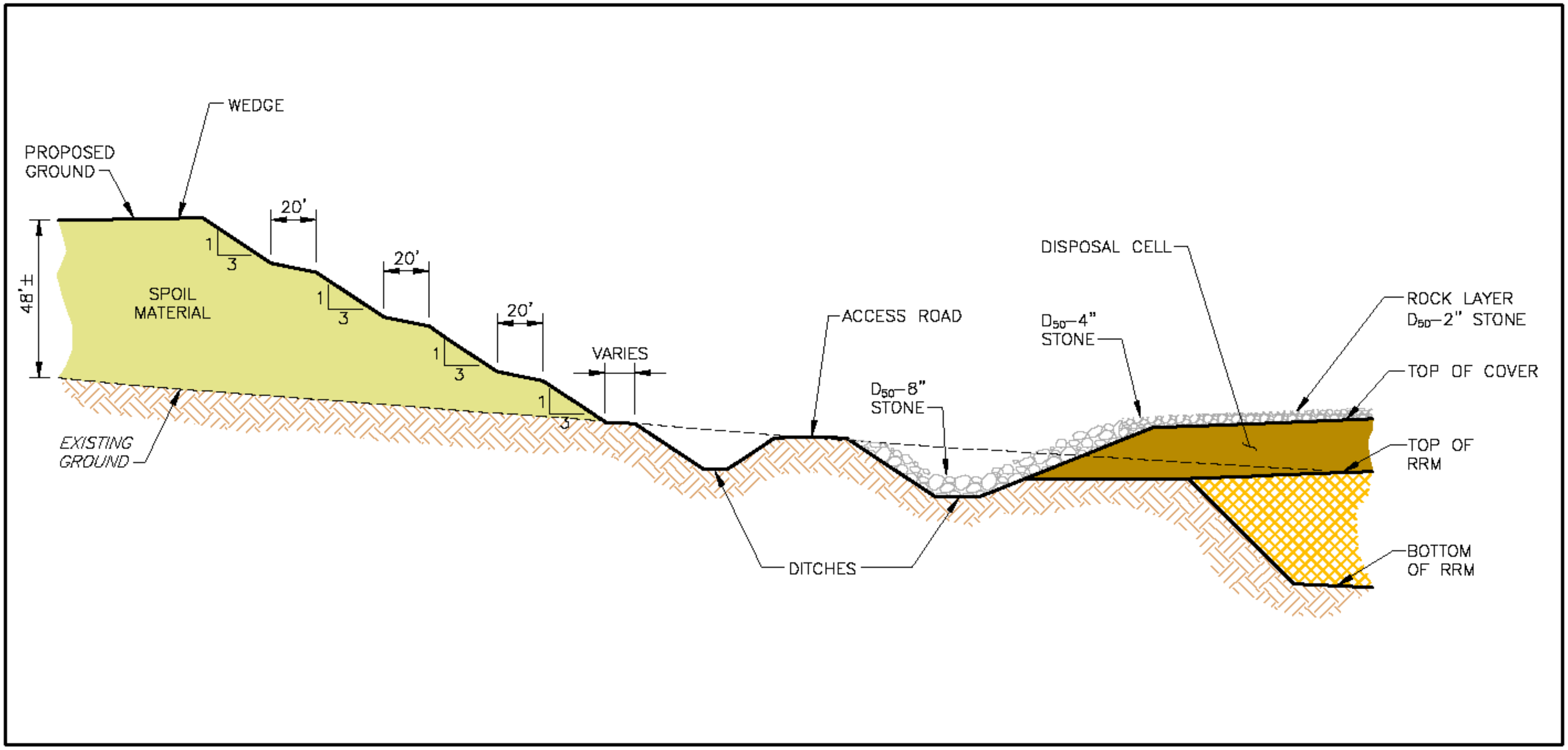

Figure 6-7. Cross Section Through the North Slope of the Cell to the Top of the Wedge

Sediment Budget

A sediment budget was developed for the ditch between the access road and the wedge using the procedures described in Section 6.4.3 for the area between the Book Cliffs and the wedge. The results of this analysis are presented in Table 6–8.

Table 6-8. Sediment Budget for the Area Between the Road Berm and the Wedge

Area	Sediment Transport Capacity (ft^3)	Sediment Yield from MUSLE (ft^3)
Channel along south side of wedge to the west	23,000	
Channel along south side of the wedge to the east	59,000	
Western portion of the south side of the wedge		3,265,000
Eastern portion of the south side of the wedge		6,417,000
Ratio of sediment supply to transport capacity (west)	143:1	
Ratio of sediment supply to transport capacity (east)	108:1	
Volume of ditch to the west	588,000 ft^3 (18% of potential sediment supply)	
Volume of ditch to the east	1,156,400 ft^3 (18% of potential sediment supply)	

These results indicate that the water flowing in the ditch along the south side of the wedge to the west and the east does not have sufficient sediment transport capacity to remove the supply of sediment from the south side slope of the wedge. A sufficient volume of sediment will erode from the south side slope of the wedge to completely fill the ditch north of the access road in approximately 180 years. Because of the geometry of the wedge and the ditch, the flow in the ditch will increase from the high point near the east-west center of the wedge and develop increasingly more sediment transport capacity as the flow proceeds downstream. The nearly uniform sediment supply along the length of the ditch and the increase in sediment transport capacity in a downstream direction will cause the bottom slope of the ditch to increase over time. This will increase the sediment transport capacity of the ditch, but it is not expected to increase enough to remove the total sediment supply from the side slope of the wedge.

Erosion from Side Slope of the Wedge

Erosion of sediment from the south side slope of the wedge was estimated with the MUSLE. Dividing the volumes in Table 6–8 by the area of the south slope of the wedge indicates that soil to a depth of approximately 12 feet will be lost from the south side slope of the wedge. Since the south side slope of the wedge will be 30 feet high at the east and west ends and 48 feet high in the center and the top of the wedge is over 230 feet wide at the west end and 150 feet at the east end, this depth of erosion, while substantial, will not threaten the integrity of the wedge.

In addition to potential erosion of the wedge by sheet and rill erosion from precipitation directly on the south side slope of the wedge, the runoff from precipitation on the south side slope is expected to form gullies on these steep slopes. The potential depth of these gullies can be estimated using the same approach as was applied to the wedge and described in Section 6.4.3.1.

The results of these calculations are summarized in Table 6–9.

Table 6-9. Summary of Calculations of Gully Depths Between the Cell and the Wedge

Description	End of South Side Slope	Center of South Side Slope
Height of embankment (ft)	30	48
Horizontal length of embankment (ft)	118	176
Length of embankment along slope (ft)	121.8	182.4
Maximum gully depth (ft)	9.6	13.2
Gully width at maximum depth (ft)	20	28.5
Maximum distance from top of slope (ft)	35	58

While the predicted depths of gullies that will form on the south side slope of the wedge over a period of 1,000 years are substantial (ranging from 9.6 to 13.2 feet), the gullies are not expected to threaten the integrity of the wedge. In each case the height of the wedge is more than three times the calculated gully depth and the minimum north-south dimension of the wedge is approximately 120 feet, much greater than the expected gully depth.

Rock in Channels

The channels on the north side of the access road carrying runoff from the south side slope of the wedge to the east and to the west will not be armored for most of their lengths because the sediment supply from the south side of the wedge will far exceed the sediment transport capacity of flow in the ditches. Beginning approximately 100 feet upstream of each end of the access road, rock will be placed in the channels to protect them against erosion from that point to the spreaders that terminate the channels. When the channels fill with sediments, the flow will leave the channels and flow southward toward and over the access road shown in Figures 6–6 and 6–7. In addition, flow from the top of the cell and the area south of the access road (north of the cell) will flow to the east and to the west in trapezoidal ditches with 3:1 side slopes on the north side and 5:1 side slopes (cell side slope) along the south side. The bottom width increases from 15 to 20 feet as the ditches extend past the edges of the cell and the south side slope transitions to 3:1. The flow in these ditches will continue along the north side of berms that extend from the cell side slopes to the spreaders. Any overflow from the ditches north of the road will also be intercepted by these ditches and berms and routed to the spreaders.

The peak flow resulting from the PMP in each of these areas has been calculated using the SCS unit hydrograph technique with an initial abstraction of 0.0 inches and a constant infiltration rate of 0.1 inches/hour. The results of these calculations are included in Table 6–10.

Table 6-10. Peak Flows from the Area Between the Wedge and the Cell for the PMP

Peak Flow from PMP	South Side of Wedge (West) (Basin E)	South Side of Wedge (East) (Basin F)	Flow from Cell (West) (Basin G)	Flow from Cell (East) (Basin H)
Drainage area (acres)	9.3	18.3	23.5	46.3
Time of concentration (min) (T_c)	23.4	35.5	25.4	42.0
Lag (min) = $0.6T_c$	14.0	21.3	15.2	25.2
Peak flow (cfs)	172.8	252.6	410.6	558.9

The D_{50} for stone erosion protection was determined using the Safety Factor Method (Nelson et al. 1986). The results of these calculations are presented in Table 6–11. Each of the channels north of the road berm is assumed to have a bottom width of 10 feet and side slopes of 3:1.

Since the ditches north of the access road are expected to fill with sediment and will overflow into the ditches south of the access road, the peak flows in the ditches south of the road have been assumed equal to the sum of the flows north and south of the road.

Table 6-11. D_{50} of the Stone Required for Erosion Protection

D50 for Erosion Protection	South Side of Wedge (West)	South Side of Wedge (East)	Flow from Cell (West)	Flow from Cell (East)
Peak Flow (cfs)	172.8	252.6	583.4	811.5
Channel Slope	.0094	.0076	.0089	.0063
D50 (inches) on 3:1 Side of Channel	3.3	3.4	5.1	4.4
D50 (inches) on Bottom of Channel	2.6	2.6	3.9	3.4
D50 (inches) on Bottom of 5:1 Side of Channel Channel			4.2	3.7
Portion of Channel Draining the South Side of the Wedge after it has Turned Southerly				
Channel Slope	.0175	.0175		
D50 (inches) on Side of Channel	5.8	7.2		
D50 (inches) on Bottom of Channel	4.5	5.6		
Channel South of Access Road beyond Cell Boundaries				
D50 (inches) on 3:1 Side of Channel			4.7	4.1
D50 (inches) on Bottom of Channel			3.6	3.2

Rock and Scour at Spreader Outlets

Flow from the channel north of the access road and from the top of the cell will combine at the spreader for discharge onto natural ground. The peak flows from the PMP have been added to estimate the peak flow from each spreader. To obtain the flow per unit width, the peak flow has been spread over a width of 100 feet. To account for potential channelization in the rock of the spreaders, the unit flow has been multiplied by three for calculation of the required D_{50} of rock for erosion protection and potential scour depth at the outlet of each spreader. The D_{50} was calculated using the Safety Factor Method assuming a channel with 3:1 side slopes, a one foot bottom width and a channel slope of 2.3 percent. Scour was calculated using the Federal Highway Administration culvert scour equations (DOT 1983) assuming flow in a V-shaped ditch with 2:1 side slopes. The results are summarized in Table 6–12. To protect the toe of the spreaders against head cutting by scour from the discharge of the PMP runoff, a 10H to 1V buried rock blanket will be constructed downstream of the toe to protect against erosion down to the expected depth of scour. The D50 of the stone in the buried rock blanket will be at least 9.7 inches at the west spreader and 11.6 inches at the east spreader.

Table 6-12. Calculated Depth of Scour at Spreader Outlets

	West Spreader	East Spreader
Peak flow from channel (cfs)	172.8	252.6
Peak flow along berm (cfs)	410.6	558.9
Combined peak flow (cfs)	583.4	811.5
Concentration factor	3	3
Design flow (cfs/ft)	17.50	24.35
Minimum rock D_{50} (in)	4.5	5.2
Estimated scour depth (ft)	3.82	4.46

6.4.3.3 Summary of Wedge Longevity Analyses

Calculations C-03 and C-04 have been performed to assess whether the wedge will continue to protect the cell during the 1,000-year design life. Three possible processes by which the integrity of the wedge might be compromised have been considered.

1. Erosion of the wedge by runoff from the area between the Book Cliffs and the wedge will tend to erode the wedge as it is routed to the southwest and northwest around the wedge and the disposal cell. The results of these calculations indicate that the total sediment carrying capacity of the runoff as it flows around the wedge is approximately 12 percent of the volume of the wedge. In addition, the sediment supply from the Book Cliffs area computed from the MUSLE will be approximately three times the sediment transport capacity of the flow around the wedge, resulting in a net gain in the volume of the wedge over the design life of the disposal cell.
2. Precipitation falling directly on the top of the wedge will run off toward the northeast and the northwest. This runoff will erode the wedge from the top. Application of the MUSLE to estimate the volume of sediment lost from the wedge through this mechanism indicates that the wedge will be reduced in average height by about three inches to four inches. With a design height of the wedge ranging from approximately 28 feet to 48 feet, this loss of soil will not threaten the integrity of the wedge.
3. The third mechanism considered is concentration of flow as it runs off the top of the wedge and the consequent formation of gullies both on top of the wedge and on the steep slopes to the northwest, the northeast, and the south. The calculations show that the gullies formed by runoff would not pose a serious threat to the integrity of the wedge.

Based on these calculations, the wedge will protect the disposal cell from runoff from the areas to the north and continue to function over the 1,000-year design life of the cell.

6.5 Rock Durability

Several sources of erosion protection rock have been evaluated and are potentially suitable for use on the cell. Rock used for erosion protection on the disposal cell must meet the specified scoring criteria listed in Table 6–13 to ensure meeting NRC durability requirements.

Table 6-13. NRC Table of Scoring Criteria For Rock Quality

Laboratory Test	Weighing Factor			Score										
	Limestone	**Sandstone**	**Igneous**	10	9	8	7	6	5	4	3	2	1	0
Specific Gravity	12	6	9	2.75	2.70	2.65	2.60	2.55	2.50	2.45	2.40	2.35	2.30	2.25
Absorption, %	13	5	2	0.10	0.30	0.50	0.67	0.83	1.0	1.5	2.0	2.5	3.0	3.5
Sodium Sulfate, %	4	3	11	1.0	3.0	5.0	6.7	8.3	10.0	12.5	15.0	20.0	25.0	30.0
LA Abrasion, % (100 revolutions)	1	8	1	1.0	3.0	5.0	6.7	8.3	10.0	12.5	15.0	20.0	25.0	30.0
Schmitt Hammer	11	13	3	70	65	60	54	47	40	32	24	16	8	0

Notes

1. Scores were derived from Tables 6.2, 6.5, and 6.7 of NUREG/CR-2642, Long-Term Survivability of Riprap for Armoring Uranium Mill Tailings and Covers: A Literature Review, 1982.
2. Weighing Factors are derived from Table 7 of "Petrographic Investigations of Rock Durability and Comparisons of Various Test Procedures," by G.W. Dupuy, Engineering Geology, July 1965. Weighing factors are based on inverse of ranking of test methods for each rock type. Other tests may be used; weighing factors for these tests may be derived using Table 7, by counting upward from the bottom of the table.
3. Test methods should be standardized, if a standard test is available and should be those used in NUREG/CR2642, so that proper correlations can be made.

