Application for Amendment of USNRC Source Material License SUA-1475, Volume 1

United Nuclear Corporation Mill Site

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Executive Summary

This document is a license amendment request (LAR) for Source Material License SUA-1475 for the UNC Mill Site near Church Rock New Mexico. The submittal provides information to support GE/UNC's request to the US Nuclear Regulatory Commission (NRC) to modify the reclamation plan described in License Condition 34 as well as the reclamation timelines defined in License Condition 35. This LAR has been prepared to address parts of the Standard Review Plan outlined in NUREG-1620. Proposed activities at the site include construction of a Repository for mine-impacted soil and debris on the licensed mill tailings disposal area. Mine waste will be removed from the Northeast Church Rock Mine Site (based on defined cleanup standards), transported to and placed in the Repository, located on the existing tailings disposal area. The Repository design includes specific procedures for protection of the existing radon barrier over the mill tailings and an erosion-resistant, evapotranspirative (ET) cover over the mine waste. Stormwater controls will be added in the immediate vicinity of the Repository and to the Pipeline Arroyo. The specific regulatory framework under the NRC and NUREG-1620, Appendix I is the placement of non-11e.(2) byproduct materials on a licensed 11e.(2) byproduct material tailings facility. This document presents the design and supporting analyses for the UNC Mill Site Repository for NRC review.

The UNC Mill Site is a non-operating uranium mill and tailings disposal site located approximately 17 miles northeast of Gallup in McKinley County, New Mexico. The Mill Site included an ore processing mill and a tailings disposal area (TDA) that covers approximately 10 and 40 hectares (25 and 100 acres), respectively. The previous reclamation plan for the TDA was reviewed and approved by NRC on March 1, 1991. This LAR describes the nearby population, local climate, site geology, geomorphology, and seismicity. The LAR includes a review of the site surface water hydrology, groundwater hydrology and current environmental monitoring for the site. A description of the tailings reclamation program previously completed for the TDA is also included.

Required design elements for the removal action described in the AOC and SOW (USEPA, 2015) include activities at both the Mine Site and Mill Site. These items are presented in their entirety in appendices to the LAR. Note that the design documents appended are titled as a *95% Design* to comply with the USEPA documents, however the design is considered complete for NRC review of this LAR. Data collected prior to initiating the Repository design is summarized, with pre-design studies appended to this LAR. Details of the design components for the Repository are appended to this LAR, with each design appendix presenting a narrative and associated calculations and analyses for review. The design components include the Repository itself (volume, stability, settlement, liquefaction), ET cover, haul roads, support facilities, and the perimeter stormwater control features. The LAR also summarizes the anticipated construction sequence and schedule, the construction specifications, the quality assurance plan, and the health and safety plan.

A Supplemental Environmental Report (SER) for this project is being prepared concurrently with the LAR. Combined, the documents include a cost-benefit analysis to compare the alternatives considered during the design process.

Abbreviations

µR/hr	microRoentgen per hour
mR/hour	milliroentgens per hour
ADWR	Arizona Department of Water Resources
amsl	above mean sea level
AOC	Administrative Settlement Agreement and Order on Consent for Design and Cost Recovery
ASTM	American Society for Testing Materials
CAP	corrective action program
CC	Construction Contractor
CERCLA	Comprehensive Environmental Response, Compensation and Liability Act
CFR	Code of Federal Regulations
cfs	cubic feet per second
CPT	cone penetration test
CQA/CQC	construction quality assurance/construction quality control
CQAO	Construction Quality Assurance Officer
CQAP	Construction Quality Assurance Plan
CS	Construction Superintendent
CSC	Construction Supervising Contractor
DOE	US Department of Energy
DSC	Design Supervising Contractor
DSHA	deterministic seismic hazard analysis
EE/CA	engineering evaluation/cost analysis
EOR	Engineer of Record
ER	Environmental Report
ET	evapotranspirative
FE	Field Engineer
FI	Field Inspector
fps	feet per second
FS	factor of safety
FSL	field screening level
GE/UNC	General Electric/United Nuclear Corporation
GMPE	Ground Motion Prediction Equations
HASP	health and safety plan
HDPE	high-density polyethylene
HSA	hollow-stem auger
IRA	Interim Removal Action
kg	kilogram
LAR	License Amendment Request

mg	milligram
Mill Site	Church Rock Mill Site
Mine Site	Northeast Church Rock Mine Site
mph	miles per hour
MWH	MWH Americas, Inc.
NDC	North Diversion Channel
NECR	Northeast Church Rock
NMED	New Mexico Environment Department
NNEPA	Navajo Nation Environmental Protection Agency
NRC	US Nuclear Regulatory Commission
NUREG	Nuclear Regulatory Guide
pcf	pounds per cubic foot
pCi	picocuries
pCi/g	picocuries per gram
$pCi/m^2/s$	picocuries per square meter per second
PDS	Pre-design Studies
PET	potential evapotranspiration
PGA	peak ground acceleration
PI	plasticity index
PM	Project Manager
PMF	probable maximum flood
PMP	probable maximum precipitation
PSHA	probable seismic hazard analysis
PTW	principal threat waste
QC	quality control
QCM	Quality Control Manager
Ra-226	radium 226
RA	Removal Action (Mine Site) or Remedial Action (Mill Site)
ROD	Record of Decision
RSO	Radiation Safety Officer
RWP	Radiation Work Permit
SA Site	Settlement Agreement Site
SER	Supplemental Environmental Report
SHA	seismic hazard analysis
SM	Site Manager
SOW	Statement of Work
SPT	standard penetration test
SWPPP	Stormwater Pollution Protection Plan
TDA	Tailings Disposal Area
TLD	thermoluminescent dosimeters

UNC	United Nuclear Corporation
USCS	Unified Soil Classification System
UHS	Uniform Hazard Spectra
USEPA	US Environmental Protection Agency

1.0 INTRODUCTION

1.1 LICENSING BACKGROUND

The Church Rock Mill Site (Mill Site) is the site of a former uranium mill owned by United Nuclear Corporation (UNC) Mining and Milling (a division of United Nuclear Corporation, now a subsidiary of General Electric), and is located on private land. The Mill Site operated from May 1977 to May 1986 under a license issued by the New Mexico Environmental Improvement Division. On June 1, 1986, the NRC assumed regulatory authority for uranium and thorium milling activities and mill tailings in the State of New Mexico (51 FR 19432; May 29, 1986) and subsequently issued Source Material License No. SUA-1475 to UNC for the Mill Site. Source Material License SUA-1475 was most recently amended (No. 53) on November 17, 2016. The tailings reclamation plan (Canonie, 1991) for the tailings disposal area (TDA) associated with the former mill was submitted by UNC on August 30, 1991 and approved by NRC on March 1, 1991.

Source Material License Condition 30 includes details for requirement of a groundwater corrective action program (CAP). License Condition 34 grants the licensee permission to construct and operate "*an enhanced evaporation system in accordance with the system described in the submittal dated June 14, 1990."* The as-built report for the evaporation ponds, constructed on the south cell, is dated March 1989 (Canonie, 1989).

UNC submitted a mill decommissioning plan dated December 29, 1988, later revised on April 10, 1990, in accordance with Condition 26 of the source material license. The mill decommissioning completion report was submitted to NRC on April 13, 1993. The report included details of the mill facilities demolition and placement of the debris into one of the former borrow pits within the TDA. Mill debris was placed in the former borrow pit (no. 2) on the east side of the Central Tailings Cell within the TDA.

Source Material License Condition 34 indicates the approved tailings reclamation plan was submitted August 30, 1991 and modified by licensee submittals dated March 5, April 10, and June 21, 1996. The three primary cells within the TDA (North, Central, and South) were reclaimed in phases and covered between 1989 and 1995. Source Material License Condition 35 states that the licensee shall complete site reclamation in accordance with the approved reclamation plan and groundwater CAP as authorized by License Condition Nos. 34 and 30, respectively. Condition 35 also includes the following schedule and target date items (not yet completed, in the area of the evaporation ponds):

- Placement of final radon barrier designed and constructed to limit radon emissions to an average flux of no more than 20 picocuries per square meter per second (pCi/m^2/s) above background – December 31, 2019.
- Placement of erosion protection as part of reclamation to comply with Criterion 6 of Appendix A of 10 CFR Part 40 – December 31, 2019.
- Projected completion of groundwater corrective actions to meet performance objectives specified in the groundwater corrective action plan – December 31, 2018.

The schedule items listed above, and in the source material license, are specific to completion of the groundwater CAP and reclamation (placement of final radon barrier and erosion protection) of the existing evaporation ponds located on the South Cell of the TDA.

The NRC issued a letter dated January 7, 2003 regarding erosion protection design concerns at the Site. During a site visit on June 13, 2002, the NRC identified "several areas of concern in relation to erosion protection" and issued the letter to document these concerns. The identified concerns included I) Sediment in Branch Swales, II) Sediment in North Upstream Diversion Channel and Poor Condition of the Road, III) Damage to Jetty IV) Erosion at Southwest End of Embankment, and V) Differential Settlement on Top of the Tailings Impoundment (eastern part of the Central Cell).

This license amendment request (LAR) is intended provide sufficient information to NRC to allow modification of the reclamation plan described in License Condition 34 and the dates in Condition 35.

1.2 PROPOSED LICENSE CONDITION CHANGES

The following modifications are proposed for amendment of License Condition 34, with the additions noted in bold print:

34. The approved tailings reclamation plan is that submitted by the licensee on August 30, 1991, and modified by licensee submittals dated March 5, April 10, and June 21, 1996 ***and September 24, 2018****. [Applicable Amendments: 10, 17, 24, 25]*

The following modifications are proposed for amendment of License Condition 35, with the additions noted in bold print:

35. The licensee shall complete site reclamation in accordance with the approved reclamation plan and ground water corrective action plan, as authorized by License Condition Nos. 34 and 30, respectively.

A. To ensure timely compliance with target completion dates established in the Memorandum of Understanding with the Environmental Protection Agency (56 FR 55432, October 25, 1991), the licensee shall complete reclamation to control radon emissions as expeditiously as practicable, considering technological feasibility, in accordance with the following schedule:

(1) Windblown tailings retrieval and placement on the pile - complete.

(2) Placement of the interim cover to decrease the potential for tailings dispersal and erosion - complete.

(3) Placement of final radon barrier designed and constructed to limit radon emissions to an average flux of no more than 20 pCi/m^2/s above background ***– During construction of the Repository and by December 31, 2026.***

B. Reclamation, to ensure required longevity of the covered tailings and ground water protection, shall be completed as expeditiously as is reasonably achievable, in accordance with the following target dates for completion:

(1) Placement of erosion protection as part of reclamation to comply with Criterion 6 of Appendix A of 10 CFR Part 40 ***– During Construction of the Repository and by December 31, 2026.***

(2) Projected completion of ground water corrective actions to meet performance objectives specified in the ground water corrective action plan - December 31, 2018.

[Applicable Amendments: 23, 39, 44, 48, 49, 50]

C. Any license amendment request to revise the completion dates specified in Section A must demonstrate that compliance was not technologically feasible (including inclement weather, litigation which compels delay to reclamation, or other factors beyond the control of the licensee).

D. Any license amendment request to change the target dates in Section B above must address added risk to the public health and safety and the environment, with due consideration to the economic costs involved and other factors justifying the request such as delays caused by inclement weather, regulatory delays, litigation, and other factors beyond the control of the licensee.

The licensee requests modification of the dates under License Condition 35 in Section A and Section B due to the schedule and requirements to fulfill USEPA's Non-Time-Critical Removal Action at the Northeast Church Rock Mine Site (2011 Action Memo; USEPA, 2011) and Record of Decision (ROD; USEPA, 2013). This is beyond the control of the licensee; and therefore, is justified under License Condition 35.C and D. EPA's decision is intended to provide added protections for public safety and the environment in the vicinity of the mine and mill. In addition, this LAR contains a major redesign of the rock jetty and other surface water management improvements that will enhance the stability of the TDA compared to its present status.

Cessation of the groundwater corrective action program (CAP) is necessary to complete EPA's removal action. Groundwater recovery operations have already been terminated in Zone 1 and the Southwest Alluvium Remedial Target Areas. While pumping continues in Zone 3, recent Annual Review Reports (Hatch-Chester, 2018) have demonstrated that it too has reached the limits of its effectiveness. UNC has filed permits to install several monitoring and sentinel wells to help support any desired administrative and/or institutional controls to enable the final closure of the groundwater CAP. Equipment and resources will be available onsite to complete closure of the evaporation ponds during construction of the Repository.

1.3 DISPOSAL OF NON-11E.(2) BYPRODUCT MATERIAL IN TAILINGS IMPOUNDMENTS (NRC REGULATORY ISSUE 2000-23)

Within Appendix I of NUREG-1620, the NRC outlines interim guidance on the Disposal of Non-atomic Energy Act of 1954, Section 11e.(2) Byproduct Material in Tailings Impoundments. Appendix I states: *In reviewing licensee requests for the disposal of wastes that have radiological characteristics comparable to those of Atomic Energy Act of 1954, Section 11e.(2) byproduct material [hereafter designated as "11e.(2) byproduct material"] in tailings impoundments, the Nuclear Regulatory Commission staff will follow the guidance set forth below.* Table 1.3-1 is excerpted from NUREG-1620 Appendix I, Attachment 1, Interim Guidance.

Table 1.3-1 Interim Guidance for the Disposal of Non-Atomic Energy Act of 1954, Section 11e.(2) Byproduct Material in Tailings Impoundments

Item	NUREG-1620 Guidance	Comments related to this LAR
1	Since mill tailings impoundments are already regulated under 10 CFR Part 40, licensing of the receipt and disposal of such material [hereafter designated as "non-11e.(2) byproduct material"] should also be done under 10 CFR Part 40.	The design for the modifications to the reclamation plan and Repository are designed in accordance with 10 CFR 40.
2	Special nuclear material and Section 11e.(1) byproduct material waste should not be considered as candidates for disposal in a tailings impoundment, without compelling reasons to the contrary. If staff believes that such material should be disposed of in a tailings impoundment in a specific instance, a request for Commission approval should be prepared.	Section 11e.(1) byproduct material is not being considered for disposal on the tailings impoundment.
3	The 11e.(2) licensee must provide documentation showing necessary approvals of other affected regulators (e.g., the U.S. Environmental Protection Agency or State) for material containing listed hazardous wastes or any other material regulated by another Federal agency or State because of environmental or safety considerations.	The design was conducted pursuant to the Administrative Settlement Agreement and Order on Consent (AOC) (USEPA, 2015). USEPA accepted the Design Deliverables and Response to Comments on May 25, 2018 (USEPA, 2018). The design deliverable was also reviewed by DOE, Navajo Nation EPA (NNEPA), and the New Mexico Environmental Department (NMED) prior to USEPA approval of the design submittal.
4	The 11e.(2) licensee must demonstrate that there will be no significant environmental impact from disposing of this material.	Refer to Section 4.0, Health, Safety, and Environment in this document.
5	The 11e.(2) licensee must demonstrate that the proposed disposal will not compromise the reclamation of the tailings impoundment by demonstrating	Refer to Section 4.0, Health, Safety, and Environment in this document.

	compliance with the reclamation and closure criteria of Appendix A of 10 CFR Part 40.	
6	The 11e.(2) licensee must provide documentation showing approval by the Regional Low-Level Waste Compact in whose jurisdiction the waste originates as well as approval by the Compact in whose jurisdiction the disposal site is located, for material which otherwise would fall under Compact jurisdiction.	The Rocky Mountain Low-level Radioactive Waste Board regulates: "*LLW waste other than: high-level radioactive waste, transuranic waste, nuclear weapons waste, uranium and thorium tailings, and wastes from mining, milling, smelting, or similar processing of ores and mineral-bearing material primarily for minerals other than radium."* The mine waste being proposed for disposal is considered non-11e.(2) waste from mining and therefore does not fall under Compact jurisdiction.
7	The U.S. Department of Energy (DOE) and the State in which the tailings impoundment is located, should be informed of the U.S. Nuclear Regulatory Commission findings and proposed action, with a request to concur within 120 days. A concurrence and commitment from either DOE or the State to take title to the tailings impoundment after closure must be received before granting the license amendment to the 11e.(2) licensee.	Upon receipt of the NRC findings and proposed action, the licensee will make a formal "request to concur" to the DOE. The DOE participated in the review process for the project design documents to date.
8	The mechanism to authorize the disposal of non-11e.(2) byproduct material in a tailings impoundment is an amendment to the mill license under 10 CFR Part 40, authorizing the receipt of the material and its disposal. Additionally, an exemption to the requirements of 10 CFR Part 61, under the authority of 10 CFR 61.6, must be granted, if the material would otherwise be regulated under Part 61. (If the tailings impoundment is located in an Agreement State with low-level waste licensing authority, the State must take appropriate action to exempt the non-11e.(2) byproduct material from regulation as low-level waste.) The license amendment and the 10 CFR 61.6 exemption should be supported with a staff analysis addressing the issues discussed in this guidance.	The licensee is submitting this LAR for approval to modify their existing license under 10 CFR Part 40 to receive non-11e.(2) byproduct material at the licensed facility. The design has been prepared to comply with 10 CFR Part 40. UNC requests an exemption from NRC to 10 CFR Part 61, under 10 CFR Part 61.6 as part of this LAR, since the placement of the material will provide added protections for public safety and the environment, to the existing licensed 11e.(2) byproduct disposal facility, as demonstrated by this design.

1.4 PROPOSED ACTION

The proposed site activities include construction of a Repository for mine-impacted soil and debris on the licensed TDA. The mine-impacted materials (exceeding defined cleanup standards) will be removed from the Northeast Church Rock (NECR) Mine Site (Mine Site) and transported to the UNC TDA. The proposed Repository will be located on the existing North and Central cells of the reclaimed TDA. The existing TDA includes a soil cover with a rock erosion protection layer. A series of surface drainage swales lined with riprap were also constructed on the cover. Because the proposed Repository will

have limited areal influence on the existing TDA, only the portions of the previously reclaimed tailings to be affected by construction of the Repository are being analyzed for long-term performance as part of this LAR. The surface grading of the Repository will tie to existing drainage swales on the cover. In locations where new stormwater flow regimes dictate changes to the channel designs, the channels are being re-sized and upgraded with appropriate erosion protections for the adjusted flow conditions.

Mine-impacted soils will be excavated and removed from the Mine Site, located approximately 0.5-mile northwest of the former Mill Site. These materials will be transported to, and disposed in, the Repository to be constructed on portions of the reclaimed North and Central cells of the TDA. The project components that pertain to the TDA are listed below:

- Construct a mine haul road to transport material to the Repository
- Remove the existing erosion protection layer on the TDA cover within the footprint of the Repository
- Enhance the Repository footprint by compacting the existing tailings radon barrier to provide separation between the Repository and underlying tailings
- Transport mine waste to the TDA for placement in the Repository
- Construct an evapotranspirative (ET) cover over the final mine waste surface of the Repository
- Upgrade the stormwater channels adjacent to the Repository
- Reconstruct the “rock jetty” area located in the Pipeline Arroyo adjacent to the west side of the TDA
- Restore and revegetate areas at the Mill Site disturbed by construction

Materials to be placed in the Repository include soil, waste rock, and mine debris (metal, concrete, wood, etc.), and removed vegetation. The mine waste cleanup level for radium 226 (Ra-226) is 2.24 pCi/g and the cleanup level for natural uranium is 230 mg/kg. Mine waste that contains 200 pCi/g or more of Ra-226 and/or 500 mg/kg or more of total uranium will be segregated from lower activity mine waste and transported to an off-site, licensed and controlled disposal or reprocessing facility. Sections 3 and 4 of the LAR provide further details on the Repository design.

The existing radon barrier above the tailings in the TDA will be modified in-place to serve as the foundation layer for the Repository. The sequence for radon barrier improvement is:

- Conduct a baseline gamma radiation survey on the existing North and Central Cell covers in the work area
- Remove the existing rock mulch and riprap (for reuse as erosion protection material)
- Fill the existing drainage swales to regrade the surface
- Moisture condition and recompact the existing radon barrier

Following preparation of the radon barrier, initial lifts of perimeter stormwater berms will be constructed using borrow soils at the edge of the waste placement areas. These berms will allow for containment of contact stormwater within the Repository during waste placement. Mine waste will then be hauled, dumped and spread in lifts for compaction. The perimeter slopes of the mine waste surface will be graded as the material is placed. Stormwater berms can be raised, as needed, to provide clean cover material over the outer slopes of the mine waste. Once berms are no longer needed for stormwater control, the berms will be graded over the mine waste surface during placement of the ET cover.

Cover placement will be conducted progressively after areas of the Repository reach their design capacity. The cover design includes an erosion protection layer consisting of a rock-soil admixture layer overlying a soil layer. The thicknesses of these layers and the sizes of the rock used for erosion protection vary based on locations on the Repository, with an overall cover thickness (including erosion protection layers) of 4 feet.

The action will also include installation of permanent stormwater controls for the Mill Site Repository using existing swales and channels constructed for the TDA with improvements and supplemental controls where necessary. The existing Pipeline Arroyo will also be stabilized with a reconstructed rock jetty with a riprap chute. Stability of the Pipeline Arroyo is

important for long-term viability of the Repository and the TDA, to address the potential for lateral southeastward migration of the arroyo that could lead to embankment erosion.

1.5 EFFECT ON EXISTING LICENSE CONDITIONS AND APPROVED RECLAMATION

This proposed action would be a modification to License Condition 34 and 35 and requires revisions to the tailings reclamation plan for the area of the TDA influenced by the Repository as well as the schedule to complete reclamation. In other areas of the TDA, not disturbed by the new design, no changes are being made to the previously-approved and implemented reclamation. The analyses included in this LAR represent an updated tailings reclamation plan for the source material license specific to modifications of the reclaimed TDA near the Repository. This LAR was prepared to address the parts of the Standard Review Plan outlined in NUREG-1620. Table 1.5-1 outlines the requirements in NUREG-1620 and the location of the information included with this LAR to address each part of the guidance. Items which remain unchanged from the previous version of the tailings reclamation plan are noted as such in the comments column and highlighted in green. Design items that are being updated with this LAR submittal are highlighted in yellow. Although the AOC and SOW from USEPA required the submittal of a *95% Design* and the appended design documents are labeled as such, the design is considered to be a complete design package for NRC review of this LAR.

Table 1.5-1 LAR Sections and Supporting Information - NUREG CR-1620 Standard Review Plan for the Review of a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Radiation Control Act of 1978

Section	Requirement	Location of Information	Comments
1.0	**GEOLOGY & SEISMOLOGY**		
1.1	**Stratigraphic Features**		
1.	The regional and site-specific stratigraphy are described in sufficient detail to produce an adequate understanding of the site-specific subsurface characteristics, including descriptions of adequate understanding of the site-specific subsurface characteristics, including descriptions of major stratigraphic units and their orientations, age relationships, thicknesses, and distribution.	Geohydrologic Report (Canonie, 1987); Appendix Z Mill Site PDS (MWH, 2014)	Characterization completed previously, no change since previous reclamation plan.
2.	Stratigraphic units are described in sufficient detail to provide input to a geotechnical stability analysis.		
3.	Descriptions of regional and site-specific stratigraphic units contain sufficient information for input to an analysis of groundwater resources and the protection thereof.		
4.	Regional stratigraphic information is discussed in sufficient detail to support site-specific information.		
5.	Descriptions of the regional and site stratigraphy are based on published literature and site data and conform to standard geological classifications.		
6.	Discussions of regional stratigraphy are adequately referenced and supported by published reports, maps, logs, and cross-sections.		
7.	Site descriptions are based on field investigations and adequate sampling to define physical and chemical properties of surface and subsurface materials such as soils and underlying geologic formations at the site.		
8.	Maps are at a scale sufficient to show the locations of all site explorations such as borings, geophysical surveys, trenches, and sample locations.		
1.2	**Structural and Tectonic Features**		
1.	Descriptions of regional and site-specific structural and tectonic features are based on published literature and gathered data.	Design Report, Appendix G, Attachment G.1 (Stantec, 2018)	An updated seismic hazard analysis was conducted with ground-motion estimates from deterministic and probabilistic procedures and is included as Attachment G.1
2.	Regional structural and tectonic features, particularly faults, are defined in sufficient detail to present and adequate understanding of the structural geologic conditions in the region surrounding the site that may have a likelihood of impacting the site stability or groundwater regime.		
3.	Site-specific structural tectonic features, particularly faults are described in sufficient detail to present adequate information for an analysis of the site stability. Information presented should address the uncertainties and variability within the site area and the potential impacts on the disposal facility.		
4.	The structural and tectonic province or provinces that influence the site seismicity are identified and described.		
5.	The tectonic history of the pertinent province(s) is discussed in sufficient detail to support an analysis of the potential for disruption of the site by tectonic activity.		
6.	Discussions are adequately referenced and are supported by maps, logs, and cross sections showing locations of all site explorations and surveys and depicting surface and subsurface structural tectonic features.		
7.	Descriptions contain discussions of age relationships of structural and tectonic features.		
1.3	**Geomorphic Features**		
1.	Descriptions of the regional and site-specific geomorphology and geomorphic processes include information sufficient to allow the reviewer to assess the nature and extent of major active processes that may modify the present-day topography of the geomorphic province(s) and the site area.	Approved Rec. Plan (Canonie, 1991)	NRC-approved Jetty design in Canonie Rec. Plan. No change since previous reclamation plan.
		Design Report, Appendix I (Stantec, 2018)	Updated Jetty Design
2.	The geomorphic features, particularly potential geomorphic hazards, are clearly delineated on topographic base maps of adequate scale to enable the reviewer to assess their occurrence and distribution.	Approved Rec. Plan (Canonie, 1991)	NRC-approved Jetty design in Canonie Rec. Plan. No change since previous reclamation plan.
		Design Report, Appendix I (Stantec, 2018)	Updated Jetty Design
3.	Descriptions are adequately referenced and are supported by published reports and maps or site data.	Approved Rec. Plan (Canonie, 1991)	NRC-approved Jetty design in Canonie Rec. Plan. No change since previous reclamation plan.
		Design Report, Appendix I (Stantec, 2018)	Updated Jetty Design
4.	The regional and site-specific geomorphology and geomorphic processes are described in sufficient detail to support an analysis of the geomorphic and geotechnical stability of the site.	Approved Rec. Plan (Canonie, 1991)	NRC-approved Jetty design in Canonie Rec. Plan. No change since previous reclamation plan.
		Design Report, Appendix I (Stantec, 2018)	Updated Jetty Design
1.4	**Seismicity and Ground Motion Estimates**		
1.	The information presented on the regional and site-specific seismicity contains sufficient detail to allow the staff to determine the vibratory ground motion (peak horizontal acceleration) at the site caused by seismic events and to further use that determination to assess the geotechnical stability of the site. The geotechnical stability of the site is sufficient to control radiological hazards for 1,000 years to the extent reasonably achievable, and, in any case, for at least 200 years.	Design Report, Attachment G.1 (Stantec, 2018)	An updated seismic hazard analysis was conducted with ground-motion estimates from deterministic and probabilistic procedures and is included as Attachment G.1.
2.	In conducting this review, the staff will consider a deterministic and/or a probabilistic seismic hazard analysis as an acceptable method for selecting the peak horizontal acceleration for a site. An analysis of the geotechnical stability of the design proposed in the reclamation plan will be based on the resultant peak horizontal acceleration (Chapter 2.0, "Geotechnical Stability," of this standard review plan).	Design Report, Attachment G.1 (Stantec, 2018)	An updated seismic hazard analysis was conducted with ground-motion estimates from deterministic and probabilistic procedures and is included as Attachment G.1.
a.	Deterministic Analysis: The use of a deterministic seismic hazard analysis is acceptable if:		
i.	Capability is determined by suitable methods, such as those outlined by Slemmons (1977).		

Section	Requirement	Location of Information	Comments
ii.	Fault length versus magnitude relationships for determining the maximum magnitude earthquake that may be produced by each capable fault or capable tectonic source are developed using acceptable approaches such as those of Slemmons, et al. (1982); Bonilla, et al. (1984); or Wells and Coppersmith (1994).		
iii.	For each maximum magnitude earthquake, the peak horizontal acceleration at the site is determined using the applicable attenuation relationship between earthquake magnitude and distance for the site. Campbell (1997); Campbell and Bozorgnia (1994); and Boore, et al. (1993, 1997) offer examples of acceptable attenuation relationships. In applying the relationship, the site-to-source distance should be the distance between the site and the closest approach of the fault.		
iv.	The peak horizontal acceleration value adopted for each capable fault or tectonic source is not less than the median value provided by the attenuation relationship. Possible soil amplification effects are considered.		
v.	To assess potential ground motion at the site from earthquakes not associated with known tectonic structures (i.e., random or floating earthquakes), the largest floating earthquakes reasonably expected within the tectonic province are identified. In addition, the largest floating earthquakes characteristic of any adjacent tectonic provinces are identified, if such earthquakes cause appreciable ground motion at the site. For each of these earthquakes, the peak horizontal acceleration at the site is calculated as stated previously, with 15 km [9 mi] used as the site-to-source distance for floating earthquakes within the host tectonic province. For floating earthquakes in other tectonic provinces, the distance between the site and the closest approach of the province boundary is used as the site-to-source distance.		
vi.	The peak horizontal acceleration for the site is the maximum value of the peak horizontal accelerations determined for earthquakes from all capable faults, tectonic sources, and tectonic provinces.		
b.	Probabilistic Analysis: The use of a probabilistic seismic hazard analysis as an alternative to the requirements of 10 CFR Part 40, Appendix A, Criterion 4(e), is acceptable, as is stated in the Introduction to Appendix A, if:		
i.	It is shown that the design proposed by the licensee will achieve a level of stabilization and containment, and a level of protection for public health and safety and the environment, which is equivalent to, to the extent practicable, or more stringent than that achieved by the requirements of 10 CFR Part 40, Appendix A.		
ii.	The licensee takes into account local conditions when estimating the seismic design of the facility because peak horizontal acceleration values are often calculated for hypothetical rock foundations. The effects of local site conditions on the peak ground acceleration are reviewed in Chapter 2.0 in the standard review plan.		
2.0	**GEOTECHNICAL STABILITY**		
2.1	**Site and Uranium Mill Tailings Characteristics**		
1.	The site stratigraphy is described in sufficient detail to provide an understanding of the site-specific subsurface features, including structural features and other characteristics of underlying soil and rock.	PDS (MWH, 2014); Design Report, Appendices G and I (MWH, 2018)	Characterization completed for the PDS Report and additional characterization for the Jetty (2017 and 2018). Other characterization of the underlying rock is based on Canonie (1987) and previous site geotechnical information.
2.	Information on regional and local faults and seismicity, as obtained from field data, published literature, and historical records is presented in sufficient detail to effectively incorporate that information into a geotechnical stability analyses. (Note: This aspect of the review should be coordinated with the geology and seismology review performed in accordance with standard review plan Chapter 1.)	Design Report, Appendix G, Attachment G.1 (Stantec, 2018)	An updated seismic hazard analysis was conducted with ground-motion estimates from deterministic and probabilistic procedures and is included as Attachment G.1.
3.	Sampling scope and techniques are appropriate and sufficient to ensure that samples collected are representative of the range of *in situ* soil conditions, taking into consideration variability and uncertainties in such conditions within the site.	PDS (MWH, 2014);	
4.	For all soils that might be unstable because of their physical or chemical properties, locations and dimensions are identified and the properties have been documented.	PDS (MWH, 2014);	
5.	Investigations (including laboratory and field testing) are conducted using appropriate standards published by the American Society for Testing and Materials or the International Society for Rock Mechanics and are sufficient to establish the static and dynamic engineering parameters of borrow materials, other materials, tailings, and underlying soil and rock materials at the site (NRC, 1978, 1979).	PDS (MWH, 2014);	ASTM standards used for field and laboratory testing
6.	A detailed discussion of laboratory sample preparation techniques is presented, when standard procedures are not used. For critical laboratory tests, details such as how saturation of the sample was determined and maintained during testing, or how the pore pressures changed are provided. A detailed and quantitative discussion of the criteria used to verify that the samples were properly taken and tested in sufficient number to define the critical soil parameters for the site is presented. In the case of tailings material (e.g., license amendment reviews), the evaluations of its strength and settlement characteristics are presented in detail.	PDS (MWH, 2014);	Sample preparation and testing procedures documented
7.	Parameter values are presented to enable evaluation of properties of mill tailings, borrow materials, other materials, and underlying soil and rock, including the following:		
a.	Compressibility and rate of consolidation	PDS (MWH, 2014);	New information on tailings, cover materials and mine spoil properties.
b.	Shear strength, including, for sensitive soils, possible loss of shear strength resulting from strain-softening	PDS (MWH, 2014);	New information on tailings, cover materials and mine spoil properties.
c.	Liquefaction potential	PDS (MWH, 2014);	New information on tailings, cover materials and mine spoil properties.
d.	Permeability	PDS (MWH, 2014);	New information on tailings, cover materials and mine spoil properties.
e.	Dispersion characteristics	PDS (MWH, 2014);	Non-dispersive materials present
f.	Swelling and shrinkage	PDS (MWH, 2014);	New information on tailings, cover materials and mine spoil properties.
g.	Long-term moisture content for radon barrier material	PDS (MWH, 2014);	New information on tailings, cover materials and mine spoil properties.
h.	Cover cracking	PDS (MWH, 2014);	New information on tailings, cover materials and mine spoil properties.