Acceptable Rock Scores

An acceptable rock score depends on the intended use of the rock. The rock's score must meet the following criteria:

- For occasionally saturated areas, which include the top and sides of the cell, the rock must score at least 50 percent or the rock is rejected. If the rock scores between 50 percent and 80 percent the rock may be used, but a larger D_{50} must be provided (oversizing). If the rock score is 80 percent or greater, no oversizing is required.
- For frequently saturated areas, which include all channels and buried slope toes, the rock must score 65 percent or the rock is rejected. If the rock scores between 65 percent and 80 percent, the rock may be used, but must be oversized. If the rock score is 80 percent or greater, no oversizing is required.

Rock Oversizing

Oversize rock as follows:

- Subtract the rock score from 80 percent to determine the amount of oversizing required. For example, a rock with a rating of 70 percent will require oversizing of 10 percent (80 percent - 70 percent = 10 percent).
- The D_{50} of the rock shall be increased by the oversizing percent. For example, a rock with a 10 percent oversizing factor and a D_{50} of 12 inches will increase to a D_{50} of 13.2 inches.
- The final thickness of any layer of oversized rock shall increase proportionately to the increased D_{50} rock size. For example, a layer thickness equals twice the D_{50}, such as when the plans call for 24 inches of rock with a D_{50} of 12 inches, if the stone D_{50} increases to 13.2, the thickness of the layer of rock with a D_{50} of 13.2 should be increased to 26.4 inches.

6.6 Rock Sources

Determination of the rock sources to be used for disposal cell aggregate and riprap requirements is an important component of the design. After selection of Crescent Junction as the site for the disposal cell and prior to final cell design, a number of potential rock sources were evaluated. Results from that investigation are provided in the *Evaluation of Aggregate and Riprap Source Areas for Rock Cover of Crescent Junction, Utah, Disposal Cell* (DOE 2007). Rock sources were evaluated from the existing LeGrand Johnson Pit for aggregate and the existing Papoose Quarry for riprap. A potentially new source for aggregate was investigated at the Green River Terrace site, and potentially new sources for riprap were investigated at the Valley City, Little Valley, Tenmile Wash, Blue Hills Road, and Cane Creek Anticline sites.

During the final design process, one additional site, Fremont Junction, was identified and is recommended as the sole rock source for both aggregate and riprap (Figure 6–8). The Fremont Junction rock source is approximately 4 miles east of Fremont Junction. Areas closer to the disposal cell may be investigated as alternative sources of durable rock, including orthoquartzite of the Cedar Mountain Formation at the Blue Hills Road site, before rock emplacement at the disposal cell.

Rock quarried from the Fremont Junction site was previously selected by DOE and approved for use by the NRC as cover material for a previously constructed Title I UMTRA disposal cell located near Green River, Utah (DOE 1988). Results presented in the following sections are based on a combination of rock evaluations of the Fremont Junction site previously conducted in 1988 by DOE for use on the UMTRA Green River disposal cell and more recent information obtained in 2007 and 2008 for use on the Crescent Junction disposal cell (Addendum F).

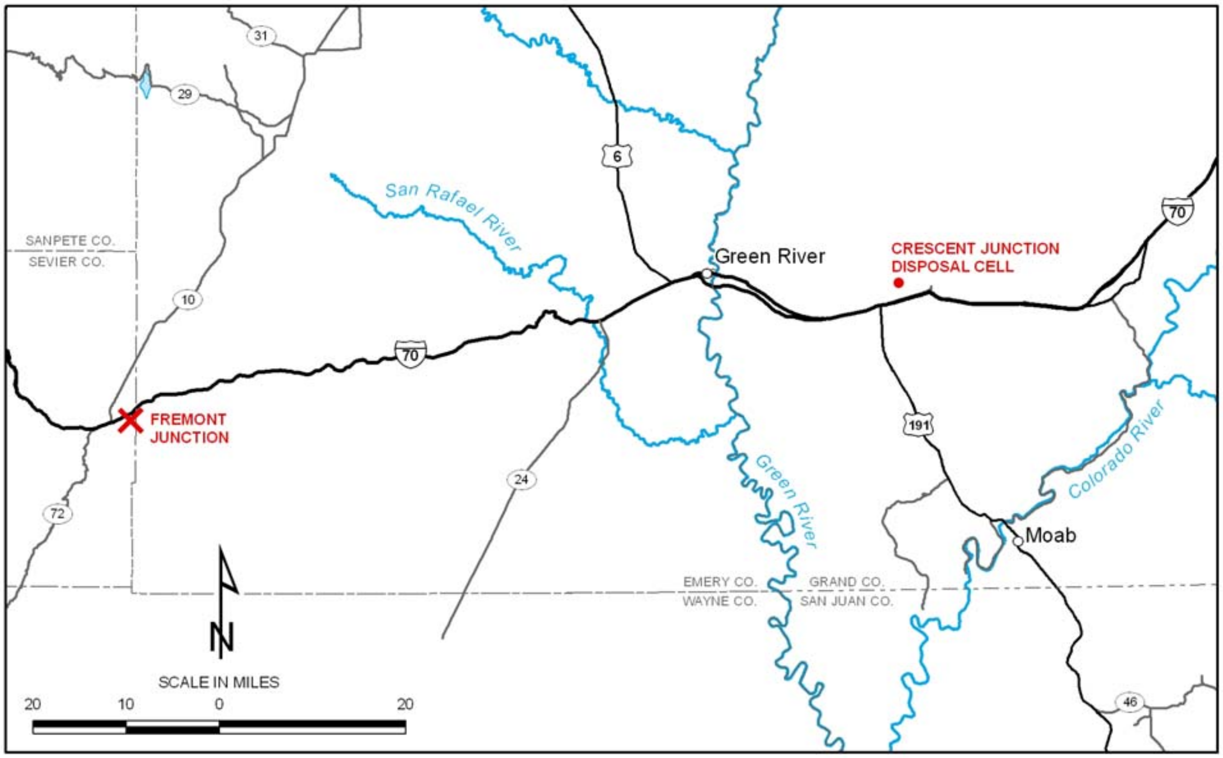

Figure 6–8. Location of Fremont Junction Site

6.6.1 Site Description

The Fremont Junction site consists of 400 acres of property owned by the State of Utah School and Institutional Trust Lands Administration (SITLA) that has been permitted for the purposes of mining ordinary sand and gravel (Figure 6–9). The property is at the east edge of Sevier County, on the Walker Flat 7.5-minute USGS topographic quadrangle, and consists of the S½ and the E½ NE¼ of Section 36, T23S, R5E. The deposit (rock source) consists of Quaternary pediment-mantle material that overlies bedrock composed of the Blue Gate Member of the Upper Cretaceous Mancos Shale (Doelling 2004). The pediment-mantle at the property forms a surface covered mostly by sagebrush that slopes gently at about 1 degree to the northeast. Elevation of the surface, which can be considered as the south part of Ivie Creek Bench (Figure 6–9), decreases from 6,640 ft at the southwest to 6,500 ft at the northeast. Characteristic of the pediment-mantle material is its significant content of subrounded vesicular basalt boulders and cobbles, which originated from flows of Neogene to as old as Oligocene age in the Fish Lake Plateau, Hilgard Mountain, and Thousand Lake Mountain areas that are 15 to 20 miles southwest and south-southwest of the property.

The following sections describe the pediment-mantle material as documented in previous investigations conducted in 1988 and observed during more recent investigations conducted in 2008. The 1988 characterization of the deposit was from an area about 1 mile northeast of the permit area. Some of the descriptions of the deposit in the area investigated in 1988 are not the same as the descriptions in the permit area observed from the 2008 evaluations. Discrepancies noted between the descriptions are also explained in the following sections.

6.6.2 Observations from Field Sampling DOE Conducted in 1988

DOE conducted sampling in 1988 to characterize and field verify the rock from the Fremont Junction site for use as cover material on the Green River UMTRA disposal cell. Samples were collected with a backhoe from three existing test pits (TP-2, TP-3, and TP-5) shown in Figure 6-9. These test pits are in material from the same pediment-mantle deposit as in Section 36, but they are outside the permit area in Section 25 (TP-2 and TP-3) and Section 30 (TP-5). Excavated material was stockpiled adjacent to each test location. Particle sizes in the stockpiles ranged in diameter from less than 1 inch to 36 inches. Representative hand-picked samples ranging in size from 8 inches to 15 inches in diameter were obtained from the stockpiles for laboratory durability tests and petrographic examination. Results of the 1988 field investigation and laboratory tests are presented in Addendum F and summarized in Section 6.6.4.

The following description of the deposit was documented during the sampling in 1988. The investigators reported that the upper portion of the deposit was divided into two distinct layers. The upper layer, which was as much as 5 ft thick, consisted of clayey sand and clayey silt. This layer was considered overburden. Immediately underlying the upper layer was a 5- to 15-ft-thick layer of mixed sand and gravel (as much as 3-inch size), cobbles (3- to 12-inch size), and boulders (larger than 12-inch size). Material gradation was reported to be variable in this lower layer. Approximately 1 to 3 ft of the uppermost zone of the lower layer contained as much as 15 percent of friable, clinker-like, weathered basalt and basalt particles with friable weathering rinds. The investigators reported that these obviously weathered basalt particles were not observed in the underlying portion of the lower layer, which had a maximum thickness of about 12 ft.

Based on visual examination of the material, DOE estimated that about 80 percent of the boulders, cobbles, and gravels in the lower bed were basalt, about 10 percent were quartz and/or quartzite, and about 10 percent were fine-grained sandstone. Reportedly, some granitic cobbles were also found in TP-3. Sandstone particles were approximately gravel-to-cobble size, and were as much as 8 inches in diameter. The 1988 investigators reported that the basalt fraction of the deposit was relatively unweathered, except for the highly weathered basalt (indicated by weathering rinds) in the upper 1 to 3 ft of the lower layer.

It should be noted that the 1988 observation of as much as 15 percent of friable, clinker-like (red), highly weathered basalt in the lower layer was not observed during the 2008 evaluations described in Section 6.6.3. Neither highly weathered basalt nor weathering rinds were observed in the recent investigations at the permit area and the area of TP-5 where some of the 1988 samples were collected. Also, the percentage of basalt boulders in the pediment-mantle deposit (95 percent) was much higher in the May 2008 evaluation than in the 1988 observation (80 percent). No granitic cobbles were seen in the pediment-mantle deposit during the May 2008 evaluation.

6.6.3 Field Sampling and Evaluations DOE Conducted in 2007 and 2008

6.6.3.1 Fall 2007 Sampling

In the fall of 2007 a hand-picked sample of gray, dense basalt was collected by DOE and submitted for laboratory durability tests. The sample (HP-1) was from a surface pile along the county road in the SW¼ Section 36 (Figure 6–9), and was from material within a few feet of the surface that had been disturbed by ditch construction along the roadside. It is believed to be representative of dense, gray basalt of the deposit available within a few feet of the surface. Results of the laboratory tests for this sample are summarized in Section 6.6.4.

6.6.3.2 February 2008 Sampling and Evaluation

DOE conducted a qualitative evaluation of the Fremont Junction deposit in February 2008 to: (1) determine the consistency of the basalt characteristics across the site, (2) evaluate the amount of overburden at various locations across the site, and (3) estimate the volume of the deposit to ensure the quantity of rock is sufficient for the cell requirements. Four pits (BH-1 through BH-4) were excavated at the site with a backhoe at the locations shown in Figure 6–9. Three of the pits (BH-1 through BH-3) are on the property in the SW¼ Section 36. Pit BH-4 is outside the property in the NW¼ Section 36, but in the same pediment-mantle material as the deposit in the property. Excavated material was stockpiled adjacent to each location. A representative sample of each of “red” and “gray” basalt was collected from the BH-1 stockpile location for laboratory durability tests. From those four pits, red basalt appeared to compose approximately 5 percent of the basalt clasts in the stockpiled material. The red basalt sampled was softer than the gray basalt, possibly because it was vesicular. Red basalt seen in the May 2008 evaluation of the overall deposit appeared to be, with few exceptions, as hard and durable as the gray basalt. Gray basalt was dense and hard and appeared to represent about 95 percent of the basalt clasts in the deposit. Results of the laboratory tests for these two samples are summarized in Section 6.6.4.

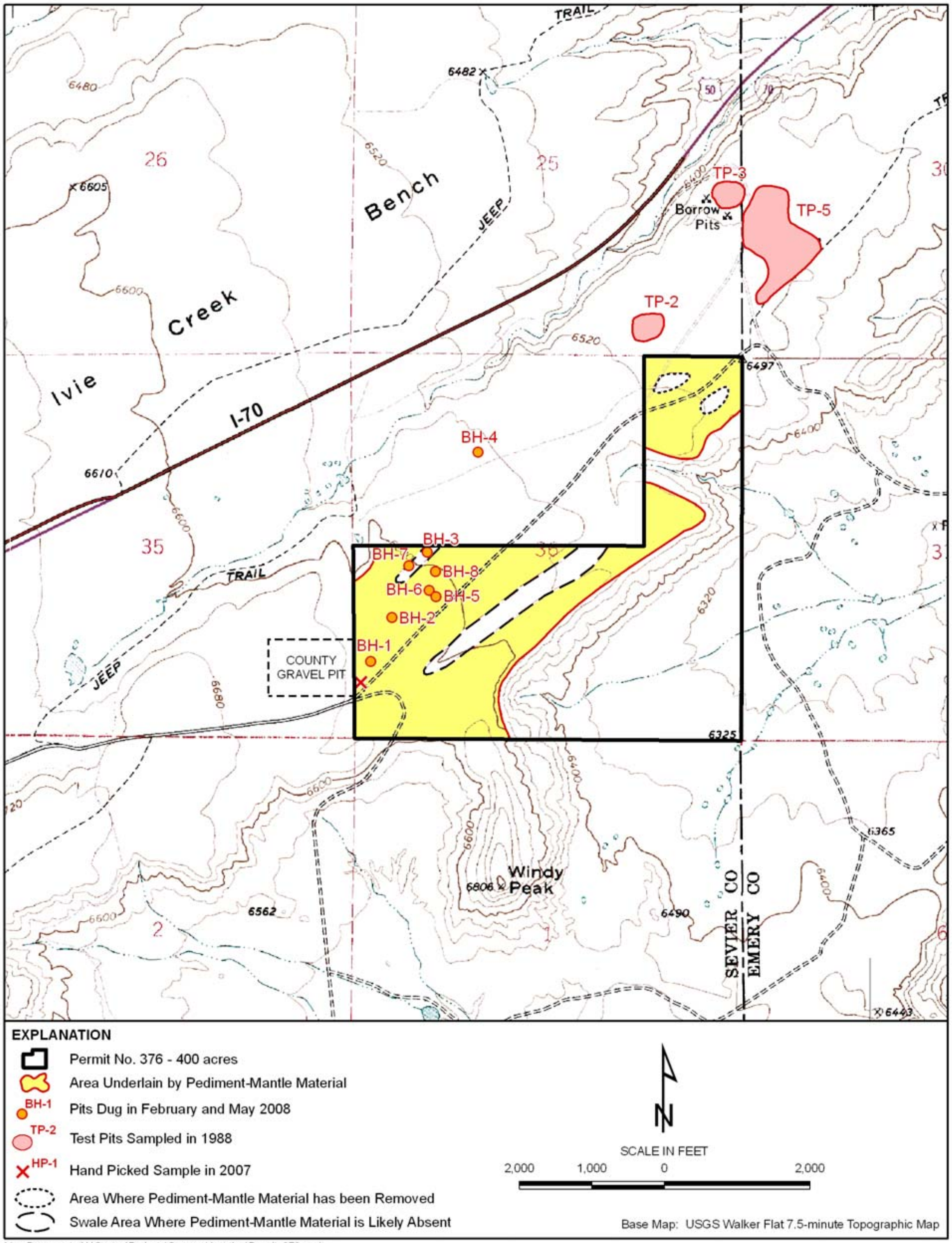

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Figure 6–9. Fremont Junction Site Permit Area, Pediment-Mantle Deposit, and Sampling Locations

In three of the four pits (BH-1, BH-2, and BH-4), the deposit was at least 20 ft thick, which was the maximum depth that could be reached by the backhoe. Only a thin part of the deposit was found in pit BH-3. At this location, weathered Mancos Shale was reached below the pediment-mantle material at a depth estimated at about half of the reach of the backhoe. Overall, the deposit contained a significant volume of rounded to subrounded, vesicular, tholeiitic basalt boulders and cobbles. The lithology of the basalt appeared relatively uniform at all pits, but the volume of basalt cobbles and boulders in the deposit varied significantly in the four pits and was visually estimated to be between 15 and 45 percent. The thickness of the overburden above the basalt deposit was variable, ranging from approximately 1 to 5 ft. A 2- to 4-ft-thick white caliche layer was just below the surface in two pits (BH-2 and BH-4).

Based on visual observations from the backhoe pits and communications with personnel who lease part of the site, the deposit varies laterally in quantity of basalt boulders. Previous quarrying in the site area has followed “channels” where the amount of boulders was greater. This approach will likely be followed when quarrying rock in this permitted area.

The total volume of usable basalt in the deposit is estimated using the following conservative assumptions: (1) only 150 acres in the permit area will be quarried, (2) the thickness is nominally 10 ft, and (3) the deposit contains 20 percent by volume of basalt cobbles and boulders. Using these assumptions, the total volume of usable basalt is estimated at 13,068,000 ft^3 (484,000 yd^3). The total volume of aggregate and riprap required by the design for the final cover is 4,368,343 ft^3 (161,790 yd^3) (Addendum C, Calculation Set E-02-C-105, Rock Cover Plan). Based on UMTRA experience the volume of estimated material that is available should be at least twice the volume required by the design. Therefore, the volume of rock at the Fremont Junction deposit is more than adequate for the Crescent Junction cell requirements.

6.6.3.3 May 2008 Evaluation

DOE conducted an evaluation of the Fremont Junction site in May 2008 to more accurately determine the thickness and lithologic character of the pediment-mantle deposit in the permitted property. Also, during the evaluation, additional samples were collected for durability testing, and evidence for natural analogs was sought to demonstrate the durability and resistance to erosion of the basalt. The Green River UMTRA disposal cell (constructed using Fremont Junction basalt approved by the NRC) was visited to compare the composition of rock in the cell cover to what is present in the Fremont Junction pediment-mantle deposit at the permit site.

Four pits (BH-5 through BH-8) were excavated with a trackhoe in the NW¼ SW¼ of Section 36 northwest of the county road (Figure 6–9). Selection of the sites for these four pits was in a part of the permit area where the pediment-mantle deposit was thought to be thick and readily accessible near the county road.

Only one pit (BH-7) did not penetrate a significant thickness of pediment-mantle material. BH-7 was excavated in a shallow swale and encountered weathered bedrock at a depth of approximately 3 ft. This pit is located along the northeast trend of the same shallow swale where nearby pit BH-3 encountered bedrock at a shallow depth. The shallow depth to bedrock from these two pits along the swale indicates that pediment-mantle material is thin to absent in such places in the permit area. Another northeast-trending swale where pediment-mantle material is thin or absent is southeast of the county road, as shown in Figure 6–9.

The parts of the permit area where a significant thickness (more that 10 ft) of pediment-mantle material is present appears to be along northeast-trending subtle ridges where subrounded gray basalt boulders lie on the surface. Boulders in these low ridges may be as large as 6 ft in diameter. Exclusion of the shallow swales and some areas in the NE¼ NE¼ of Section 36 where the pediment-mantle material has been removed in previous quarrying indicates that 160 to 165 acres of the permit area contains pediment-mantle material.

Pediment-mantle material is covered by a variable thickness of overburden, as seen in three of the pits (BH-5, BH-6, and BH-8). Overburden is as much as 8 ft thick. It contains in places a white calcified horizon that is 1 to 3 ft thick, one or more reddish petrocalcic horizons that are relict soil layers as much as 2 ft thick composed mainly of loess (silt-sized material), and variable amounts of basalt clasts (and other rock types) that have caliche crusts as much as 0.5 inch thick on the underside of the clast. Overburden contains finer-grained material than the underlying pediment-mantle deposit and has a smaller percentage of basalt clasts. No weathering rinds or other significant effects of erosion were seen on the basalt clasts. Possibly, the descriptions of weathering rinds made during the 1988 investigation were interpreted to be the thick caliche crusts on some basalt clasts.

Beneath the overburden, the fluvially deposited pediment-mantle material consists of a variable amount (15 to 45 percent by volume) of subrounded cobbles and boulders of dense to vesicular basalt and other rock types such as sandstone, limestone, chert, and quartzite. The matrix (material smaller than 3 inches in diameter) of this matrix-supported deposit consists of gravel, sand, and silt. Fluvial conditions during deposition of the material control the variable amounts of finer grained material (especially sand) seen in the deposit. The pediment-mantle material in places appears to have been deposited in one or more cycles expressed by graded beds - coarse basalt cobbles and boulders at the base grading upward to finer sand and gravel. The cycles expressed by graded deposition may be separated by a thin reddish layer composed of silt indicating soil formation between periods of fluvial deposition.

Approximately 95 percent of the cobbles and boulders in the pediment-mantle material consist of gray basalt. Cobbles 1 to 2 ft in diameter are common, but boulders may be as large as 6 ft in diameter. For the remaining 5 percent of cobbles and boulders, approximately half (or 2 to 3 percent) consist of red basalt, which can be dense or vesicular and as large as 4 ft in diameter. No weathering rinds were seen on either the gray or red basalt. Effects of weathering are minor and were seen in only a few basalt boulders, which had spalls resulting from exfoliation and cracks resulting from frost wedging. The gray basalt and red basalt are from different sources; gray basalt is derived mainly from sources about 10 miles to the southwest and the red basalt is derived from a source seen around Hogan Pass about 12 miles south-southwest of the site.

The other half of the remaining 5 percent (2 to 3 percent) of cobbles and boulders consists of soft, friable sandstone that can be tan, light gray, or red and as much as 3 ft in diameter. Also represented is light gray to white limestone, gray chert, and white quartzite, all of which are mostly less than 1 ft in diameter. Sandstone and limestone cobbles and boulders are soft and nondurable, whereas the chert and quartzite cobbles appeared to be at least as hard and durable as the basalt.

The amount of cobbles and boulders that are not basalt was found to be about 5 percent in several test pits southeast of the county road that had been excavated and filled in during the past year. This amount is slightly larger than observed at the pits that had been more recently excavated (BH-5, BH-6, and BH-8).

Natural Analogs of Basalt Durability

Natural analogs provide insight from present and past environments for long-term performance of engineered covers for tailings piles. Durability and resistance to erosion of the basalt in the Fremont Junction pediment-mantle deposit are demonstrated by the following natural analogs that are listed and discussed below, each in a separate paragraph.

1) Geomorphology of the pediment-mantle deposit that armors and protects the soft Mancos Shale bedrock from erosion.
2) Development of wind-fluted surfaces on exposed basalt boulders creating ventifacts.
3) Development of rock varnish on exposed basalt boulders.
4) Lichen cover on the surface of many exposed basalt boulders.
5) Development of pedogenic carbonate morphology on the underside of buried basalt boulders and cobbles.
6) Performance of basalt cobbles and boulders from the same pediment-mantle deposit used at the Green River UMTRA disposal cell.

The pediment-mantle mapped by Doelling (2004) that covers the soft Blue Gate Member of the Mancos Shale bedrock forms an armored geomorphic surface at the permit site that is the south part of the Ivie Creek Bench (Figure 6–9). The hard, resistant basalt cobbles and boulders provide the main armoring for this deposit that formed probably by a combination of fluvial and pro-glacial mudflow processes (Smith et al.1997). The age of deposition for the initial flows of material over the Mancos Shale pediment may be estimated by applying an average rate of downcutting for this part of the Colorado Plateau of 0.6 ft per thousand years (Willis 1992). The present course of Ivie Creek just north of Ivie Creek Bench is about 300 ft below the surface forming the top of Ivie Creek Bench. This downcutting rate would indicate that the surface of Ivie Creek Bench may be as old as approximately 500,000 years. Therefore, basalt clasts within the pediment-mantle deposit may have been in their present position for as much as 500,000 years. The lack of weathering rinds on the basalt boulders indicates their resistance and durability during their residence time in the pediment-mantle deposit.

Some exposed basalt boulders have surfaces that have been fluted from erosion by wind-driven sand. The dominant lineation of the fluting indicates a wind direction from the west. Fluting by wind erosion takes hundreds to thousands of years to form. Formation of these basalt ventifacts was probably during a warm, dry period known as the Altithermal, which occurred after the last glacial epoch in early Holocene time (possibly 8,000 to 10,000 years ago). This fluting attests to the long-term durability of these basalt boulders.

Rock varnish appears on many of the exposed basalt boulders as a dark coating. The thin varnish coating consists mainly of magnesium, which is available during periods of eolian activity and made immobile by bacterial processes during moist periods. The most recent period of relatively high eolian activity occurred during the dry Altithermal period in the early Holocene. The formation of rock varnish could have occurred later during moist periods in the middle to late

Holocene. The probable age of formation of the rock varnish on the basalt boulders several thousand years ago indicates their long-term resistance to weathering.

Several types of colorful lichen cover mainly the north aspect of many of the exposed basalt boulders. The most common lichen seen is orange, called the flame lichen (Caloplaca ignea). Also seen are the gray-green lichens: sagebrush rim-lichen (Lecanora garovaglii) and varying rim-lichen (Lecanora argopholis). Lichens have slow growth rates, usually less than 1 mm/year, and some live as long as 1,000 years. Growth of lichen on many boulders indicates the boulders have been in place for hundreds of years. In some boulders, lichen growth has covered fluted surfaces created by eolian erosion, indicating the lichens are younger than the eolian activity.

Basalt cobbles and boulders buried at depths of 3 to 6 ft in the overburden material commonly have a white calcium carbonate crust as much as 0.5 inch thick on the undersides of the basalt clasts. This pedogenic calcium carbonate development in the soil profile exhibits characteristics of Stage II carbonate morphology (Birkeland 1999). This stage of development of calcic horizon takes tens of thousands of years to form. Cobbles and boulders with this carbonate crust show no evidence of weathering rinds, indicating that these clasts have been in place for many thousands of years without noticeable weathering effects.

Material from the pediment-mantle deposit just northeast of the Fremont Junction permitted area was used in construction of the Green River UMTRA disposal cell in 1988 and 1989. Cobbles and boulders (as large as 4 ft in diameter) in the drainage channel of the cell have shown no signs of weathering during the past 20 years. The amount of non-basalt cobbles and boulders in the drainage channel is approximately 10 percent, which is significantly more than what was observed from test pits at the source rock site. Although there is a higher percentage of non-basalt material in the drainage channel, this material appeared to be performing well, with few signs of degradation.

6.6.4 Laboratory Test Results

6.6.4.1 Petrography and X-ray Diffraction Analysis

Four samples of basalt collected in 1988 at the Fremont Junction site (Figure 6–9) were submitted for laboratory petrographic thin-section and X-ray diffraction analysis (Addendum F). A qualitative description of the samples from the 1988 petrographic report is summarized in Table 6–14.

Table 6–14. Description of Basalt Samples Obtained From the Fremont Junction Site Area in 1988

Sample ID	Surface Weathering	Toughness
TP-2A	moderate	fairly tough
TP-2B	moderate	fairly tough
TP-5A	moderate	fairly tough
TP-5B	fairly severe	not exceptionally tough

Results of the petrographic thin-section analysis indicate all the basalt samples were mechanically stable (fairly free of structural defects) except for sample TP-5B, which exhibited internal weathering deposits.

The mineralogical composition determined by X-ray diffraction analysis is summarized in Table 6–15. All the samples lacked significant amounts of deleterious minerals (calcite, chlorite, clays, olivine, and feldspathoids); therefore, they should be chemically stable for thousands of years.

Table 6–15. Mineralogical Composition Determined by X-ray Diffraction Analysis of Basalt Samples Obtained From the Fremont Junction Site Area in 1988

Sample ID	Weight Percent by Mineralogy						
	Labradorite	Augite	Quartz	Magnetite	Olivine	Oxyhornblende	Rutile, apatite
TP-2A	76	15		5	1		3
TP-2B	77	12		5		3	3
TP-5A	67	15	9	4		1	4
TP-5B	72	19		6	1		2

6.6.4.2 Durability Testing

Laboratory testing for the NRC quality criteria presented in Section 6.5 (see Table 6–13) was conducted for the basalt samples collected in 1988 (Addendum F). In some cases there was not an adequate amount of material available to perform the entire suite of rock quality tests, but the results are included for completeness. Laboratory testing for the NRC quality criteria was also conducted for the more recent field sampling conducted in 2007 and 2008 (Figure 6–9). Laboratory durability tests that include bulk specific gravity, adsorption, sodium sulfate soundness, LA abrasion loss, and Schmidt rebound (hammer) are summarized in Table 6–16.

Results of the laboratory tests presented in Table 6–16 were used to calculate the NRC rock quality scores using the scoring criteria presented in Section 6.5 (see Table 6–13). The calculated NRC scores are summarized in Table 6–17.