Section	Requirement	Location of Information	Comments
8.	Soil stratigraphy and relevant parameters that are used in the geotechnical evaluations (settlement, stability, liquefaction potential, etc.) are discussed in detail.	PDS (MWH, 2014); Appendix G (Stantec, 2018) Appendix Y, Consolidation and Groundwater Report (Dwyer, 2018);	
9.	Records of historical groundwater-level fluctuations at the site as obtained from monitoring local wells and springs and/or by analysis of piezometer and permeability data from tests conducted at the site are presented in sufficient detail to effectively incorporate the information into geotechnical stability analyses. (Note: This aspect of the review should be coordinated with the hydrogeologic characterization review performed according to standard review plan Chapter 4.0.) The information should be sufficient to provide the required input for the design of the facility and to enable the reviewer to assess compliance with the regulatory requirements, such as site features contributing to waste isolation; facility location with respect to an active fault; and reasonable assurance of control of radiological hazards to be effective for 1,000 years to the extent reasonably achievable, and in any case, for at least 200 years.	Groundwater Corrective Action, Annual Review Report (Hatch-Chester 2018);	Or most current version, for groundwater levels. No changes to the existing site extents.
		PDS (MWH, 2014);	Previous records and recent characterization used in stability analyses
2.2	**Slope Stability**		
1.	Slope characteristics are properly evaluated.		
a.	Cross sections and profiles of natural and cut slopes whose instability would directly or indirectly affect the control of residual radioactive materials are presented in sufficient number and detail to enable the reviewer to select the cross sections for detailed stability evaluation.	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	
b.	Slope steepness is a minimum of five horizontal units (5h) to one vertical unit (1v) or less. The use of slopes steeper than 5h:1v is considered an alternative to the requirements in 10 CFR Part 40, Appendix A, Criterion 4(c). When slopes steeper than 5h:1v are proposed, a technical justification should be offered as to why a 5h:1v or flatter slope cannot be constructed. Appropriate compensating factors and conditions are incorporated in the slope design for assuring long-term stability. In addition, the application must contain an evaluation showing the economic benefit of slopes steeper than 5h:1v as well as a demonstration of equivalent protection.	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Repository slopes are 5h:1v or less steep
c.	Locations selected for slope stability analysis are determined considering the location of maximum slope angle, slope height, weak foundation, piezometric level(s), the extent of rock mass fracturing (for an excavated slope in rock), and the potential for local erosion.	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Critical locations selected, no rock slopes
2.	An appropriate design static analysis is presented.		
a.	The analysis includes calculations with appropriate assumptions and methods of analysis (NRC, 1977). The effect of the assumptions and limitations of the methods used is discussed and accounted for in the analysis. Acceptable methods for slope stability analysis include various limit equilibrium analysis or numerical modeling methods.	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Appropriate methods of analysis used (SLOPE/W program, Morgenstern-Price and Spencer methods)
b.	The uncertainties and variability in the shape of the slope, the boundaries and parameters of the several types of soils and rocks within and beneath the slope, the material properties of soil and rock within and beneath the slope, the forces acting on the slope, and the pore pressures acting within and beneath the slope are considered.	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Uncertainties considered
c.	Appropriate failure modes during and after construction and the failure surface corresponding to the lowest factor of safety are determined. The analysis takes into account the failure surfaces within the slopes, including through the foundation, if any.	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Appropriate failure modes included
d.	Adverse conditions such as high water levels from severe rain and the probable maximum flood are evaluated.	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Adverse conditions considered, arroyo flooding.
e.	The effects of toe erosion, incision at the base of the slope, and other deleterious effects of surface runoff are assessed.	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Erosion and runoff considered
f.	The resulting safety factors for slopes analyzed are comparable to the minimum acceptable values of safety factors for slope stability analysis given in NRC Regulatory Guide 3.11 (NRC, 1977).	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Calculated factors of safety higher than accepted minimum values
3.	Appropriate analyses considering the effect of seismic ground motions on slope stability are presented		
a.	Evaluation of overall seismic stability, using pseudostatic analysis or dynamic analysis, as appropriate (U.S. Army Corps of Engineers, 1977; NRC, 1977). Alternatively, a dynamic analysis following Newmark (1965) can be carried out to establish that the permanent deformation of the disposal cell from the design seismic event will not be detrimental to the disposal cell. The reviewer should verify that the yield acceleration or pseudostatic horizontal yield coefficient necessary to reduce the factor of safety against slippage of a potential sliding mass to 1.0 in a “Newmark-type” analysis has been adequately estimated (Seed and Bonaparte, 1992).	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Pseudo-static method used due to low seismicity
b.	An appropriate analytical method has been used. A number of different methods of analysis are available (e.g., slip circle method, method of slices, and wedge analysis) with several variants of each (Lambe and Whitman, 1979; U.S. Army Corps of Engineers, 1970b; NRC, 1977; Bromhead, 1992). Limit-equilibrium analysis methods do not provide information regarding the variation of strain within the slope and along the slip surface. Consequently, there is no assurance that the peak strength values used in the analysis can be mobilized simultaneously along the entire slip surface unless the material shows ductile behavior (Duncan, 1992). Residual strength values should be evaluated if mobilized shear strength at some points is less than the peak strength. The reviewer should ensure that appropriate conservatism has been incorporated in the analysis using the limit equilibrium methods. The limit equilibrium analysis methodologies may be replaced by other techniques, such as finite element or finite difference methods. If any important interaction effects cannot be included in an analysis, the reviewer must determine that such effects have	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Appropriate methods used (Morgenstern-Price and Spencer), with consideration of residual shear strengths

Section	Requirement	Location of Information	Comments
	been treated in an approximate but conservative fashion. The engineering judgment of the reviewer should be used in assessing the adequacy of the resulting safety factors (NRC, 1983a, b).		
c.	For dynamic loads, the dynamic analysis includes calculations with appropriate assumptions and methods (NRC, 1977; Seed, 1967; Lowe, 1967; Department of the Navy, 1982a,b,c; U.S. Army Corps of Engineers, 1970a,b, 1971, 1972; Bureau of Reclamation, 1968). The effect of the assumptions and limitations of the methods used is discussed and accounted for in the analysis.	--	N/A Dynamic loading not applicable, see 7d below
d.	For dynamic loads, a pseudostatic analysis is acceptable in lieu of dynamic analysis if the strength parameters used in the analysis are conservative, the materials are not subject to significant loss of strength and development of high pore pressures under dynamic loads, the design seismic coefficient is 0.20 or less, and the resulting minimum factor of safety suggests an adequate margin, as provided in NRC Regulatory Guide 3.11 (NRC, 1977).	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Pseudostatic analyses used, seismic coefficient is 0.20 g
e.	For pseudostatic analysis of slopes subjected to earthquake loads, an assumption is made that the earthquake imparts an additional horizontal force acting in the direction of the potential failure (U.S. Army Corps of Engineers, 1970b, 1977; Goodman, 1989). The critical failure surface obtained in the static analysis is used in this analysis with the added driving force. Minimum acceptable values for safety factors of slope stability analysis are given in Regulatory Guide 3.11 (NRC, 1977).	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Pseudostatic analyses used, with comparison of calculated factors of safety with appropriate minimum values
f.	The assessment of the dynamic stability considers an appropriate design level seismic event and/or strong ground motion acceleration, consistent with that identified in Chapter 1 of this review plan. Influence of local site conditions on the ground motions associated with the design level event is evaluated. The design seismic coefficient to be used in the pseudostatic analysis is either 67 percent of the peak ground acceleration at the foundation level of the tailings piles for the site or 0.1 g, whichever is greater.	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Pseudostatic analyses used with 67% of the PGA.
g.	If the design seismic coefficient is greater than 0.20 g, then the dynamic stability investigation (Newmark, 1965) should be augmented by other appropriate methods (i.e., finite element method), depending on specific site conditions.	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Seismic coefficient is 0.20 g
h.	In assessing the effects of seismic loads on slope stability, the effect of dynamic stresses of the design earthquake on soil strength parameters is accounted for. As in a static analysis, the parameters such as geometry, soil strength, and hydrodynamic and pore pressure forces are varied in the analysis to show that there is an adequate margin of safety.	Design Report, Appendix G, Attachment G.2 (Stantec, 2018)	Seismic effects on shear strengths considered
i.	Seismically induced displacement is calculated and documented. There is no universally accepted magnitude of seismically induced displacement for determining acceptable performance of the disposal cell (Seed and Bonaparte, 1992; Goodman and Seed, 1966). Surveys of five major geotechnical consulting firms by Seed and Bonaparte (1992) indicate that the acceptable displacement is from 15 to 30 cm [6 to 12 in.] for tailings piles. The reviewer should ensure that this criterion is also augmented by provisions for periodic maintenance of the slope(s).	Design Report, Appendix G, Attachment G.4 (Stantec, 2018)	Seismic settlement is calculated in the design.
j.	Where there is potential for liquefaction, changes in pore pressure from cyclic loading are considered in the analysis to assess the effect of pore pressure increase on the stress-strain characteristics of the soil and the post-earthquake stability of the slopes. Liquefaction potential is reviewed using Section 2.4 of this review plan. Evaluations of dynamic properties and shear strengths for the tailings, underlying foundation material, radon barrier cover, and base liner system are based on representative materials properties obtained through appropriate field and laboratory tests (NRC, 1978, 1979).	Design Report, Appendix G, Attachment G.6 (Stantec, 2018)	Liquefaction of saturated materials is evaluated
k.	The applicant has demonstrated that impoundments will not be located near a capable fault on which a maximum credible earthquake larger than that which the impoundment could reasonably be expected to withstand might occur.	Geohydrologic Report (Canonie, 1987); Approved Rec. Plan (Canonie, 1991) Design Report, Appendix G, Attachment G.1 (Stantec, 2018)	Regional faults described in design. Siting of tailings impoundment previously approved. No change to siting since previous reclamation plan.
4.	Provision is made to establish a vegetative cover, or other erosion prevention, to include the following considerations:		
a.	The vegetative cover and its primary functions are described in detail. This determination should be made with respect to any effect the vegetative cover may have on reducing slope erosion and should be coordinated with the reviewer of standard review plan Chapter 3. If strength enhancement from the vegetative cover is taken into account, the methodology should be appropriate (Wu, 1984).	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018)	Erosion protection in the design does not rely on vegetation
b.	In arid and semi-arid regions, where a vegetative cover is deemed not self-sustaining, a rock cover is employed on slopes of the mill tailings. If credit is taken for strength enhancement from rock cover, the reviewer should confirm that appropriate methodology has been presented. The design of a rock cover, where a self-sustaining vegetative cover is not practical, is based on standard engineering practice. Standard review plan Chapter 3 discusses this item in detail.	Design Report, Appendix G.7 (Dwyer, 2018)	Rock mulch cover surface designed.
5.	Any dams meet the requirements of the dam safety program if the application demonstrates the following :		
a.	The dam is correctly categorized as a low hazard potential or a high hazard potential structure using the definition of the U.S. Federal Emergency Management Agency.	-----	N/A; repository is not categorized as a dam
b.	If the dam is ranked as a high hazard potential, an acceptable emergency action plan consistent with the Federal Emergency Management Agency guide (U.S. Federal Emergency Management Agency, 1998) has been developed.	-----	N/A; repository is not categorized as a dam
6.	The use of steeper slopes as an alternative to the requirements in 10 CFR, Part 40, Appendix A, will be found acceptable if the following are met :		
a.	An equivalent level of stabilization and containment and protection of public health, safety, and the environment is achieved.	-----	N/A; maximum slopes are 5h:1v
b.	A site-specific need for the alternate slopes and an appropriate economic benefit are demonstrated.	-----	N/A: maximum slopes are 5h:1v

Section	Requirement	Location of Information	Comments
2.3	**Settlement**		
1.	Computation of immediate settlement follows the procedure recommended in NAVFAC Section 7.1 (Department of the Navy, 1982). If a different procedure is used, the basis for the procedure is adequately explained. The procedure recommended in NAVFAC Section 7.1 (Department of the Navy, 1982) for calculation of immediate settlement is adequate if applied incrementally to account for different stages of tailings emplacement. If this method is used, the reviewer should verify that the computation of incremental tailings loading and the width of the loaded area, as well as the determination of the undrained modulus and Poisson's ratio, have been computed and documented. Settlement of tailings arises from compression of soil layers within the disposal cell and in the underlying materials. Because compression of sands occurs rapidly, compression of sand layers in the disposal cell and foundations must be considered in the assessment of immediate settlement. However, the contribution of immediate settlement to consolidation settlement cannot be ignored. Clay layers and slime undergo instantaneous elastic compression controlled by their undrained stiffness as well as long-term inelastic compression controlled by the processes of consolidation and creep (NRC, 1983a).	Appendix Y, Consolidation and Groundwater Report (Dwyer, 2018); Design Report, Appendix G, Attachment G.3 (Stantec, 2018)	Immediate settlement of unsaturated and coarse-grained tailings would occur incrementally during repository construction.
2.	Each of the following is appropriately considered in calculating stress increments for assessment of consolidation settlement :		
a.	Decrease in overburden pressure from excavation	-----	N/A; minimal excavation planned
b.	Increase in overburden pressure from tailings emplacement	Appendix Y, Consolidation and Groundwater Report (Dwyer, 2018);	Increase in overburden pressure from placement of mine spoils. The amount of porewater released from the tailings due to this loading is evaluated.
		Design Report, Appendix G, Attachments G.3, G.4 (Stantec, 2018)	Increase in overburden pressure on tailings from placement of mine soils.
c.	Excess pore-pressure generated within the disposal cell	Appendix Y, Consolidation and Groundwater Report (Dwyer, 2018);	The amount of porewater released from the tailings due to this loading is evaluated.
		Design Report, Appendix G, Attachments G.3, G.4 (Stantec, 2018)	
d.	Changes in groundwater levels from dewatering of the tailings	-----	N/A; no tailings dewatering planned
e.	Any change in groundwater levels from the reclamation action	-----	N/A; no change in groundwater levels from action
3.	Material properties and thicknesses of compressible soil layers used in stress change and volume change calculations for assessment of consolidation settlement are representative of *in situ* conditions at the site.	Appendix Y, Consolidation and Groundwater Report (Dwyer, 2018);	
		Design Report, Appendix G, Attachments G.3, G.4 (Stantec, 2018)	The material properties and thicknesses of the tailings used in the consolidation analyses were based on CPT testing and drilling, sampling, and testing of tailings beneath the repository footprint.
4.	Material properties and thicknesses of embankment zones used in stress change and volume change calculations are consistent with as-built conditions of the disposal cell,	Appendix Y, Consolidation and Groundwater Report (Dwyer, 2018);	
		Design Report, Appendix G, Attachments G.3, G.4 (Stantec, 2018)	Properties are consistent, where applicable
5.	Values of pore pressure within and beneath the disposal cell used in settlement analyses are consistent with initial and post-construction hydrologic conditions at the site.	Appendix Y, Consolidation and Groundwater Report (Dwyer, 2018);	Pore pressures used are from drilling, sampling, and testing of tailings under current conditions.
		Design Report, Appendix G, Attachments G.3, G.4 (Stantec, 2018)	
6.	Methods used for settlement analyses are appropriate for the disposal cell and soil conditions at the site. Contributions to settlement by drainage of mill tailings and by consolidation/compression of slimes and sands are considered. Both instantaneous and time-dependent components of total and differential settlements are appropriately considered in the analyses (NRC, 1983a,b,c). The procedure recommended in NAVFAC DM 7.1 (Department of the Navy, 1982) for calculation of secondary compression is adequate.	Appendix Y, Consolidation and Groundwater Report (Dwyer, 2018);	Terzaghi consolidation theory was used for this analysis.
		Design Report, Appendix G, Attachments G.3, G.4 (Stantec, 2018)	Terzaghi consolidation theory was used for this analysis.
7.	The disposal cell is divided into appropriate zones, depending on the field conditions, for assessment of differential settlement, and appropriate settlement magnitudes are calculated and assigned to each zone.	Appendix Y, Consolidation and Groundwater Report (Dwyer, 2018);	
		Design Report, Appendix G, Attachments G.3, G.4	Settlement analyses reflect appropriate conditions in areas where fine-grained, near-saturated tailings are present.

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8.	Results of settlement analyses are properly documented and are related to assessment of overall behavior of the reclaimed pile.	Appendix Y, Consolidation and Groundwater Report (Dwyer, 2018);	Settlement analyses documented
		Design Report, Appendix G, Attachments G.3, G.4	Settlement analyses documented
9.	An adequate analysis of the potential for development of cracks in the radon/infiltration barrier as a result of differential settlements is provided (Lee and Shen, 1969).	Design Report, Appendix G, Attachment G.5	Cracking potential of the existing radon barrier analyzed
2.4	**Liquefaction Potential**		
1.	Applicable laboratory and/or field tests are properly conducted (NRC, 1978, 1979; U.S. Army Corps of Engineers, 1970, 1972).	PDS Report (MWH, 2014);	Appropriate laboratory tests and field (CPT) conducted.
2.	Data for all relevant parameters for assessing liquefaction potential are adequately collected and the variability has been quantified.	Design Report, Appendix G, Attachment G.6	Parameters for liquefaction appropriate.
3.	Methods used for interpretation of test data and assessment of liquefaction potential are consistent with current practice in the geotechnical engineering profession (Seed and Idriss, 1971, 1982; National Center for Earthquake Engineering Research, 1997). An assessment of the potential adverse effects that complete or partial liquefaction could have on the stability of the embankment may be based on cyclic triaxial test data obtained from undisturbed soil samples taken from the critical zones in the site area (Seed and Harder, 1990; Shannon & Wilson, Inc. and Agbabian-Jacobsen Associates, 1972).	Design Report, Appendix G, Attachment G.6	
4.	If procedures based on laboratory tests combined with ground response analyses are used, laboratory test results are corrected to account for the difference between laboratory and field conditions (NRC, 1978; Naval Facility Engineering Command, 1983).	-----	N/A; not used due to low seismicity
5.	The time history of earthquake ground motions used in the analysis is consistent with the design seismic event.	-----	N/A; not used due to low seismicity
6.	If the potential for complete or partial liquefaction exists, the effects such liquefaction could have on the stability of slopes and settlement of tailings are adequately quantified.	Design Report, Appendix G, Attachment G.6	Evaluated in terms of full liquefaction.
7.	If a potential for global liquefaction is identified, mitigation measures consistent with current engineering practice or redesign of tailings ponds/embankments are proposed and the proposed measures provide reasonable assurance that the liquefaction potential has been eliminated or mitigated.	-----	N/A; no potential for global liquefaction
8.	If minor liquefaction potential is identified and is evaluated to have only a localized effect that may not directly alter the stability of embankments, the effect of liquefaction is adequately accounted for in analyses of both differential and total settlement and is shown not to compromise the intended performance of the radon barrier. Additionally, the disposal cell is shown to be capable of withstanding the liquefaction potential associated with the expected maximum ground acceleration from earthquakes. The licensee may use post-earthquake stability methods (e.g., Ishihara and Yoshimine, 1990) based on residual strengths and deformation analysis to examine the effects of liquefaction potential. Furthermore, the effect of potential localized lateral displacement from liquefaction, if any, is adequately analyzed with respect to slope stability and disposal cell integrity.	Design Report, Appendix G, Attachment G.6.	
2.5	**Design of Disposal Cell Cover Engineering Design**		
1.	Detailed descriptions of the disposal cell material types [e.g., Unified Soil Classification System (Holtz and Kovacs, 1981)] and/or soil mixtures (e.g., bentonite additive) and the basis for their selection are presented. An analysis is included demonstrating that an adequate quantity of the specified borrow material has been identified at the borrow source. The information on borrow material includes boring and test pit logs and compaction test data. The soils that are considered suitable include the Unified Classification System Classes CL, CH, SC, and CL-ML, with desirable characteristics and limitations as listed in Table 3-1 of the "Construction Methods and Guidance for Sealing Penetrations in Soil Covers" (Bennett and Homz, 1991; Bennett and Kimbrell, 1991). The preferred material for the low-permeability layers is inorganic clay soil. This soil should be compacted to a low saturated hydraulic conductivity of at least 1 x 10 _ 7 cm/sec. For drainage layers, cobble types GW, GP, SP, and SW are recommended, with GW and GP being the preferred types (Bennett, 1991). Measures for resisting cracking, heaving, and settlement, and providing protection from burrowing animals, root penetration, and erosion over a long period of time are described.	PDS Report (MWH, 2014), Design Report Appendices H, J (Stantec, 2018)	Borrow characterization is described in the PDS report, further detail is included in Appendix H. Construction requirements outlined in the Specifications.
		Design Report, Appendix G, Attachment G.7 (Dwyer, 2018) Section 7 Drawings (Stantec, 2018)	The cover system planned for the repository is an evapotranspirative (ET) cover, which is more effective in the site environment than a compacted low-permeability material.
2.	A sufficiently detailed description of the applicable field and laboratory investigations and testing that were completed, and the material properties (e.g., permeability, moisture-density relationships, gradation, shrinkage and dispersive characteristics, resistance to freeze-thaw degradation, cracking potential, and chemical compatibility, including any amendment materials) are identified (U.S. Army Corps of Engineers, 1970, 1972; Fermulk and Haug, 1990; NRC, 1978, 1979; Lee and Shen, 1969; Spangler and Handy, 1982).	PDS Report (MWH, 2014)	Laboratory testing of cover materials followed appropriate references.
3.	Details are presented (including sketches) of the disposal cell cover termination at boundaries, with any considerations for safely accommodating subsurface water flows.	Design Report, Volume II (Stantec, 2018)	Drawings
4.	A schematic diagram displaying various disposal cell layers and thicknesses is provided. The particle size gradation of the disposal cell bedding layer and the rock layer are established to ensure stability against particle migration during the period of regulatory interest (NRC, 1982).	Design Report, Appendix G, Attachment G.7 (Dwyer, 2016a)	
		Design Report, Volume II (Stantec, 2018)	Drawings
5.	The effect of possible freeze-and-thaw cycles on soil strength and radon barrier effectiveness is adequately considered (e.g., Aitken and Berg, 1968). If the region experiences prolonged freezing, the disposal cell cover may be affected by the freeze-thaw cycle. During freezing, ice crystals and lenses can form in the soil, causing heaving. On the other hand, during melting and thawing, the soil may lose its bearing capacity because of development of supersaturated conditions (Spangler and Handy, 1982). Major factors affecting growth of ice in soil are the temperature below the freezing point, the capillary characteristics of the soil, and the presence of water. The reviewer should check whether the soil is susceptible to frost heave, considering that uniformly graded soils	-----	N/A The existing radon barrier, where it will be part of the repository, will be protected from frost by the new repository.

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	containing more than 10 percent of particles smaller than 0.02 mm and well-graded soils with more than 3 percent of particles smaller than 0.02 mm are susceptible (Holtz and Kovacs, 1981; Spangler and Handy, 1982). After many freeze-thaw cycles, the soil may become a loose collection of aggregates with significantly reduced overall strength.		
6.	A description is given (with sketches) of any penetrations (e.g., monitoring wells) through the disposal cell system, including details of penetration sealing and disposal cell cover integrity. Bennett and Kimbrell (1991) suggest methods for seal design that are acceptable.	-----	N/A; no penetrations through the repository system are planned.
7.	An adequate analysis is presented of the potential for development of cracks in the disposal cell cover as a result of differential settlement and shrinkage. Note that cracking issues associated with settlement are discussed in standard review plan Section 2.3.3.	Design Report, Appendix G, Attachment G.5 (Stantec, 2018)	Cracking analyses for the existing radon barrier was completed.
8.	An adequate description of the geomembranes and their major properties (e.g., physical, mechanical, and chemical) is provided if low permeability geomembranes are proposed as a part of the disposal cell cover. Methods for installation of the membranes in accordance with the manufacturer's recommendations are discussed. The shear strength of the interface between compacted clay and geomembranes used in the stability analyses under both static and dynamic loads is noted. The expected service life of the geomembrane is analyzed.	-----	N/A; no geomembranes are planned in the repository, to comply with USNRC long-term performance standards.
9.	Information on site characterization, slope stability, settlement, and liquefaction used in the disposal cell cover design appropriately reflects the staff evaluation, and therefore, constitutes inputs that would contribute to the demonstration of disposal cell design compliance with the regulations.	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018)	Information is presented to demonstrate compliance with NRC performance standards
		Design Report, Appendix G (Stantec, 2018)	Information is presented to demonstrate compliance with NRC performance standards
2.6	**Construction Considerations**		
1.	Engineering drawings are complete and clearly show the design features (e.g., embankments, riprap, and channels).	Design Report, Volume II (Stantec, 2018)	
2.	Sources and quantities of borrow material are identified, are shown to have been adequately characterized and quantified through field and laboratory tests and are demonstrated to be adequate for meeting the geotechnical design requirements for the disposal cell (NRC, 1978, 1979). The background levels of contamination in the borrow materials, if any, are properly established.	PDS (MWH, 2014); Design Report, Appendix H (Stantec, 2018)	Borrow characterization and volume estimates.
		Approved Rec. Plan (Canonie, 1991)	Background levels of contamination in borrow areas documented. No change since previous tailings reclamation.
3.	Methods, procedures, and requirements for excavating, hauling, stockpiling, and placing of contaminated and non-contaminated materials and other disposal cell materials are provided and are shown to be consistent with commonly accepted engineering practice for earthen works (Department of the Navy, 1982a,b; Denson, et al., 1987). Material placement and compaction procedures are adequate to achieve the desired moisture content (drying, if needed) placement density and permeability. Recommendations made in NUREG/CR5041 (Denson, et al., 1987) for gradation, placement, and compaction necessary to achieve design drainage rates and volumes, prevent internal erosion or piping, and allow for collection and removal of liquids are acceptable. Compaction specifications include restrictions on work related to adverse weather conditions (e.g., rainfall, freezing conditions). Specifications for controlling the mixture of fine tailings (slime) with sand tailings are consistent with commonly accepted engineering practice and testing programs for determination of engineering properties of this mixture.	Design Report, Appendix J (Stantec, 2018)	Specifications
4.	A plan for embankment construction is presented, that demonstrates embankments can be constructed in accordance with the design.		No embankment construction planned.
5.	Plans, specifications, and requirements for disposal cell compaction are supported by field and laboratory tests and analyses to assure stability and reliable performance.	Design Report, Appendices G, J, V (Stantec, 2018)	
6.	Testing and surveying programs to determine the extent of cleanup required are adequate. The contamination cleanup plan includes the method for determining the extent of the contaminated area and a confirmation program to demonstrate that the contaminated material has been removed. Details of the site cleanup (radiological aspects) are addressed in standard review plan Chapter 5.0.	Design Report, Appendix T	Restricted tailings area. Appendix T would apply to the mine haul road.
7.	A plan for settlement measurement is provided that is satisfactory for producing representative settlement data throughout the area of the disposal cell. Settlement measurement stations are of sufficient coverage and are strategically placed to yield adequate information for determination of total, differential, and residual settlements. Monitoring monuments are designed to be durable. The reviewer should also determine the reasonableness of the proposed monitoring frequency in accordance with NUREG/CR3356 (NRC, 1983). In the past, the staff has determined that the final radon barrier may be emplaced once 90 percent of expected settlement has occurred.	Design Report, Appendix G (Stantec, 2018)	Settlement monitoring program details are included in the design.
8.	All tailings and contaminated materials at the site can be placed within the planned configuration of the stabilized pile.	Design Report, Appendices C, G (Stantec, 2018)	
9.	Procedures, specifications, and requirements for riprap, rock mulch, and filter production and placement are provided and are shown to be consistent with commonly accepted engineering practice and the design specifications (NRC, 1977, 1982).	Design Report, Appendices G, H, I, J, V (Stantec, 2018)	
10.	The construction sequence is described and demonstrated to be adequate to achieve the intended configuration for the tailings, particularly when tailings are to be relocated to new areas of the reclaimed pile. The proposed time to completion has been shown to be reasonably achievable, and the construction schedule provides for completing the radon barrier as expeditiously as practical after ceasing operations in accordance with an approved reclamation plan.	-----	N/A; covering of tailings is not applicable for the repository, since the radon barrier over the existing tailings will not be penetrated or removed.
11.	The vegetation program or rock cover design is described and demonstrated to be adequate (Wu, 1984; NRC, 1982).	Design Report, Appendix G.7 (Dwyer, 2018)	Erosional stability is provided by the rock mulch on the cover surface. A vegetation plan has been developed but is not relied upon for erosional stability.