Table 6–16. Laboratory Test Results for Rock Samples Obtained From the Fremont Junction Site Area in 1988, 2007, and 2008

Sample Location	Lab ID	Sample Description	Laboratory Test Result				
			Bulk Specific Gravity (gm/cm^3)	Absorption (%)	Sodium Sulfate Soundness Loss (%)	LA Abrasion Loss, 100 Cycles (%)	Schmidt Hammer
Collected in 1988							
TP-2A	115	Moderately Weathered Basalt Cobbles and Gravels	2.554	1.333	4.5	NA[a]	NA
TP-2B	116	Moderately Weathered Basalt Cobbles	2.52	1.91	1.2	5.5	29
TP-3A	117	Granitic and Basaltic Cobbles	2.607	1.38	3.2	NA	NA
TP-3B	118	Basalt Cobbles	2.587	1.76	1.7	NA	NA
TP-5A	119	Moderately Weathered Basalt Cobbles and Gravels	2.612	1.486	3.9	6.4	NA
TP-5B	120	Fairly Severe Weathered Basalt Cobbles	2.54	1.739	3.7	6.7	30
Collected in 2007							
HP-1	113874 113876 113877	Gray Basalt	2.694	1.4	0	7.6	30
Collected in 2008							
BH-1A	118900 118901 118902	Gray Basalt	2.679	1	0.9	7	30
BH-1B	118897 118898 118899	Red Basalt	2.444	1.5	0.6	8.3	17

[a] Not analyzed

Table 6–17. NRC Scores for Quality of Rock Samples Obtained From the Fremont Junction Site Area

Sample Location	Lab ID	Sample Description	Weighted Test Score					Total score	Max Score	Final Score
			Specific Gravity	Absorption	Sodium Sulfate	LA Abrasion	Schmidt Hammer			
Collected and Tested in 1988										
TP-2A	115	Moderately Weathered Basalt Cobbles and Gravels	54	8.6	91.3	NA[a]	NA	153.9	220	70.0%
TP-2B	116	Moderately Weathered Basalt Cobbles	48.6	6.4	108.9	7.7	10.8	182.4	260	70.2%
TP-3A	117	Granitic and Basaltic Cobbles	63.9	8.4	97.9	NA	NA	170.2	220	77.4%
TP-3B	118	Basalt Cobbles	60.3	7	106.7	NA	NA	174	220	79.1%
TP-5A	119	Moderately Weathered Basalt Cobbles and Gravels	64.8	8	94.6	7.2	NA	174.6	220	79.4%
TP-5B	120	Fairly Severe Weathered Basalt Cobbles	52.2	7	95.7	7	11.4	173.3	260	66.7%
Collected and Tested in 2007										
HP-1	113874 113876 113877	Gray Basalt	80.1	8.4	110	6.5	11.7	216.7	260	83.3%
Collected and Tested in 2008										
BH-1A	118900 118901 118902	Gray Basalt	77.4	10	110	6.8	11.4	215.6	260	82.9%
BH-1B	118897 118898 118899	Red Basalt	35.1	8	110	6	6.3	165.4	260	63.6%
weighting factor			9	2	11	1	3			
max score			90	20	110	10	30			

[a] Not analyzed

All the final NRC scores summarized in Table 6–17 are greater than 65 percent except for the "red" basalt sample collected in 2008. This sample characterized by red color has a relatively lower rock quality score of 63.6 percent. Red basalt represents only a small portion (approximately 2 to 3 percent) of the deposit. In addition, the majority of the red basalt observed in May 2008 at the deposit appears to have the same durability quality as the gray basalt. The highest final rock scores of approximately 83 percent are characterized by gray basalt. Approximately 95 percent of the deposit is estimated to contain gray basalt of high quality.

DOE also performed durability testing for quality control purposes on the Fremont Junction basalt in 1988 and 1989 during production of two different sizes of riprap rock material (Type A and B) used in construction of the cover for the Green River disposal cell (DOE 1991). Four sampling and testing events were conducted. Tests were performed prior to emplacement of rock, after the first-third and second-third quantities were produced, and after completion of rock production activities. Only the average, low, and high final scores were reported; laboratory test results for individual samples are not available. The average final score for the four durability sample results for the Type A riprap was 85; the low score was 78 and the high score was 90. The average final score for the four durability sample results for the Type B riprap was 83; the low score was 80 and the high score was 90. These results are presented in the final completion report prepared for the Green River disposal cell (DOE 1991) and are provided in Addendum F.

6.6.5 Summary of Fremont Junction Rock Source

Based on NRC rock durability criteria noted above, the basalt in the pediment-mantle material at the Fremont Junction permitted area has been selected as the source for the Crescent Junction disposal cell cover composed of aggregate and riprap. Although the distance to the disposal cell (95 miles) from Fremont Junction is significant, a conservative estimate of the volume of the deposit indicates that adequate material of high durability is available to cover critical areas of the disposal cell.

6.7 Rock Selection During Production

Specifications that describe the requirements for sampling, testing, and placing rock riprap and aggregate materials are provided in Addendum B. The means by which the sampling, testing, and placement of the erosion protection rock will be controlled, verified, and documented are identified in the Remedial Action Inspection Plan (Addendum E). Following these procedures and specifications and the rock production procedures presented below will ensure that the rock used is generally homogeneous and absent of characteristics that would adversely affect the durability of the overall cover system.

6.7.1 Summary of Rock Characteristics

Cobbles and boulders of basalt at the Fremont Junction permitted site in Section 36 occur in fluvially-deposited pediment-mantle material of Quaternary age that can be as much as 20 ft thick. Two layers of the material are recognized – a surface layer that can be considered as overburden and the underlying layer of the main pediment-mantle material.

The surface layer is as much as 8 ft thick and consists mainly of fine-grained material (sand and silt) and a small amount of cobbles composed of basalt and other rock types. A white calcified horizon 1 to 4 ft thick is present in places. One or more reddish, calcium carbonate-cemented, relict soil layers as much as 2 ft thick may be present.

The main pediment-mantle layer can be as much as 20 ft thick. This deposit is matrix-supported and contains from 15 to 45 percent by volume of subrounded cobbles and boulders of dense to vesicular basalt and other rock types. The matrix is gravel, sand, and silt. One or more cycles of graded beds may occur in which coarse cobbles and boulders grade upward to finer grained gravel and sand. Approximately 95 percent of the cobbles and boulders (as large as 6 ft in diameter) are gray basalt of high durability quality. For the remainder, about half (2 to 3 percent) are red basalt, about half (2 to 3 percent) are soft, friable, tan to light gray or red sandstone, and a trace amount (< 1 percent) are light gray to white limestone, hard gray chert, and white quartzite. Red basalt, chert, and quartzite appear to be at least as high quality as the gray basalt, but sandstone and limestone clasts are softer and less durable.

6.7.2 General Quarrying Operations

Overburden will be stripped and stockpiled at a designated location that is separate from other sources before the first phase of rock production (Addendum B). The thickness of the overburden is expected to be 3 to 5 ft in most places, but it may be as much as 8 ft. Overburden consists mainly of sand and silt that may be cemented by calcium carbonate in places; cobbles of basalt and other rock types may also occur in small amounts.

Below the overburden is the main pediment-mantle deposit, which consists mainly of subrounded cobbles and boulders of basalt. The cobbles and boulders are in a matrix of gravel, sand, and silt. In places, the bedding of the material is graded – coarse cobbles and boulders grading upward to finer gravel and sand. This main pediment-mantle deposit will be excavated to the top of bedrock, which is the Blue Gate Member of the Mancos Shale. This geologic contact will be identified visually to guide the total depth of the excavation. The contact can be identified by a change in color and bedding structure that are exhibited by the characteristic sedimentary features of the weathered shale. Material from this lower zone will be excavated and loaded directly into the crusher at the quarry. Rock which is retained on the screens in the crusher is then moved by conveyor and placed in primary stockpiles to be further processed and that is separate from other sources. Reject material will be moved by conveyor and stockpiled separate from other sources.

To meet the rock cover requirements of the disposal cell, the material is initially crushed to provide angularity, help remove lower quality material (crushed away), and provide appropriate sizes to meet the gradations specified in Addendum B (see Table 3). Sizes smaller and larger than specified in Addendum B will not be retained. Rock from the first pass through the crusher and screens is segregated and placed in primary stockpiles as described above.

Samples from the primary stockpiles will be collected and tested to determine the rock quality and any oversizing that may be required. Lower quality rock such as sandstone and limestone will be extracted from the stockpile to assure than no more than 10 percent by volume is present in the final product. Sampling and testing will be conducted every 10,000 yards according to the guidance provided in Section 6.5 and specified in Addendum B.

Production of the sizes required to achieve the gradations specified in Addendum B (see Table 3) will be performed by further processing of the primary stockpiles that have been tested and the results have demonstrated that the material meets the NRC rock quality criteria. The crusher plant uses a gyratory cone hammer that is sized to meet the gradation requirements for each size specification. Rock from the secondary crushing operation is screened, sorted, and stockpiled by conveyor at a designated location that is separate from other sources. Only one or two gradation sizes will be produced at a time depending on the construction need. Reject material from the secondary crushing operation will be moved by conveyor and stockpiled separate from other sources.

6.7.3 Personnel Involved in Rock Selection and Testing

A qualified geologist that is knowledgeable of the rock being produced, its durability requirements, and potential heterogeneities in the rock that would lead to production of unsuitable erosion protection rock will be primarily responsible for rock selection and NRC rock durability testing. The geologist will visual examine the rock for potential heterogeneities that could lead to production of unsuitable material, including the presence of weathering rinds, significant chemical alteration of phenocrysts, and an increase in deleterious mineralogy (i.e. olivine). The geologist will periodically inspect and observe the quarry operation, provide training of the quarry operator in the visual selection of the basalt, visually inspect the segregated primary stockpiles at the crusher plant, and select representative rock samples to be submitted to a commercial geotechnical laboratory for NRC durability testing.

The geologist will work closely with the construction manager/project engineer to communicate how the rock samples were selected for testing, to interpret NRC durability tests results, and to identify any unacceptable rock that should be excluded. Lower quality material (e.g. sandstone and limestone) will be extracted to assure that no more than 10 percent by volume is present in the final product. The construction manager/project engineer has overall responsibility for performing gradation tests on the processed stockpiles for the aggregate and for each type of riprap in accordance with the specifications in Addendum B and for identifying the stockpiled processed rock for loading and transport for placement operations.

6.7.4 Rock Selection Procedure

The overall goal of the rock selection procedure is to minimize the potential that rock unsuitable for use in long-term erosion protection is used on the cover for the Crescent Junction cell. Five gradations of material ranging in size from D_{50} of 12-inch, 8-inch, 6-inch, 4-inch, and 2-inch will be produced at the Fremont Junction site for the rock cover. The rock selection procedures incorporate the construction details for placement of the rock described in Section 7.0 and the procedures and construction specifications, including gradations, specified in Addendum B. The locations and sizes where the material is placed on the cell cover are provided in Addendum C (Calculation Set E-02-C-105, Rock Cover Plan). Quality Control personnel will visually inspect the rock production operations and verify that the rock is produced and placed in accordance with the plans and specifications as described in the Remedial Action Inspection Plan (Addendum E).

The primary and secondary crushing, screening, and sorting processes used ensures that the rock produced will be fairly homogeneous and the visual portions will be representative of the entire stockpile. The processing (vibratory and dynamic hammers) and screening also ensures the lower quality material (e.g. sandstone and limestone) will be removed (crushed away) so that the potential is low for significant unacceptable rock to be present in the produced aggregate or riprap material. In the unlikely event that a stockpile contains significant unacceptable rock, the lower quality material will be extracted to assure that no more than 10 percent by volume is present in the final product.

The quarry operational protocols have been prepared so that the basalt rock is segregated, stockpiled, and tested prior to production of the final product. Only primary stockpiles that meet the NRC rock quality criteria will be used to produce the final product. These procedures will help ensure that only basalt that meets the NRC rock quality criteria will be used on the Crescent Junction cell.

Based on the rock quality testing summarized in Section 6.6, it appears that most of the basalt rock will score 80 percent or higher; therefore, oversizing will not be required for use in either critical or non-critical areas of the cell. Rock that scores between 65 and 80 percent will be oversized according to the guidance provided in Section 6.5 (Rock Durability) and as specified in Addendum B for use in critical areas of the cell, such as areas that are frequently saturated including all channels, poorly-drained and buried toes and aprons, control structures, and energy dissipation areas. Rock that scores less than 65 percent will be rejected for use in critical areas of the cell.

Rock that scores between 50 and 80 percent will be oversized according to the guidance in Section 6.5 (Rock Durability) and as specified in Addendum B for use in non-critical areas of the cell, such as occasionally-saturated areas including top slopes, side slopes, and well-drained toes and aprons. Rock that scores less than 50 percent will be rejected for use in non-critical areas of the cell.

End of current text

7.0 Disposal Cell Design and Construction Details

This section summarizes the disposal cell design, based on information presented in Sections 4.0, 5.0, and 6.0 of the RAS. Design features and considerations relevant to compliance with EPA regulations include the following:

- Geotechnical stability – consideration of factors including site stratigraphy, and evaluation of performance for slope stability, settlement, and liquefaction (Section 4).
- Radon attenuation – evaluation of the disposal cell cover for acceptable radon emanation under long-term conditions. The typical UMTRA Project cover design will be used (Section 5).
- Surface water hydrology and erosion protection – acceptable performance was evaluated under long-term conditions (represented by using the PMP) (Section 6).

Section 8.0 discusses the relevant cell design criteria with respect to ground water protection.

7.1 Disposal Cell Design

Figure 7–1 shows the disposal cell footprint and existing and proposed site features. Typical cross sections through the disposal cell are shown in Figures 7–2 and 7–3. The disposal cell will cover approximately 230 acres, and will be constructed partially below grade. The anticipated depth of excavation will vary between 10 to 20 feet below grade. The northern edge of the disposal cell will be excavated into the existing grade with only the cover being above existing grade. The southern edge of the disposal cell will be excavated to approximately 20 feet below the existing grade. There will be a berm of approximately 20 feet in height along the southern edge of the disposal cell. A berm along the east and west edges of the disposal cell will vary in height from 0 to 25 feet above the existing grade. The top surface of the disposal cell will slope upward from the north to approximately the quarter point and then slope down to the southern edge. The side slopes of the disposal cell are designed with maximum slopes of 5:1 (20 percent).

The current design volume of the cell is estimated at 11.15 million yd^3. This accounts for RRM from the tailings pile, subpile, and contaminated soils on the processing site and vicinity properties, which primarily surround the processing site.

The area of the cell and depth of excavation have been calculated to accommodate the RRM volume, such that sufficient materials generated from cell excavation are used for embankment and cover material. The volume of material to be excavated within the footprint of the cell is 3.19 million yd^3 of colluvial material and 3.46 million yd^3 of weathered Mancos Shale. The embankments require 0.46 million yd^3 of fill while the cover design requires 2.26 million yd^3 of fill. There will be 3.93 million yd^3 of excess material. Most of the excess material will be placed to the north between the cell and the Book Cliffs to divert surface water away from the cell. A ditch will be constructed between the excess material placement and the cell to divert surface water that accumulates there to the east and west away from the cell. The remainder of the excess material will be used for final site grading.

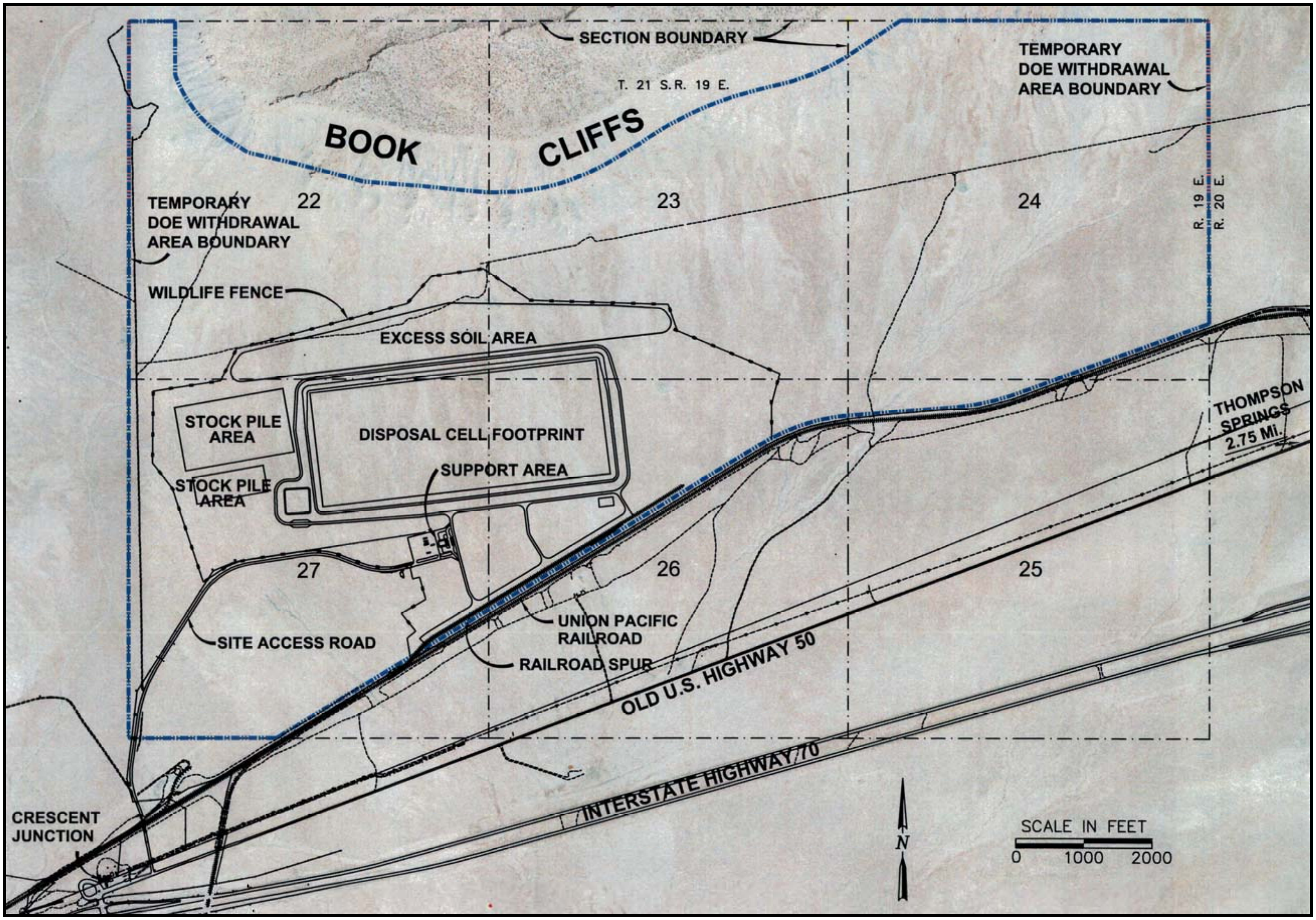

Figure 7-1. Crescent Junction Disposal Cell Footprint and Existing and Proposed Site Features

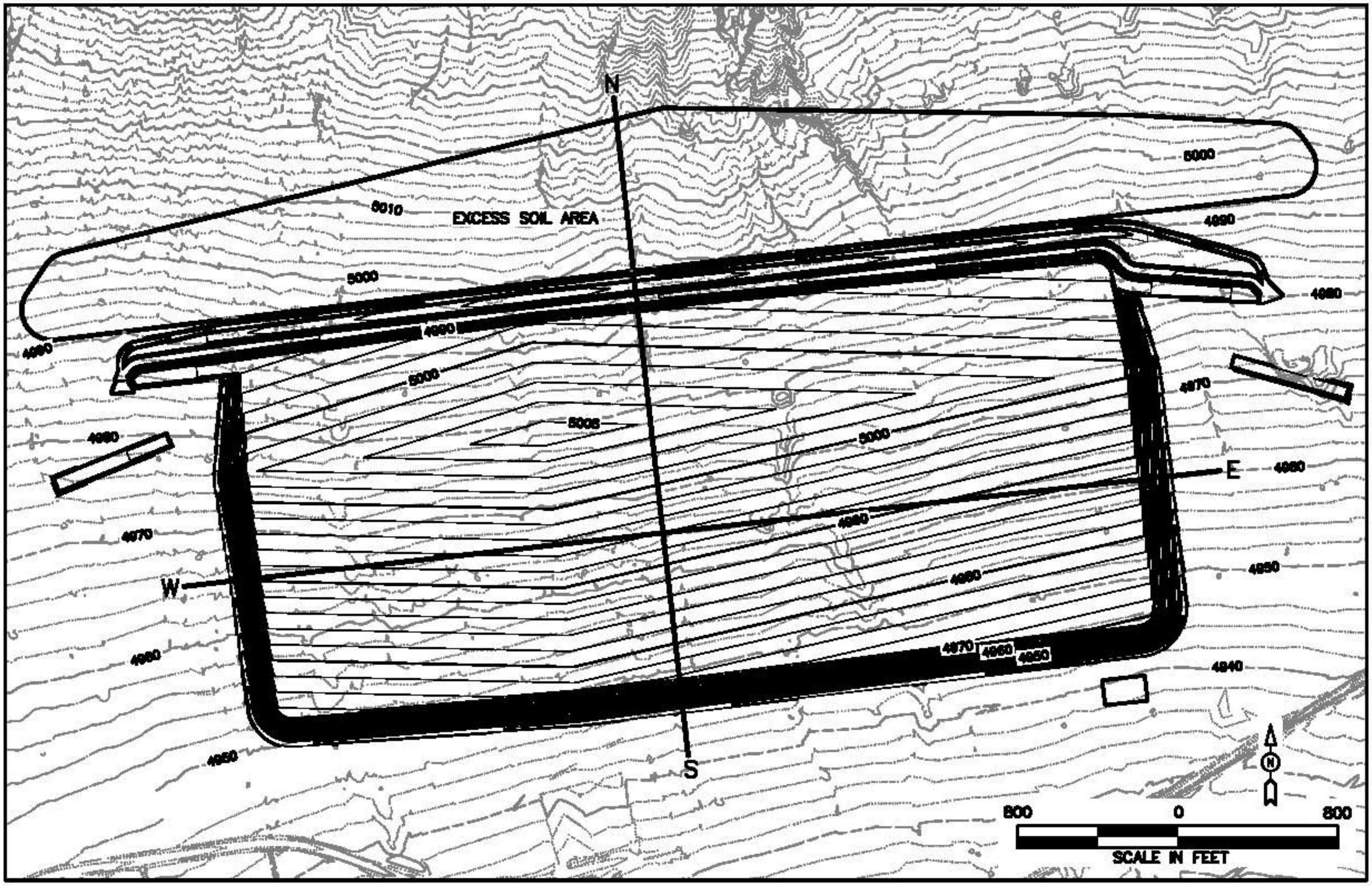

Figure 7-2. Disposal Cell Layout with Typical Cross-Section Locations

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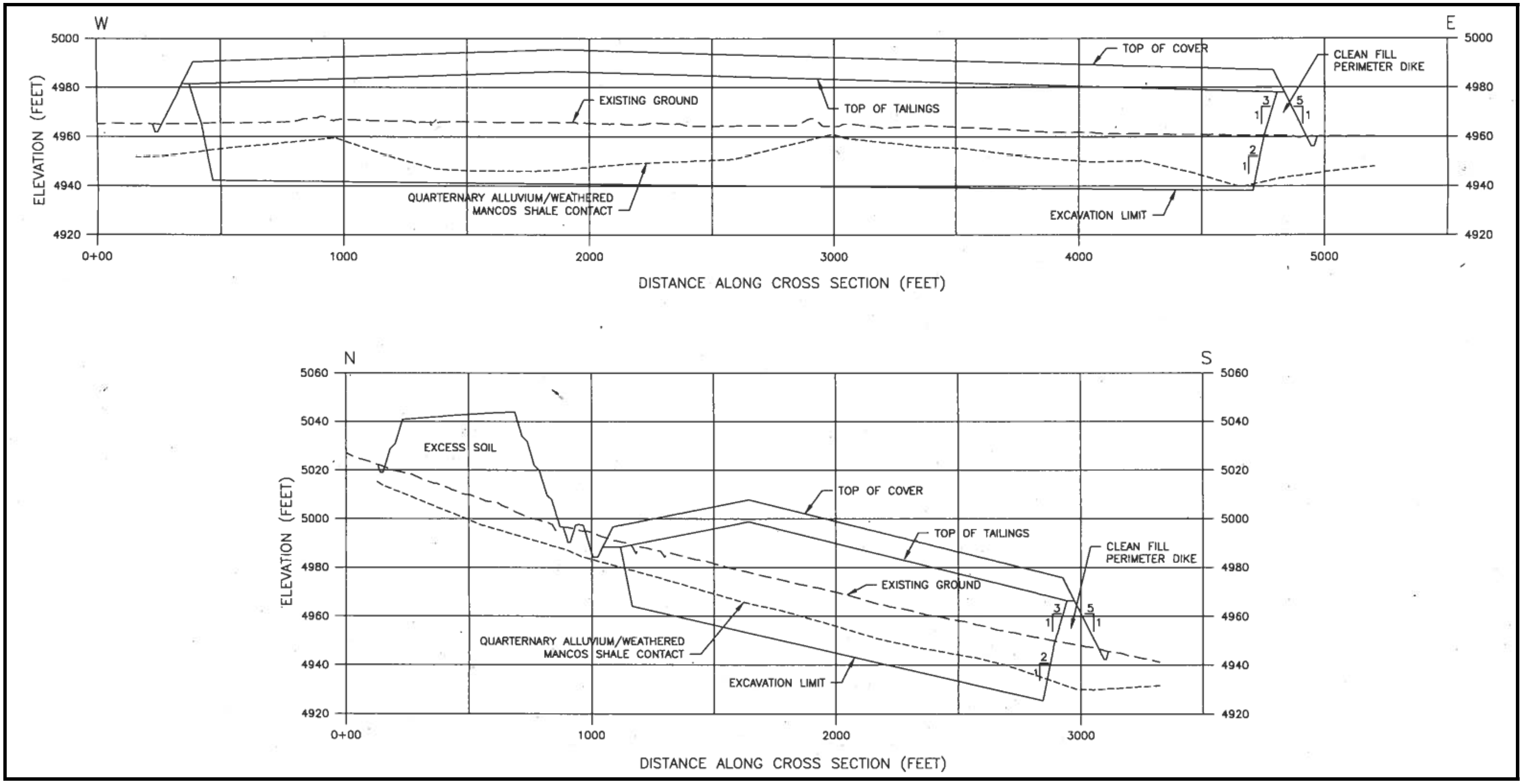

Figure 7-3. Typical Cross Sections for Crescent Junction Disposal Cell

The UMTRA cover design (Figure 5-1) will be used. This multi-component cover system serves to resist erosion, promote runoff, limit infiltration into the RRM, minimize radon emissions, reduce long-term maintenance, minimize animal and human intrusions and reduce the risk to human health and the environment.

The first layer to be placed onto the RRM is the interim cover. The layer is uncontaminated native alluvial material. It acts as a protective cover during the time frame from when the tailings have been placed to the desired design elevation, to the time when the radon barrier and subsequent layers are placed. The interim cover acts as a temporary blanket that helps protect the tailings from wind and precipitation erosion during this interim time period as well as reducing the radon emanation from the RRM for worker and environmental protection. It provides a clean buffer on which equipment and personnel can traverse and on which the radon barrier can be placed and compacted.

The second layer, the radon barrier, reduces the flux of radon from the RRM to less than 20pCi/m2/s as required by EPA The compacted layer is constructed from conditioned on-site weathered Mancos Shale excavated from the disposal cell footprint. The radon barrier also reduces or eliminates the infiltration of moisture from precipitation events. The radon barrier is discussed in detail in Section 5.0.

The third layer to be placed is the infiltration and biointrusion barrier. This layer of coarse-grained (gravelly-stone) material goes on top of the radon barrier and has three basic functions. It provides positive drainage of any surface water that seeps through the upper layers of the cover and transmits the infiltration to the side slopes of the cover. The conductivity of the infiltration layer will be several orders of magnitude higher than the underlying compact clay layer beneath it, and the preferential flow path for water will be through the infiltration layer to the cover perimeter. The infiltration layer serves as a capillary break, limiting the head pressure that can occur above the radon barrier layer. A second function of this layer is that it provides a gravel/stone barrier against burrowing animals. Finally, the layer provides a break in the soil regime to discourage root growth into the radon barrier. In the event that deep-rooted plants, such as greasewood, occupy the site, increased maintenance may be required to remove these deep-rooted plants so they do not root into the radon barrier and provide pathways for water to infiltrate into the RRM.

The frost protection layer is the next layer to be placed and ensures that the freeze/thaw cycle of the site will not adversely affect the radon barrier. The material is common fill which will be generated during the excavation for the disposal cell. Addendum D, Calculation C-13 is a check of the freeze/thaw depth. This calculation estimates the freeze/thaw depth to be 45 inches for a recurrence interval of 200 years. Therefore, the thickness of this layer along with the rock armoring and infiltration/ biointrusion layers provide four feet of protection.

The outer-most layer of the cover is the rock armoring. This layer is the first line of defense against erosion and protects all of the underlying cover layers and RRM. Rock used for erosion protection on the disposal cell must meet NRC durability requirements. The rock armoring is discussed in detail in Section 6.

A schematic depiction of the disposal cell in relationship to surrounding geologic and hydrogeologic features is shown in Figure 7 – 4. As discussed in Section 8.0, the cell construction and site hydrogeology is anticipated to effectively isolate the RRM from the uppermost Dakota aquifer. The stable geologic, seismic, and geomorphic setting of the site will ensure adequate control of the RRM for the design life of the cell. Details of the disposal cell design can be found in Addendums B and C. Calculations for the disposal cell are in Addendum D.