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12.	Appropriate quality control provisions are provided to ensure that the construction will be in accordance with the reclamation plan. The descriptions of the methods, procedures, and frequencies by which the construction materials and activities are to be tested and inspected are reasonable and appropriate records will be maintained (NRC, 1983).	Design Report, Appendices J, V, (Stantec, 2018)	
13.	Tailings are placed below grade, or the licensee has demonstrated that the above-grade disposal design provides reasonably equivalent isolation of the tailings from natural erosional forces. Tailings pile topographic features take into account wind protection and vegetation cover.	Design Report, Appendix G (Stantec, 2018);	The design demonstrates isolation of the mine waste from natural erosive forces and presents a stable final configuration on the existing tailings.
2.7	**Disposal Cell Hydraulic Conductivity**		
1.	A sufficient technical basis is provided for the design hydraulic conductivity (K) value for the disposal cell. The hydraulic conductivity is minimized by compacting fine-grained soil for a sufficient depth above the stabilized tailings. Natural borrow soils having insufficient silt and clay content to effectively reduce the hydraulic conductivity of the barrier can be amended with bentonite for improved effectiveness. (Note that construction issues are discussed separately using standard review plan Section 2.6.)	Design Report, Appendix G.7 (Dwyer, 2018)	Infiltration control is provided by the ET cover. Tailings are isolated from the repository by the existing radon barrier.
2.	A field testing program adequate to verify the constructability of the disposal cell with a design hydraulic conductivity $K<10^{-7}$ cm/sec is provided unless the reclamation plan demonstrates that field testing is not required (Benson and Daniel, 1990; NRC, 1979). To meet to the U.S. Environmental Protection Agency (EPA) groundwater standards, designers of disposal cells for mill tailings sites are proposing increasingly smaller design hydraulic conductivity (K) values. It is not unusual for laboratory permeability test values to yield results of 10^{-8} to 10^{-10} cm/sec. Such tests are performed on compacted soil samples considered by the design engineer to represent the soil to be used for the disposal cell. However, several technical papers (Rogowski, 1990; Panno et al., 1991; Benson and Daniel, 1990) have raised serious questions concerning the exclusive use of laboratory testing for demonstrating hydraulic conductivity values in those cases in which a radon barrier K-value less than 10^{-7} cm/sec is specified. On the basis of these technical papers, field testing is necessary to confirm the radon barrier hydraulic conductivity, since construction operations and soil material variability can create preferred pathways, joints, seams, holes, and flaws that effectively increase the value of this parameter. Test results should take into consideration the variability and uncertainty in site conditions and material properties. The test results should be properly documented and available for inspection.	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018) Design Report, Appendix G (Stantec, 2018)	Infiltration control is provided by the ET cover. PDS data supports the design assumptions.
3.	An appropriate quality control program is followed for the field testing to determine hydraulic conductivity (NRC, 1983). For all cases in which $K<10^{-7}$ cm/sec and the test fill program requirement has been defined, specifications and related documents (Remedial Action Inspection Plan, etc.) will require an adequate quality control program. An acceptable quality control program should contain mechanisms to ensure that as-built construction duplicates the test fill construction techniques on the cell barrier (NRC, 1983). The objective of the quality control program will be to provide assurance that uniform and high-quality construction of the cell barrier has been achieved. Records for implementation of the quality control program during the construction of the cell barrier should be properly maintained and available for inspection.	Design Report, Appendix G, V, J (Stantec, 2018)	The existing radon barrier will be improved in place. CQAP and specifications will include details and requirements for field testing.
4.	A reasonable construction schedule is proposed. The proposed construction schedule should promote completion of the radon barrier as expeditiously as practical after ceasing operations in accordance with a written, Commission-approved reclamation plan.	Design Report, Appendix K (Stantec, 2018)	
3.0	**SURFACE WATER HYDROLOGY AND EROSION PROTECTION**		
3.1	**Hydrologic Description of Site**		
1.	The description of structures, facilities, and erosion protection designs is sufficiently complete to allow independent evaluation of the impact of flooding and intense rainfall.	Design Report, Appendix I (Stantec, 2018)	General description of surface water hydrology (Design Report, Appendix I, Figure I.7-1 through I.7-10)
2.	Site topographic maps are of good quality and of sufficient scale to allow independent analysis of pre- and post-construction drainage patterns.	Design Report, Appendix I (Stantec, 2018)	
3.	The reclamation plan contains sufficient information for the staff to independently evaluate the hydraulic designs presented. In general, detailed information is needed for each method that is used to determine the hydraulic designs and erosion protection provided to meet NRC regulations. NUREG 1623 (NRC, 1998) discusses acceptable methods for designing erosion protection to provide reasonable assurance of effective long-term control and thus conform to NRC requirements. NUREG 1623 (NRC, 1998) also provides discussions and technical bases for use of specific criteria to meet the 1,000-year longevity requirement, without the use of active maintenance. Specific design methods are provided and form the primary basis for staff review of erosion protection designs.	Design Report, Appendix I (Stantec, 2018)	The newer version of NUREG 1623 (NRC, 2002) was used for guidance on analysis methods.
3.2	**Flooding Determinations**		
1.	The designs conform to the suggested criteria in Appendix D to NUREG 1623 (NRC, 1998). NUREG 1623 (NRC, 1998) discusses acceptable methods for designing erosion protection to provide reasonable assurance of effective long-term control and to meet NRC requirements. It also presents discussions and technical bases for use of specific criteria to meet the 1,000-year longevity requirement without the use of active maintenance. Acceptable design methods are presented and form the primary basis for staff review of erosion protection designs. These methods were derived from regulatory requirements, other regulatory guidance, staff experience, and various technical studies.	Design Report, Appendix I (Stantec, 2018)	The probable maximum precipitation has been used for evaluation of erosional stability for the repository and channels to meet the 1,000-year longevity requirement.
2.	Information pertinent to computation of the design flood is submitted in sufficient detail to enable the staff to perform an independent flood estimate, specifically : • Model input parameters are adequate. • Staff and the reclamation plan estimates of flood levels and peak discharges are in agreement. • Computational methods for design flood estimates are adequate.	Design Report, Appendix I (Stantec, 2018)	
1.	"Worst conditions" postulated in the analysis of upstream dam failures are (1) an approximate 25-year flood on a normal operating reservoir pool level coincident with the dam-site equivalent of the earthquake for which the remedial action project is designed, (2) a flood of about one-half the severity of a probable maximum flood on a normal reservoir pool level coincident with the dam-site	-----	N/A

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	equivalent of one-half of the earthquake for which the remedial action project is designed; and (3) a probable maximum flood (or design flood) on a normal reservoir pool. Surface Water Hydrology and Conditions 1 and 2 are applied when the dam is not designed with adequate seismic resistance; Condition 3 is applied when the dam is not designed to safely store or pass the design flood. If the proposed design is based on less than a probable maximum flood event, the licensee offers reasonable assurance of conforming to the stability requirement of 200 years. Dam failure analyses are either realistic or conservative, and include locations and sizes of upstream dams, instantaneous failure (complete removal) of the dam embankment and compute the peak outflow rate.		
3.3	**Water Surface Profiles, Channel Velocities, And Shear Stresses**		
	The proposed designs conform to the suggested criteria in Appendix D to NUREG 1623 (NRC, 1998). NUREG1623 (NRC, 1998) discusses acceptable methods for designing erosion protection to provide reasonable assurance of effective long-term control and to comply with NRC requirements. This document also contains discussions and technical bases for use of specific criteria to meet the 1,000-year longevity requirement without the use of active maintenance. Specific design methods are presented, and reasonable similarity to these methods forms the primary basis for staff acceptance of erosion protection designs. Specifically :	Design Report, Appendix I (Stantec, 2018)	
1.	Localized flood depths, velocities, and shear stresses used in models for rock size determination or soil cover slope analysis conform to the guidance presented in Appendix D to NUREG 1623 (NRC, 1998).	Design Report, Appendix I (Stantec, 2018)	
2.	For off-site flooding effects, computational models have been correctly and appropriately used and that the data from the model have been correctly interpreted.	Design Report, Appendix I (Stantec, 2018)	
3.	Acceptable models and input parameters have been used in all the various portions of the flood analyses and that the resulting flood forces have been adequately accommodated.	Design Report, Appendix I (Stantec, 2018)	
3.4	**Design of Erosion Protection**		
	The proposed designs conform to the suggested criteria in NUREG 1623 (NRC, 1998) discusses acceptable methods for designing erosion protection to provide reasonable assurance of effective long-term control and to comply with NRC requirements. This document also contains discussions and technical bases for use of specific criteria to meet the 1,000-year longevity requirement without the use of active maintenance. Specific design methods are presented, and reasonable similarity to these methods forms the primary basis for staff acceptance of erosion protection designs. NUREG 1623 (NRC, 1998) updates and expands the final staff technical position (NRC, 1990). If active maintenance is proposed as an alternative to the designs suggested above, such an approach will be found acceptable if the following criteria are met :	Design Report, Appendix G.7 (Dwyer, 2018) Design Report, Appendices I and W (Stantec, 2018)	Cover erosion protection design. No active maintenance is anticipated.
1.	The maintenance approach must achieve an equivalent level of stabilization and containment and protection of public health, safety, and the environment	Design Report, Appendix W	No active maintenance is anticipated.
2.	The licensee must demonstrate a site-specific need for the use of active maintenance and an economic benefit	Design Report, Appendix W	No active maintenance is anticipated.
3.	The licensee must provide funding for the maintenance by increasing the amount of the required surety. The staff should determine if the licensee's estimate of funding required for active maintenance is adequate. The licensee should also work with the long-term custodian to assess any additional funding requirements related to long-term surveillance and monitoring	30% Design Report, Appendix W	No active maintenance is anticipated.
3.5	**Design of Unprotected Soil Covers and Vegetative Soil Covers**		
	The designs conform to the suggested criteria in NUREG 1623 (NRC, 1998). NUREG 1623 (NRC, 1998) discusses acceptable methods for designing erosion protection to provide reasonable assurance of effective long-term control and thus meet NRC requirements. This document also provides discussions and technical bases for use of specific criteria to meet the 1,000-year longevity requirement without the use of active maintenance. Specific acceptance criteria for many of the review areas are presented and form the primary basis for staff review of erosion protection designs. These criteria were derived from regulatory requirements, other regulatory guidance, staff experience, and various technical references. If active maintenance is proposed as an alternative to the designs suggested above, such an approach will be found acceptable if the following criteria are met:	Design Report, Appendix G.7 (Dwyer, 2018)	The repository cover surface is designed to provide erosional stability based on a rock mulch and not on vegetation or unprotected soil
1.	The maintenance approach must achieve an equivalent level of stabilization and containment and protection of public health, safety, and the environment	Design Report, Appendix W	No active maintenance is anticipated.
2.	The licensee must demonstrate a site-specific need for the use of active maintenance and an economic benefit	Design Report, Appendix W	No active maintenance is anticipated.
3.	The licensee must provide funding for the maintenance by increasing the amount of the required surety. The licensee should also work with the long-term custodian to assess any additional funding requirements related to long-term surveillance and monitoring	Design Report, Appendix W	No active maintenance is anticipated.
4.0	**PROTECTING WATER RESOURCES**	Protection of water resources related to the repository's influence on the tailings area is demonstrated by the quantified seepage presented in the Consolidation and Groundwater Report (Dwyer, 2018). Based on the fill placement not influencing the existing groundwater, the checklist items in Section 4.0 of this checklist are tied to the groundwater corrective action program and not a change in conditions resulting from the construction of the repository. Therefore, Section 4.0 is not applicable to the request for source material license amendment.	
5.0	**RADIATION PROTECTION**		
5.1	**Cover Radon and Gamma Attenuation and Radioactivity Content**		
5.1.3.1	**Radon Attenuation**		
1.	The one-dimensional, steady-state gas diffusion theory for calculating radon flux and/or minimum cover thickness is used. An acceptable analytical method for determining the necessary cover thickness to reduce radon flux to acceptable limits or to determine the long-term radon flux from the proposed cover is the computer code RAECOM (NRC, 1984) and the comparable RADON code (NRC, 1989). The main difference between the two codes is that RADON does not have an optimization for cost-benefit. The staff will use the RADON code to verify the analysis. Other methods that estimate the average surface radon release from the covered tailings may be acceptable, if it can be shown that these methods produce reliable estimates of radon flux.	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018)	The RADON model was used in radon emanation analyses

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2.	With the RAECOM and RADON computer codes, the radon concentration above the top layer is either set to a conservative value of zero or a measured background value is used. The precision number (the level of computational error that is acceptable) is set at 0.001.	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018)	These settings were used in the analyses
3.	The estimates of the material parameters used in the radon flux calculations are reasonably conservative, considering the uncertainty of the values. For all site-specific parameters, supporting information describing the test method and its precision, accuracy, and applicability is provided. The basis for the parameter values and the methods in which the values are used in the analyses are adequately presented. Moisture-dependent parameter values are based on the estimated long-term moisture content of the materials at the disposal site (e.g., radon emanation coefficient and diffusion coefficient). The materials testing programs employ appropriate analytical methods and sufficient and representative samples were collected to adequately determine material property values for both cover soils and contaminated materials. In the absence of sufficient test data, conservative estimates are chosen and justified. The quality assurance program for parameter data is adequate and the data will be available for inspection. All parameter values are consistent with anticipated construction specifications and represent expected long-term conditions at the site.	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018)	Material density and long-term moisture content values in the analyses were consistent with long-term values from material placement specifications and long-term site conditions.
4.	The estimate of the tailings thickness is determined from estimates of total tailings production and the tailings areal extent, from boring logs, or changes in elevation from pre- to post-operation. Either the estimated thickness of a tailings source is used, or alternatively, the RADON code default value of 500 cm (16.4 feet) is used (NRC, 1989).	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018)	The profiles evaluated are based on the mine spoils or the actual thickness of tailings near the edges of the repository.
5.	Dry bulk densities of the cover soils and tailings material are determined from Standard Proctor Test data (ASTM D 698) or Modified Proctor Test data (ASTM D 1557). Radon barrier materials are usually compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D 698 or to a minimum of 90 percent of the maximum dry density as determined by ASTM D 1557. Field or placement densities to be achieved following the construction specifications are used in the calculations. If the pile is stabilized in place, the in-situ bulk density for the tailings is used in the analysis. Porosities are measured by mercury porosimetry or another reliable method, or the method for estimating the porosity of cover soils and tailings materials using the bulk density and specific gravity given in Regulatory Guide 3.64 (NRC, 1989) is used. If a portion of the modeled cover could be affected by freeze-thaw events, that portion is represented in the model with lower density and corresponding higher porosity values than the unaffected portion. The U.S. Army Corps of Engineers (1988) and the DOE (1988) have demonstrated that freeze-thaw cycles can increase the permeability of compacted clay by 40 to 300 times the original value. For fine-grained soils with some sand (50-percent fines), the DOE conservatively estimated that freeze-thaw cycles could lower the density by 14 percent (DOE, 1992). Also see the discussion in Section 2.5.3 of this standard review plan.	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018) PDS (MWH, 2014)	Material density and long-term moisture content values in the analyses were consistent with long-term values from material placement specifications and long-term site conditions. These long-term values incorporate the effects of freeze-thaw and wetting-drying cycles and are represented by densities equivalent to approximately 85 to 90 percent of ASTM D 698 and significantly drier than optimum moisture content. Data is based on the PDS and the site geotechnical data collected at the Mill Site.
6.	The long-term moisture content that approximates the lower moisture retention capacities of the materials or another justified value is used. Estimated values for the long-term moisture content can be compared with present in situ values to assure that the assumed long-term value does not exceed the present field value. Borrow samples can be taken at a depth of 120 to 500 cm (3.9 to 16.4 feet), but not close to the water table, and the borrow site conditions should be correlated to conditions at the disposal site.	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018)	Long-term moisture content values were used in the analyses, based on measured moisture content at depth, measured values at wilting point, and correlations with clay content in NRC (1984).
7.	Values for Ra-226 activity (pCi/g) are measured directly from tailings samples and other large volume sources of contaminated material by radon equilibrium gamma spectroscopy (allow at least 10 days for the sealed sample to equilibrate), wet chemistry alpha spectrometry, or an equivalent procedure. If the tailings are fairly uniform in Ra-226 content and the Ra-226 and uranium (U-238) in the ore were approximately in equilibrium, the Ra-226 activity can be estimated from the average ore grade processed at the site, as discussed in Regulatory Guide 3.64 (NRC, 1989). Generally, tailings should be sampled at 60- to 90-cm [2- to 3-ft] intervals to a depth of 366 cm [12 ft], including representative sampling of slime tailings. More than one layer of contaminated material is represented in the flux model if there are significant differences in Ra-226 content with depth. Since the disposal cell performance standard deals only with radon generated by the contaminated material, it is acceptable to neglect the Ra-226 activity in the cover soils for modeling flux, provided the cover soils are obtained from materials not associated with ore formations or other radium-enriched materials. If deep {below 61 cm [2 ft]} cover layers contain elevated Ra-226 or Th-230, that material layer is represented in the flux model.	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018)	Ra-226 activity concentrations used in the analyses are based on measured values for mine spoils and tailings.
8.	The emanation coefficient has been obtained by using methods provided in Nielson et al. (1982) and properly documented, or otherwise set to the reasonably conservative (for most soils) code default value of 0.35. A value of 0.20 may be estimated for tailings based on the literature, if supported by limited site-specific measurements.	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018)	The conservative default emanation value of 0.35 was used, in the absence of measured values.
9.	The radon diffusion coefficient, D, represents the long-term properties of the materials. The D value can be determined from direct measurements. The soil should be tested at the design compaction density, with a range of moisture content values that includes the lower moisture retention capacity of the soil so that a radon breakthrough curve can be obtained (DOE, 1989). The calculation of diffusion coefficient, based on the long-term moisture saturation, and porosity, as proposed in Regulatory Guide 3.64, Section C.1.1.5 (NRC, 1989) and the optional calculation in the RADON code, is acceptable.	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018)	The calculated value of radon diffusion coefficient from NRC (1989) was used, in the absence of measured values.
10.	The estimated soil cover thickness in the reclamation design is such that the calculated average long-term radon flux is reduced to a level that meets the requirement in 10 CFR Part 40, Appendix A, Criterion 6(1).	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018)	The flux requirement was met in the cover thickness calculations.
5.1.3.2	**Gamma Attenuation** Most radon barriers should be thick enough to reduce the gamma level of the disposal cell to background. To demonstrate compliance with this aspect of Criterion 6(1), the cover gamma attenuation is calculated based on the shielding value of the cover soil. Alternatively, the licensee commits to (1) measure the gamma level at 1 meter above the completed cover (or radon barrier) with at least one measurement per acre and (2) demonstrate that the average gamma level for the cell is comparable to the local background value.	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018)	The planned cover thickness of 4 feet is sufficient for attenuation of gamma radiation from the mine spoils. The existing radon barrier is sufficient for attenuation of gamma radiation form the underlying tailings.

Section	Requirement	Location of Information	Comments
5.1.3.3	**Cover Radioactivity Content** At least the upper 61 cm [2 ft] of the disposal cell cover will contain levels of radioactivity essentially the same as surrounding soils, as demonstrated by an appropriate procedure. The data will be in the reclamation completion report if not available for the reclamation plan.	Design Report, Appendix G, Attachment G.7 (Dwyer, 2018) PDS (MWH, 2014)	Borrow areas for cover materials have been evaluated for radioactivity. Analytical data in the PDS.
5.2	**Decommissioning Plan for Land and Structures**		
1.	The plan contains procedures to identify and place within the disposal cell, all soils on, and adjacent to, the processing site that are in excess of the standards in Part 40, Appendix A, Criterion 6(6), due to site activities. The plan is substantiated by the radiological characterization data and site history.		N/A – the material to be placed in the Repository is non- 11e.(2).
2.	Appropriate soil background values (different geological areas may need separate background values) for Ra226, and for U-nat, Th-230, and/or Th-232 as appropriate, have been proposed with supporting data.		N/A to LAR
3.	If elevated levels of uranium or thorium are expected to remain in the soil after the Ra-226 criteria have been met, the licensee has used the radium benchmark dose approach in Appendix H for developing decommissioning criteria.		N/A to LAR
4.	To ensure consistency of measurements, instrumentation and procedures used for soil background analyses and the Ra-gamma correlation are the same or very similar to those proposed to provide verification data. The instrumentation has the appropriate sensitivity and procedures are adequate to provide reliable data.		N/A to LAR
5.	A detailed quality assurance and quality control plan for all aspects of decommissioning is provided. In addition to the basis for accepting or rejecting data, a procedure for sampling additional grids when a verification Ra-226 sample fails to meet the standard is provided.		N/A to LAR
6.	Final verification (status survey) procedures are adequate to demonstrate compliance with the soil and structure cleanup standards. Survey instruments are specified and will be properly calibrated and tested. The proposed verification soil sampling density takes into consideration detection limits of sample analyses, the extent of expected contamination (unaffected area would have fewer measurements than affected areas), and limits to the gamma survey for the potentially contaminated area to be sampled. The gamma guideline value to be used for verification has been appropriately chosen. Also, there is a commitment to provide the verification soil Ra-226-gamma correlation and the number of grids that had additional removal because of excessive Ra226 values, to confirm that the gamma guideline value was adequate. The plan provides adequate data collection beyond the expected excavation boundary (buffer zone).	Design Report Appendix G (Stantec, 2018)	Verification survey procedures for the Repository are described in Appendix G.
7.	The plan indicates the location of records important to decommissioning, discusses protection of health and safety, and demonstrates that decommissioning will be completed as soon as practicable.		N/A to LAR
8.	The decommissioning cost estimate is itemized in sufficient detail and a basis (source) for each cost is provided. The total cost is reasonable for the area of the site and the expected decommissioning activities.	-----	A surety cost estimate will be prepared separately.
9.	The plan adequately describes the non-radiological hazards of decommissioning to human health and the environment as required by 10 CFR Part 40, Appendix A, Criterion 6(7). The licensee must maintain a financial surety, within the specific license, for the surface reclamation and decommissioning, with the surety sufficient to recover the anticipated cost and time frame for achieving compliance, before the land is transferred to the long-term custodian. Guidance on establishing financial surety is presented in NRC (1988, 1997b). Appendix C to this standard review plan provides an outline of the cost elements appropriate for establishing surety amounts for conventional uranium mills. Any staff assessment of surety amounts is reasonably consistent with the applicant's.	-----	Updated surety to be provided to NRC.
5.3	**Radiation Safety Controls and Monitoring**		
1.	The RP identifies the radiation safety concerns that are unique to reclamation and decommissioning activities. These concerns include characterization of radiation hazards associated with inhalation of resuspended tailings material or yellowcake, gamma exposure from working close to tailings, and inhalation of radon gas and its progeny (decay products) emanating from tailings material.	Design Report, Appendices L, Q, R (Stantec, 2018)	
2.	The RP describes any changes to an existing radiation safety or monitoring program that would be necessary to ensure worker or public safety during reclamation or decommissioning activities.	Design Report, Appendices L, Q (Stantec, 2018)	
3.	Regular wetting and/or phased stabilization efforts are used for control of windblown tailings material or yellowcake dust.	Design Report Appendices Q, J (Stantec, 2018)	Plans in place for control of dust from mine waste
4.	Any proposed changes to established monitoring programs will meet acceptable criteria of the applicable parts of Regulatory Guide 8.22, ".Bioassay at Uranium mills" (NRC, 1988) and Regulatory Guide 8.9, Revision 1, "Acceptable Concepts, Models, Equations, and Assumptions for a Bioassay Program" (NRC, 1993), or an acceptable justification is provided for selecting an alternative approach.	Design Report, Appendix L (Stantec, 2018)	
5.	The existing or proposed workplace airborne radiological monitoring program is consistent with applicable parts of Regulatory Guide 8.25, "Air Sampling in the Workplace" (NRC, 1992) and Regulatory Guide 8.30, "Health Physics Surveys in Uranium Mills" (NRC, 1983), or an acceptable justification is provided for selecting an alternate approach. The monitoring program is sufficient to provide adequate protection of workers from radon gas exposures to maintain compliance with the inhalation limits in Part 20. If sampling locations will be revised, the RP contains one or more maps of the site that indicate the location of samplers for airborne radiation and provides the criteria for determining the revised locations.	Design Report, Appendices L, Q (Stantec, 2018)	
6.	The existing or proposed contamination control program is consistent with the guidance on conducting surveys for contamination of skin and of personal clothing presented in Regulatory Guide 8.30 (NRC, 1983).	Design Report, Appendix L (Stantec, 2018)	

Section	Requirement	Location of Information	Comments
7.	The existing or proposed environmental radiological monitoring program is consistent with applicable parts of Regulatory Guide 4.14, "Radiological Effluent and Environmental Monitoring at Uranium Mills" (NRC, 1980), or an acceptable justification is provided for selecting an alternative approach. The licensee has adequately considered site-specific aspects of climate and topography in determining locations of offsite airborne monitoring stations and environmental sampling areas so that detection of maximum offsite concentrations of windblown tailings material and contamination from any other significant transport pathways applicable to the site is ensued.	Design Report, Appendix L (Stantec, 2018)	
8.	The proposed radiation protection program contains plans for documentation of exposures to all monitored workers and contractors and for availability of exposure records in a single location for inspection. The program provides for record-keeping that meets the requirements of 10 CFR 20.2102; at least annual review of the program content and implementation; and implementation of the ALARA requirements of 20.1101(d).	Design Report, Appendix L (Stantec, 2018)	
9.	The applicant commits to verifying the radon barrier effectiveness and to maintaining adequate records of this verification as required by 10 CFR Part 40, Appendix A, Criterion 6(4).	Design Report, Appendices G, Q (Stantec, 2018)	Radon testing method for the completed Repository cover described in Appendix G
Appendices			
A	Relationship of 10 CFR Part 40, Appendix A Requirements to Standard Review Plan Sections	LAR	CFR performance criteria table in Section 3.4 of the LAR document.
B	Guidance to the USNRC Staff for Reviewing Historical Aspects of Site Performance for License Renewals and Amendments		
C	Outline Recommended by The U.S. Nuclear Regulatory Commission Staff for Preparing Site-Specific Facility Reclamation and Stabilization Cost Estimates for Review	LAR	Described in section 4.11, to be submitted separately.
D	Guidance to The U.S. Nuclear Regulatory Commission Staff for Reviewing Long-Term Surveillance Plans	Appendix W (Stantec, 2018)	Operation, Monitoring, and Maintenance Plan
E	Guidance to The U.S. Nuclear Regulatory Commission Staff on The License Termination Process for Licensees of Conventional Uranium Mills		N/A
F	Guidance to The U.S. Nuclear Regulatory Commission Staff on Effluent Disposal at Licensed Uranium Recovery Facilities: Conventional Mills		N/A
G	National Historic Preservation Act and Endangered Species Act Consultations	Supplemental Environmental Report (INTERA, 2018)	
H	Guidance to The U.S. Nuclear Regulatory Commission Staff on The Radium Benchmark Dose Approach		N/A
I	Regulatory Issue Summary 2000–23	Section 1.3 of the LAR	
J	Technical Evaluation of Appendix A Criteria	Design Report Appendices G, I, J, U (Stantec, 2018)	Refer also to the table in Section 3.4 of the LAR document.
K	Content and Format for Alternate Concentration Limit Applications		N/A

The Repository would affect the completed construction for the North and Central Cells and would result in an updated and supplementary construction completion report to complement any reports already submitted to NRC for the North and Central Cell reclamation. The Repository design and analyses assumes that the existing evaporation ponds will be reclaimed, and the previously planned and approved cover will be completed in that area. Reclamation of the evaporation ponds is not included in the LAR.

The following items within the reclaimed tailings area have been analyzed due to changes in the proposed configuration of the North and Central Cells and are being submitted as part of this LAR for the modified reclamation design:

- Site Seismic Hazard Analysis (SHA)
- Slope stability: North and Central Cells (portions), near the Repository
- Tailings liquefaction: North and Central Cells (portions), near the Repository
- Tailings consolidation: North and Central Cells (portions), near the Repository
- Cover erosion protection: North and Central Cells (portions), for the Repository cover
- Cover radon modeling: North and Central Cells (portions), for the Repository cover
- Stormwater channel improvements and erosion protections in the following areas where runoff is expected to be influenced by the addition of the Repository:
    - existing Branch Swale C
    - North Upstream Diversion
    - North Cell Drainage Channel
    - Dilco Hill
    - Runoff Control Ditch
    - Pipeline Arroyo (jetty)

The proposed design represents a modification of the reclaimed TDA. Existing features of the approved tailings reclamation are incorporated into the design. In addition, specific erosional stability issues of concern (identified by NRC) have been also addressed as part of the design.

2.0 SITE CHARACTERISTICS

2.1 SITE LOCATION AND LAYOUT

The Mill Site is a non-operating uranium mill site located approximately 17 miles northeast of Gallup in McKinley County, New Mexico (Figure 1). The Mill Site included an ore processing mill and a TDA which covers approximately 10 and 40 hectares (25 and 100 acres), respectively. The Mill Site encompasses Section 2, Township 16 North, Range 16 West and extends to Section 36, Township 17 North, Range 16 West, which is also owned by UNC. The Mill Site is located near the end of NM Highway 566 (NM 566) which extends from I-40 near Church Rock, NM to the NECR Mine Site where the highway ends. NM 566 is located within Pipeline Canyon. The former Mill Site is located between the Mine Site and NM 566, on the west side of the highway. Elevations at the Mine and Mill Sites range from about 6,900 to 7,200 feet (ft) above mean sea level (amsl). The Pipeline Arroyo is a drainage feature crossing the Mill Site, located on the east side of NM 566, between the highway and the TDA. The elevation of the Mill Site is approximately 6,970 ft amsl.

The existing site features and infrastructure are shown in Figure 2. Two soil borrow pits were previously excavated within the Central Tailings Cell as a source of borrow soil for construction of the tailings impoundment. Borrow Pit 1 was located near the center of the Central Tailings Cell and Borrow Pit 2 was located on the east side of the Central Cell. The existing evaporation ponds are located on the South Tailings Cell. The rock jetty, a buried riprap slope, is located northeast of the evaporation ponds, outside of the tailings area, and perpendicular to Pipeline Arroyo. Dilco Hill is a prominent natural rock outcrop located east of the North Tailings Cell and is one of the highest points of elevation on the site. The upstream diversion channels (North and South) are located along the east/southeast side of the tailings area and the North Upstream Diversion channel was constructed by cutting through the rock on the east side of Dilco Hill.

2.2 POPULATION DISTRIBUTION AND LAND USE

The city of Gallup, approximately 20 miles southwest of the site, is the largest population center within McKinley County. The county is sparsely populated with a 2010 census population density of 13.1 people per square mile, as compared to 16.9 people per square mile in the state. In 2010, 44 percent of the population was Native American (including Navajo, Zuni, and Hopi) and 32 percent was Hispanic or Latino. The Mill Site is in a sparsely populated area of McKinley County, where the nearest residence is approximately one mile northwest from the center of the TDA. The land surrounding the Mill Site includes Navajo Nation Reservation (north), Tribal Trust Land (east, west), and Indian Allotment Land (south).

An annual land use survey (grazing, residence, wells, etc.) is conducted for a 2-mile area surrounding the Mill Site. The survey report is submitted to the NRC, as required in License Condition 31 of the source material license. There have been minimal changes to area ownership and use over the past twenty years. The number of homesites within two miles of the Mill Site has decreased by two homesites over the past 10 years. In 2007, the 36 nearby homesites included one UNC employee homesite in Section 2, 11 homesites in Section 10, and 24 homesites in the northern portion of the two-mile radius (Navajo Reservation) (UNC, 2008). In 2017, the 34 nearby homesites included one unoccupied homesite in Section 2, eleven homesites in Section 10, and 22 homesites on the Navajo Reservation (UNC, 2018). Other changes to land use since 2010 include: installation, sampling, and monitoring of new wells, and installation and monitoring of the bioventing system in the northern portion and adjacent to Section 35 (NECR Mine Site) on Navajo Reservation land.

The project is expected to require an average of 40 full-time workers over an estimated duration of three years. These will primarily include heavy-equipment operators and laborers.

2.3 HISTORIC, SCENIC, AND CULTURAL RESOURCES

The Repository design and proposed construction consider historic, scenic and cultural resources that exist near the TDA and within the limits of proposed project disturbance area. The Supplemental Environmental Report (SER) (INTERA, 2018) includes specific details on these resources. Cultural resources were not described previously in the approved Reclamation Plan (Canonie, 1991).

2.4 CLIMATE

The site is situated in an arid to semi-arid climate with sunshine no more than 50 percent of the time throughout the year and is subject to erosion due to high-intensity precipitation events and significant wind. The climate for the region is summarized using measurements taken at the Gallup Municipal Airport and reported by the Western Regional Climate Center (WRCC, 2016).

2.4.1 Temperature

Average annual temperatures at the site range from 31 degrees Fahrenheit (31°F) (-1° Celsius, C) to 67°F (20°C). Temperatures during the year average approximately 49°F (10°C). Maximum temperatures at the site range from 42°F in December/January to 85°F in July. Minimum temperatures at the site range from 17°F in January to 56°F in July. Extreme daily temperatures can reach as high as 95°F and as low as -20°F.

2.4.2 Wind and Evaporation

Climatological data from the site indicate that winds are generally moderate, originating from the west and southwest. Wind frequency and velocity are usually highest during the spring. Average annual daily wind speed is 7.0 mph with an average maximum 2-minute wind speed of 22 mph and an average peak gust of 27.3 mph. Net pan evaporation rates obtained at the Gallup Ranger Station between 1966 and 1975 show an average annual evaporation of approximately 62.5 inches per year, which exceeds net precipitation by approximately six times (WRCC, 2016). The potential evapotranspiration (PET) is typically highest during July and August.

2.4.3 Precipitation

Historical weather data for the Gallup, NM area and surrounding weather stations were evaluated from 1897 to 2016. Average precipitation at the Gallup airport is about 11 inches per year (28 cm) per the Western Regional Climate Center (WRCC, 2016). The most extreme climate year occurred in 1906 when the area received 23.8 inches (60.5 cm) of precipitation. The site receives an average of 30.6 inches of snowfall annually.

An updated probable maximum precipitation (PMP) event was calculated for the analyses completed for this LAR. The PMP 1-hour precipitation value of 6.5 inches was used in the cover and stormwater design analyses. The PMP storm depths and distributions were developed using the Arizona Department of Water Resources (ADWR) PMP Evaluation Tool (ADWR, 2013). The tool provides PMP depths and distributions for three different storm types: (1) local convective storms, (2) remnant tropical storms, and (3) general frontal storms. The tool provides PMP depths for the local convective storm PMP at 1-hour intervals for storm durations between 1 hour and 6 hours. The design discharge is discussed in more detail in Section 4.1.1 and calculations are in Appendix I.1.

2.5 GEOLOGY AND SEISMOLOGY

Stantec updated geologic and geomorphic information from previous studies for the site (Canonie, 1987, 1991) and prepared a summary of available geotechnical data for the site (MWH, 2014). Additional geotechnical studies were completed to collect additional susbsurface data near the TDA. In addition, studies of regional seismology for the probable seismic hazard analysis (PSHA) and geomorphology of the arroyo were completed as part of the design and are appended to this LAR.

2.5.1 Stratigraphic Features

The Mill Site is in the San Juan Basin region of the Colorado Plateau, where Cretaceous and Mesozoic sediments crop out in the area. The stratigraphy of the San Juan Basin region is characterized by Mesozoic age sediments deposited in and adjacent to the western margin of a transgressing and regressing Late Cretaceous sea. The sedimentary rocks in the area consist of primarily sandstones, shale, siltstone/mudstone, and coal. The sandstones were deposited in fluvial, eolian, and nearshore marine environments.

The bedrock units at the Mill Site in descending order are the Dilco Coal Member, the Upper Gallup Sandstone, and the Upper D-Cross Tongue of Mancos Shale. The Dilco Coal Member is a sequence of alternating sandy siltstones, sandstones, coals, and carbonaceous shales. The thickness of this unit varies from 0 to 300 feet within the Site area. The Upper Gallup Sandstone is divided into three units in the Site area: Zone 3, an upper sandstone; Zone 2, a shale and coal parting member; and Zone 1, a lower sandstone unit. Figure 3 shows the regional geology of the Site.

Erosion during the Pleistocene epoch carved valleys into the Cretaceous bedrock, which have since filled with alluvium. Bedrock subcrops are in contact with this alluvium in the Pipeline Arroyo and across the TDA. The thickness of the alluvium on site varies up to 150 feet thick and underlies most of the tailings at the Site. The alluvium consists primarily of silty sands and sandy silts with gravel.

2.5.2 Structural and Tectonic Features

The Colorado Plateau is characterized by large regions of folding with broad uplifts and intervening basins. Structural deformation of the plateau consisted of development of a series of uplifts and downwarps and later large-scale, northwest trending folds with associated small-scale folding and faulting. The Site is located at the juncture of several of these major fold structures: the San Juan Basin, the Zuni Uplift, and the Defiance Uplift. The Site lies on the Chaco slope, which forms the northeast edge of the Zuni Uplift and the southwest rim of the San Juan Basin (Canonie, 1987). The Site is in the vicinity of three local structural features (see Figure 3) that are related to the regional structures listed above. These local features are:

- Pipeline Canyon Lineament
- Fort Wingate Lineament
- Pinedale Monocline

Monoclinal folds are the most distinctive smaller-scale structures that have been identified at the Site. These folds occur throughout the plateau and commonly form the boundaries of the larger uplifts and basins. Large-scale faulting is uncommon in the southeastern portion of the plateau and, therefore, has little control over groundwater flow in the region. Small-scale joints and fractures, especially those related to the monoclines, are prevalent and affect groundwater flow. An orthogonal fracture pattern striking north-northeast and west-northwest is also common in the plateau and is evident in the sandstone outcrops throughout the Site (Canonie, 1987).

2.5.3 Geomorphic Features

The landscape of the Pipeline Arroyo Watershed is comprised of upland mesas and buttes that flow steeply over rock outcrops into alluvial valley bottoms that form ephemeral channels. Mesas and hillslopes are vegetated with a mixture of grasses,

shrubs, and trees. Alluvial drainages contain limited vegetation. The mesas and buttes are comprised of sandy clay loam to loamy soils with medium to high runoff potential. Transitions from mesas and buttes to valley floors are dominated by rock outcrops and soil cover consisting of sandy clays. These regions have significant slopes and have high runoff potential. The alluvium valley floor that forms the ephemeral channels “consists of fine sand interfingered with layers of silty clay” that “overlies sedimentary bedrock” (USGS, 1994).

The Pipeline Arroyo is an existing ephemeral arroyo that runs along the northwest side of the TDA. Stability of the Pipeline Arroyo is important for long-term viability of the Repository and the TDA, as lateral southeastward migration of the arroyo could create embankment erosion, with potential for significant erosion to threaten the integrity of the TDA. An area of concern along the Pipeline Arroyo is the rock outcrop (nick point) and buried rock “jetty” constructed during the TDA reclamation (Canonie, 1991). The jetty was constructed as a buried riprap slope, located perpendicular to, and between, the TDA and the Pipeline Arroyo. Progressive scour and undermining of the jetty has led to ongoing concerns that loss of the jetty will result in uncontrolled lateral scour in the arroyo toward the tailings embankment. Other than the erosional pathway, the Pipeline Arroyo downstream of the jetty appears stable (based on aerial imagery), with some historical deepening and widening, but with no lateral movement. Although historical images show no lateral movement in the last decade, further downcutting in the pathway and undercutting of the banks could cause episodic bank failures and pathway shifting toward the TDA. That the pathway would shift far enough to the east to threaten the TDA embankment is unlikely; however, the available bedrock information indicates that migration will not be limited by a bedrock control. Besides the erosional pathway, the engineered arroyo channel between the jetty and the southern end of the TDA has been stable with no meandering since at least 1981; although, similar to the erosion pathway, lateral migration will not be limited by bedrock. A large meander bend in the Pipeline Arroyo exists downstream of the TDA. Details and supporting data are presented in Appendix I.

Aerial images from as early as 1954 show the historical development of the Pipeline Arroyo in the limits of the TDA:

- In 1954 (Appendix I, Figure I.7-1), the Pipeline Arroyo does not appear to be influenced by mining or other anthropogenic activities. Two branches of the arroyo are evident. The main branch of the arroyo originates to the east of the current alignment of the arroyo upstream of the rock outcrop and converges to the current alignment near the rock outcrop. Downstream of the rock outcrop, the arroyo runs in a nearly straight alignment that is offset to the southeast of the post-reclamation (1991) alignment and aligns with the current head-cut erosional feature downstream of the rock jetty. The arroyo downstream of the rock outcrop shows significant down-cutting; whereas, little or no down-cutting is evident above the rock outcrop, indicating that the rock outcrop (referred in earlier documents as the nick point) has historically provided upstream grade control. The tributary branch of the Pipeline Arroyo runs under the present-day TDA and combines with the main branch downstream of the TDA.
- By 1962 (Appendix I, Figure I.7-2), a water control dam was constructed across both branches of the Pipeline Arroyo near the rock outcrop. Alluvial deposits are apparent upstream of the dam. Downstream of the dam, the alignment and headcutting appears unchanged from 1954.
- By 1978 (Appendix I, Figure I.7-3), the water control dam was removed. Upstream of the rock outcrop, the alignment of the Pipeline Arroyo had shifted to the west compared to its 1954 alignment, and the North Cell of the TDA had been constructed over the 1954 alignment. Downstream of the rock outcrop, the arroyo had cut back to its 1954 alignment.
- By 1981 (Appendix I, Figure I.7-4), the Pipeline Arroyo downstream of the rock outcrop had been engineered to a channel approximately 100 feet to 150 feet to the northwest, and the topography in the area of the original (1954) alignment had been graded to slope away from the TDA.
- In 1991 (Appendix I, Figure I.7-5), the Pipeline Arroyo continued to follow the engineered alignment but with evidence of some downcutting and widening in the arroyo channel.

- By 1997 (Appendix I, Figure I.7-6), the rock jetty had been constructed and keyed into the rock outcrop. The jetty appears effective in controlling the upstream grade. Downstream of the outcrop and rock jetty there was evidence of significant downcutting and widening of the engineered arroyo channel, but little lateral movement of the engineered arroyo channel. A large headcut is also apparent near the southern end of the South Cell of the TDA that extends from the engineered arroyo channel toward the original (1954) alignment of the Pipeline Arroyo. A less-developed headcut is also apparent approximately 475 feet downstream of the rock outcrop and jetty.
- By 2005 (Appendix I, Figure I.7-7), a drainage cut is apparent that runs from the rock jetty to where the headcut downstream of the rock jetty is apparent in 1997. The drainage cut appears to be caused by stormwater avulsing the engineered arroyo channel at the rock jetty and flowing perpendicular to the rock jetty (southeast toward the TDA). The cut follows the approximate location of the original Pipeline Arroyo alignment for about 475 feet (at the location of the headcut apparent in 1997) where it makes a 90-degree bend and reconnects with the engineered arroyo channel.
- The 2009, 2011, and 2014 images (Appendix I, Figure I.7-8 to I.7-10) show continued development of the drainage cut apparent in 2005. No lateral migration of the channel is apparent upstream of the rock jetty or downstream of the drainage cut.

2.5.4 Seismicity and Ground Motion

A seismic hazard analysis (SHA) was conducted and is included as Appendix G.1. The historical earthquake record for the study area contains earthquakes from 1887 through 2016 and provides a general overview of the seismicity of the area. Seismicity is defined as events with a moment magnitude (M_w) greater than or equal to 2.5 ($M_w \geq 2.5$). The historical seismic events were compiled from two sources: the Petersen Catalog (Petersen et al., 2014), which was used to compile earthquakes in the project region from the beginning of the catalog (1887) to 2012; and the Advanced National Seismic System (ANSS) Comprehensive Catalog (ComCat) (USGS, 2017), which was used to compile events in the region after 2012. The earliest recorded event for the site area in the catalogs occurred in 1887, and the largest event in the catalogs is a M_w 6.5. The final combined catalog used in the PSHA included 413 earthquakes, where over 99 percent of the earthquakes have a relatively small magnitude of $M_w < 5.0$.

Ground Motion Prediction Equations (GMPEs) were applied to earthquakes to estimate design ground motion at the Mill Site. GMPEs are mathematical expressions that define how seismic waves propagate from the source to the site. GMPEs estimate ground motion as a function of magnitude, distance, and site conditions (e.g. soil, rock, or V_{s30}). The relationships are derived by fitting equations to data obtained by strong-motion instruments for a specific region.

Ground motions at the Mill Site were calculated by a PSHA for the average horizontal component of motion in terms of peak ground acceleration (PGA). The shear wave velocity, V_{s30}, was estimated in the top 100 feet of the original ground surface for both an alluvium site and soft rock (sandstone) site. Measurements of shear wave velocity via cone penetration testing (CPT) for the alluvium material, and these values were used in the PSHA. A V_{S30} value of 275 m/s was used for the alluvium. No site-specific shear wave velocity measurements are available for the sandstone and, therefore, published values for sandstone were used in the PSHA. A V_{S30} value of 566 m/s was used for the sandstone material. The PSHA also used an average of the alluvium and sandstone values in the analysis, which resulted in V_{S30} of 420 m/s. The three values were selected to represent the range of alluvium thickness within the foundation.

The V_{S30} value of 275 m/s for the alluvium resulted in the highest mean PGA of 0.30 for the 10,000-year return period. For this shear wave velocity, the mean magnitude was calculated to be M_w 5.8 at a mean distance of 26 km and the modal magnitude was calculated to be M_w 5.5 at a modal distance of 20 km (12.4 miles).

A deterministic seismic hazard analysis (DSHA) was also conducted for the site. Calculations were performed in an Excel spreadsheet using the same GMPEs that were used in the PSHA. The lowest V_{S30} value of 275 m/s was used in the DSHA calculations. The seismic sources evaluated in the DSHA included: the unsegmented Nacimiento fault, the interbasin faults on

the Llano de Albuquerque, the unsegmented Jemez-San Ysidro fault, and the unsegmented San Felipe fault. The DSHA results for the four considered faults are similar with PGA values for the 84th percentile ranging from 0.04 to 0.07 g.

The results of the site-wide SHA indicate the mean PGA for long-term conditions is estimated to range from 0.25 g to 0.30 g. The PGA values are associated with an average return period of 10,000 years, or a probability of exceedance of 2 percent to 10 percent for a design life of 200 to 1,000 years, respectively. Comparing the DSHA results to the PSHA results for a V_{S30} of 275 m/s, the Uniform Hazard Spectra (UHS) for the 10,000-year return period is well above the 84th percentile of the Nacimiento fault, which had the highest ground motions of the sources considered in the DSHA.

2.6 SURFACE WATER HYDROLOGY

The UNC Site is located west of the Continental Divide in the Rio Puerco Basin on the Colorado Plateau. Alluvial drainages show limited vegetation. The mesas and buttes are comprised of sandy clay loam to loamy soils with medium to high runoff potential. Transitions from mesas and buttes to valley floors are dominated by rock outcrops and limited soil cover consisting of sandy clays. These regions have significant slopes and have very high runoff potential.

The Mill Site is within the Pipeline Arroyo Watershed, which drains into the North Fork of the Rio Puerco drainage. The Pipeline Arroyo Basin above the UNC Site has a drainage area of approximately 18 square miles. The North Fork of the Rio Puerco drainage basin drains 280 square miles above the confluence of the Pipeline Arroyo. The Pipeline Arroyo Basin above the UNC Site boundary has a maximum relief of approximately 800 feet. Upland areas consist of relatively flat mesas with extremely steep sideslopes. Channel slopes of the arroyo vary considerably (0.0018 to 0.053 feet per foot) from its headwaters to its confluence with the North Fork Rio Puerco and are dependent on local bedrock controls, such as the nickpoint near the TDA. The “nickpoint” is one specific location within the arroyo where the outcropping sandstone extends toward and into the channel and constricts flow.

Downstream from the nickpoint, the channel slope steepens to as high as 0.053 ft/ft and the channel cross section is narrow and deeply incised into the previously deposited alluvial sediment. There are no surface water bodies, diversions, or control structures downstream of the Mill Site, within the Pipeline Arroyo. The channel upstream of the nickpoint is relatively wide and has a braided stream pattern due to the nickpoint, which provides a channel base-control section. The channel slope above the nickpoint is as low as 0.0018 ft/ft.

2.7 GROUNDWATER HYDROLOGY

Prior to mining and milling activities, the near-surface bedrock units and the alluvium in the valley were unsaturated. Due to relatively low precipitation and high evaporation, infiltration in the area is limited. Conditions at the Site changed in 1968 as a result of mine water discharge into the arroyo and tailings placement in the TDA. This discharge partially saturated the alluvium and Zones 3 and 1 of the Upper Gallup Sandstone near the TDA, creating an artificial hydrologic regime, referred to as the “artificial system” (Canonie, 1987). During operations, mine water discharge produced year-round surface water flow in the arroyo, which continued until February 1986. Prior to mine dewatering, no continuous groundwater system existed in the near-surface geologic formations in the site area. Evidence for this condition is provided by the construction logs for the NECR mine shaft located northwest of the Mill Site, as well as drilling logs for deep geotechnical boreholes located in the tailings area and completed for design of the TDA (Canonie, 1987). During milling operations, mine water discharges to the Arroyo began to recharge the alluvium and the Gallup Sandstone, creating a temporary, artificially saturated groundwater system. The recharge of seepage from tailings disposal became superimposed on the artificial system. The seepage created a mound on top of the artificial system which changed flow paths in the immediate vicinity of the TDA.

The most recent Groundwater Corrective Action, Annual Review Report for the Site (Hatch-Chester, 2018) states:
“Groundwater is present in the Southwest Alluvium as a result of the infiltration of water historically discharged into the

Pipeline Arroyo after having been pumped from the Quivira and NECR mines to facilitate their construction and operation. This water percolated into the alluvium and created temporary saturation in the vicinity of the tailings impoundments, which has diminished gradually over time. The detailed history of infiltration of mine-dewatering groundwater, into the alluvium and the subcrop of Zone 3 and Zone 1, has been incorporated into the Site groundwater flow model (Chester Engineers, 2012c, 2014b). UNC submitted the groundwater flow model report previously to NRC as part of License Amendment No. 52 in order to revise the groundwater protection standards in the source material license. This amendment request was approved by NRC. *This temporary saturation caused by discharged mine-dewatering groundwater is the recognized Southwest Alluvium background water (EPA, 1988a; 1988b; 1998; 2008). The level of saturation has been declining since groundwater pumping in connection with historical mine operations ceased in 1986. As a result, the flanks of the alluvial valley and the northern property boundary alluvium have completely de-saturated and, by 2000, a 31 percent saturation loss had been observed further to the south (Earth Tech, 2000d)." "Groundwater levels in the Southwest Alluvium continued to decline (with periodic fluctuations observed) in 2017, indicating that the artificially recharged zone of saturation continues to become naturally dewatered as the groundwater drains down the arroyo."*

The Groundwater Corrective Action, Annual Review Report includes the most recent Potentiometric Surface Map for the Southwest Alluvium, or the alluvium located along Pipeline Arroyo on the southwest side of the TDA. Figure 4 shows the contours for groundwater in the Southwest Alluvium with an approximate groundwater elevation of 6,865 feet, in the alluvium to the west of the Central Cell. These groundwater elevations are about 90 feet below the base of the proposed Repository on the north side and about 100 feet below the base of the Repository on the south side. The Annual Review Report also states: "*The distribution of the groundwater suggests the likelihood that the northern portion of the groundwater system, upgradient of the Nickpoint and including Well 509 D, may have become "detached" or ponded (i.e., lost hydraulic continuity) from the groundwater to the south."*

The potentiometric surface map of the groundwater in the Zone 3 sandstone (Hatch-Chester, 2018) is included as Figure 5. The 2017 annual report states: *"The saturated thickness measured in Zone 3 wells has declined by 79 percent on average since the third quarter of 1989….Groundwater levels in Zone 3 continued to decline in 2017, indicating that the zone of anthropogenic saturation continues to diminish as the groundwater drains down the dip of the bedrock layers. Extraction well pumping since 2005 has locally accelerated the rate of water level decline in northern Zone 3."* The Zone 3 sandstone is partially saturated on the northeast side of the TDA, with groundwater surface elevations ranging from about 6,885 feet near the north edge of the North Cell to 6,910 feet, to the east of Dilco Hill. While the Zone 3 Sandstone extends to near the ground surface beneath the North Cell, these groundwater surface elevations are 50 feet (or more) beneath the base of the proposed Repository. Additionally, shallow sandstone in the North Cell provides separation from (and elimination of) any stress-induced load effects to groundwater from the addition of mine waste in this area of the Repository because the vertical loading is carried by the incompressible rock. In this particular area, the fill placement does not result in a reduction in void ratio in the foundation.

The potential for impacts from the Repository construction on the underlying groundwater condition within the alluvial soil was evaluated in the Consolidation and Groundwater Report (Dwyer, 2018) attached as Appendix Y. The report evaluated the reduction in porosity of the underlying tailings and partially saturated alluvium as a result of the stress increase from fill placement of the mine materials on the TDA. The report concludes that although consolidation and reduction of porosity will occur, "*…there is no drainage impact into the underlying groundwater. That is, there is no increase in flux into the underlying groundwater from the tailings impoundment.*" The findings of the report also show*…that the new ET cover prevents flux while the existing cover potentially allows small amounts of percolation. Consequently, the addition of the mine waste and the new ET Cover should help reduce the potential future groundwater impacts from the impoundment."*

The calculations completed in the Consolidation and Groundwater Report (Dwyer, 2018) assume depths to saturated alluvium beneath the proposed Repository based on a water levels encountered while drilling in the TDA in 2013 (MWH, 2014). Groundwater elevations ranging from approximately 6,883 to 6,887 feet, in the alluvium beneath the former borrow

pit areas were encountered at that time. These borings were located in the deeper parts of the infilled fluvial channel located beneath the TDA. While this is likely water that remains within the alluvium from continued drainage, these groundwater elevations generally align with the Zone 1 Potentiometric Surface Map, for the east side of the TDA, included with the most recent Annual Review Report for the Site (Hatch-Chester, 2018), see Figure 6. The deeper alluvial deposits on the east side of the Central Cell are in contact, in some areas with the Zone 1 Sandstone. The Hatch Chester (2018) report states: "*Earlier groundwater flow in Zone 1 was approximately eastward, reflecting groundwater mounding and recharge from the borrow pits and the alluvium to the west. Since the dewatering of Borrow Pit No. 2 and termination of mine-dewatering groundwater discharge into Pipeline Arroyo, the former mounding has dissipated.*"

2.8 ENVIRONMENTAL MONITORING

Existing biological and ecological environments have been studied for baseline conditions and to evaluate necessary post-construction restoration. The Mill Site is located on arid mixed grass and shrubland communities located in deep alluvial soils. The proposed haul road to transport mine waste from the Mine Site to the Repository will cross piñon–juniper (PJ) woodland community and grass/shrubland communities, and will transverse the bottomland ecosystem (Cedar Creek, 2017). Two baseline vegetation and biological resource evaluations were conducted for the Mill Site Repository design (Cedar Creek, 2014) and the Environmental Data Gap Report (INTERA, 2017). Further description of the environmental impacts is included in the SER (INTERA, 2018).

2.9 RADIOLOGICAL MONITORING

2.9.1 Approved Reclamation Plan

As part of the original reclamation plan design, a radiological survey of the Church Rock facility was conducted to assess the radiological characteristics of the Site and to form a basis for reclamation planning, in accordance with NRC regulations (Appendix A of 10 CFR 40). The radium levels in background soils were presented in the Approved Reclamation Plan (Canonie, 1991). Appendix A of 10 CFR 40 stipulates that remediation is generally required when the Ra-226 concentration, averaged over areas of 100 square meters, exceeds the background level by 5 pCi/g in the first 15 cm of soil or 15 pCi/g in any 15 cm layer of soil below the first 15 cm.

Canonie determined background Ra-226 concentrations for site soils through sampling and laboratory analyses of soils taken from areas unaffected by uranium processing activities. Background gamma ray exposure rates were also determined. A systematic gamma ray exposure survey of the Site was conducted, and soil samples were collected for laboratory determination of Ra-226 concentrations where gamma ray exposure rates were observed to exceed 4 milliroentgens per hour (mR/hr) above background (NUREG-2954; NRC, 1983a). The laboratory-determined concentrations of Ra-226 were correlated with gamma ray exposure readings, and boreholes were drilled in areas of concern to estimate Ra-226 concentrations in layers of soil below the first 15 cm.

During the original tailings reclamation, three background areas were surveyed to estimate pre-operational radiological conditions at the mill and TDA as a basis for determining the radiological impact of site operations. The background areas were selected on the basis of distance and direction from the tailings and their locations opposing the predominant wind direction. The weighted overall mean of the background Ra-226 measurements was 0.78 pCi/g with a standard deviation of 0.53 pCi/g. A value of 1.0 pCi/g Ra-226 was selected as the background concentration for the Site. Therefore, the remediation criteria identified in Appendix A of 10 CFR 40 indicated that 6 pCi/g Ra-226 was the acceptable limit for surface Ra-226 activity and 16 pCi/g Ra-226 activity was the target acceptable limit for depths greater than 15 cm. Gamma ray exposure rate measurements were also conducted at each background area. The mean (corrected) background external gamma ray exposure rate was found to be 15 μR/hr with a standard deviation of 12 percent. The mean background gamma ray exposure

rate was used as the basis for identifying areas around the TDA and mill facilities that required more extensive evaluation. Areas showing gamma ray exposure rates of 4mR/hr or greater above background (i.e., greater than or equal to 19 mR/hr) were subjected to further assessment (shielded readings, borehole logging, and/or soil sampling/analyses). The 6 pCi/g Ra-226 was correlated to gamma ray exposure rate of 23 mR/hr. This value was used as the action level for determining Ra-226 at the surface in lieu of soils laboratory analyses.

Radon barrier placement on the tailings piles was completed in 1996 and radon flux measurements were performed September 25, 1996. The NRC reviewed the report and requested more information in September 1997. UNC provided the requested additional information in January 1998. NRC determined that the appropriate procedures were used, and that quality assurance data were within the acceptable limits. The average measured radon flux of 112 locations was 5.7 pCi/m^2/s, compared to the limit of 20 pCi/m^2/s, demonstrating that the barrier met the requirements of Criterion 6(2) for the areas tested. These measurements did not include the evaporation ponds, since reclamation of the ponds is not yet complete.

Table 2.9.1 compares key design criteria and analyses used by Canonie in the approved reclamation plan design with the current design criteria and analyses presented by Stantec for the 2018 LAR. The 1991 design did not include settlement analyses or cover cracking analyses for the tailings impoundment cover. In both cases the designs are intended to be effective for 1000 years.

Table 2.9-1 Comparison of Design Criteria and Analyses

Design Criteria	Canonie (1991) NRC-Approved Tailings Reclamation Plan	Stantec LAR 2018 Design
Probable Maximum Precipitation (in), for the one-hour storm	8.41 (HMR 49, 1984)	6.14 (ADWR, 2013) (Appendix I)
Peak Ground Acceleration (Horizontal Acceleration Coefficient, g)	No PGA defined in design (0.05), NRC review (1997) concluded a PGA of 0.196 (0.131) was appropriate for the Site.	0.30 (0.20) (Appendix G, Attachment G.1)

2.9.2 Ongoing Compliance Monitoring

NRC conducts announced inspections of the Church Rock facility, generally at least bi-annually, as part of the original reclamation plan. Based on the most recent report available, the following aspects of radiological monitoring of the facility were inspected (NRC, 2018):

- Management Organization and Controls – a review of staffing adequacy to ensure compliance with license and regulatory requirements. NRC found that routine site inspections, adverse site condition corrections, annual audits, and land use surveys were conducted in accordance with regulatory and license requirements.
- Radiation Protection and Operator Training – a review of oversight and control of licensed activities of the facility to verify compliance with 10 CFR Part 20 and license requirements. NRC found that the radiation protection program was implemented per license and regulatory requirements. Records indicated that no worker was assigned an occupational exposure since the last inspection, all workers received required training, and no contamination problems were identified.
- Radioactive Waste Management – a review of applicable records, interview of Church Rock facility representatives, and site tour to verify that an effective radioactive waste management program is in place and maintained. NRC found that radioactive wastes were managed in accordance with license requirements. An eradication program to remove deep-rooted plants from the vicinity of the tailings impoundment was voluntarily implemented. Stantec core drilled in the area of the jetty and the work was conducted in accordance with the work plan. The site staff continued to control access to the restricted area using fences, gates, and postings.

- Effluent Control and Environmental Protection – a review of the effluent and environmental protection programs put in place to ensure compliance with license and regulatory requirements. NRC found that the groundwater corrective action and monitoring programs were being implemented in accordance with license requirements. Semi-annual effluent and environmental monitoring reports are submitted to the NRC in compliance with License No. SUA-1475, Amendment 34, Conditions 12 and 30. The data specifies the concentration of each principle radionuclide released to unrestricted areas in water effluent.

The radiation monitoring instruments at the Site are routinely calibrated and the radiation monitoring program is in effect but has been in standby status awaiting the final evaporation pond closure reclamation activity. The required radiation monitoring program for Repository construction will be conducted under a Radiation Work Permit (RWP).

2.10 SITE RECLAMATION ACTIVITIES

The proposed reclamation plan was developed by Canonie and originally submitted in June 1987. NRC approved the plan in March 1991 with several technical changes from the original plan submittal. The Reclamation Plan had undergone review with subsequent revisions over nearly four years, and UNC implemented several components of the plan during this time as directed by the NRC. UNC implemented the following actions:

- Interim stabilization of tailings, control and cleanup of wind-blown tailings in accordance with License Conditions 16 and 33
- Decommissioning of the mill in accordance with License Conditions 26 and 33
- Collection of tailings seepage in accordance with License Condition 30
- Construction of an enhanced evaporation system in accordance with License Condition 32

UNC implemented the interim stabilization portion of the reclamation plan beginning in 1989 in accordance with NRC directives. The interim stabilization plan focused on the elimination of significant pathways for potential release, such as the seepage routes and the air route via wind-blown tailings and radon emanation. The infiltration from precipitation was minimized by regrading and recontouring the TDA and conduits of potential seepage migration were eliminated by plugging selected wells. An interim soil cover and revegetation was placed in some areas to eliminate wind-blown tailings, to reduce infiltration of precipitation, and to reduce radon flux from the tailings (Canonie, 1990). The interim cover made up one foot of the 1.5-foot radon attenuation soil cover specified in the design. Final reclamation activities were conducted after interim stabilization activities and included the following:

- Completed backfilling and grading of Borrow Pit No. 2
- Placed the final radon attenuation soil cover and the erosion protection cover over the tailings areas
- Constructed surface water control channels, diversion ditches, drainage swales, Pipeline Arroyo low-flow channel, and the buried rock jetty
- Revegetated disturbed areas and secure reclaimed areas

Final reclamation plans were designed to meet the objectives of Appendix A of 10 CFR 40, to the extent practicable by minimizing final slopes containing and controlling major flood events, minimizing radon emanation from the TDA and maximizing the long-term stability of the reclaimed site. The final tailings area radon attenuation soil cover was designed to provide reasonable assurance that control of radiological hazards would be effective for 1,000 years and that releases of Rn-222 to the atmosphere would not exceed an average release rate of 20 pCi/m^2/s, to the extent practicable, throughout the design life of the cover. The final radon attenuation soil cover was designed using the RADON computer model to have a total thickness of 1.5 feet (1.0 foot during interim stabilization and 0.5 foot during final reclamation). The design, submitted to the NRC in 1987, used soil characteristics determined from field explorations conducted during the reclamation investigation and

previous site investigations. The design was then modified using actual field construction data obtained during the interim cover placement over the North and Central cells. The cover design also included a 0.5-foot soil/rock matrix layer to protect against water and wind erosion.

A description of the previously-completed South Cell reclamation is not included in the LAR since the proposed modifications to the approved reclamation are limited to the North and Central Cells.

2.10.1 North and Central Cells

The final construction of the radon attenuation layers over the North and Central Cells consisted of 21 and 18 inches of compacted soil, respectively. The radon attenuation layer was designed to reduce the long-term radon flux from the underlying tailings to 20 pCi/m^2/s. The Reclamation Plan (Canonie, 1991) design specified 18 inches of compacted soil for the radon attenuation layer. However, radon exit flux measurements conducted after completion of interim stabilization indicated that some areas of the Central Cell had higher than anticipated radon flux values. To compensate for the higher values, the thickness of the radon attenuation layer was increased from 18 to 21 inches in the Central Cell. The erosion protection cover consists of 6 inches, or more, of a soil/rock matrix placed on top of the radon attenuation soil cover. The soil/rock matrix was constructed by placing a minimum of 3 inches of rock mulch over the completed radon attenuation soil cover, then placing a 4- to 6-inch layer of random soil material over the rock mulch. The soil was then forced into the rock mulch voids to form the soil/rock matrix. The soil/rock matrix was designed to promote surface water runoff and protect the underlying radon attenuation soil layer from wind and water erosion. The erosion protection cover was constructed over the entire areas of the Central and North Cells except for the drainage swales, which were riprapped in accordance with the Reclamation Plan. Figure 2 includes the existing site features.

Surface water control structures constructed on the Central Cell included Branch Swales A, B, C, D, and H. The branch swales are shallow, riprapped ditches located on top of the Central Cell designed to convey runoff from the reclaimed tailings area. The swales consist of shallow, trapezoidal ditches with 3H:1V sideslopes, and riprap armor along the bottom and sideslopes. Swales C, D, and H were excavated into coarse tailings sands, which were overexcavated and replaced with fill soil. The radon attenuation layer was placed through the bottom and sideslopes of these swales. Placement of a radon attenuation layer was not required in Swales A and B because they were constructed in native soils and bedrock south of the tailings area. The North Cell Drainage Channel is located along the east side of the North Cell and was designed to carry the runoff from North Cell Branch Swales E, F, and G and Central Cell Branch Swales A, B, C, and D. Swale H was designed to carry runoff to the South Cell Drainage Channel.