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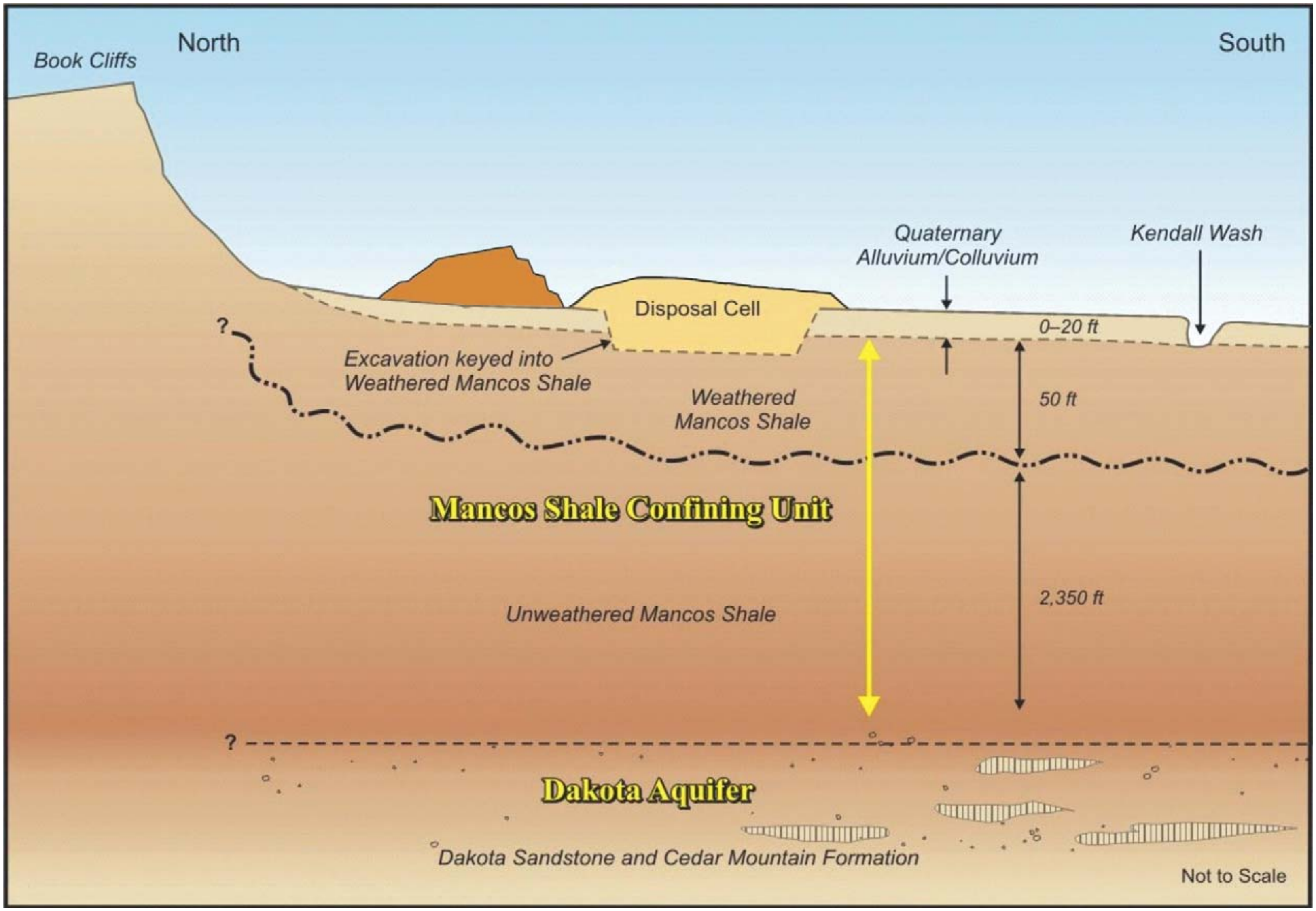

Figure 7-4. Schematic Diagram of Crescent Junction Disposal Cell and Surrounding Geologic and Hydrogeologic Features

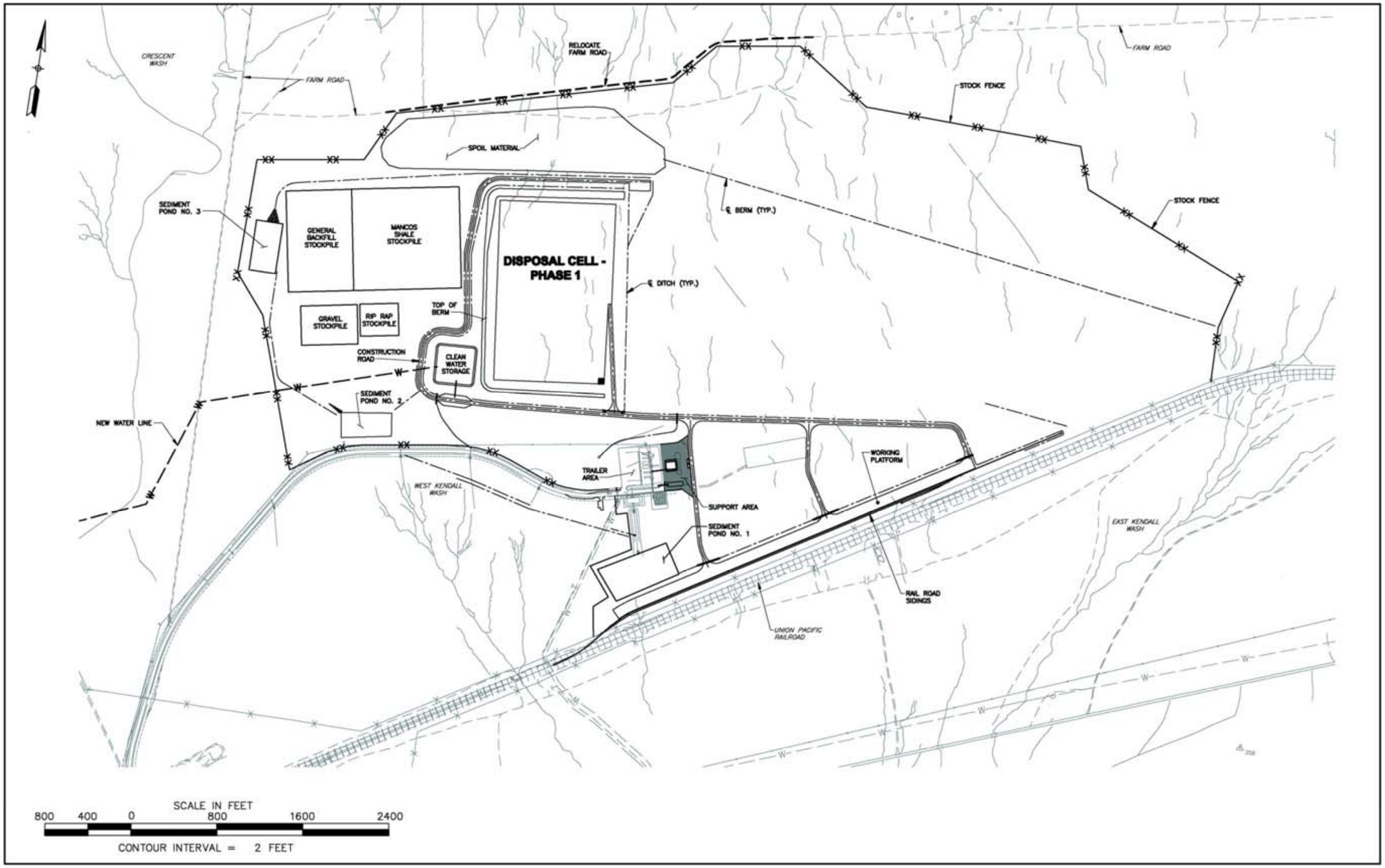

Figure 7-5. Phase 1 of Disposal Cell Construction

7.2 Construction Details

7.2.1 Phased Construction

The construction of the disposal cell will be performed in stages in order to prevent excessive areas of the cell from remaining open while waiting arrival of the shipments of contaminated material. The phased construction also minimizes the amount of contaminated material exposed in the disposal cell. Construction of the first phase of the disposal cell will commence on the western side of the cell and progress towards the east. The area of this first construction effort is roughly 2,000 feet in the north-south direction and 1,000 feet in the west-east direction (Figure 7–5). Subsequent phases of cell construction will continue eastward.

The eastern edge of the disposal cell is considered to be the flexible component of the cell. In the event that a larger volume of contaminated material is encountered at or under the Moab tailings pile, the eastern edge of the cell could move further to the east to accommodate the extra material. Current projections include two feet of contaminated soil below the tailings pile. If the volume is less than estimated, the cell's eastern wall could be moved to the west and thus shorten the length of the cell. Determination of this volume will be better estimated as the excavation at the tailings pile extends into the contaminated sub-pile material.

Initial Crescent Junction Site construction will include installation of infrastructure components such as the construction of a water pipeline from the Green River, 21 miles to the west of the site, a storage pond for construction water roads; and support facilities, including the transfer yard where containers from the Moab Site will be off-loaded onto trucks.

7.2.2 Cell Excavation

Excavation of the disposal cell will include segregation of the various materials encountered into stockpiles for future cover component use as well as for general backfill and construction of the protective wedge on the north side of the cell. The surface layer will be removed and placed into a 'topsoil' stockpile for future restoration purposes. The alluvial soils and Mancos Shale will be used in the construction of the outer berms of the cell. The weathered Mancos Shale also will be conditioned and stockpiled for future use as the radon barrier layer of the cell cover. Excess soil will be used to construct the protective wedge to the north of the cell.

The excavation will extend to a maximum depth of roughly 20 feet in the footprint of the cell. Per the design drawings in Addendum C, the excavation will be a minimum of two feet into the Mancos Shale. If there are small areas or pockets where Mancos Shale is not encountered at the specified drawing location for the cell bottom, the non-Mancos material will be undercut two feet and replaced with Mancos Shale. Equipment used for the excavation will include scrapers, excavators, and dozers for ripping the Mancos where needed.

Along the eastern edge of Phase 1, an interim berm will be installed to separate the RRM placement area from the adjacent uncontaminated zone.

For the first phase, once the excavation is complete and the western portion of the cell berm is built, the construction will transition to placement of the RRM from the Moab Site. If warranted by schedule needs, excavation and placement sequencing can be performed concurrently as long as measures are taken to prevent cross-contamination. Interim berms may be placed to segregate the "clean" construction from the RRM placement activities.

7.2.3 Placement of Contaminated Materials in Disposal Cell

RRM to be placed in the disposal cell include mill tailings, interim cover soils, starter embankment soils, contaminated subsoils beneath the tailings, vicinity property materials, and mill debris. All of these materials are from the Moab uranium mill.

The primary RRM materials are the mill tailings generated from operation of the Moab Processing Site. The tailings were generated as a residue from milling operations for recovery of uranium. The tailings (sand to silt-sized materials) were discharged as slurry into an impoundment constructed and operated adjacent to the Moab mill. The impoundment was operated as a side-hill structure, with an earthen starter embankment constructed on the downhill side. Tailings were contained within the impoundment by a perimeter embankment constructed with tailings, and raised in stages in an upstream manner (Vick 1990). The tailings slurry was discharged along the perimeter embankment by spigotting, resulting in the coarse fraction of tailings (tailings sands) settling out along the perimeter, and the fine fraction of tailings (tailings slimes) settling out in the interior of the impoundment. The tailings have been classified for characterization and excavation as tailings sands (primarily sand-sized material), tailings slimes (primarily silt-sized material), and transitional material (a mixture of silts and sands). The shear strength and handling properties of the tailings vary with material type, from the sands (with a water content by dry weight of approximately 10 percent) to the slimes (with a water content by dry weight of over 100 percent).

The remaining materials to be placed in the disposal cell consist of soils and debris. The soils are primarily alluvial materials (sand to boulder-sized material) that were used for starter embankment material and interim cover material, and comprise the subsoils beneath the impoundment. Debris that was buried in the impoundment includes (1) structural debris, tanks, pressure vessels, and other material from demolition of the Moab mill; (2) pipe and supporting trestle material from operation of the tailings impoundment; and (3) wick drain material from recent tailings dewatering operations.

All of the contaminated material will be excavated at the Moab Site, placed in tight containers, and transported from Moab to Crescent Junction. The containers will be off-loaded onto six-wheel drive, off-road articulated trucks. A ramp into the cell will allow for the loaded containers to be end dumped over jersey barriers into the southwestern portion of the cell footprint. Dozers, using the global positioning system (GPS) integrated computer aided earthmoving system (CAES) system, will then spread and compact the material in lifts per the specifications. Quality control personnel will perform required testing and verification per the Remedial Action Inspection Plan (RAIP) in Addendum E. Placement will commence in the southwest corner of the cell floor, proceed north, and then work east.

The objective of material placement in the disposal cell will be to minimize subsequent settlement by compacting compressible materials and filling void spaces within and around incompressible materials (e.g., debris). The first phase of the excavation and placement of contaminated materials will have minimal debris. Specifications for RRM placement are found in Addendum B, 31-00-20 R1.

7.2.4 Transient Drainage

Although the tailings will be dried to near–optimum moisture conditions prior to being placed in the disposal cell, the average moisture content of the tailings will probably be biased on the wet side of optimum, leaving enough residual moisture to drain from the RRM under the influence of gravity. Furthermore, post-construction consolidation of the RRM will release water as the consolidation proceeds. These two components of released water constitute what is called transient drainage.

As the first phase of construction and placement proceeds, any released water will be collected in a sump at the southeastern most region of the bottom of the cell. The accumulated water can be used for dust control for the construction activities. Subsequent phases of excavation, construction, and RRM placement will likewise capture water in sumps. As a segment of the cell is completed, transient drainage will be monitored with the installation of standpipes tapping into sumps positioned at four locations at the interior south edge of the disposal cell (Figures 7–6 and 7–7).

In the event that transient drainage accumulates in a sump and reaches some action level, DOE will pump the fluid out through a standpipe. After the disposal cell is constructed, and no further water accumulates in the sump, DOE will remove the standpipes or abandon them in place.

In addition to monitoring the standpipes, DOE will monitor for the presence of ground water and tailings fluids at well locations 0202, 0203, 0205, and 0210 (Reference Figure1, page 4, Attachment 3, Ground Water Hydrology). These wells were drilled in 2006 to depth of approximately 300 feet as part of the Crescent Junction site characterization. Prior to placing RRM in the disposal cell, DOE will recomplete the four wells mentioned above as monitoring locations. These wells will be used as indicators of tailings cell performance and to determine if cell leakage is occurring and if so, to determine if leakage is occurring as predicted in the RAP model. The recompleted wells will be screened through the weathered Mancos shale and slightly into the weathered Mancos Shale.

For the first three years following the start of RRM placement DOE will monitor annually for the presence of water in Wells 0202, 0203, 205 and 0210. If water is detected additional chemical analysis will be done on tailings fluid indicator constituents (i.e. uranium, ammonia). After three years following the start of RRM placement DOE will monitor for the presence of water every third year. Chemical analysis will be performed if water is detected.

7.2.5 Placement of Cover

An interim cover will be placed on the tailings where the full height of placement has been achieved. This cover material will be placed with a dozer equipped with the CAES system and a smooth drum vibratory roller. The equipment will push the interim cover material ahead of it so that the equipment will not become contaminated nor cross-contaminate the interim cover layer. This interim layer is uncontaminated and acts as a sacrificial layer that protects the RRM from erosion by wind and water and also minimizes worker exposure.