Surface water control structures constructed on the North Cell included Branch Swales E, F, and G and the North Cell Drainage Channel located along the east side of the North Cell. Branch Swales E, F, and G are shallow ditches located on top of the North Cell which slope gradually from west to east, at a grade of less than one percent. The swales consist of shallow, trapezoidal ditches with 3H:1V sideslopes, with riprap armor along the bottom and sideslopes of the swales. Swales E, F, and G were excavated into coarse tailings sands, which were overexcavated and replaced with fill soil. The radon attenuation layer was placed over the bottom and sideslopes of the swales. The North Cell Drainage Channel was designed to carry the runoff from Branch Swales E, F, and G plus the runoff from additional branch swales on the Central Cell. The North Cell Drainage Channel has a 10-foot-wide flat bottom with 3H:1V sideslopes and is lined with riprap. The lower reach of the channel is a curved channel that connects the upper and middle reaches to an existing natural channel located between the north edge of the tailings pile and an elevated roadway. The lower reach gradually widens from the 10-foot-wide bottom to a 30-foot-wide bottom.

The South Cell Drainage Channel was constructed from Swale I to the Pipeline Arroyo, a distance of approximately 1,600 feet. The channel is trapezoidal shaped with a 10-foot bottom and sideslopes of 3H:1V. The upper 200 feet of the drainage channel was lined with two layers of bedding material and 15 inches of riprap. The lower portion of the channel did not require riprap because it was completed in competent bedrock. The existing North Diversion Ditch was lined with bedding material and riprap

at two locations where the NRC expressed concern regarding the long-term potential for erosion. These locations were at two sharp curves immediately south of the Central Cell.

The Runoff Control Ditch is located immediately west of the TDA and was designed to collect surface water runoff from the west embankment of the tailings area. The Runoff Control Ditch was constructed to be 2 feet deep with a 10-foot-wide bottom and 3H:1V sideslopes, lined with 3 inches of bedding material and 3 inches of riprap. The southern portion of the Runoff Control ditch intersects the South Cell Drainage Channel and was constructed in bedrock.

The North Cell Drainage Channel was constructed along the perimeter of the North Cell and Borrow Pit No. 2. The channel was constructed with a 10-foot-wide bottom and 3H:1V sideslopes and designed to collect surface water runoff from the drainage swales. Bedding layers were placed on subgrade material and 15 inches of riprap was placed on top of the bedding material. The non-tailings area north of the North Cell was also covered with riprap in 1995.

2.10.2 Pipeline Arroyo and the Jetty

The Pipeline Arroyo was designed to ensure that the probable maximum flood (PMF) would pass without damage to the TDA. Previous studies of the flood stability of the Pipeline Arroyo indicated the probability of existing channel migration, large-scale scouring and avulsion of the existing channel during large-scale flood events. Based on hydrologic modeling by Canonie, the plan proposed in 1987 called for the Pipeline Arroyo channel and embankments to be excavated, reconfigured, and protected to stabilize the channel and provide protection of the TDA for a 1,000-year period, to the extent practicable. The design approach was then changed in response to NRC comments for the final design presented in Canonie (1991). The design involved artificially extending the “nickpoint” across the alluvial plain to the tailings embankment by constructing a buried rock jetty, allowing the arroyo channel to meander naturally in a low-flow channel upgradient from the nickpoint.

The constructed buried rock jetty consists of a stone-filled trench to the east edge of the arroyo approximately 150 feet north of the nickpoint outcrop. A trench was constructed from the subcrop in the west bank of the arroyo to the protective bench along the tailings embankment toe. The trench was 45 feet wide and varied in depth from 9 feet at the west bank of the arroyo to 28 feet at the protective bench. The final excavated depth was determined by the depth to the Gallup Sandstone subcrop. The trench was backfilled with crushed rock in 3-foot lifts, then loose soil in 12-inch lifts was backfilled and compacted after each lift of crushed rock was placed. The rock and soil lifts were constructed to the elevation of the existing channel.

A low-flow channel was constructed within the Pipeline Arroyo from the buried jetty upstream for approximately 6,000 feet. The channel is 30 feet wide by 2 to 4 feet deep and was designed to enhance the flow capabilities of the existing channel. South of the buried jetty, the existing drainage channels were cleaned out and obstructions were removed.

2.10.3 Former Borrow Pits

During construction of the tailings dams, two borrow pits were excavated in the Central Cell Area. During operations, the pits were used for water management and settling of fines. At the time reclamation began, Borrow Pit No. 1 (located to the west of Borrow Pit No. 2) was mostly filled with fine tailings. Reclamation of Borrow Pit No. 1 consisted of fill placement and grading activities that were consistent with the grading design plan. Due to higher than anticipated settlement after the interim stabilization measures were completed in 1991, additional cover was placed in the area in 1992 (Canonie, 1992).

Borrow Pit No. 2 was used for solution storage during milling operations and the remaining solution was neutralized following conclusion of operations. The pit was then used for water storage during groundwater remediation and pumpback, prior to commissioning of the evaporation ponds. The pit was pumped dry by mid-1991 and the reclamation Contractor placed a working surface of fill material in the bottom of the pit, prior to placement of demolition debris.

Reclamation of Borrow Pit No. 2 consisted of backfilling the pit with decommissioned mill materials (demolition debris) and soil, completing the radon attenuation soil cover and erosion protection cover, and constructing drainage swales over the reclaimed surface. In July 1991, backfilling operations began by placing and compacting a layer of soil at the base of Borrow

Pit No. 2. A layer of mill debris approximately 2.5 feet thick was placed over the base soil layer. The materials were covered with 2 feet of soil, which was compacted and worked down into the mill materials. Additional soil was added to bring the total thickness of compacted cover to 1.5 feet above the mill materials. Decommissioned mill material placed in 1991 included structural steel and siding from demolished buildings, process equipment, piping, tanks, wooden staves, and barrels containing lab ore samples.

In 1992, an average of 3.5 feet of decommissioned mill materials were placed in Borrow Pit No. 2. The materials consisted primarily of concrete from the process area foundations, sumps, and floors. The materials were covered with 2 feet of soil, which was compacted and worked into the mill debris. Additional soil was added until the total thickness of the new cover was about 4 feet. A small volume of decommissioned materials including piping, solution pumps, and other miscellaneous items, was placed in the southeast corner in 1993. The materials were covered with compacted soil. In 1994, a small amount of remaining miscellaneous mill equipment was placed in a mound in the center of Borrow Pit No. 2. This mound was covered with soil, and the entire borrow pit was backfilled to near the surface. This fill was referred to as the interim cover and was compacted to form the base for the radon attenuation cover. In 1995, several tanks were cut up and placed in Borrow Pit No. 2, and 5,000 cubic yards of contaminated soil that was removed during swale construction was placed in the borrow pit. These materials were buried beneath the clean soils that made up the interim cover. The borrow pit area was compacted for construction of the radon attenuation layer.

The radon attenuation layer over Borrow Pit No. 2 consists of 18 inches of compacted soil. The radon attenuation layer was designed to reduce the long-term radon flux from the underlying tailings to 20 pCi/m^2/s. The erosion protection cover consists of 6 inches or more of a soil/rock matrix placed on top of the radon attenuation soil cover. The soil/rock matrix was constructed by placing a minimum of 3 inches of rock mulch over the completed radon attenuation soil cover, then placing a 4- to 6-inch layer of random soil material over the rock mulch. The soil was then forced into the rock mulch voids to form the soil/rock matrix. The soil/rock matrix was designed to promote surface water runoff and protect the underlying radon attenuation soil layer from wind and water erosion. The erosion protection cover constructed over the area of Borrow Pit No. 2 was similar to the cover constructed of the remainder of the tailings area.

Surface water control structures associated with Borrow Pit No. 2 include Branch Swales A, B, and C and the North Cell Drainage Channel. The branch swales are shallow riprapped ditches located on top of Borrow Pit No. 2 and were designed to convey runoff from the reclaimed tailings area to the North Cell Drainage Channel. The swales were excavated, the subgrade was prepared, and the radon attenuation layer was constructed over the bottom and sideslopes of the swales. Sedimentary bedrock and native soils were encountered in sections of the swales. The radon attenuation cover was not placed in these areas. Bedding material and riprap material was placed in the branch swales in accordance with the design.

2.10.4 Evaporation Ponds

Two synthetically-lined evaporations ponds were constructed as part of the groundwater CAP from 1988 to 1989. The pond system was used for storage and evaporation of groundwater pumped from near the tailings disposal site. Each pond has a bottom surface area of 5 acres and a total storage capacity of approximately 10 million gallons and a total depth of 6 feet and interior side slopes of 4H:1V. Under normal maximum operating conditions, with necessary freeboard, the maximum volume of water that is stored in each pond is 6.4 million gallons. The evaporation pond system was constructed on the south cell of the TDA due to the available surface area and the large depression that existed in the south end of the cell. Coarse and fine-grained tailings and alluvial soils were excavated primarily from the north and west ends of each pond and were used as fill for each pond's southern embankment. A geotextile and a synthetic liner were installed in each of the ponds following final grading of the ponds. The geotextile was placed first, and the synthetic liner was placed over the geotextile.

To accommodate the potential for some areas of the pond to settle under the weight of water, all slack in the synthetic liner was pulled into the pond bottom and distributed throughout the pond as wrinkles. After excess slack was pulled in the ponds, the perimeter of the geotextile and synthetic liner was buried in an anchor trench. Fill material was backfilled within the trench

over the geotextile and liner and was compacted. Sandbags were placed on 50-foot centers throughout the pond bottom on top of the liner to prevent uplift of the liner.

Site operations currently add water, pumped from the Mill Site Well, and piped to the Evaporation Ponds to minimize dust from the pond liners and protect the integrity of the high-density polyethylene (HDPE). This process is completed regularly to maintain water levels in the ponds. Once the CAP is terminated, any remaining water will be allowed to evaporate, and the ponds will be decommissioned. Remaining 11e.(2) materials on site will be interred in the ponds and the NRC-approved tailings cover for the South Cell will be installed to complete reclamation of the South Cell. For the proposed Repository design, Stantec has assumed the Evaporation Ponds will be reclaimed and the South Cell cover will be completed as originally designed.

3.0 DESCRIPTION OF PROPOSED ACTION

3.1 USEPA (CERCLA) REMOVAL ACTION

The required design elements for the removal action described in the AOC and SOW (USEPA, 2015) include activities at both the Mine Site and Mill Site. The selected remedy is to remove mine waste material from the Mine Site and dispose of the mine waste at the nearby Mill Site on the TDA. The Selected Remedy is designed to be protective of human health and the environment, to comply with the USEPA Action Memorandum: Request for a Non-Time-Critical Removal Action at the Northeast Church Rock Mine Site, McKinley County, NM, Pinedale Chapter of the Navajo Nation (2011 Action Memo; USEPA, 2011) and Record of Decision, United Nuclear Corporation Site, McKinley County, New Mexico, USEPA ID NMD030443303; Operable Unit: OU 02, Surface Soil Operable Unit (ROD; USEPA, 2013), and to fulfill the requirements of the AOC SOW (USEPA, 2015).

3.2 PRE-DESIGN STUDIES

The Pre-Design Studies (PDS) completed for the Mill Site Tailings Impoundment are included as Appendix Z. The PDS included:

- Updated topographic survey
- Geotechnical evaluation of the tailings impoundment
- Borrow Material Investigation
- Stockpile Material Investigation
- Vegetation evaluation
- Bio-intrusion evaluation
- Known cultural resources summary
- Inventory of Mill Site debris
- Visual inspection and survey Branch Swales

Data collected during the PDS was used to supplement the existing site data. The primary objective was to collect the data necessary to complete the analyses for the Repository.

3.3 MINE WASTE REMOVAL

The Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) Removal Action (RA) for the Mine Site includes excavating approximately 1 million cubic yards of mine waste and disposing of it at a Repository at the Mill Site. Material with a Ra-226 concentration greater than 200 pCi/g has been designated a principal threat waste (PTW) by EPA and will not be disposed in the Repository. The performance standard for identification and action levels of mine waste is defined as soils and debris with Ra-226 concentrations above the field screening level (FSL) of 2.24 pCi/g. Removal and placement volumes will be tracked on a regular basis during the removal action.

Information obtained from sampling, field screening, and laboratory analyses of the Mine Site material during the 2014 Mine Site PDS was used to estimate volumes of soil and mine debris. The neat-line excavation surface was compared to the existing ground surface, resulting in an excavation volume of 725,240 cubic yards. The volume includes 32,200 cubic yards of PTW material that will be disposed offsite. A net volume of approximately 693,040 cubic yards (725,240 cubic yards less

32,200 cubic yards) will be excavated and moved to the TDA. The Mine Site removal activities are described in Appendix C. The Repository is designed for a capacity of up to 1.03 million cubic yards with the flexibility to adjust for lesser volumes.

3.3.1 Mine Waste Properties (Soil)

The mine waste was characterized based on results of laboratory tests performed on samples of mine waste collected during the Mine Site PDS (MWH, 2014). The sampled mine waste has gravimetric water contents ranging from 2 to 20 percent with an average of 8 percent. Dry densities of the samples tested ranged from 84 to 111 pounds per cubic foot (pcf), with an average dry density of 97 pcf. Maximum Proctor dry densities ranged from 102 to 125 pcf, with an average of 119 pcf, and optimum water contents ranged from 9 to 21 percent with an average of 12 percent. Mine waste is specified in the design to be placed in horizontal lifts and compacted to 90 percent relative compaction per standard Proctor. The mine waste is assumed to have similar index properties (fines content and Atterberg limits) as the cover soils to be used from the proposed borrow areas. This is based on similarities of geotechnical properties (specific gravity, max dry density and optimum water content) to the borrow soils. Details of mine waste properties with radiological (activity levels) for the waste by removal areas are included in Appendix G.

3.3.2 Mine Waste Properties (Debris)

Surface mine debris and structures will be removed and placed in the Repository. Approximately 25,600 cubic yards of mine debris, 12,800 cubic yards of which have been accounted for in the mine waste volume determination, will also be placed in the Repository. Stantec added a conservative estimate of an additional 10,000 cubic yards of mine debris to account for trees and other vegetation, totaling 22,800 cubic yards of Mine Site debris. The debris types include mixed/buried waste, concrete, wood, metal, rubber, plastic, and vegetation. To the extent possible, excavated mine waste materials and debris from the mine will be loaded directly into haul trucks, transported, and placed within the Repository. Debris that will be disposed of in the Repository will be size-reduced (crushed), spread in thin lifts, and filled over and around with mine waste material after placement to minimize void spaces and associated settlement. Vegetation debris will be removed from the Mine Site to facilitate the removal action. This consists of grasses, shrubs, and trees. The vegetation debris will be shredded and/or chipped at the Mine Site and then hauled for placement in the Repository. The shredded vegetation debris will be spread with soil and not permitted to be placed in nested layers. Mine Site debris removal is described in Appendix C.

3.4 REPOSITORY PERFORMANCE CRITERIA

The following table lists the performance standards applicable to the design of the Repository on the existing tailings areas and revisions to the tailings reclamation plan. The identifying numbers correspond to performance standards tables already included in the design documents, as provided to USEPA.

Table 3.4-1 NRC Performance Criteria

Identifying Number*	Location of Performance Standard Requirement	Topic	LAR Appendix or Document	Performance Standard	Comments
46	10 CFR 61	Closure	G, J, U	10 CFR 61 52(a)(9)	
47	10 CFR 61	Environmental Monitoring	SER (INTERA, 2018)	10 CFR 61 53(a)	
48	10 CFR 61	Environmental Monitoring	W	10 CFR 61 53(b)	
49	10 CFR 61	Environmental Monitoring	E, G, Q	10 CFR 61 53(c)	
50	10 CFR 61	Environmental Monitoring	W	10 CFR 61 53(d)	

Identifying Number*	Location of Performance Standard Requirement	Topic	LAR Appendix or Document	Performance Standard	Comments
51	10 CFR 40, Appendix A	Financial	Financial Surety Agreement	10 CFR 40, Appendix A Criterion 9	Surety to be updated separately
52	10 CFR 40, Appendix A	Financial	Financial Surety Agreement	10 CFR 40, Appendix A Criterion 10	Mill Operator Fee
53	10 CFR 40, Appendix A	Monitoring	W, SER (INTERA, 2018)	10 CFR 40, Appendix A Criterion 7	
54	10 CFR 40, Appendix A	Monitoring and Maintenance	W	10 CFR 40, Appendix A Criterion 12	
55	10 CFR 61	Performance Objectives	G, I	10 CFR 61 44	
56	10 CFR 40, Appendix A	Radiation Protection	G, W	10 CFR 40, Appendix A Criterion 6A	
57	10 CFR 40, Appendix A	Repository Design	G, W	10 CFR 40, Appendix A Criterion 6	
58	10 CFR 40, Appendix A	Repository Design	G, I	10 CFR 40, Appendix A Criterion 1	
59	10 CFR 61	Repository Design	B, E, G, I	10 CFR 61 41	
60	10 CFR 61	Repository Design	G, I	10 CFR 61 51(a)(1), 51(a)(4), 51(a)(5), and 51(a)(6)	
61	10 CFR 61	Repository Design		10 CFR 61 52(a)(6)	See Performance Standards associated with 10 CFR 20.1301 and 1302.
62	10 CFR 40, Appendix A	Repository Design	G, I, J, U	10 CFR 40, Appendix A Criterion 4	
63	10 CFR 40, Appendix A	Repository Design	G	10 CFR 40, Appendix A Criterion 3	
64	10 CFR 40, Appendix A	Repository Design	G	10 CFR 40, Appendix A Criterion 5	
65	10 CFR 61	Repository Design	-	10 CFR 61 51(a)(2) and 51(a)(3)	See Performance Standards associated with 10 CFR 61.41 through 44
66	10 CFR 61	Repository Design	-	10 CFR 61 52(a)(3)	See Performance Standards associated with (a)(4) through (a)(11) of this section.
67	10 CFR 40, Appendix A	Site and Byproduct Material Ownership	-	10 CFR 40, Appendix A Criterion 11	Title transfer to the DOE.
68	10 CFR 61	Waste Disposal	G, J	10 CFR 61 52(a)(10)	
69	10 CFR 61	Waste Disposal	C, G, J	10 CFR 61 52(a)(11)	
70	10 CFR 61	Waste Disposal	J	10 CFR 61 52(a)(4), 52(a)(5), 52(a)(7), and 52(a)(8)	

Identifying Number*	Location of Performance Standard Requirement	Topic	LAR Appendix or Document	Performance Standard	Comments
71	10 CFR 40, Appendix A	Waste Disposal	C	10 CFR 40, Appendix A Criterion 2	
75	40 CFR 192	Repository Design	G, I	40 CFR 192 32(b)	
76	40 CFR 192	Repository Design	G, I	40 CFR 192 02(b)	
77	40 CFR 192	Health and Safety	L	40 CFR 192 41	
78	40 CFR 192	Health and Safety	L	40 CFR 192 43	
79	40 CFR 192	Repository Design	A, G, I	40 CFR 192 20(d)	
80	40 CFR 192	Repository Design	A, G	40 CFR 192 02(a) and 02(b)	
82	10 CFR 20	Health and Safety	L	10 CFR 20 (All)	
99	10 CFR 61	Waste Disposal	J	10 CFR 61 56(b) through 56(b)(3)	
100	10 CFR 61	Health and Safety	L	10 CFR 61 43	See Performance Standards associated with 10 CFR 20 and 10 CFR 61.41
101	10 CFR 61	Site Controls and Security	B, G	10 CFR 61 42	
102	10 CFR 61	Licensing	-	10 CFR 61 30(a)(5)	The DOE will assume responsibility for Mill Site ICs following license transfer.
103	10 CFR 61	Institutional Controls	Financial Surety Agreement	10 CFR 61 59	Groundwater surveillance is addressed under the UNC Groundwater Corrective Action Plan.
104	10 CFR 61	Licensing	B, E, G, I	10 CFR 61 23(b)	See Performance Standards associated with 10 CFR 61.41
105	10 CFR 61	Licensing	B, G	10 CFR 61 23(c)	See Performance Standards associated with 10 CFR 61.42
106	10 CFR 61	Licensing	-	10 CFR 61 23(d)	See Performance Standards associated with 10 CFR 20
107	10 CFR 61	Licensing	-	10 CFR 61 23(e)	Current ICs at the Mill Site are adequate to protect public health and safety. The DOE will assume

Identifying Number*	Location of Performance Standard Requirement	Topic	LAR Appendix or Document	Performance Standard	Comments
					responsibility for ICs following license transfer.
110	40 CFR 61	Air Quality Standards	G, Q, W	40 CFR 61 192	
143	10 CFR 61	Waste Disposal	No Additional Licensee Action Required	10 CFR 61 52(a)(1)	Segregation is unnecessary. See Performance Standards associated with 10 CFR 61.56(b)
144	10 CFR 61	Waste Disposal	No Additional Licensee Action Required	10 CFR 61 52(a)(2)	No Class C wastes will be placed in the repository.
151	10 CFR 61	Air Quality	G, Q, W	40 CFR 61 92	
169	40 CFR 192	Administrative	No Additional Licensee Action Required	40 CFR 192 40	This clause does not contain requirements or standards.
170	40 CFR 192	Administrative	No Additional Licensee Action Required	40 CFR 192 34	This clause does not contain requirements or standards.
171	40 CFR 192	Corrective Action Program	No Additional Licensee Action Required	40 CFR 192 33	This clause does not contain requirements or standards.
172	40 CFR 192	Administrative	No Additional Licensee Action Required	40 CFR 192 42	This clause is an administrative option and does not contain substantive requirements for the respondent.
208	40 CFR 192	Administrative	No Additional Licensee Action Required	40 CFR 192 10 and 11	These clauses do not contain requirements or standards.
209	40 CFR 192	Administrative	No Additional Licensee Action Required	40 CFR 192 30 and 31	These clauses do not contain requirements or standards.
210	40 CFR 192	Administrative	No Additional Licensee Action Required	40 CFR 192 00 and 01	These clauses do not contain requirements or standards.
211	40 CFR 61	Administrative	No Additional Licensee Action Required	40 CFR 61 90 and 91	These clauses do not contain requirements or standards.
227	40 CFR 61	Emissions Monitoring	Q, W	40 CFR 61 93 through 95	
229	40 CFR 61	Administrative	No Additional Licensee Action Required	40 CFR 61 97	This clause does not contain requirements or standards.

Identifying Number*	Location of Performance Standard Requirement	Topic	LAR Appendix or Document	Performance Standard	Comments
233	40 CFR 192	Groundwater	No Additional Licensee Action Required	40 CFR 192 12(c)	This clause pertains to ground-water monitoring and corrective action. Groundwater monitoring, and corrective actions are addressed under Operable Unit 1 for the Site and are not addressed in this Remedial Design.
235	40 CFR 192	Guidance for Implementation	No Additional Licensee Action Required	40 CFR 20 through 23	These clauses do not contain requirements or standards for the respondent.
236	40 CFR 192	Habitable Buildings	No Additional Licensee Action Required	40 CFR 192 12(b)	This clause pertains to habitable buildings. The Settlement Agreement (SA) site does not include any habitable buildings.
240	40 CFR 192	Monitoring and Corrective Action	No Additional Licensee Action Required	40 CFR 192 03 and 04	This clause pertains to groundwater monitoring and corrective action. Groundwater monitoring and corrective actions are addressed separately.
247	40 CFR 192	Radiation Protection	C, G, T	40 CFR 192 12(a)	
248	40 CFR 61	Radiation Protection	W	40 CFR 61 223	
254	40 CFR 192	Repository Design	G, W	40 CFR 192 32(a)	
307	40 CFR 125	Water Treatment	No Additional Licensee Action Required	40 CFR 125 ALL, except those explicitly identified in other rows	The design does not incorporate active water treatment.
308	10 CFR 61	Licensing	No Additional Licensee Action Required	10 CFR 61 30	This clause pertains to the transfer of the source material license from the licensee to the site owner. This is not relevant to the LAR.

3.5 SUBSURFACE CONDITIONS

The TDA materials were evaluated for geotechnical engineering properties during the PDS (MWH, 2014). The objective of the study was to evaluate subsurface conditions for the placement of the Repository. Figure 7 shows the locations of the sampling locations completed for the Mill Site PDS on the TDA.

3.5.1 Existing Cover

Ten of the twelve test pits excavated on the existing impoundment during the Mill Site PDS (MWH, 2014) exhibited three distinct material types. An upper, soft layer of sandy clay material was observed, ranging from 3 to 8 inches thick. Below the upper sandy clay layer, a layer of rock mulch (crushed basalt) was observed, ranging in thickness from 3 to 4 inches. This combined layer is considered the admixture layer of the existing cover. Gradations were conducted on samples of the soil-rock admixture layer of the existing impoundment cover. The median diameter (D_{50} value) of the samples tested ranged from 0.1 to 13 mm (less than or equal to 0.5 inches). The gravel fraction of these samples was then evaluated for potential reuse as erosion protection at the Site. The gravel was tested for durability in general accordance with guidelines for long-term performance outlined by NRC. Based on information provided in NUREG-1623 (NRC, 2002), rock for use in areas defined as critical areas must meet a score of 65 percent or greater and require oversizing if the score is less than 80 percent. The basalt samples tested from the existing cover scored 94 percent and therefore would not require oversizing for reuse as erosion protection rock.

Below the admixture, or rock mulch layer, a clay material was encountered and is considered the radon barrier layer of the existing cover. At two test pit locations (MWH, 2014), only sandy clay was encountered. Test pits did not extend beyond the radon barrier. The clay layer beneath the admixture layer on the existing impoundment cover was sampled and tested for geotechnical properties. Clay layer samples are classified as low-plasticity clay (CL) with fines contents ranging from 51 to 69 percent. Water contents of the samples tested are between 6 and 11 percent (by mass) and up to 4 to 6 percent below the optimum water contents measured from the standard Proctor tests (MWH, 2014).

3.5.2 Tailings

The Mill Site PDS (MWH, 2014) characterized the subsurface materials using 32 CPTs and 8 hollow-stem auger borings, across all three cells. Nineteen of the 32 CPTs encountered fine-grained tailings in contact with the alluvium, and in three of the 32 CPT locations, coarse-grained tailings were in contact with the alluvium. In the North Cell, the coarse tailings ranged from 0 to 17 feet thick, interlayered coarse/fine tailings from 0 to 4.5 feet thick, and fine-grained tailings from 0 to 2 feet thick.

In the Central cell coarse-grained tailings ranged from 0 to 27 feet thick, interlayered coarse/fine tailings from 0 to 26 feet thick, and fine-grained tailings from 0 to 1 foot thick. At Borrow Pit No. 1, the coarse tailings ranged from 6 to 23 feet thick, the interlayered coarse/fine tailings from 0 to 23 feet thick, and the fine-grained tailings from 0 to 30 feet thick. Around the perimeter of Borrow Pit 1, coarse grained tailings ranged from 8 to 14 feet thick, interlayered coarse/fine tailings from 0 to 7 feet thick, and fine-grained tailings from 3.5 to 6 feet thick. At Borrow Pit 2, fine-grained tailings were encountered ranging from 2 to 11.5 feet thick.

The CPT and borehole results within the tailings areas indicated the presence of coarse-grained materials above, and in some cases interlayered with, fine-grained materials. The upper coarse-grained materials were found to have low water contents and were partially saturated. The fine-grained samples were also partially saturated, although with a higher moisture content and degree of saturation than the coarse-grained material. No free water was identified in the tailings during the PDS drilling program.

The fine-grained tailings were generally characterized as inorganic clays of low to high plasticity (USCS CL and CH). The plasticity index of the fine-grained tailings ranged from 27 to 61 percent. The coarse-grained tailings were classified as silty sands or sand silt mixtures (SM) and one was classified as a poorly graded sand (SP) and were non-plastic. The interlayered

coarse/fine tailings were classified as inorganic clays of low to medium plasticity (CL) and clayey sands to clay (SC/CL) with a plasticity index ranging from 17 to 24 percent.

3.5.3 Alluvium

The alluvium and fill generally consists of silty sands in the top portion of the borings. The alluvium grades to silts, then to clay with interbedded lenses of sands and silts. The sand was predominantly fine-grained sand, but occasional lenses of coarse-grained sands and gravels were encountered. Free water was encountered during drilling at three boring locations, TI-B3, TI-B10, and TI-B11 within the alluvium. In TI-B3, water was encountered at an elevation of approximately 6,903 feet; in TI-B10 at 6,883 feet; and in TI-B11 at 6,885 feet.

3.5.4 Sandstone

The underlying bedrock surface was updated with the results of the PDS drilling. Depressions in the bedrock surface that were previously interpreted to be isolated depressions were interpreted to be paleochannels with the updated results. One paleochannel encompasses, and is in general alignment, north-south, with the Pipeline Arroyo, and another is a tributary to that paleochannel that trends east-west through Borrow Pits 1 and 2 in the impoundment. The paleochannels are represented by a deeply eroded bedrock surface and thick alluvial deposits underlying the Pipeline Arroyo and the Central Cell. This is consistent with the general interpretation of the subsurface features and geology made by Canonie (1987).

3.6 REPOSITORY LAYOUT

The Repository layout includes a design capacity of 1,033,500 cubic yards of mine waste with the ability to accommodate variations in the mine waste volume between about 955,000 cubic yards and 1,110,000 cubic yards. The Repository will be located within the limits of the existing UNC Site TDA on the North and Central cells and is set back a minimum of 50 feet eastward from the western embankment (existing tailings dam) to limit traffic loading from haul traffic and stress from fill placement on the western embankment. The fill thickness of mine waste over the former borrow pits is limited to reduce the potential for differential settlement of the final Repository cover surface. The area of maximum fill thickness is located over the area of shallow sandstone that extends between the North and Central Cells, to minimize settlement. Figure 8 shows isopachs of the fill thickness of the Repository and the overall thickness of the fill plus the thickness of the underlying tailings.

The existing radon barrier covering the tailings in the TDA will be prepared to serve as the foundation layer for the Repository. The erosion protection layer overlying the radon barrier (consisting of a nominal 6-inch-thick layer of soil and rock) will be removed and reused for Repository cover construction. The erosion protection layer and existing rock (D_{50}=3 inches) in the swales within the Repository footprint will also be removed. The rock from the swales will either be combined with the rock taken from the erosion protection layer and reused on the new cover or used for erosion protection on other areas of the Site. The residual soil from the existing erosion protection layer will be reused for Repository cover construction, to fill in the swales located on the existing cover, or for general fill around the Repository. See Section 7 Drawings, details on 7-09.

The existing clay radon barrier will be modified in-place and serve as the “low-permeability layer” located between the placed mine waste and the existing tailings. The upper 6 inches of material (erosion protection layer) will be removed from the cover, prior to moisture conditioning and re-compaction of the radon barrier. The purpose of the layer is to eliminate the potential to collect water resulting in a “bathtub effect”. This layer will be constructed of earth materials, rather than synthetic, to eliminate failure risk associated with punctures and rips, and will be moisture conditioned and compacted in place. Refer to Appendix G for details on preparation of the Repository subgrade.

The Repository design and layout has been incorporated into the existing site stormwater control features of the previously reclaimed TDA. The design includes a sloping cover surface to existing Branch Swale C located on the east and south sides of the Repository. On the north side, surface flow from the Repository will be directed into the North Cell Drainage Channel,

which directs flow around the TDA from the east side to the north side and outlets to the Pipeline Arroyo. On the west side of the Repository, following construction of the Repository cover, clean fill will be added to the existing TDA cover, with similar erosion protection (rock admixture) to the Repository cover, to convey sheet surface flow from the new cover to the crest of the TDA embankment and down into the existing Runoff Control Ditch. The Runoff Control Ditch will be modified, and the riprap will be enlarged (see Appendix I).

On the southwest side of the Repository cover, clean fill will be required to fill in existing Branch Swale D, which currently drains to the northeast. Stormwater off the Repository surface will flow to the southwest. The surface of this fill area will include a similar erosion protection layer to the Repository cover. Flow from this small catchment area of the Repository will be directed onto the existing TDA cover and to existing Branch Swale H which flows to the south. Branch Swale H is planned to be reestablished once the evaporation ponds are decommissioned and removed. The Section 9 Drawings show the stormwater control designs for the Repository and they are described in Appendix I.

3.7 REPOSITORY COVER DESIGN

The cover system design is included as Appendix G.7. The cover system design report includes analyses for erosional stability of the cover, ET cover design, water balance, infiltration, radon emanation, and bio-intrusion. The cover design consists of an erosion protection layer including a soil layer with rock admixture overlying a soil layer. The thicknesses of these layers and the sizes of the rock used for erosion protection vary based on locations on the Repository. The layout for the different erosion protection layers and the cover design details for the different cover sections are shown on Drawing 7-09 in the Section 7 Drawings.

3.7.1 Cover Materials

Materials to be used for cover construction will consist of soil from the onsite borrow areas and stockpiles, and erosion protection rock both reused from the existing TDA cover and imported from an offsite rock quarry. Borrow materials are described in Appendix H. The ET cover has been modeled and designed based on the soil properties of the on-site borrow areas. Construction specifications have been developed to provide quality assurance and material consistency for the materials from the borrow areas that are used for cover construction. Based on the relatively uniform geotechnical properties of the soils from the borrow areas and the topsoil stockpile, soils from any of these sources may be used in any order, for cover construction. Mixing of the borrow soil materials is not required. A comparison of available borrow sources is included in Appendix H, Section H.4.1.9.3 and Figures H.4-1 and H.4-2. Stantec completed additional drilling and sampling in the jetty excavation area to further characterize the jetty soils for use as a borrow source. The material is currently being tested for suitability in the laboratory and results will be ready late-Fall 2018.

Rock to be used for the erosion protection layers on the cover will vary in size (1.5 to 3 inches), depending on location and slope length. The erosion protection layers will consist of a mixture of soil and 33 percent rock by volume and a rock slope on the east side of the Repository. The rock sources have been selected to meet NRC requirements for durability. Borrow soils and durability testing of candidate rock sources for the project are summarized in Appendix H. Granite and limestone riprap can be sourced from a quarry located near Gallup, New Mexico. Rock quality testing from quarry samples (provided in Appendix H) indicate that these local rock sources meet NRC (NRC, 2002) durability requirements. The granite could be used without upsizing, while the limestone would require a 5 percent increase in the median design diameter to account for long-term degradation potential (see Appendix H, Section H.1).

3.7.2 Erosion Protection Design

Appendix G.7 includes the erosion protection designs for the ET cover admixture and the perimeter fill material (transition areas) located on the west and southwest sides of the Repository. Erosion protection designs for the 20 percent slope located on the east side of the Repository are included in Appendix G.8. Due to the steepness, this slope was designed as a riprap

slope rather than as an ET cover with an admixture layer. The rock size for the 20 percent slope is a minimum D_{50} of 1.5 inches and is similar to the design rock sizes for other areas of the Repository (D_{50} of 3 inches or less) due to the length of the slope and the small catchment area. The rock along the 20 percent slope will also be mixed with a limited volume of soil to allow for vegetation establishment on the slope.

3.7.3 Radon Flux Measurements

The Repository cover is designed and will be constructed to limit the release of radon-222 (radon) to the atmosphere, not exceeding an average release rate of 20 pCi/m^2/s. The ET cover design report (Appendix G.7) includes radon emanation calculations for the cover. Following construction of the erosion protection layer, radon emission testing will be conducted to verify that the design and construction of the cover is effective at limiting radon flux using the method described in 40 CFR part 61, Appendix B, Method 115. Section 2 of Method 115 describes the radon flux measurements.

Consistent with Method 115, a single set of radon flux measurements will be made over the entire Repository. The Repository will be considered a region within the existing TDA. Method 115 specifies a minimum of 100 measurements for a region. Radon flux measurements over the Repository will be made at 102 equally spaced locations (grid nodes of 150-foot square grid cast over the Repository). A radon measurement procedure, similar to the detailed measurement procedure provided in Appendix A of USEPA 520/5-85-0029 will be used to measure the radon flux on the Repository. This radon flux measurement procedure involves adsorption of radon on activated charcoal in large-area canisters. The radon canisters will be placed on the surface of the pile and allowed to collect radon for a 24-hour period. The radon flux measurements will not be initiated within 24 hours of a rainfall and will not be performed if the ambient temperature is below 35°F, or if the ground is frozen. The radon collected on the charcoal will then be measured by gamma-ray spectroscopy.

The mean of the radon flux measurements will be calculated for verification of the 20 pCi/m^{2}s radon emission performance standard for the Repository. Results of the individual radon flux measurements with locations will be included in the as-built report for the completed Repository.

3.7.4 Repository Cover Gamma Exposure Rate Measurement

Criterion 6(1) of 10 CFR Part 40, Appendix A specifies that: "*Direct gamma exposure from the tailings or wastes should be reduced to background levels."* A direct gamma radiation survey will be performed following placement of the ET cover to verify that the direct gamma exposure attains the required ambient background levels. The results of this survey will be compared to the survey conducted prior to removal of the erosion protection layer on the existing TDA, described in Section 4.5.1. The direct gamma radiation survey will be performed at the same 102 locations as the radon flux measurement locations described in the previous section. The direct gamma radiation survey will be conducted during radon canister placement for radon emission testing. The gamma survey will consist of a one-minute static gamma measurement at each location over the Repository area. The gamma radiation levels will be measured in exposure rate micro Roentgen per hour (µR/hr). The static gamma radiation survey will be conducted using a 2x2 NaI(Tl) scintillation detector interfaced with a scaler/rate meter. The mean of the direct gamma exposure rates will be calculated for comparison to the background levels. Results of the individual direct gamma exposure rate measurements with locations will be included in the as-built report for the Repository. Details on the confirmation survey procedures are included in Appendix T.

4.0 HEALTH, SAFETY, AND ENVIRONMENT

4.1 SURFACE WATER DRAINAGE AND DIVERSION

The design intent of the stormwater controls at the Mill Site is to prevent stormwater from impacting the TDA. As a result of the proposed action, modifications to the existing stormwater controls will be required. This section summarizes and Appendix I presents the design basis for these proposed modifications and others that will reduce sediment accumulation in channels (see Section 4.1.2). Appendix I also includes the evaluations and designs for:

- Improvements to the North Diversion Channel, which is located along the south and east side of the TDA (see Section 4.1.3).
- Improvements to the drainage of the alluvial floodplain area north of the North Cell of the TDA, and improvements to the North Cell Drainage Channel located north of the North Cell of the TDA (see Section 4.1.4)
- Evaluation and mitigation designs for the Pipeline Arroyo stabilization upgradient of, and adjacent to, the Repository area, and specifically improvements to the existing buried rock "jetty" (see Section 4.1.4).

4.1.1 Design Discharge

The design event for the Mill Site stormwater controls and cover erosion protection is the PMF. The design for the Pipeline Arroyo Stabilization evaluated a range of flood events and provides protection that can statistically be expected to *"…be effective for one thousand years, to the extent reasonably achievable, and, in any case, for at least 200 years…"* (40 CFR §192.32). Stantec estimated the design flood event by simulating runoff hydrographs for a corresponding design storm event, where the design storm event was developed, as a center peaking rainfall distribution that included the peak rainfall intensity for every duration from 5 minutes to 24 hours, for design storm frequency or the PMP intensity for all durations from 10 minutes to 6 hours.

The calculated design flows incorporate new methods that were not available when the previous TDA reclamation plan was developed (Canonie, 1991). The PMP depth and distribution presented in this document were calculated using the recently-developed PMP Tool prepared for the Arizona Department of Water Resources (ADWR, 2013) while the previous TDA reclamation plan utilized Hydrometeorological Report (HMR) 49 (Hansen et al., 1984). The ADWR PMP study, which incorporates the Pipeline Arroyo watershed, accesses a larger precipitation database and newer analytical techniques that were not available in the development of HMR 49. The ADWR PMP tool produces gridded PMP values using a grid spacing of approximately 2.5 square miles to allow site-specific estimation of precipitation depths. Similarly, other frequency-based storm hyetographs were developed with site-specific precipitation intensity-duration information as recommended in the recent National Engineering Handbook (NRCS, 2015). Finally, the updated design discharge estimates compute rainfall losses using the Green-Ampt method (Green and Ampt, 1911) which provide physically-based estimates of losses during different storm intensities and storm durations. The methods and assumptions used to develop these different model inputs are discussed in Appendix I.1.

Stantec estimated the PMF at various Mill Site stormwater control locations using a numerical rainfall-runoff model (HEC-HMS 4.2.1). The model development methods and simulation results for the Mill Site stormwater hydrology are presented in Appendix I.1. Stantec developed five hydrologic models to facilitate estimation of flood flows for different locations, conditions, and storm events. These models are summarized in Table 4.1-1 and the development methods and simulation results for the models are provided in Appendix I.1.

Table 4.1-1 Summary of Developed Hydrologic Models

Hydrologic Model	Peak Flows Simulated	Related Design Analyses/Model Uses
Pipeline Arroyo Watershed Model for Existing Conditions	PMF and 2-, 5-, 100-, 200-, 1,000-, 10,000-year events	Used to evaluate the existing hydraulic conditions within the Pipeline Arroyo
Pipeline Arroyo Watershed Model for Post-Repository Conditions	PMF and 2-, 5-, 100-, 200-, 1,000-, 10,000-year events	Evaluation of Pipeline Arroyo Stabilization Alternatives (riprap sizing, erosional protection, energy dissipation efficiency), Evaluation of upper pipeline arroyo hydraulics (flood extents for design events in reach adjacent to TDA), Computational Fluid Dynamics modeling of rock jetty
Mill Site Sub-Catchments Model for Post-Repository Conditions	PMF and 2-, 5-, 100-, 200-, 1,000-, 10,000-year events	Hydraulic Analysis of North Diversion Channel, Repository Channel Capacity and Erosional Stability, Lower East Repository Channel Sediment Transport Competency, and Hydraulic Analysis of Mine Site Outlet Channel

4.1.2 Repository Stormwater Controls

The proposed stormwater controls for the Repository use existing swales and channels previously constructed for the TDA with improvements and supplemental controls where necessary to conform to performance standards. These stormwater controls are summarized in Figure 9 and shown with detail on Drawings 9-01 and 9-02, include the East Repository Channel and related sediment controls and drainage improvements for the south and west side of the Repository. Calculations for the design of the Repository stormwater controls are provided in Appendix I.2, and filter compatibility calculations for the granular filters below the channels are included in Appendix I.3.

4.1.2.1 East Repository Channel and Related Sediment Controls

The proposed East Repository Channel will run along the south and east perimeter of the Repository. Stations 0+00 to 34+60 of the proposed East Repository Channel follow the current alignment of existing Branch Swale C and Stations 34+60 to 41+39 are aligned with the existing upper reach of the North Cell Drainage Channel (see Drawings 9-02 through 9-04). The design objectives for the East Repository Channel are to provide capacity and scour protection against the PMF, and pass sediment delivered to the channel.

4.1.2.2 East Repository Channel Capacity and Scour Protection

The hydraulic calculations (see Appendix I.2) show the following requirements for the East Repository Channel to conform to performance standards:

- Stations 0+00 to 18+30 – No improvements are required to the existing Branch Swale C.
- Stations 18+50 to 28+30 – The required median (D_{50}) riprap size is 3.0 inches. The existing D_{50} riprap size in this reach of the swale is 1.5 inches. Thus, a larger riprap size is required and excavation of some material below the riprap layer will be necessary to accommodate the larger riprap.
- Stations 28+30 to 34+60 (downstream of the confluence with existing Branch Swale B) – The required D_{50} median riprap size is 9.0 inches.
- Station 34+60 to 41+39 – The required D_{50} median riprap size is 9.0 inches. The design also includes modifying the cross-section of the existing channel in this reach to increase the sediment transport capacity of the channel.

Existing Branch Swale C between approximately Stations 0+00 and 18+30 is constructed over tailings and a radon barrier. Because no channel improvements are required in this reach, the radon barrier will not be impacted by the proposed design for the East Repository Channel. Filter compatibility calculations show that a two-layer granular filter is required to meet filter criteria for the subgrade and various riprap sizes (Appendix I.3).

4.1.2.3 East Repository Channel Sediment Control Features

Sediment accumulation along existing Branch Swale C at the base of the south side of Dilco Hill has created localized high points in the swale that reduce the swale capacity and are promoting further sediment deposition. Sediment has also accumulated in the upper reach of the North Cell Drainage Channel (future East Repository Channel) where an erosional feature from Dilco Hill empties into the channel. The apparent sources of the sediment are bare areas on the south side of Dilco Hill and an erosional feature on the northwest side of Dilco Hill.

The design for the East Repository Channel proposes several controls to reduce sediment delivery to and increase the sediment transport capacity of the channel:

- Two interceptor channels will be constructed on Dilco Hill. The interceptor channels will reduce sediment delivery from Dilco Hill by cutting the overland flow length. The interceptor channels will also divert stormwater runoff and sediment from Dilco Hill into the lower reach of the East Repository Channel, which is designed for improved sediment transport capacity (see Drawing 9-02).
- A rock check dam will be constructed at the base of the erosional feature where it empties into the East Repository Channel. The check dam will decrease sediment loading to the lower reach of the East Repository Channel (see Drawing 9-02).
- The lower reach of the East Repository Channel will be constructed to modify the base of the existing channel cross-section from flat to triangular (see details on Drawing 9-04). The triangular section will improve the sediment transport capacity of the channel by nearly three times and will have sufficient capacity to pass sediment delivered from the two Dilco Hill drainage channels.

Calculations demonstrating the sediment transport capabilities of the channels on Dilco Hill are provided in Appendix I.4.

4.1.2.4 Repository South and West Side Drainage

This design includes no new drainage channels or swales on the west side of the Repository. Instead, the Repository cover will be extended to the existing north-flowing portion of the existing Runoff Control Ditch that runs along the west side of the TDA. The north portion of the Runoff Control Ditch will be extended south to capture drainage from the southwest side of the Repository. Hydraulic calculations indicate that the existing Runoff Control Ditch has sufficient capacity to convey the post-Repository PMF flow but that the riprap size will need to be increased to a D_{50} of 3 inches (see Appendix I.2) to maintain erosional stability during the PMF. Appendix G describes the Repository cover design.

The design includes no new channels or swales on the southwest side of the Repository (west of the proposed head of the East Repository Channel). Stormwater draining from the southwest side of the Repository will drain to existing Branch Swale H (see Figure 9). Currently, Branch Swale H has no outlet point. Branch Swale H was originally designed to drain to the south and tie into the South Diversion Channel. The alignment of the future tie-in reach of Branch Swale H is through the existing evaporation ponds and will be completed following removal and reclamation of the ponds. The hydraulic calculations show that the existing Branch Swale H has capacity for the post-Repository PMF (see Appendix I.2). Design modifications are not being made to Branch Swale H or the downstream South Diversion Channel. These structures will remain as originally designed and constructed under the NRC-approved tailings reclamation plan.

4.1.3 North Diversion Channel

The North Diversion Channel (NDC) is an existing earthen conveyance channel that intercepts stormwater runoff from native upgradient watersheds to the south and east of the TDA and diverts it to the alluvial floodplain area north of the TDA. The upper and middle reaches of the NDC have a mild slope (approximately 0.005 ft/ft) and are constructed with an earthen embankment on the left channel bank (i.e., between the channel and the TDA). The lower (northernmost) portion of the

channel is cut through Dilco Hill and has steeper channel slopes (approximately 0.03 ft/ft). The NDC has some areas of minor aggradation, but overall appears to function according to its design intent.

Hydraulic modeling of the PMF through the NDC shows that, in its current condition, the NDC can convey the PMF with no overtopping (see Appendix I.5); however, an area of concern for long-term loss of channel capacity is near where the channel turns from east to north. In this location, the channel embankment is breached by a dirt road that crosses the channel. The road is causing sediment deposition where it crosses the bottom of the channel. The proposed improvements will re-grade the road to allow the channel embankment to be reconstructed and to maintain a constant channel invert slope (see Figure 9). With the proposed improvements, the hydraulic analysis shows that the NDC has more than 1 foot of freeboard during the PMF under the estimated condition of future vegetation overgrowth in the channel.

The design also includes two rock check dams on the right (south) bank in the east-west portion of the NDC (see Figure 9). The purpose of the check dams is to trap sediment at the outlets of two tributary catchments to the NDC that have historically delivered sediments to the NDC.

The hydraulic model simulations predict that the PMF flow will be sub-critical in all but the lower NDC reach; however, the predicted PMF velocities in all reaches of the NDC are high (over 10 feet per second [fps]), and channel and bank scour is possible during extreme flood events. The depth of scour is, however, unlikely to compromise the embankment, which is over 80 feet wide at the base. The model predicts super-critical flows with velocities up to 29 fps for the lower reach of the NDC that is cut through Dilco Hill, but excessive scour in the reach is not expected because the channel is excavated through rock.

4.1.4 Pipeline Arroyo Flood Extents and the North Cell Drainage Channel

The Pipeline Arroyo Watershed above (north of) the TDA is approximately 18 square miles in area. The estimated PMF in the arroyo reach that runs along the TDA is 27,600 cubic feet per second (cfs) (see Appendix I.1). Appendix I, Figure 4.1-2 shows the floodplain extents for the PMF and the 100-year and 5-year floods, estimated with a two-dimensional hydraulic model (HEC-RAS) (see Appendix I.6). The simulated flood extents show that the 5-year storm will be contained in the Pipeline Arroyo, but that the 100-year flood and the PMF will overtop the banks. The estimated floodplain extents for the 100-year flood and PMF include Pipeline Canyon Road that parallels the arroyo, north of the TDA. The estimated PMF floodplain extents are also estimated to encroach on the north edge of the TDA and the base of the Repository. The previous PMF evaluation (Canonie, 1991) predicted similar PMF flood extents (also shown in Appendix I, Figure 4.1-2). Note that PMF flood extents predicted by Canonie previously do not account for the Repository; whereas the flood extents estimated in the current design do.

Results of the two-dimensional hydraulic model also show that the North Cell Drainage Channel will be inundated during the PMF. Under existing conditions, the downstream portion of the North Cell Drainage Channel could experience velocities on the order of 5 ft/s. To reduce velocities in the North Cell Drainage Channel in large flood events, and thus decrease potential for scour at the base of the Repository, the road that runs along the north bank of the North Cell Drainage Channel will be raised as a protective berm to hydraulically isolate the North Cell Drainage from the alluvial area to the north (see modeling results in Appendix I.6).

4.2 EROSIONAL STABILITY OF THE JETTY AREA

4.2.1 Review of Bedrock Depths and Quality

Information collected from previous drilling, tailings reclamation (Canonie 1991), cone penetration testing (MWH, 2014), and geotechnical borings for design (Stantec, 2017 and 2018; Appendix I.8) indicate that the bedrock surface dips steeply to the southeast near the rock jetty. Depths to bedrock increase between the Pipeline Arroyo and the TDA dam, with a maximum depth of over 100 feet (Appendix I.8). The exposed sandstone bedrock along the existing portion of the Pipeline Arroyo at the

location of the rock outcrop is highly weathered and friable, with severe scour into the sandstone bedrock created by flood events (Appendix I, Photo 4-5). Rock core obtained from the geotechnical borings performed indicate that the underlying rock consists of zones of sandstone, shale, and coal of similar quality as the exposed outcrop. The rock would be subject to substantial scour if exposed, unprotected, to a series of annual peak floods.

4.2.2 Assessment Summary

The undercutting downstream of the jetty location has exposed the existing jetty rockfill and threatens to progressively, or abruptly, fail the toe of the jetty (Appendix I, Photo 4.2-1). The undercutting is at the head of an “erosional pathway” headcut that appears to have originated about 450 feet downstream and has created a preferential flow pathway away from the engineered section of the Pipeline Arroyo over the rock outcrop (Appendix I, Figure 4.2-11). Based on the review of historical images and a site tour, the cause of the erosional pathway appears to be flood waters pushing away from the rock outcrop and into the softer fill material behind the jetty in an alignment that closely follows the alignment of the pre-mine Pipeline Arroyo. The headcut has scoured to the bedrock near the toe of the jetty, and future flooding through the erosional pathway will likely dislodge the jetty rock from the toe, leading to collapse of the jetty sometime in the future. A failure of the jetty may not put the TDA embankment at immediate risk of failure but could result in a loss of grade control at the rock outcrop, leading to episodic head-cutting of the Pipeline Arroyo upstream from the location of the existing jetty.

The rock sizes observed in the existing jetty structure and shown in the as-built documents indicate that the jetty was not designed to protect against flows overtopping the structure. The median design rock size is 6 inches (Canonie, 1991). Stantec’s hydraulic simulations suggest that floods with an annual return interval of between 10 years and 100 years would exceed the capacity of the upstream arroyo channel. In such a flood event, flooding would overtop the jetty and a breach-type failure of the jetty would be likely. Thus, an eventual failure of the jetty, either by undercutting or overtopping, is likely.

If the jetty were to fail, lateral migration of the Pipeline Arroyo upstream of the rock outcrop, though not certain, is possible. Under a failure scenario, the downcutting that has occurred below the rock outcrop could progress upstream; although, historically, the rock outcrop appears to have provided grade control against upstream headcutting so that upstream headcutting and lateral migration might be slow and limited.

Regardless of the failure scenarios presented, currently the jetty is functional, and the Pipeline Arroyo upstream of the jetty appears stable, with no evidence of scour or lateral migration of the channel. Other than the erosional pathway, the Pipeline Arroyo downstream of the jetty also appears to be stable (based on aerial imagery), with some historical deepening and widening, but with no lateral movement. Prediction of how far the erosional pathway might migrate further toward the TDA is difficult. Historical images show only deepening of the pathway with no lateral movement in the last decade, but the further downcutting in the pathway and undercutting of the banks could cause episodic bank failures and pathway shifting toward the TDA. That the pathway would shift far enough to the east to threaten the TDA embankment is unlikely; however, the available bedrock information indicates that migration will not be limited by bedrock control in that direction. Besides the erosional pathway, the engineered arroyo channel between the jetty and the southern end of the TDA has been stable with no meandering since at least 1981, although, similar to the erosional pathway, lateral migration will not be limited by bedrock. A large meander bend in the Pipeline Arroyo does exist just downstream of the TDA.

4.2.3 Design Description for the Riprap Chute

Through the design process, the replacement of the existing rock jetty with a riprap chute was selected. Drawings 9-09 through 9-11 show the design of the Riprap Chute, and the hydraulic evaluations and riprap sizing calculations for the chute are provided in Appendix I.7. The chute crest extends from just downstream of the rock outcrop on the right bank of the Pipeline Arroyo (looking downstream) to the embankment of the TDA. This extent is sufficient to capture flows from the PMF. The chute will slope longitudinally at 5.3 percent for about 56 feet vertically, where the flood flows will discharge into a sunken riprap basin. A 5.3 percent slope was selected over steeper slopes that would have less excavation volumes because the 5.3

percent slope grades the chute beyond the steep drop in the arroyo bed (see Appendix I, Figure 4.2-12). This drop appears to be a headcut in the channel bottom, presenting the concern that if the drop migrates to the base of the chute, it could potentially undercut the chute stability. By grading the chute beyond this drop, the 5.3 percent slope chute eliminates this concern.

The median riprap diameter for the chute is 27 inches. The factor of safety (FS) of the median design riprap size (D_{50d}) to the median riprap size at the threshold of displacement (D_{50f}) can be computed as:

$$FS = \frac{D_{50d}}{D_{50f}}$$

The hydraulic analysis (Appendix I.7) demonstrates that these riprap sizes will provide a factor of safety for the PMF of slightly greater than 1.0. Flood events between the 10,000-year flood and 100-year flood are estimated to have greater factors of safety as shown in Table 4.2-2 (see also Appendix I.7).

Table 4.2-1 Estimated Factors of Safety against Riprap Failure for Various Flood Events

Flood Event		PMF	10,000-Yr	1,000-Yr	200-Yr	100-Yr
Exceedance Probability (%)	in 1,000-years	0.05	9.5	63	99.3	~100
	in 200-years	0.001	2.0	18	63	87
D_{50f} (inches)	Maximum inside of critical zone	25.5	20.4	18.1	16.0	14.2
	Maximum outside of critical zone	20.8	16.7	13.4	12.3	10.2
Factor of Safety	Inside of critical zone	1.06	1.32	1.49	1.69	1.90
	Outside of critical zone	1.30	1.62	2.01	2.20	2.65

Notes:
1) Exceedance probability is the probability that the designated flood event will be exceeded in a given time period (1,000 years and 200 years)
2) Exceedance probability of PMF is estimated using an assumed recurrence interval of 2×10^6 years based on regression of simulated flood events
3) Factor of safety values computed using a median riprap diameter of 27 inches assuming the specific gravity of the rock is 2.6
4) D_{50f} = median riprap size at the threshold of displacement

A sunken riprap basin is designed at the toe of the chute, with a depth of 2 feet and a length of about 100 feet. The hydraulic modeling shows that the hydraulic jump on the chute will be a submerged jump and controlled by the downstream constriction (see Appendix I.7). Therefore, the jump length will not be influenced by the outlet basin length. To account for potential changes in downstream conditions, the length of the outlet basin was designed by assuming that a free jump would form at the toe of the chute and have a length of six times the sequent flow depth (Chow, 1959) for the PMF, or approximately 15 feet.

The side slopes adjacent to the TDA embankment will slope toward the weir at 5H:1V, and the side slopes on the right side of the weir will be cut back into rock at a slope of 2.5H:1V. The side slopes will be rock-armored with rock having a median diameter (D_{50}) of 3 inches to provide erosion protection from incidental rainfall and runoff.

Samples from the 2016 geotechnical characterization (Stantec, 2017) indicate that the existing subgrade below the proposed chute is composed primarily of fine-grained soils (silty clay, silty sand, sandy silt). To prevent washout of the subgrade soil, a granular filter is included between the riprap and the underlying soil. Two filter layers are required to prevent loss of subgrade below the chute (see Appendix I.3).

The design provides no flood controls in the arroyo downstream of the chute outlet basin. The historical imagery of the area (Section 4.2.2) shows no evidence of lateral migration of the downstream arroyo; however, post-closure monitoring downstream is recommended to identify possible instabilities with the potential to migrate back toward the riprap basin.

4.3 GEOTECHNICAL STABILITY

4.3.1 Slope Stability Analyses

Static and pseudo-static slope stability analyses were conducted for the Repository. The slope stability analysis calculation brief, which includes a figure showing the cross-sections locations used for the analyses, is included in Appendix G.2. Three cross-sections were selected for stability analyses (shown in Appendix G, Figure G.8-1). The cross sections were selected as representative of the maximum loading conditions, critical slope geometry, and maximum fill height for the Repository. The cross-sections are located along the Repository slopes to represent loading conditions on the existing TDA and embankment, to evaluate design slopes of the final Repository cover slopes, and to evaluate the global stability of the final Repository and existing TDA embankment. Limit equilibrium slope stability analyses were performed using the GeoStudio software SLOPE/W (Geoslope International, 2016). Material properties and the geometry and stratigraphy of the selected cross-sections were based on findings from previous field investigations and laboratory analyses conducted during the PDS. The analysis evaluated both circular and block-type failure surfaces along the selected cross-sections.

The critical (lowest) calculated factors of safety for both static and pseudo-static loading conditions for each of the cross sections from the model outputs were evaluated against the required design factors of safety given by the NRC design guidance documents. A summary of the static and pseudo-static slope stability results is provided in Table 4.3-1. The calculated factors of safety are greater than the recommended minimum factors of safety for each case evaluated. Additional slip surfaces and factors of safety are presented and summarized in Appendix G.2 for deep failures to bedrock, shallow failures, and failures that exit at the bottom of the cover and/or embankment for each analysis.

Table 4.3-1 Summary of Slope Stability Results

Cross Section	Failure Type	Loading Condition	Minimum Required Factor of Safety (1)	Calculated Factor of Safety
Cross Section A – Southwest Slope	Circular	Static	1.5	9.9
		Pseudo-Static	1.0	1.8
Cross Section A – Northeast Slope	Circular	Static	1.5	2.8
		Pseudo-Static	1.0	1.3
	Block	Static	1.5	2.8
		Pseudo-Static	1.0	1.3
Cross Section B – Repository Slope	Circular	Static	1.5	8.1
		Pseudo-Static	1.0	1.7
Cross Section B – Existing Dam	Circular	Static	1.5	2.4
		Pseudo-Static	1.0	1.2
Cross Section B – Arroyo Flood	Circular	Static	1.2	2.6
Cross Section C – North Slope	Circular	Static	1.5	3.2
		Pseudo-Static	1.0	1.7
Cross Section C – North Slope (Entry/exit)	Circular	Pseudo-Static	1.0	1.7
Cross Section C – Arroyo Flood	Circular	Static	1.2	2.6

Cross Section	Failure Type	Loading Condition	Minimum Required Factor of Safety (1)	Calculated Factor of Safety
Cross Section C – South Slope	Circular	Static	1.5	10.3
		Pseudo-Static	1.0	1.8

1) NRC Regulatory Guide 3.11 (NRC, 2008)

4.3.2 Liquefaction

The potential for liquefaction of saturated tailings and alluvium beneath the proposed Repository during the design seismic event was evaluated for the design. The analysis was performed for the most critical condition (a design seismic event after completion of Repository construction) and based on the subsurface profile at the time the PDS field sampling was conducted. The liquefaction analysis calculation brief is included as Appendix G.6, with the method and results summarized below.

The liquefaction triggering analysis evaluated the potential for liquefaction of saturated tailings or underlying alluvium beneath the Repository, which may result in damage to the existing TDA radon barrier or compromising the effectiveness of the Repository to isolate mine waste. A liquefaction screening evaluation (Bray et al., 2009) was performed to identify zones of tailings or soil that may be susceptible to liquefaction. One-dimensional profiles were developed for analysis, based on conditions observed during the Mill Site PDS field investigation and modified to reflect proposed loading conditions. Identified zones of potentially susceptible materials within these profiles, as identified by the screening analysis, were evaluated for liquefaction potential using simplified liquefaction triggering analysis methods (Idriss and Boulanger, 2008; Youd et al., 2001).

The liquefaction triggering analysis used data collected during the CPT program, drilling and standard penetration testing (SPT), and laboratory testing to calculate the FS against liquefaction for potentially susceptible zones of saturated material below the Repository. The primary liquefaction analysis used the results of CPTs. SPT results, where available, were used to provide secondary data against which the results of the CPT-based analyses were checked. The liquefaction triggering analyses incorporated supplemental data from laboratory testing and were performed according to the methods outlined in Idriss and Boulanger (2008) and Youd et al. (2001). The FS was calculated as the average of the FS values calculated by each of the analysis methods.

The liquefaction screening evaluation identified eight samples, out of 33 samples screened, that were moderately susceptible to liquefaction. The remaining 25 samples that were screened were not susceptible to liquefaction. Of the samples representing zones that were saturated or nearly saturated, two zones had minimum average calculated FS values below the acceptable FS criteria (per NRC, 2008) of 1.0 (at 0.9). These zones consisted of fine-grained tailings relatively deep in the tailings profile (33 to 45 feet depth) within Borrow Pit 1 (from borings B8 and B10).

The interlayered nature of the materials in Borrow Pit 1, and the proximity of the samples with low-FS values to each other, indicate that this zone of potential liquefaction represents a small percentage of the overall Repository foundation. The depth of these zones is also significantly below the depth of critical failure surfaces generated from pseudo-static slope stability analysis, along selected cross-sections, that include these zones.

The potential for liquefaction of the tailings or alluvium beneath the Repository footprint was evaluated using accepted screening and analysis methods. The potential for liquefaction only applies to materials that are saturated or nearly saturated and in a relatively loose condition. Due to the unsaturated condition of most of the underlying tailings and alluvium beneath the Repository footprint, the screening and analysis identified only two zones of materials at depth in Borrow Pit 1 that were moderately susceptible to liquefaction. The depth and localized nature of these two zones pose a risk for minor amounts of additional post-earthquake consolidation settlement, but no slope stability concerns are anticipated.

Liquefaction-induced settlement was estimated using the results of the liquefaction analysis and field data. Liquefaction-induced settlement occurs following a seismic event during which liquefaction occur. The soil may experience a volume change as the excess pore water dissipates and the soil particles rearrange themselves. The method used for estimating this volume change is outlined in Idriss and Boulanger (2008), Section 4.4 “Post-liquefaction Reconsolidation Settlement”. The liquefaction induced settlement calculations and results are included in Appendix G.4. Based on the field data and the results of the calculations, the potential for liquefaction-induced settlement at the Site is contained in a localized area and occurs at a depth where surficial expression and damage to the radon barrier or ET cover is considered unlikely.