The subsequent layers of the cover will be placed on top of the interim cover. The radon barrier will be installed in lifts per the specifications listed in Addendum B. Equipment that will be used for this layer includes scrapers, compactors, and dozers equipped with GPS. After a segment of the radon barrier is approved, the next layer will be installed. The gravel for the infiltration and biointrusion barrier layer will be installed using belly dump trucks and dozers. The frost protection, and rock, or riprap final cover layers will all be placed similarly with belly dump trucks and dozers.

The transition from the top cover to the side slopes is shown in Figure 7-8. Precipitation off of the top slope will flow through a 10 foot “apron” of larger rock onto the side slope erosion protection riprap. The underlying layers of the cell cover will interconnect to the side slopes at the same angle as the top slope, which will drain precipitation that infiltrates into the cover layers. The infiltration layer is much more permeable and will serve to drain excess infiltrated water to the outside slope.

7.3 Testing and Inspection

The RAIP provides details of the methods, procedures, and frequencies by which construction materials and activities are to be tested and inspected to verify compliance with the design specifications. Quality assurance requirements will be in accordance with the RAIP, the Quality Assurance Program Plan, and the approved design specification requirements. Addendum E contains the RAIP.

Placement of the RRM into the disposal cell will be monitored using a system called CAES. The system can be used to monitor compaction efforts and slope placement in real time using a differential GPS-based method that provides continuous logs of disposal operations. CAES has been successfully used in various waste disposal sites such as the Clive Facility in Utah, since June 2005; the White Street Landfill, in Greensboro, North Carolina, for the past 2 years; the Sprint Landfill, Fort Bend County, Texas, since January 2004; and the Olinda-Alpha Landfill, County of Orange, California.

7.4 Construction Sequence

Excavation of the first phase of the Crescent Junction disposal cell and construction of the clean-fill berms will begin in 2008. RRM placement is currently scheduled to commence in the spring of 2009 and an interim cover will be placed on RRM sections as they reach final designed height. Final cap placement on Phase 1 is currently scheduled for 2012. Subsequent construction phases have been forecast to culminate in cell completion in early FY 2026. Funding and other government rulings may impact the overall schedule.

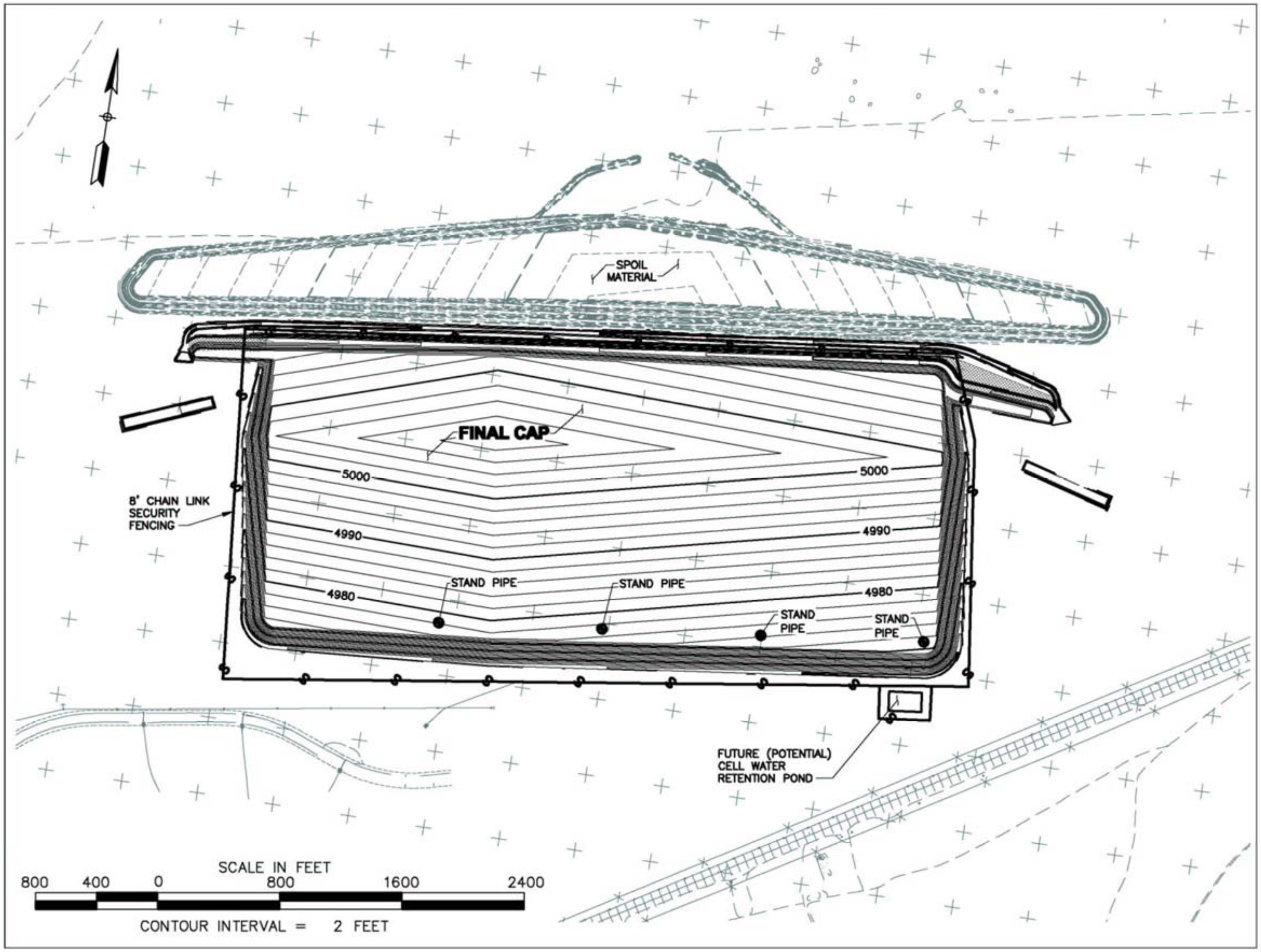

Figure 7-6. Location of Stand Pipes in the Disposal Cell

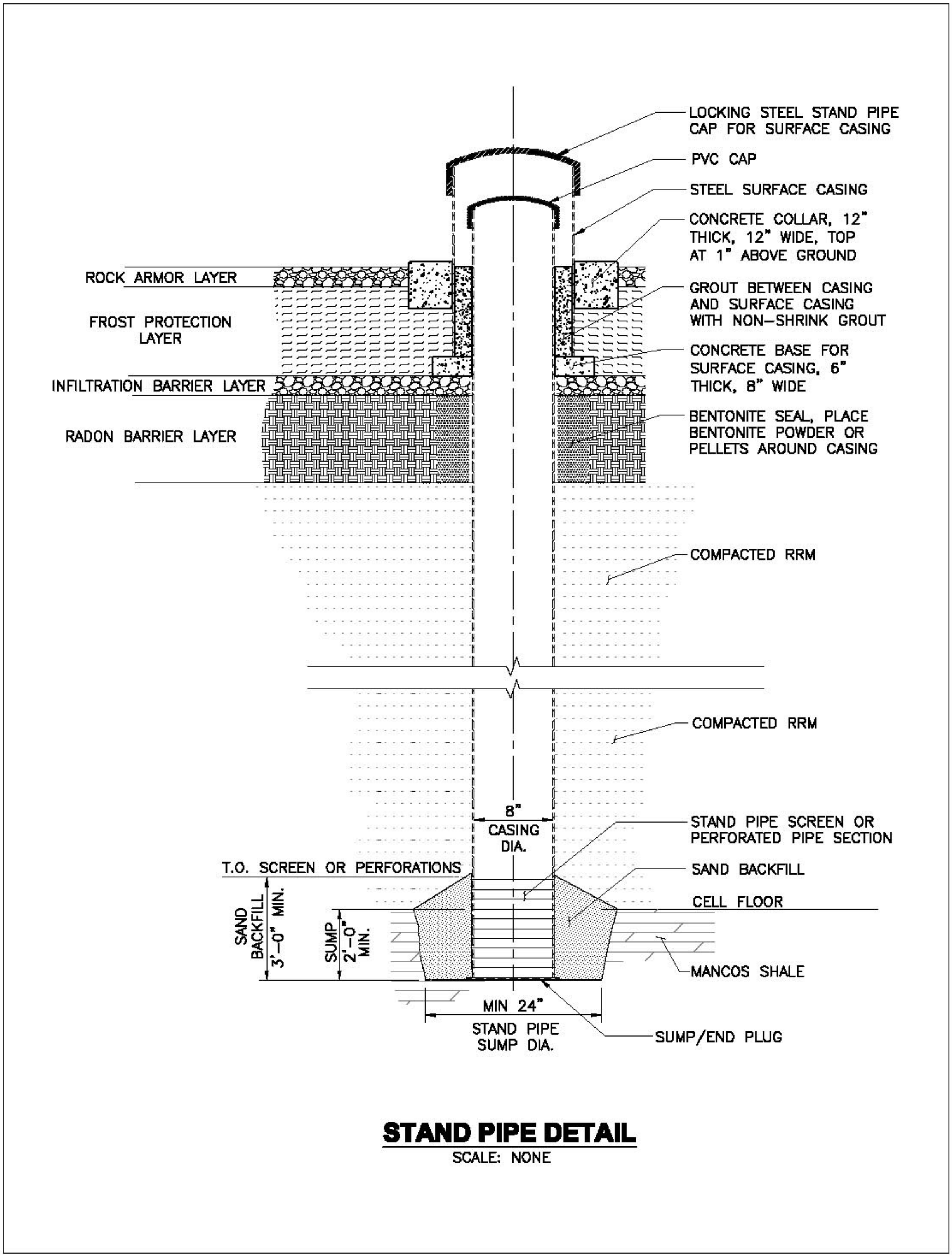

Figure 7-7. Details of Stand Pipe

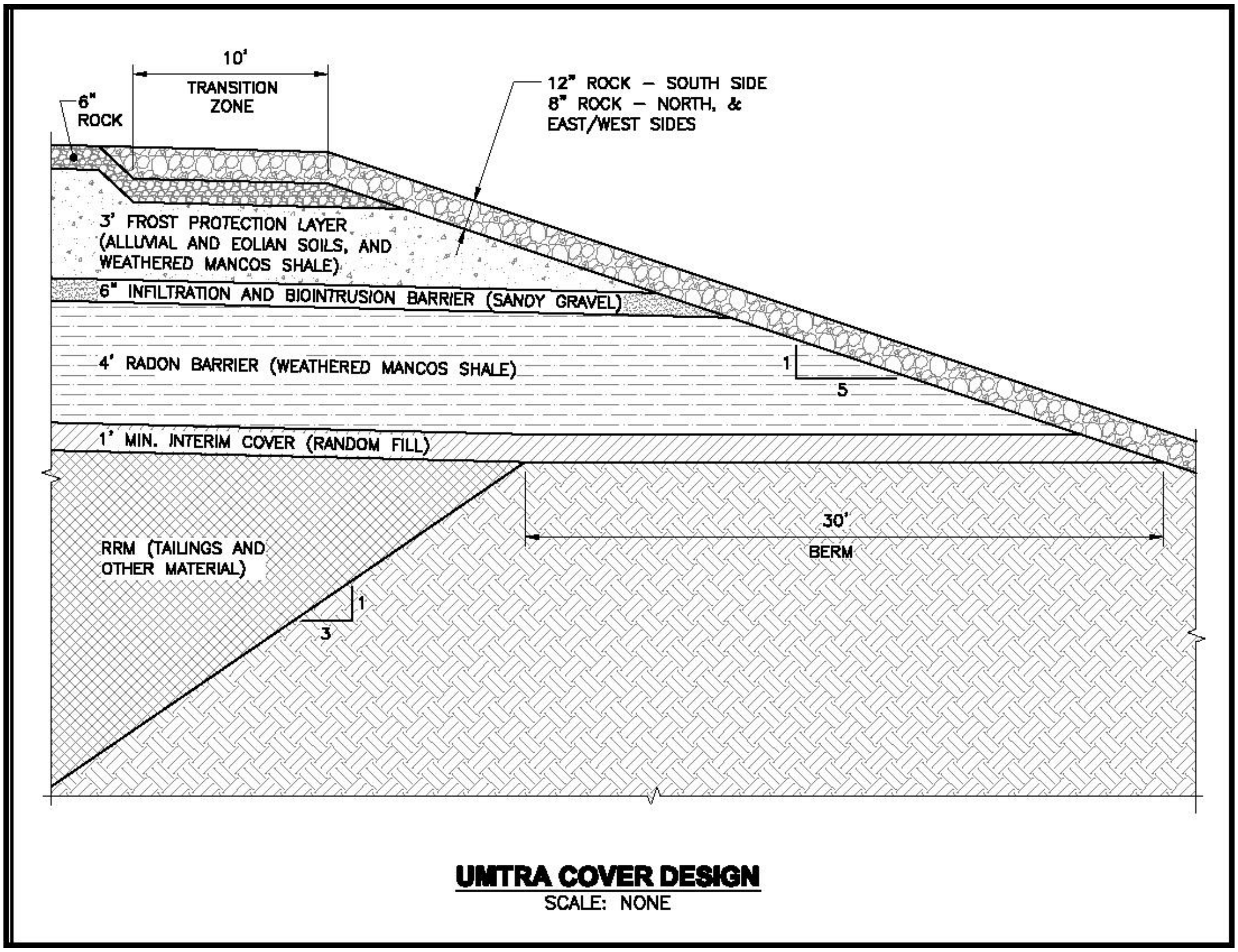

Figure 7-8. Transition of Disposal Cell Top Cover to Side Slopes

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8.0 Water Resources Protection

This section presents the water resources protection strategy for the Crescent Junction disposal cell. Many key features and characteristics presented and described previously in this document have led to the selection of hydrogeologic isolation as the appropriate means of ensuring protection of ground water beneath the disposal cell. The effectiveness of hydrogeologic isolation precludes the need for a ground water monitoring and corrective action program for the site and ensures that ground water in the uppermost aquifer will remain isolated from any cell-derived water during the design life of the disposal cell.

DOE has characterized the hydrogeologic units, hydraulic and transport properties, geochemical conditions, and water use at the Crescent Junction Disposal Site. Major points are summarized below. Details of hydrogeologic characterization are provided in Section 3.0 of this document and Attachment 3 of the RAP. Additional information supporting the water resources protection strategy is provided in Attachment 4 of the RAP.

8.1 Summary of Key Hydrogeologic Site Features

The Crescent Junction Disposal Site is located in an area with a very arid desert climate. The site receives an average of 9.1 inches of annual precipitation; pan evaporation rates are 60 inches per year. Precipitation events tend to be brief and intense, followed by rapid evaporation. Test pits excavated during field investigations at the site showed no visible evidence of saturation.

The bedrock beneath the disposal site is Mancos Shale, which is an important regional confining unit composed primarily of mudstones having a very low hydraulic conductivity. Highly to slightly weathered Mancos Shale in a layer 20 feet to 100 feet thick overlies the much thicker unweathered Mancos Shale. About 2,400 feet of confining Mancos Shale separates the uppermost Dakota aquifer from the ground surface.