4.4 TOTAL AND DIFFERENTIAL SETTLEMENT

The settlement analyses conducted for the Repository includes immediate settlement, primary consolidation, secondary consolidation, and seismic induced settlement. The analyses were conducted to evaluate settlement due to placement of the mine waste and cover material on the existing TDA. The settlement analyses calculation briefs, except for seismically-induced settlement, are provided as Appendix G.3. The seismically-induced settlement calculation brief is provided as Appendix G.4. Figure G.8-1 shows the borehole and CPT locations, and the fill thicknesses used to conduct the analyses (see also Figure 8). The proposed surfaces for the top of mine waste and top of cover for the Repository were used to determine the proposed thickness of fill at each CPT and/or borehole location within the Repository. These fill thicknesses were used to determine the increase in stress within the Repository at each location.

Based on the filling plan progressing from north to south, the north slope of the Repository will be filled to design waste grade during early stages of filling. The north slope of the Repository would be completed several months before completion of fill placement in the southeast corner. During placement to the design waste grade and covering with a temporary cover of clean soil, settlement will be monitored, by survey while fill placement continues in other areas of the Repository. Settlement monitoring data will be collected during the construction period and compared to the predicted consolidation settlements to verify that no additional grading mitigation measures are necessary prior to completion of the final cover.

4.4.1 Immediate Settlement

Immediate settlement was calculated for one-dimensional settlement, following the guidelines presented in the NAVFAC 7.01 design manual (Department of the Navy, 1986), based on guidance in NUREG-1620 (NRC, 2003). The immediate settlement of the TDA surface near the perimeter of fill placement was evaluated to address potential impacts of cover cracking of the existing radon barrier as a result of differential settlement. Immediate settlement of the upper unsaturated materials (including the radon barrier) would occur rapidly and incrementally with each layer of mine waste and will therefore not impact the long-term performance of the Repository cover. Cover cracking of the existing radon barrier is discussed in Section 4.4.5.

Immediate settlement was calculated at three locations (B15/CPT-15, CPT-26, and B1/CPT-01) on the southwest slope of the Repository. These locations were selected because of proximity to the area where the Repository fill will transition directly to an area where the existing radon barrier will remain in-place and unmodified. The primary focus for evaluating the immediate settlement is to determine the contribution to differential settlement near the perimeter. Within the interior of the Repository the immediate settlement will occur incrementally as fill is placed and will not affect the Repository. The results of the immediate settlement analysis were used to determine the extent of the impacts from Repository construction on the existing TDA cover. A summary of the results of the immediate settlement analysis are present in Table 4.4-1.

Table 4.4-1 Summary of Immediate Settlement Results

Location	Immediate Settlement (ft)
CPT-01	0.1
CPT-15	1.0
CPT-26	0.6

4.4.2 Primary Consolidation

Settlement during and following active construction of the Repository is anticipated to result from primary consolidation caused by fill placement and the resulting dissipation of porewater pressures in the fine-grained tailings. Primary consolidation was calculated using a one-dimensional consolidation settlement analysis following the guidelines presented in the NAVFAC design manual 7.01 (Department of the Navy, 1986) and based on guidance in NUREG-1620 (NRC, 2003). Each of 25 CPT locations within the Repository footprint were used to estimate the primary consolidation. The soil profiles were created using data from CPT and borehole testing locations conducted during the PDS (see Appendix G, Figure G.8-1). Consolidation was calculated using the fill thickness from the Repository design at the locations of the CPT hole locations. Seven of the 25 locations included boreholes paired with the CPT.

Because only the fine-grained tailings are near saturation, the primary consolidation calculations only include settlement results for fine-grained tailings. In locations where the estimated settlement is presented as 0.0, no near-saturated layers of fine-grained tailings layers were encountered. The primary consolidation was estimated for each location where fine-grained tailings were encountered in the subsurface profile, if the profile information along with the laboratory data indicate the fine tailings are near saturation (85 percent degree of saturation or greater). Additionally, in locations where interlayered fine and coarse tailings were encountered, the overall thickness of this material was assumed to behave as fine tailings to present a more conservative estimate of primary consolidation settlement totals for the location. The calculated primary consolidation at each location is summarized in Table 4.4-2, and the calculations are included in Appendix G.3.

4.4.3 Secondary Consolidation

Settlement estimates for the completed ET cover surface resulting from secondary consolidation (creep) are also included in the estimated overall total settlement. Secondary consolidation was calculated using a one-dimensional settlement analysis following the guidelines presented in the NAVFAC design manual 7.01 (Department of the Navy, 1986) and based on guidance in NUREG-1620 (NRC, 2003). The same soil profiles used for primary consolidation were used to estimate the secondary consolidation at 25 locations within the Repository footprint. The soil profiles were created using data from CPT and borehole testing locations conducted during the PDS. The secondary consolidation was calculated for each fine-grained tailings layer in each CPT vertical soil profile location. The summation of the secondary consolidation in each fine-grained tailings layer at one location resulted in the total estimated secondary consolidation at that location. The secondary consolidation calculations include estimates of secondary consolidation for locations where the fine-grained tailings are near saturation. The calculated secondary consolidation at each location is presented in Table 4.4-2. In locations where the estimated settlement is presented as 0.0, a saturated or near saturated fine-tailings layer was not encountered. The overall total (primary plus secondary) consolidation estimates are also presented in Table 4.4-2.

Appendix G, Figure G.9-1 shows the estimated total amounts of consolidation settlement expected to occur during the construction period as well as over the design life of the Repository. Immediate settlement is not included in the totals shown in Appendix G, Figure G.9-1 since immediate settlement will occur prior to the completion of the cover. In addition, a percentage of the consolidation settlement totals shown on the figure will occur prior to completion of the final surface of the cover as fill is placed. Therefore, the amounts shown represent upper limits for the post-construction settlement totals. The figure presents total estimated settlements based on profiles from the CPT and borehole locations as well as settled surface contours showing expected changes to the cover surface grading if the estimated total settlement were to occur. The contouring indicates changes to the design slopes will occur on the south facing slope and a portion of the east facing slope (over the east and south edge of the former borrow pit). However, based on the calculations, predicted settlements will not result in slope reversal or areas of ponding on the cover.

Table 4.4-2 Summary of Primary, Secondary, and Total Consolidation Results

Location	Primary Consolidation (ft)	Secondary Consolidation (ft)	Total Primary and Secondary Consolidation Settlement (ft)
CPT-01 (TI-B1)	0.02	0.05	0.1
CPT-02 (TI-B2)	0.13	0.03	0.2
CPT-04	0.17	0.04	0.2
CPT-05	0.00	0.00	0.0
CPT-06	1.03	0.22	1.3
CPT-08 (TI-B8)	0.51	0.21	0.7
CPT-09	1.13	0.34	1.5
CPT-10 (TI-B10)	1.08	0.29	1.4
CPT-11 (TI-B11)	0.08	0.14	0.2
CPT-12	0.02	0.03	0.1
CPT-13	0.00	0.00	0.0
CPT-14	0.20	0.05	0.3
CPT-15 (TI-B15)	0.00	0.00	0.0
CPT-16	1.34	0.32	1.7
CPT-17	0.40	0.09	0.5
CPT-18	1.45	0.35	1.8
CPT-19	0.59	0.21	0.8
CPT-20	0.19	0.12	0.3
CPT-23 (TI-23)	0.00	0.00	0.0
CPT-24	0.00	0.00	0.0
CPT-25	0.00	0.00	0.0
CPT-26	0.00	0.00	0.0
CPT-27	0.00	0.00	0.0
CPT-28	0.00	0.00	0.0
CPT-29	0.00	0.00	0.0

4.4.4 Seismic Settlement

Seismic settlement calculations were prepared to estimate the potential settlement that may occur within the footprint of the proposed Repository as a result of the design seismic event. Analysis of one-dimensional stratigraphic profiles was performed using the design seismic event which was characterized by the parameters presented in the PSHA (see Appendix G.1). The seismic settlement analysis used data collected during CPTs, hollow-stem auger (HSA) drilling, and laboratory testing from the PDS to estimate the magnitude of potential seismic settlement within the footprint of the proposed Repository.

Six one-dimensional stratigraphic profiles were developed and analyzed as part of the seismic settlement analysis. These six locations correspond with locations where shear wave velocities were measured during CPT. The stratigraphic profiles were developed based on conditions observed during the Mill Site PDS (MWH, 2014) field investigation and modified to reflect proposed Repository construction (placement of mine waste and the Repository cover). During the Mill Site PDS field investigation, eight boreholes were “paired” with, and drilled adjacent to, CPT locations. Seven of these paired locations are within the footprint of the proposed Mill Site Repository, and shear wave velocity measurements were recorded during CPT at six of those locations. A summary of the seismic settlement analyses results is presented in Table 4.4-3 and the calculation

brief is included as Appendix G.4. These amounts of settlement are considered within the tolerable limits (6 to 12 inches) of seismic deformation for tailings impoundments described in NUREG-1620 (NRC, 2003).

Table 4.4-3 Potential Seismic Settlement Resulting from the Design Seismic Event

Borehole ID	Seismic Settlement (ft)
TI-B1/CPT-01	0.07
TI-B2/CPT-02	0.12
TI-B8/CPT-08	0.08
TI-B10/CPT-10	0.12
TI-B11/CPT-11	0.13
TI-B15/CPT-15	0.09

4.4.5 Cover (Radon Barrier) Cracking

The differential settlement, at the most likely location for cover cracking (along the tie-in between the proposed Repository and the existing TDA cover), was used to estimate the potential for cover cracking of the existing radon barrier. The cover cracking analysis calculation brief is included as Appendix G.5. The analysis was performed for the southwest edge of the proposed Repository, where the new cover ends on the existing TDA cover. Similar to the immediate settlement, these locations were selected because they are closest to the area where the Repository fill will transition directly to an area where the existing radon barrier will remain in-place and unmodified. Cover cracking of the exiting radon barrier is calculated using the location with the maximum anticipated differential settlement between the new cover and the existing cover. Around the remainder of the Repository perimeter, the new cover extends to an existing swale or drainage channel on the east and south or to the west apron which extends to the top of the dam. On the north edge of the Repository, the new cover will extend beyond the limits of regraded tailings (Canonie, 1991) and to approximately the top of the North Drainage Channel. The purpose of the analysis is to determine if the stress increase from the fill placement results in detrimental differential settlement at the edge of the Repository that will negatively affect the radon barrier outside the Repository.

Using the overall total combined predicted settlements (immediate, primary consolidation, secondary consolidation, and seismic induced settlement) for subsurface profiles from TI-B15/CPT-15, CPT-26, and TI-B1/CPT-01, differential settlement was estimated between the southwest slope of the Repository and the radon barrier located immediately beyond the edge of the Repository. For CPT-26, where seismic settlement was not estimated, the estimate for the seismic settlement from TI-B15/CPT-15 was used in the total. These three locations are beyond (TI-B1/CPT1) or near the edge of waste placement (TI-B15/CPT-15, CPT-26) where the new cover material will transition directly to the existing radon barrier. The maximum differential settlement was estimated for each of three locations over the distance to the edge of the proposed cover and is summarized in Table 4.4-4.

Table 4.4-4 Estimated Differential Settlement and Cover Cracking Potential near the Edge of the Repository

Borehole ID	Estimated Total Differential Settlement (ft)	Horizontal Distance to Edge of Cover (ft)	Resulting Slope Reduction (%)	Horizontal Strain (%)
TI-B1/CPT-01	0.26	80	0.33	0.009
TI-B15/CPT-15	1.17	180	0.65	0.008
CPT-26	0.67	210	0.32	0.003

To evaluate the potential for cracking in the existing radon barrier a relationship between tensile strain and plasticity index (PI) of the soil is used (Morrison-Knudson, 1993). The PI value for the existing radon attenuation layer was estimated to be 16 percent, calculated as the average of the measured PIs of ten radon barrier samples collected during the PDS (MWH, 2014). Using this value for PI, the minimum estimated horizontal tensile strain that will induce cracking is 0.10 percent. The resulting

slope reduction from the estimated maximum differential settlement predicted near the edge of the Repository was then used to calculate horizontal movement for the 21-inch-thick radon barrier in the central cell. The horizontal movement was then doubled based on Gourc et al. (2010) and Rajesh and Viswanadham (2010). The values of peak horizontal movement were then used to estimate peak horizontal strain, which was calculated to be less than 0.01 percent or one-tenth of the maximum allowable horizontal strain to prevent cracking for the three locations evaluated. These results indicate cover cracking will not occur for the proposed conditions.

4.4.6 Stress Influence Analysis

The shape of the Repository concentrates the greatest fill thicknesses near the middle of the layout, with thickness decreasing from the center to the perimeter. Due to the gently sloping surface of the Repository and only minimal fill thickness around the perimeter, induced stresses beyond the edge of the Repository (due to fill placement within the Repository footprint) are expected to be minimal and will not influence the areas of the TDA outside of the Repository footprint. Due to the configuration of the Repository, there would be no compressive stresses that would extend outward from the edges of the Repository.

4.5 MATERIAL ISOLATION

The existing cover radon barrier will be reused as the bottom layer and the ET cover will be built over the mine waste extending down to this bottom layer (see Appendix G for details and analyses).

4.5.1 Baseline Gamma Survey

Prior to removal of the erosion protection material from the cover of the existing TDA within the Repository, a baseline gamma radiation survey will be conducted on the surface. The survey will provide a location-specific ambient background level for comparison with the completed Repository cover (see Section 3.5.4). The gamma survey will consist of a GPS-based one-minute static survey at each node of a 150-foot square grid over the Repository area. The gamma radiation levels will be measured in exposure rate (µR/hr). The static gamma radiation survey will be conducted using a 2x2 NaI(Tl) scintillation detector interfaced with a scaler/rate meter. The scaler/rate meter will be integrated with a DGPS/controller to log the radiation levels and their corresponding position coordinates. Following removal of the erosion protection layer and prior to re-compaction of the radon barrier surface, a visual inspection of the cover will be completed to verify that tailings were not exposed during the removal process. The tailings are generally gray in color while the surficial materials are typically brown. This inspection will be completed prior to initiating compaction of the layer and prior to placement of mine waste in the Repository. If the visual inspection indicates tailings have been exposed, a post-excavation (one-minute) static gamma survey, similar to the baseline survey, can be performed to determine extent of the material exposed. If tailings are exposed, the radon barrier would be excavated in the vicinity of one of the branch swales so that the tailings can be placed back below the radon barrier. The radon barrier extends down beneath the erosion protection in each of the branch swales. Removing the radon barrier within one of the swales would provide storage volume beneath the grade of the surrounding radon barrier. Additional details on the methods for the baseline survey are included in Appendix T.

4.5.2 Repository Subgrade Preparation

The existing radon barrier above the tailings in the TDA will be prepared to serve as the foundation layer for the Repository. The erosion protection layer overlying the radon barrier (consisting of a nominal 6-inch-thick layer of soil and rock) will be removed and reused for Repository cover construction. The rock will be screened from the soil to separate it. The erosion protection layer and existing rock (D_{50}=1.5 inches) in the swales within the Repository footprint will also be removed. The rock from the swales will either be combined with the rock taken from the erosion protection layer and reused on the new cover or used for erosion protection on other areas of the Site. The residual soil from the existing erosion protection layer will be reused for Repository cover construction, to fill in the swales located on the existing cover, or for general fill around the Repository.

To achieve a stable foundation for mine waste placement and maintain a zone of separation from the underlying tailings, the following preparation tasks for the existing radon barrier beneath the footprint of the Repository will be competed:

Rock mulch and riprap removal. The rock mulch on the surface of the radon barrier will be excavated and stockpiled for use on the Repository cover. Rock mulch excavation will be conducted carefully to minimize removal of radon barrier material. Riprap currently lining the swales would also be removed. The swales will be filled to grade with the soil portion of the existing rock mulch. The excavated surface of the radon barrier would be regraded where necessary to smooth the surface for compaction and establish proper grades.

Radon barrier compaction. The specifications require the upper 6 inches of the radon barrier to be compacted to 95 percent relative compaction per standard Proctor, at a water content less than the optimum water content with the objective of an unsaturated hydraulic conductivity of less than $1x10^{-7}$ cm/sec for the layer. Additional reworking or excavation or ripping into the radon barrier is not recommended due to the potential for contact with, and exposure of, underlying tailings. Material properties of the existing radon barrier are described in the As-Built Reclamation Reports (Canonie, 1991, 1992, 1994, and 1995; Smith, 1996 and 1997) from construction of the radon barrier, as well as in the test pit sampling of the cover in the PDS report (MWH, 2014).

With the mine waste and Repository cover in place, the flux of meteoric water through the Repository would be sufficiently low that no zone of saturation would develop on the surface of the radon barrier, and the radon barrier would remain in an unsaturated condition (see Appendix G.7). The ET performance of the Repository cover will aid in maintaining unsaturated conditions in the radon barrier.

Water application. Water will be added, as necessary, for dust control during rock mulch removal and radon barrier compaction. The addition of water may be required in order to achieve the desired level of compaction. Water will also be applied, on a limited basis, to the compacted radon barrier surface for dust control during initial mine waste hauling and placement.

4.5.3 Mine Waste Placement Sequence

Following preparation of the radon barrier, clean borrow soils will be used to construct perimeter stormwater berms at the edge of the mine waste placement area within the Repository. These berms will allow for containment of contact water within the Repository during waste placement (additional detail in Appendix G Section G.5.3). As construction progresses, these berms will be incorporated into the construction of the soil cover layer. The stormwater containment berm design is described in Section G.5.3.1 of Appendix G.

Excavated mine waste will be hauled from the Mine Site, and initially placed and compacted directly on the prepared radon barrier. Because, the mine waste will be placed from the north to south across the Repository and will be spread in lifts for compaction; initially the stormwater containment berms will be constructed on the north side of the Repository. The perimeter slopes of the compacted mine waste surface will be extended as the mine waste surface is raised and the perimeter stormwater berms can be adjusted to maintain containment of contact water runoff from the outer slopes of the Repository.

The Mine Site removal sequence is divided into six phases and Drawings 7-02 through 7-04 show, in concept, plan and section views of where each of the five phases (excluding the PTW phase) of removal from the Mine Site would be placed within the Repository. Three locations (Sandfill 1 (Area 7), Sediment Pad (Area 6), and Pond 1 (Area 9) at the Mine Site have mine waste activity levels (for radium-226) greater than the weighted average (by volume) of the overall volume of mine waste. A summary of the activity levels for the mine removals is presented in Table 4.5.1. The volume of these three areas combined account for less than 13 percent of the estimated total volume of mine waste to be placed in the Repository. The activity levels were estimated from the analytical data in the PDS report, (excluding the PTW). Of the five phases of removal and placement proposed, the majority of the volume is included in Phase 3. Areas 6, 7, and 9 (Sediment Pad, Sandfill 1, and Pond 1) would be removed in three different phases (Phases 2, 3, and 4). These three volumes will be dispersed within the other lower

activity materials removed and then placed during that same phase. Further, the specifications for Mine Waste Excavation and Disposal limit the placement of the materials from these three Mine areas to greater than 200 feet from the outer slope of the Repository.

Table 4.5-1 Mine Waste Weighted Average Activity Levels by Volume

Mine Removal Phase	Mine Area(s)	Average of 75th Percentile Activity (pCi/g)	Estimated Removal Volume (CY)
2	1	2.4	14,763
2	2	26.7	37,005
2	3	11.4	67,138
2	4	5.2	13,771
2	7	50.5	40,917
3	6	86.3	23,086
3	8	24.2	422,473
4	5 and 11	9.8	6,399
4	9	85.2	29,302
4	10	10.2	7,758
5	12	7.1	34,272
6	13	6.5	28,356
		Weighted Average by Volume (75th percentile vales) = 26.5 pCi/g	

4.5.4 Repository Cover Placement

As described in Section 3.7, the designed ET cover thickness is based on limiting infiltration and maximizing evapotranspiration. The cover thickness has been analyzed for acceptable rates of radon-222 emanation at the cover surface, based on the weighted average (by volume) radium-226 activity levels of the mine waste. Refer to Appendix G.7 for specifics on the radon emanation calculations. The Design Drawings (Section 7) show a Repository layout for storage of approximately 1.03 million cubic yards of mine material. The design capacity for the Repository is based on the removal estimate for soil and debris (waste) from the Mine Site. The placement sequence allows for flexibility in the final volume by making adjustments to the top surface (at approximately 2 percent slope) and how this surface ties into the existing TDA cover on the south side of the Repository. The 1.03 million cubic yards capacity provides for approximately 30 percent contingency storage.

The cover design includes the combination of a lower soil layer, of varying thickness depending on location, and an upper admixture layer consisting of a mixture of soil with varying percentages of rock. The lower soil layer will be placed and compacted over the graded mine waste slope surface. The erosion protection admixture layer will then be placed over the lower soil layer. The Construction Contractor will have the option of mixing the soil and rock layers on or off the cover.

4.6 GROUNDWATER COMPLIANCE

Because the Consolidation and Groundwater Report (Dwyer, 2018) concluded that flux resulting from the loading by placement of mine waste does not result in a change in flux to the saturated alluvium beneath the proposed Repository, there are no groundwater quality impacts that could occur as a result of the NECR removal action. Groundwater protection standards defined in license amendment No. 52 (NRC, 2015) would still apply to the site. Further details on the current groundwater quality data and annual compliance monitoring is in the Annual Review Report (Hatch-Chester, 2018).

4.7 CONSTRUCTION SEQUENCING

4.7.1 Construction Support Facilities

The contractor will provide labor, equipment, and materials to develop the construction support facilities as shown on the drawings. A general layout and grading plan for the construction facilities is shown on the drawings. These facilities will be constructed at the Mill Site, at the Mine Site, and near the TDA. The primary Decontamination Facilities will be located at the former Mill Site and will contain (1) a vehicle decontamination area, and (2) a personnel decontamination area. The vehicle decontamination area will be paved with asphalt concrete and will drain to a collection sump. The collection sump will collect water from vehicle decontamination procedures, and stormwater runoff from the paved decontamination pad. A heated structure will be provided for screening equipment storage. Personnel decontamination facilities will include: scanning equipment, showers, lockers (for changing from civilian clothes into work clothes and vice versa), restroom, and laundry facilities. The support facilities are shown in the Section 2 Drawings and described in Appendix B.

Support facilities will provide contamination control through segregation of contaminated and non-contaminated materials and activities. Support facilities will be organized in areas using the following terms and definitions for the various work areas:

- Support: Area(s) free of contamination
- Controlled: Area(s) with potential contamination
- Exclusion: Area(s) with contamination subject to removal (Mine Site and Repository)
- Decontamination Area: The transition area between the Controlled area and the Support area. This is where personnel enter and exit the Controlled Area and where most decontamination activities take place.

4.7.2 Haul Roads

The contractor will construct and maintain construction access roads and haul roads as required for use by the contractor and as indicated on the drawings. The project will require a construction haul road crossing at NM 566 as indicated on the drawings and the contractor will provide and maintain a relocated temporary connection to Pipeline Canyon Road at NM 566 for use by public traffic. The contractor will make its own investigation of the condition of available public and private roads and for clearances, restrictions, bridge load limits, and other limitations affecting transportation and ingress and egress to the site of the work. Roads that are installed, or existing roads that are improved, for construction access will be removed and restored following use, as directed by the engineer. Each temporary access road will be winterized prior to demobilization for temporary winter weather shutdowns. Construction of the haul roads also includes construction of temporary stormwater controls to manage runoff and contact water. The haul roads are shown in the Section 3 Drawings and described in Appendix D.

The contractor will provide flaggers, spotters and all other personnel required for traffic control activities not otherwise specified. The contractor will develop and implement a traffic control system for the Mine Waste Haul road traffic control.

4.7.3 Radon Barrier

Prior to initial hauling of mine waste to the Repository location, the existing radon barrier will be prepared as described in Section 4.5.2. Although unlikely, during the process of removing rock from the cover layer and swales, there is a potential that earthwork on the cover could expose existing tailings. In general, tailings can be identified by a light gray color and differs in appearance from the soil cover. If the contractor suspects that tailings are exposed during the process, work must stop so the radiation safety officer (RSO) may conduct a radiological scan of the ground surface in the work area. Material that is confirmed to be tailings must be returned to beneath the existing radon barrier. This will be accomplished by excavating an area of the existing cover approved by the engineer (likely in one of the existing swales), placing the material, and recompacting the radon barrier in 6-inch conditioned lifts over the material. The radon barrier in the area of exposed tailings will be reconstructed in compacted lifts to match the original design. The work area must then be confirmed by scans that the

tailing material has been removed from the surface. The design specifications (Appendix J) include details for addressing this contingency.

4.7.4 Temporary Stormwater Controls

The contractor will establish temporary stormwater controls at the Repository site to control contact water that falls on the mine waste once placed. The controls will include a temporary stormwater control berm constructed from clean borrow soils. The berm will be required to extend along the perimeter of the Repository layout and downgradient of any areas where mine waste will be placed, to contain stormwater runoff. Additional temporary stormwater controls will be required along the haul road, in the borrow areas, in the support zones and all other disturbance areas. The contractor is required to submit for Engineer approval a Stormwater Pollution Prevention Plan (SWPPP) prior to initiating work. Appendix E describes the temporary stormwater controls for the project.

4.7.5 Haul and Place Mine Materials

To the extent practicable, excavated materials will be direct loaded into haul vehicles, transported, and placed within the Mill Site Repository without stockpiling. The Mine Site excavation plan is included as Appendix C and the Section 3 Drawings. When direct haul is not practical, excavated materials will be temporarily stockpiled at approved locations for impacted soils as defined by the engineer prior to haul and placement. Unclassified mine waste will be disposed of in the Repository constructed on the tailings disposal area as shown on the drawings. Soils and sediments will be hauled, dumped and spread in uniform horizontal lifts with a maximum loose thickness of 12 inches and compacted to minimum 90 percent Standard Proctor density as determined by ASTM D698. Compacted moisture is specified to be dry of optimum. Appropriate mechanical compaction equipment will be used to obtain the required compaction. The project specifications are in Appendix J.

Soils removed from mine Area 7 (Phase 2) Sediment Pad, Area 6 (Phase 3) Sandfill 1, and Area 9 (Phase 4) Pond 1 will not be placed within 200 feet of the outer cover slope of the Repository. Demolition debris will be comingled with the soils, to minimize void spaces and will be distributed throughout designated areas of the backfill to avoid concentrated areas of debris. Per the specifications, no debris will be placed within 100 feet of the Repository perimeter, within 3 feet of the bottom of the cover layers, or within 2 feet vertically from the radon barrier (a lift of mine waste soil must be placed for separation). The maximum size of demolition debris will not exceed 20 feet in the longest dimension. Smaller dimensions may be necessary for loading, handling, hauling, and placement of material in the disposal cell. Compressible materials will be crushed and then covered with backfill. Incompressible materials will be placed in the disposal area, with the void spaces outside of the materials filled with backfill. Progressive Covering of Mine material in the Repository will be implemented to limit contact with stormwater and avoid dust. The proposed fill sequence is shown in the Section 7 Drawings.

Cover Soil and General Fill will be placed over compacted mine waste as shown on the drawings. The soil cover material will be placed in successive horizontal lifts of loose material not more than 12 inches in depth. Each lift will be spread uniformly and moistened, or scarified, as necessary to achieve the specified compaction. For the admixture layer, lifts to be mixed in-place will be no thicker than 9 inches once compacted, following mixing. Each lift must meet specifications (density and moisture) before placement of additional materials. The cover and fill soils will be sourced from one of the on-site borrow areas. The proposed borrow sources are described in Appendix H and the borrow area grading plans are shown in the Section 8 Drawings.

4.7.6 Stormwater Channels Including Jetty Improvements

Following completion of the Repository cover, stormwater control improvements will be implemented to the existing channels near the North and Central cells. Excavation of ditches and channels will be finished by cutting accurately to the cross sections, grades, and elevations shown on the drawings and will not be excavated below grades shown. Riprap, primarily imported from offsite, will be placed for erosion protection as shown on the drawings. Because the jetty excavation, is being considered a source of borrow soil for the Repository cover construction, the excavation in this area may proceed in parallel

with mine waste placement and progressive covering. The stormwater control design is described in detail in Appendix I and the design is shown in the Section 9 Drawings.

4.7.7 Revegetation

After the ET cover has been constructed and the stormwater control features have been completed, the disturbed areas will be seeded with a site-specific native vegetation mix. The revegetation plan is described in Appendix U and the revegetation areas at the Mill Site are shown in Drawing 10-02.

4.8 SCHEDULE

The proposed construction schedule is included in Appendix K. Work would begin with establishing the proper support facilities and work zone controls. This includes early works, haul and access roads, Repository preparation activities, and several drainage improvement activities near the Repository. This allows for the establishment of drainage controls around the Repository and development and adjustment of dust control measures prior to transporting mine waste. The removal of PTW materials is planned as one of early-phase activities during material excavation, however this will not impact the licensed facility.

Mine waste removal is based on an average haul capacity of 2,700 cubic yards per day, assuming the use of a fleet of between five and ten 30-cubic-yard articulated trucks, operating 7 hours per day. The use of articulated haul trucks is assumed, because the Mine Site excavation areas are not well suited to high volume excavation fleets such as scrapers or large haul trucks. Sufficient excavation and placement capacity was assumed to support haul capacity. Haul volumes are increased by 25 percent for loose, in-truck volumes.

Cover construction consists of hauling and placement of the cover soils and the erosion protection layer. This activity (cover construction) is expected to be conducted in phases, concurrent with mine waste placement. For scheduling purposes, it is shown as a continuous activity concurrent with mine waste placement. Jetty construction was assumed to be completed at the same time as cover placement completion (concurrent finish). A concurrent finish with cover placement results in Jetty excavation after depletion of at least one other borrow area. Riprap production was assumed to occur in the year prior to jetty construction. Conservative durations of 130 days (6 months) were estimated for both jetty excavation and riprap placement. There is not expected to be a formal winter shutdowns of construction operations; however, an approximate two-week weather delay has been included within each winter season (indicated by split tasks on the Gantt chart).

4.9 QUALITY ASSURANCE

Stantec prepared a Construction Quality Assurance Plan (CQAP) to accompany the design which is included as Appendix V. The following roles and responsibilities are described in more detail in Appendix V. The CQAP is intended to define the structure of the quality assurance operations and additional details, including entity names, will be incorporated into this plan once the construction team and the contractor's organization is further defined.

4.9.1 Key Personnel and Responsibilities

The following roles are described in the CQAP for providing quality assurance during construction:

The **Design Supervising Contractor (DSC)** is the company retained by GE/UNC to provide design and engineering services for design of the Project. The DSC coordinates development of the design with GE/UNC, USEPA and NRC. Stantec (formerly MWH), is a licensed design firm retained by GE/UNC as the DSC. The Engineer of Record (EOR) is an employee of the DSC and is the Professional Engineer licensed in the State of New Mexico ultimately responsible for accepting the work as complete, and for approving design modifications or deviations.

The **Construction Supervising Contractor (CSC)** would be the company retained by GE/UNC to provide professional construction quality assurance services in connection with the construction of the project, including its qualified personnel. The CSC would be independent of, and oversee, the Construction Contractor (CC) on behalf of GE/UNC (including review of the CC's proposed means and methods) and serve as the primary point of contact with the CC with regard to construction quality. The CSC would be responsible for implementation of this CQAP.

GE/UNC would retain the **Construction Contractor** to provide labor, materials, and equipment required to construct the project in accordance with the project documents. The CC is responsible for scheduling, coordinating, and planning the construction work (e.g., the means and methods). The CC is responsible for the quality of their constructed work product as well as the necessary inspections and tests required to verify that work complies with the design drawings and technical specifications.

The GE/UNC **Project Manager (PM)** shall be the individual responsible for executing the project on behalf of GE/UNC. The PM is an employee of GE/UNC and is the primary point of contact between GE/UNC and the USEPA or NRC and has financial contractual authority and final decision-making authority on all matters related to the project.