Vertical travel times for ground water to migrate from the surface to the uppermost aquifer have been estimated at 3,330 to 33,300 years, far exceeding the 1,000-year maximum design life for the disposal cell. In addition, modeling of geochemical processes that are likely to occur as ground water moves through the subsurface indicates that attenuation of ammonia, and to a lesser degree uranium, would probably lengthen the break-through times for these constituents.

There are no known ground water discharge points within one mile to two miles of the site. Some local water users obtain water from springs located seven mile upgradient of Thompson Springs, Utah; the source of these springs is in the Mesaverde Group, which is stratigraphically above the bedrock units at the disposal site. There is no use of the limited water occurring in the Mancos Shale in the vicinity of the disposal site. Ground water is pumped from wells ranging from 800 feet to 1,200 feet deep near Canyonlands Field (Grand County Airport), which is 15 miles south of the disposal site. The nearest major source of surface water is the Green River, 20 miles west of the disposal site. Geologic and hydrologic features of the disposal site are discussed in greater detail in Attachments 2, 3, and 4.

8.2 Summary of Key Disposal Cell Design Features

The radon barrier and drainage layer are the most important design features affecting ground water resources protection. The cover design is based on the UMTRA Project “checklist” cover (DOE 1989) to ensure that the cover will perform as required and meet the 200- to 1,000-year design life, given site-specific conditions. A clean fill dike is incorporated as part of the design to prevent lateral water migration. Temporary standpipes to monitor transient drainage are discussed in Section 7.0 of this document.

The radon barrier will have a hydraulic conductivity of nominally 1×10^{-7} cm/s (NRC 1993; p 23), which is conservative in that it does not rely on limiting infiltration. So-called "bathtubbing" will be prevented by constructing the cover with an average hydraulic conductivity that is much lower than that of the underlying weathered Mancos Shale. The cover design should be effective for more than 1,000 years.

8.3 Disposal Standards and Compliance Strategy

DOE has demonstrated that the hydrogeologic characteristics of the Crescent Junction Site, combined with the disposal cell design will ensure that any leachate draining from the cell would take thousands to tens of thousands of years to reach the uppermost Dakota aquifer. This indicates that disposal of tailings in the Crescent Junction disposal cell would meet the 40 CFR 192 ground water protection requirements of being "effective for up to 1,000 years to the extent reasonably achievable, and, in any case, for at least 200 years."

Because leachate from the disposal cell is not projected to reach the uppermost aquifer, constituent concentrations in the uppermost aquifer would not exceed background levels during the period of cell performance. All seepage would be contained within the Mancos Shale confining unit. Based on site hydrogeology and cell design, leachate from the cell is expected to migrate vertically into the Mancos Shale; no surface discharge is anticipated. Hydrogeologic isolation of the cell from the uppermost aquifer and from the surface would ensure protection of human health and the environment for the design life of the cell.

Because of the effectiveness of hydrogeologic isolation, no constituents of concern need to be identified or ground water concentration limits established. No monitoring needs to be conducted to ensure protection of the ground water, and no point of compliance is required. Likewise, no corrective action plan for ground water is necessary.

8.4 Disposal Cell Components and Longevity

Provisions in 10 CFR 192.20 require that control of RRM and listed constituents be designed to be effective for up to 1,000 years, to the extent reasonably achievable, and, in any case, for at least 200 years. In addition, it is required that there be a reasonable assurance the radon-222 in air will be controlled to specific standards and that listed constituents not exceed specific ground water concentration limits.

The design of the disposal cell at Crescent Junction has been configured to meet the standards in the regulations considering the appropriate technical guidance. The disposal cell components are constructed from natural materials that have been sufficiently characterized to ensure a thorough understanding of their long-term performance. These materials are to be placed in conditions that take advantage of natural processes to reduce the effects of weathering and erosive forces such that the requisite reasonable assurance of long-term performance is achieved. Specific DOE and NRC technical guidance and methods have been used in developing the disposal cell design (e.g., DOE 1989, NRC 1989a, and NRC 1993).

9.0 Processing Site Cleanup

9.1 Radiological Cleanup

Extensive field sampling and radiological surveys have been conducted to determine the extent and degree of contamination at the Moab Processing Site. Attachment 1, Appendix I contains data pertaining to materials contained within the tailings pile.

9.1.1 Radiological Site Characterization

Attachment 5, Appendix M, contains details for limits of RRM exceeding EPA standards within DOE's property boundaries on the former processing site. The total volume of contaminated materials being used for estimating the size of the disposal cell is 12.0 million yd^3. Measurements of background radioactivity near the Moab Site and measurements of existing radiological conditions are summarized in Table 9–1 and in Attachment 1, Appendix K.

Table 9-1. Background Radioactivity and Radiological Conditions at the Moab Site

Description	Range	Average
Gamma Exposure Rate		
Background	11−15 μR/h	12 μR/h
Above tailings pile	60–830 μR/h	−
Off-pile	14–4,500 μR/h	−
Radon-222 in Air		
Background	0.4−1.3 pCi/L	0.7 pCi/L
Flux from tailings pile	2–318 $pCi/m^2/s$	104 $pCi/m^2/s$
Soil Concentrations		
Background radium-226	0.4−1.7 pCi/g	0.8 pCi/g
Total uranium	0.5−2.6 pCi/g	1.2 pCi/g
Tailings pile radium-226	13−2,195 pCi/g	707 pCi/g
Off-pile radium-226	1–1,283 pCi/g	−

μR/h = microroentgens per hour

RRM volume to be disposed of comprises a number of separate quantities: the tailings pile, the remediated off-pile soils, the remediated vicinity property materials, and the subpile soils (contamination below the pile from leaching and infiltration). The tailings pile volume was calculated using the aerial survey data from 2005 and the existing ground contours that were confirmed using borehole and CPT test data. These data were then used to cut cross sections through the pile and to calculate the volume of the tailings pile. The cross sections and the geotechnical data were then used to estimate the quantities of the three principal soils types: sands, sand-slime mixes (transitional), and slimes. A volumetric weight and moisture content was then calculated for each area of the pile; these calculations provided an estimate of the dry weight and water weight of each type of material. The in-place volume for the 130-acre tailings pile was calculated to be 9.9 million yd^3 using average maximum dry densities and moisture contents for each material type. Because of the varying moisture content between the sands, transitional material, and slimes, the weight of the material will vary as it is excavated, transported, and dried to near optimum moisture for compaction.

The subpile volumes were determined by advancing boreholes through the bottom of the tailings into underlying alluvial soils. Radium-226 activities were measured every foot to determine the maximum depth of contaminated soils that require removal. This thickness was multiplied by the footprint area of the pile to determine the volume of subpile contamination. Because of the expense to drill through the pile and the goal to not contaminate substrate, only a few borings were drilled. As a result of limited data, two extra feet of material was added to the volume estimate based on lessons learned at remediating other UMTRA Project sites. The volumes of the tailings pile and contaminated subpile soils are estimated in Attachment 1, Appendix I.

Approximately 700,000 yd^3 of off-pile RRM has been estimated over the 439-acre area within the DOE property boundary. This volume includes the areas within the highway rights-of-way, but excludes the area within the footprint of the tailings pile. Depths of contamination for the area range from 6 inches to 20 feet below grade. Concentrations of radiological contaminants range up to 1,283 pCi/g for radium-226, up to 1,154 pCi/g for total uranium, and up to 779 pCi/g for thorium-230. The details of the extent of contamination off-the pile is presented in Attachment 5, Volume II, Appendix M.

Although properties adjacent to the processing site are being assessed for extent of contamination, there is little evidence of tailings leaving the processing site and contaminating vicinity properties in the city of Moab. Consequently, an estimate of 120,000 yd^3 is being used for potential cleanup of vicinity properties adjacent to the processing site and those possibly located in the city. This amount should not vary enough to impact the final cell design.

9.1.2 Standards for Cleanup

DOE is committed to removing contaminated materials and placing them in an engineered disposal cell such that all EPA standards in 40 CFR 192 are met. The standards require that average surface (top 15 cm) radium-226 concentrations must be equal or less than five pCi/g plus background average of 0.8 pCi/g and average subsurface (below 15 cm) radium-226 concentrations must be equal or less than 15 pCi/g plus background in each 100 square meter (m^2) area. All disturbed areas will be restored for adequate control of surface drainage. All excavations are either backfilled to original grade or with a minimum of six inches of fill. Some excavations are not backfilled and are subsequently remediated to five pCi/g to meet the surface standard. Where removal of contaminated materials is not practical or feasible, application of supplemental standards may be considered according to 40 CFR 192.21.

Th-230 undergoes radioactive decay to produce Ra-226. During remediation of UMTRA Project mill sites, a thorium standard was developed which ensures that, as Th-230 decays, the Ra-226 standard will not be exceeded over a 1,000-year performance period. The allowable amount of Th-230 is dependent upon the concentration of Ra-226 left in place. Table 9-2 shows the relationship of Th-230 to Ra-226 left in place. Thorium is measured by laboratory alpha spectrometry.

Table 9-2. Authorized Limits for Soils, Including Background

Remediation Goals				
	Surface (including background)		**Subsurface (including background)**	
Ra-226	5.8 pCi/g		15.8 pCi/g	
Th-230	Ra-226 (pCi/g)	Th-230 (pCi/g)	Ra-226 (pCi/g)	Th-230 (pCi/g)
	1.0	14.6	1.0	43.2
	2.0	12.7	2.0	41.2
	3.0	10.9	3.0	39.5
	4.0	9.0	4.0	37.6
	5.0	7.2	5.0	35.7
	5.8	5.8	6.0	33.9
			7.0	32.0
			8.0	30.2
			9.0	28.3
			10.0	26.5
			11.0	24.6
			12.0	22.8
			13.0	20.9
			14.0	19.1
			15.0	17.2
			15.8	15.8
Total Uranium (pCi/g)	Case-by-Case Basis		Case-by-Case Basis	

The Moab mill site will be remediated and soils verified for Ra-226 and Th-230. The Moab vicinity properties will be remediated for Ra-226.

9.1.3 Verification of Cleanup

Excavation control monitoring will be conducted during remedial action to ensure that the 5 pCi/g and 15 pCi/g above background radium-226 standards are met for surface and subsurface soils, respectively. Engineered design drawings will be developed to depict the depth of contamination and requirements for remediation. Gamma readings and soil samples will be taken to guide the depth and extent of excavation, preventing both under excavation and over excavation. Supplemental Standards applications will require NRC approval.

After completion of excavation, a verification measurement of the residual radium-226 concentration in each 100 m^2 area will be performed. The intent of the verification survey is to provide reasonable assurance that the remedial action has complied with the standards.

Final verification surveys will be performed to document average radium-226 concentrations on all 100 m^2 areas remediated. Nine-plug composite verification soil samples will be collected from each 100 m^2 area remediated and analyzed by on-site gamma spectroscopy to verify compliance with EPA standards. The gamma spectroscopy system shall have an accuracy of plus or minus 30 percent of the standard at the 95 percent confidence level for a sample with concentration equal to the standard. Ten percent of all verification samples are sent to an independent laboratory for verification of radium-226 and thorium-230 concentrations. When soil containing a significant fraction of small rocks is encountered, the radium-226 concentration determined by gamma spectroscopy will be corrected using approved procedures, such as Procedure 3, Section 4.7 in the Field Services Procedures Manual (STO 203).

A GPS/gamma scanning system may be used for verification in lieu of soil sampling every 100-m^2 grid. Automated gamma measurements would be taken over 100 percent of all accessible remediated areas and the exposure rate data stored in a computer. Soil samples will be taken during the excavation control process to develop a correlation between the exposure rate readings and radium-226 concentrations. A minimum of five percent of the soil grids will be composite sampled during verification to confirm the gamma to radium correlation.

Supplemental standards may be applied in areas where excessive environmental harm or worker risk outweighs the benefits of attaining the established soil cleanup standards. Based on known conditions, potential uses of supplemental standards include areas under asphalt of the state and federal highways, around high-pressure gas lines and high voltage electric lines, on steep (inaccessible) hillsides, around the Union Pacific rail track, below the water surface of the Colorado River, and around significant archaeological features.

If thorium-230 is detected in significant concentrations after radium-226 has been removed to the EPA standards, a supplemental standard under criterion (f) of 40 CFR 192.21 will be imposed. For thorium-230 contamination, the supplemental standard will be to reduce the thorium-230 concentration to a level such that the radium-226 concentration in 1,000 years, including residual and ingrown radium-226, will not exceed 15 pCi/g in subsurface soil.

Independent radiological surveillances and health and safety audits will be conducted by DOE and its Technical Assistance Contractor during remedial action to ensure that all activities are conducted to meet federal, state, local, and UMTRA Project standards and guidelines. Quality control and quality assurance requirements and procedures are in place to ensure that adequate cleanup and subsequent verification are properly implemented and documented.

9.2 Ground Water Cleanup

Ground water contamination and conditions at the Moab Processing Site were described and evaluated in the SOWP (DOE 2003). Ground water remediation was also evaluated in the EIS for the Moab Site (DOE 2005), in which the preferred alternative was identified as active ground water remediation. An interim action for ground water cleanup was initiated in 2003 and has been operating and expanded since that time. A final decision regarding long-term ground water cleanup approaches and remediation goals will be deferred until a later date and documented in a subsequent Ground Water Compliance Action Plan (GCAP) according to the requirements of 40 CFR 192.

Human health and the environment will not be affected by deferring the final ground water remedial action in the uppermost aquifer (alluvium) at the Moab Processing Site until the cleanup of RRM has been completed. The interim action ground water program is limiting discharge of contaminants to the environment and there is no public exposure to the ground water as the existing wells are being used for either monitoring or ground water cleanup.

The main concern regarding contaminated ground water at the Moab Processing Site is how its discharge to the Colorado River might affect surface water quality and, in turn, affect potential habitat for endangered fish that are known to be present in that segment of the river. The current ground water and surface water monitoring programs at the Moab Processing Site are focused on these concerns and will be continued as deemed necessary during and beyond the remediation process. Several different tasks are currently being carried out by DOE as required by the

U.S. Fish and Wildlife Service's final Biological Opinion, issued as part of the Final EIS for the site (DOE 2005).

9.2.1 Ground Water Cleanup Standards

Because of the high natural salinity of ground water at the Moab Processing Site, the alluvial aquifer qualifies for supplemental standards. Ground water cleanup is only required to be protective of human health and the environment where it could affect other usable water bodies (e.g., the Colorado River). Therefore, the focus of ground water cleanup efforts has been on improving surface water quality rather than meeting numerical ground water standards.

9.2.2 Cleanup Demonstration

Specific cleanup goals and means of demonstrating that they have been met will be discussed in the future in the GCAP for the Moab Processing Site. Results of ongoing monitoring at the site will be used to help formulate this approach.

9.2.3 Ground Water and Surface Water Monitoring Programs

Several different types of monitoring are ongoing at the processing site. These include routine surface water and ground water sampling, interim action surface water and ground water sampling, and biomonitoring of the Colorado River.

End of current text

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