The **Site Manager's (SM)** primary responsibility is to administer the CC's and CSC's contracts. The SM would be an employee, or contract employee, of GE/UNC and the primary point of GE/UNC contact for the CC's and CSC's key personnel. The SM would report to the GE/UNC PM.

The **Construction Superintendent (CS)** is an employee of the CC and is the individual responsible for executing the construction activities as required by the construction contract. The CS coordinates scheduling, construction crews, procurement, and all on-site CC personnel report to the superintendent.

The **Construction Quality Assurance Official (CQAO)** is an employee of the CSC and will coordinate field implementation of this CQAP. The CQAO would be responsible for assembling, tracking, and storing CQA/CQC related documentation. The primary duty of the CQAO is to confirm and document that the project is implemented in accordance with the approved design.

The **Field Engineer's (FE)** primary responsibility is to assist the SM with administration of the CC contract and ensure the project is performed in accordance with the design plans and Technical Specifications. The FE reviews CQC testing documentation with the CC, engineers, and inspectors.

The **Quality Control Manager (QCM)** is responsible for daily on-site implementation of the CC's QC system. CC staff may include QC Technicians to support the QCM. The QCM may assume the role of the QC Technician.

4.9.2 Construction Quality Control

The construction contractor is required to establish a quality control (QC) system to perform sufficient inspection and tests of all work. Quality control includes inspection, sampling and testing, and associated requirements of the specifications. The contractor must review and comply with the CQAP. The CC's QC system shall be established for all construction except where the Technical Specifications provide for specific compliance tests by laboratories employed by GE/UNC. The CC's QC system shall specifically include all testing required by the various section of the Technical Specifications. The CC shall be responsible for establishing a system of daily test reports that will document all CQC test results. Test results from each day's work period shall be submitted to the FE prior to the start of the next day's period. The CC's responsible technician and the CC CQM shall sign the daily test reports. The FE will review test results daily and identify any non-conforming test results for discussion with the CC regarding potential corrective action.

4.9.3 Meetings

The FE will plan, and participate in, a pre-construction meeting prior to initiating major work components. The purpose of pre-construction meetings is to resolve uncertainties following award of the construction contract, but prior to the start of

construction. At a minimum, the meeting will be attended in person or via telephone by GE/UNC (or GE/UNC's representative), the EOR, the CQAO (Supervising Contractor), the FE, and CC Project Manager or superintendents and QCM. The FE will also plan, and participate in, weekly construction coordination meetings held at the site during construction. Additional coordination meetings may be scheduled by either the SM or CC staff. The FE will document the meeting and distribute minutes to all attendees within three working days of the meeting. At a minimum, progress meetings will be attended by the FE, the CC QCM, and the CC superintendent.

4.9.4 Monitoring and Testing

Mandatory hold points are established in the Technical Specifications (Appendix J) for certain key construction activities. At these points, the CC will notify the CQA staff that the construction activity, or portion of an activity, is ready for inspection. At each hold point noted above, the FE must review the item and provide approval that the completed activity meets specifications. Approval shall be verbally given to the CS and noted in the FE's daily report. If an item is found to be deficient, the procedures of Appendix V, Section V5.3 shall be followed.

The CQA staff will review CC activities to document technical compliance in identification, handling, storage, packaging, preservation, and delivery of materials, parts, assemblies, and end products with either the Technical Specifications, manufacturers' recommendations, or generally accepted practices.

CQA staff will monitor the CC to confirm that material identification and management requirements are met. Products and materials shall be traced from receipt through installation. Documentation such as project control checklists, material receipts, sample and test documentation, and reports will verify that the applicable material/item is received and installed. Technical specifications and/or procedures define product identification and management requirements, which generally include the following:

- Construction materials or equipment intended for project use are identified and segregated until inspection confirms that they conform to technical and quality requirements.
- Materials or equipment are traceable to documents attesting to their conformance with technical requirements that are stated in the Technical Specifications or Construction Drawings.

Appendix V, Sections V.5.2.4 through V.5.2.11 include lists of items in the drawings, technical specifications, or design documents that the CQAO personnel must verify, review, and/or document during construction. Daily reports will be completed by the Field Inspectors (FIs) when they are on-site. All FIs will be assigned field books or tablets by the FE that will be labeled with a unique number. The FIs, including the FE, will record all field observations and the results of field tests in their assigned field book or tablet. When not in use, all field books will be kept in the FE field office. After each field book is filled (or at the end of the project), the field books will be returned to the FE.

Each daily report or page of the field book will be numbered, dated, and initialed by the FI. The remaining individual entries will be prefaced by an indication of the time at which they occurred. If the results of test data are being recorded on separate sheets, this will be noted. Field test results will be provided to the FE and other specified representatives of the CQAO team, electronically in spreadsheet format (MS Excel), within 24 hours of the tests being completed. The FE will prepare and sign a daily construction report. The report will include a summary of the CC's daily construction activities. Supporting inspection data sheets will be attached to the daily report where needed.

4.10 RADIATION SAFETY

Site control will be implemented to prevent unauthorized, untrained, or unprotected personnel from entering the site. Areas where construction activities associated with mine-impacted soils and material are conducted will be designated as the

Exclusion and Controlled areas for radiation protection. Some areas will be designated as separate controlled areas within the Exclusion Area. The haul road from the Mine Site to the TDA will be designated as a Controlled Area.

The project presents an added component of radiation protection for the public, due to the mine waste haul road crossing public NM 566. The highway crossing will be controlled during haul operations with a traffic control signal system. Control measures will include mud grates at each end of the haul road, where trucks will be screened and loose contamination above the field screening level will be brushed or scraped from the trucks. Additional mud grates will be installed on each side of the highway. The highway will be protected from haul traffic using a heavy-duty tarp, which will be removed when haul traffic is stopped to allow public vehicle traffic to continue along the highway. Traffic flaggers and a signal system will be used to control traffic at the crossing. Areas where the radioactivity level is not high enough to require radiation protection may be designated as a Clean Area and used for activities such as work breaks, eating, drinking, smoking, etc.

The RSO will conduct general work area monitoring to assess potential radiation exposures to workers. Monitoring instruments such as alpha scintillometers, gamma scintillometers, gamma radiation exposure rate meters and Geiger-Mueller detectors will be function-checked prior to use each day. The RSO will calibrate the radiation monitoring equipment annually. Air monitoring will be conducted at upwind and downwind locations for internal and external radiation. Perimeter air monitoring for internal and external radiation exposure to individual members of the public will be conducted using track etch radon monitors and environmental thermoluminescent dosimeters (TLDs) exposed continuously at the perimeter air monitoring stations.

Several Appendices to this LAR address radiation safety for the project. The health and safety plan (HASP) for Stantec staff is included as Appendix L. Appendix Q is the Air Monitoring Plan for monitoring the work areas for fugitive dust. Appendix R is the Release Contingency and Prevention Plan for managing and cleanup of spills or materials out of containment. Appendix W is the Operations Maintenance and Monitoring Plan for required inspections and repairs following completion of construction.

4.11 FINANCIAL ASSURANCE

In accordance with License Condition 25 and following NUREG-1620 Appendix C, an updated baseline financial surety estimate will be prepared for the site. Except for inflationary adjustments, the baseline surety for the site was last updated in 2004. To account for revisions to the reclamation plan described in this LAR as well as other site activities since 2004, the surety will be updated and submitted to NRC separately. The updated surety estimate, which will account for additional construction on and around the TDA, will be submitted to NRC at least 90 days prior to the proposed start of construction for the project.

5.0 EVALUATION OF ALTERNATIVES

5.1 REMOVAL ACTION ALTERNATIVES

Alternatives for addressing the mine material at the Mine Site were evaluated during the USEPA engineering evaluation/cost analysis (EE/CA) (USEPA, 2009) process and the remedy to dispose of mine materials on the existing TDA was selected based on that process. The following alternatives were considered:

1. No action
2. Excavation and disposal at an offsite treatment, storage, and disposal facility (TSDF) of all NECR wastes
3. Consolidation and covering of mine wastes on the Mine Site
4. Construction of an above-ground capped and lined repository at the Mine Site
5. Consolidation of mine wastes with a cap and a liner at the Mill Site facility either in an existing tailings cell or in a newly-constructed repository.

Alternatives 3 through 5 included the option (A) to dispose of material defined by USEPA as PTW at an offsite hazardous waste disposal facility or an alternative appropriate facility, rather than in the onsite Repository. USEPA's preferred alternative was Alternative 5(A) with the additional disposal of PTW offsite. USEPA estimated the total cost of the preferred alternative to be $44,300,000, with an estimated construction duration of four years, per the referenced document.

5.2 SELECTED ALTERNATIVE

The SER (INTERA, 2018) includes specific details on the alternatives analysis within the design process completed for the Repository and describes design decisions and selections of specific components within the Repository design considered. The alternatives that were considered within the design process included the following:

- No action
- Proposed action
- Conveyance – use of a conveyor system instead of hauling by trucks
- Material sourcing for cover – use of the excavated soils from the Jetty as a primary borrow source
- Disposal of PTW – transport and disposal of PTW in Clive, UT rather than reprocessing of this material at the White Mesa Mill facility, near Blanding, UT.

5.3 COST ANALYSIS FOR ALTERNATIVES

Order of magnitude costs were estimated for the additional construction costs associated with the options for conveyance, borrow sources, and the disposal rather than the re-processing of the PTW material. Table 5.3-1 summarizes the estimated additional costs associated with these three elements beyond the cost for the base design. Aside from cost, each presents additional environmental issues, for which they were primarily excluded from selection during the design process. The costs of these additional environmental impacts are not accounted for here. These design alternatives are further described in the SER.

Table 5.3-1 Summary of Estimated Alternatives Costs

Alternative	Estimated Additional Construction Costs
Conveyance – transport of mine waste to the Repository by conveyor, rather than haul trucks	+$1M
Material sourcing – cover soil from the original four borrow areas vs. borrow only from the Jetty excavation	+$3M
Disposal of PTW at an offsite disposal facility – rather than reprocessing at the White Mesa Mill	+$5M

The conveyance option evaluated the additional cost associated with installing a conveyor system from the Mine Site to the Repository. The conveyor would be installed on a direct line from the mine to the north end of the North Cell. The assumptions include the use of haul trucks at each end of the conveyor to move material to and from the start and end points. The additional cost is based on the conveyance of 1M CY of material from the Mine Site.

The material sourcing option compared using all four of the original borrow areas (North, South, East and West) for cover soil and then reclaiming the borrow areas upon completion with using only the Jetty excavation as the source for cover soil. This option assumes in both cases the jetty excavation must still be completed, however, the soils from the Jetty would be wasted in one of the other borrow areas for the option where the jetty soil is not used for borrow.

The third alternative compares hauling the PTW to the nearest RCRA/hazardous waste disposal facility near Clive, UT versus hauling the PTW to the White Mesa Mill for reprocessing. The estimated additional cost for this alternative compares the use of covered rear-dump trucks using the shortest travel routes (not limited to major interstates). Further details on the cost comparison calculations as well as the associated environmental impacts of each are provided in the SER (INTERA, 2018). The overall project construction cost estimate will be incorporated with the updated surety estimate.

6.0 ENVIRONMENTAL APPROVALS AND CONSULTATIONS

6.1 RADIOACTIVE MATERIAL LICENSE

Material License No. SUA-1475, License Condition 13 states *"Before engaging in any activity likely to cause an environmental impact not previously assessed by the NRC, the licensee shall prepare and record an environmental evaluation of such activity. When the evaluation indicates that such activity may result in a significant adverse environmental impact that was not previously assessed or that is greater than that previously assessed, the licensee shall provide a written evaluation of such activities and obtain prior approval of the NRC in the form of license amendment."*

6.2 APPROVED RECLAMATION PLAN

The NRC Approved Reclamation Plan (Canonie,1991) for the tailings impoundment does not reference a separate environmental evaluation prepared prior to the construction activities completed to cover the tailings impoundment.

6.3 SUPPLEMENTAL ENVIRONMENTAL REPORT

The original Environmental Report (ER) for the Mine Site and Mill Site was submitted by UNC in 1975 (UNC, 1975). An additional ER was submitted by D'Appolonia (1981) in support of a license renewal application (D'Appolonia, 1981). Each of these documents were issued to the State of New Mexico, prior to the state becoming an agreement state with the NRC.

A SER has been prepared for submittal to NRC along with this LAR to evaluate the environmental aspects of the proposed project. The SER (INTERA, 2018) is being submitted under separate cover, to address the environmental impacts related to construction of the Repository and related improvements on the existing tailings impoundment.

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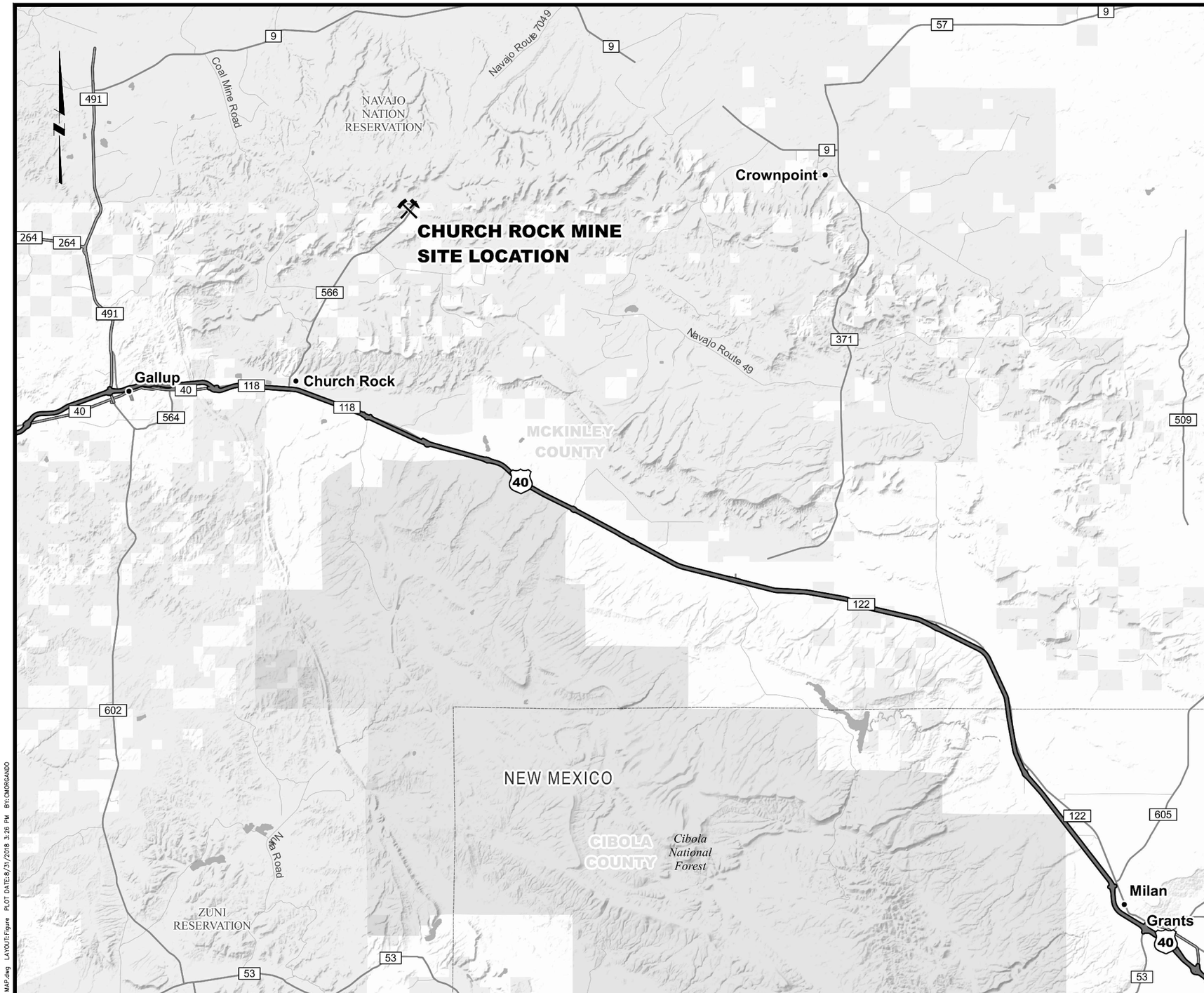
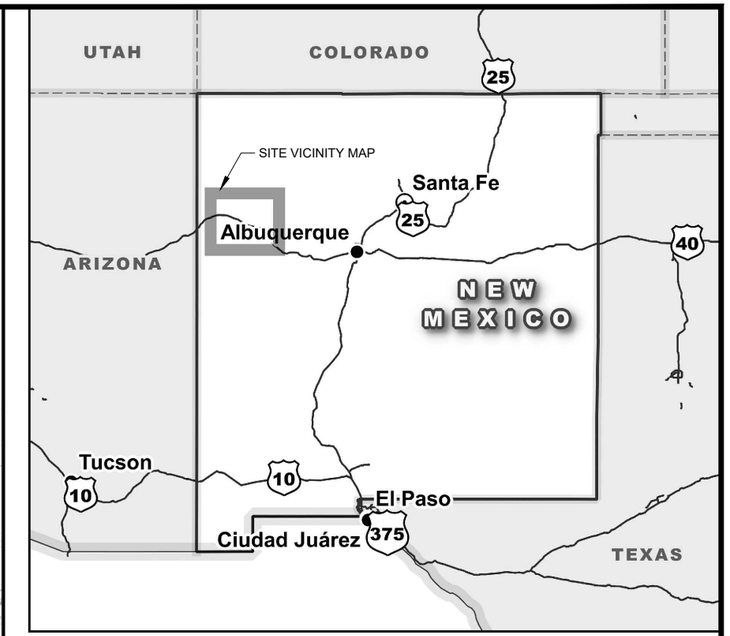
FIGURES



SITE VICINITY MAP

5 0 5 Miles



SITE LOCATION MAP

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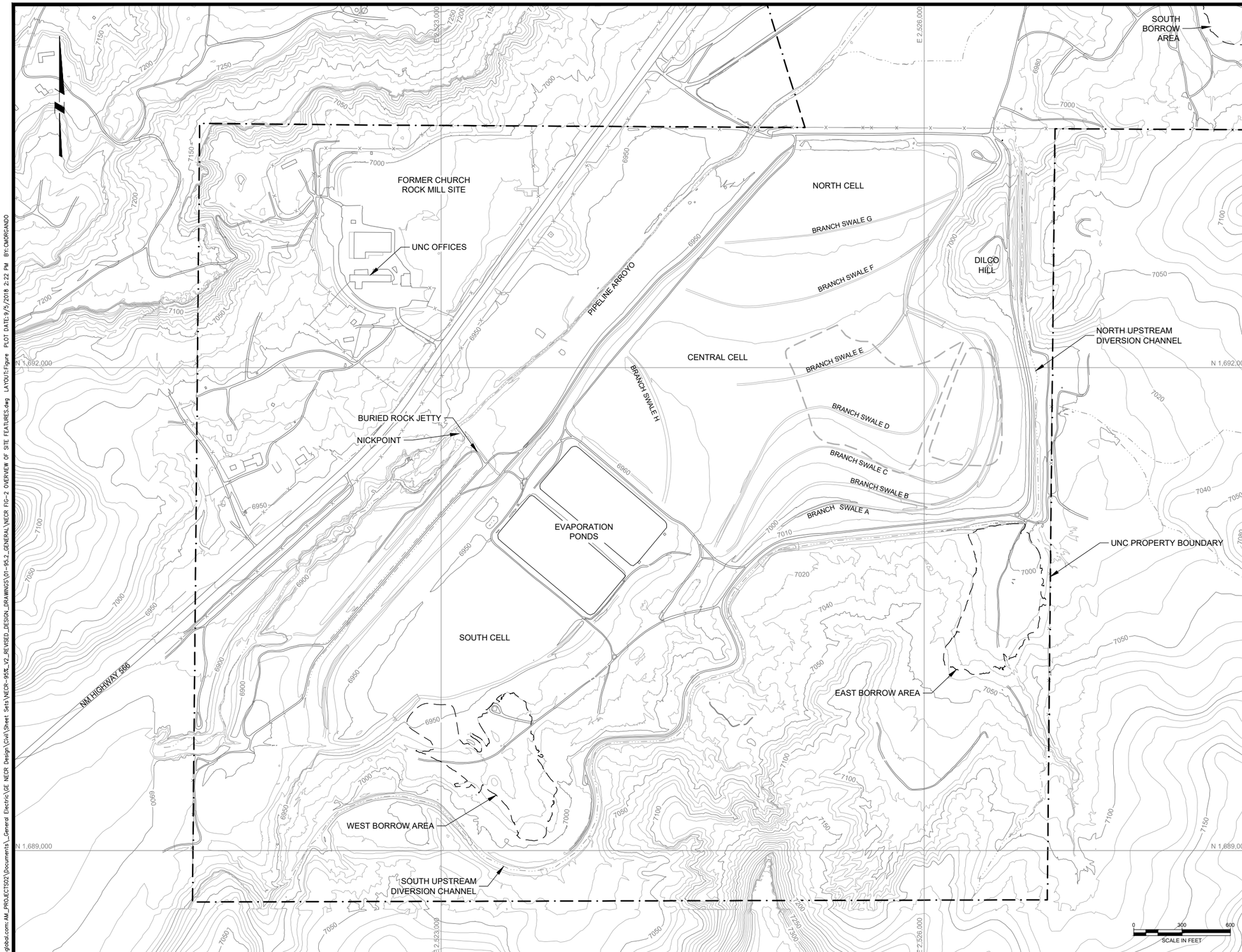
C:\pwworkdir\d0484697\NEOR FIG-1 LOC MAP.dwg LAYOUT:Figure PLOT DATE:8/31/2018 3:26 PM BY:CMORGANDO



LEGEND:

- 7200 — EXISTING GROUND SURFACE CONTOUR & ELEVATION, FEET
- UNC PROPERTY BOUNDARY
- EXISTING ROAD
- NATURAL DRAINAGE
- EXISTING FENCE
- FORMER BORROW PIT BOUNDARY
- STRUCTURES

DESIGNED K REED
CHECKED S DOWNEY
APPROVED J CUMBERS

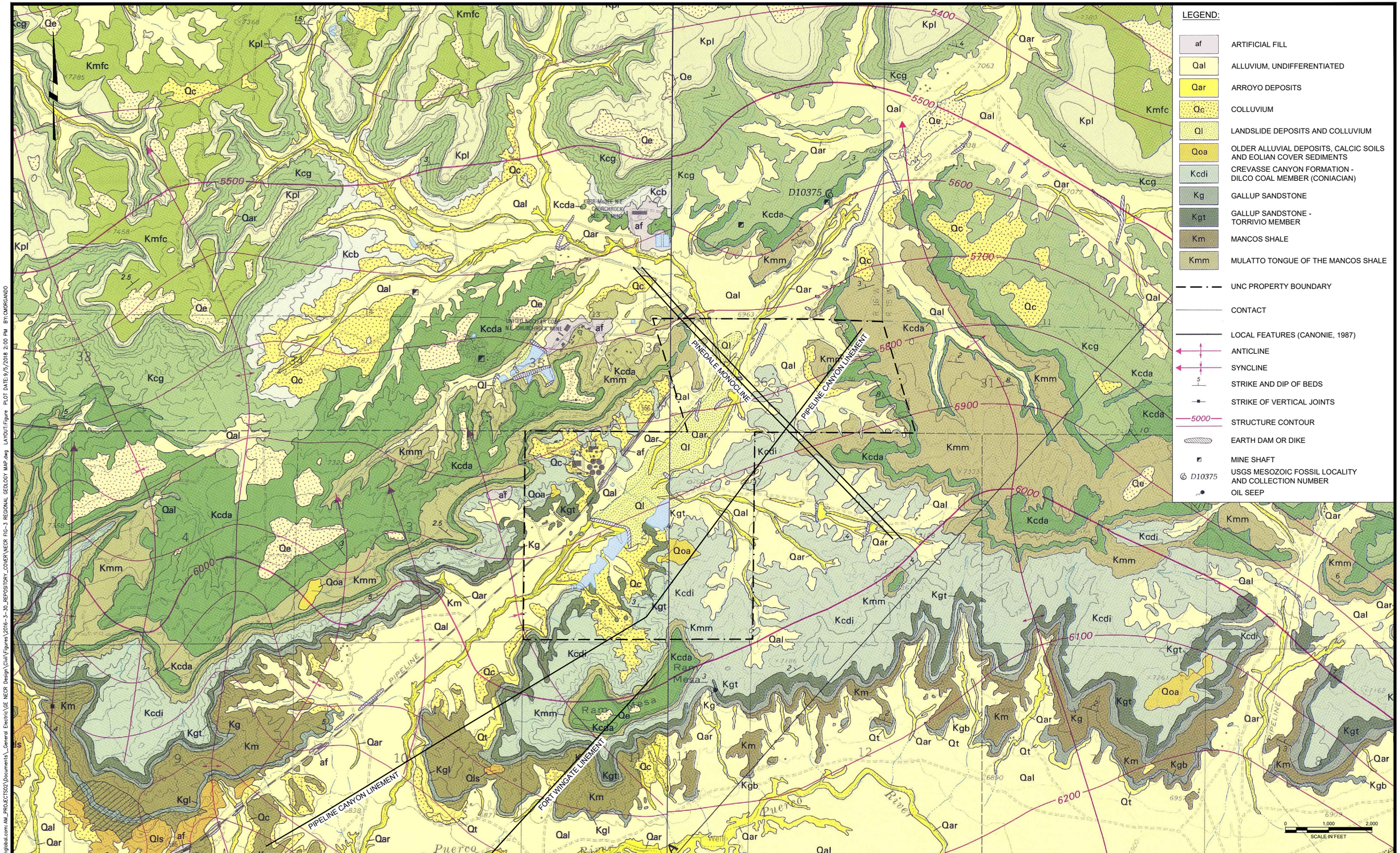
Stantec

UNITED NUCLEAR CORPORATION AND NORTHEAST CHURCH ROCK MINE
McKINLEY COUNTY, NEW MEXICO

LICENSE AMENDMENT REQUEST SUA 1475
EXISTING SITE CONDITIONS

FIGURE
2
233001048
SEPTEMBER 2018

pw:\\smerpwin103.mwlglobal.com:AM_PROJECTS02\Documents_General Electric\GE NECR Design\Civil\Sheet Sets\NECR-95%_V2_REVISED_DESIGN_DRAWINGS\01-95.2_GENERAL\NECR FIG-2 OVERVIEW OF SITE FEATURES.dwg LAYOUT:Figure PLOT DATE:9/5/2018 2:22 PM BY:CMORGANDO



REFERENCE:
USGS GEOLOGICAL MAP OF THE OAK SPRING QUADRANGLE
McKINLEY COUNTY, NEW MEXICO (1984)

DESIGNED J CUMBERS
CHECKED K REED
APPROVED C STRACHAN

Stantec

UNITED NUCLEAR CORPORATION AND NORTHEAST CHURCH ROCK MINE
McKINLEY COUNTY, NEW MEXICO

LICENSE AMENDMENT REQUEST SUA 1475
REGIONAL GEOLOGY MAP

FIGURE
3
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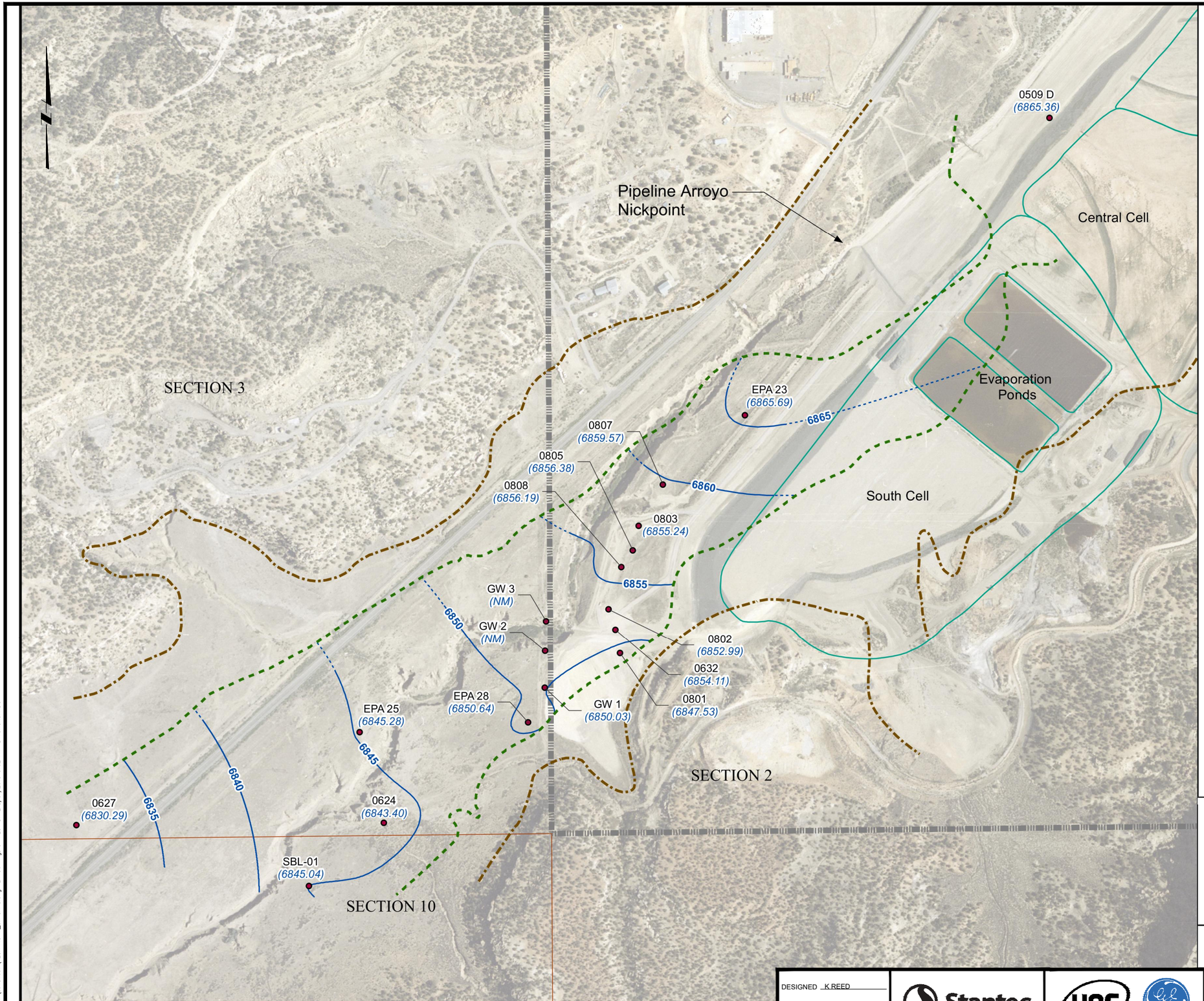
SEPTEMBER 2018

pw:\\amerpwint03.mwhglobal.com:AM_PROJECTS02\Documents_General Electric\GE NECR Design\Civil\Figures\2016-3-30_REPOSITORY_COVER\NECR FIG-3 REGIONAL GEOLOGY MAP.dwg LAYOUT:Figure PLOT DATE:9/5/2018 2:00 PM BY:CMORGANDO



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Legend

- Southwest Alluvium Monitoring Well
- Groundwater Elevation Contour
- Inferred Groundwater Elevation Contour
- Approximate Extent of Alluvium
- Approximate Extent of Saturated Alluvium
- Property Boundary
- Section Boundary
- Cell Boundary

Notes:

1. Groundwater elevation values are displayed in feet above mean sea level.
2. Well names are displayed with black text.
3. Groundwater elevations are shown with blue text and enclosed in parentheses.
4. Aerial photo taken on August 1, 1996.
5. NM – Not Measured

0 250 500 1,000
Feet

Southwest Alluvium Potentiometric Surface Map, October 2017

United Nuclear Corporation Church Rock Site, Church Rock, New Mexico

HATCH CHESTER
Member of the Hatch Group

DESIGNED K REED
CHECKED S DOWNEY
APPROVED J CUMBERS

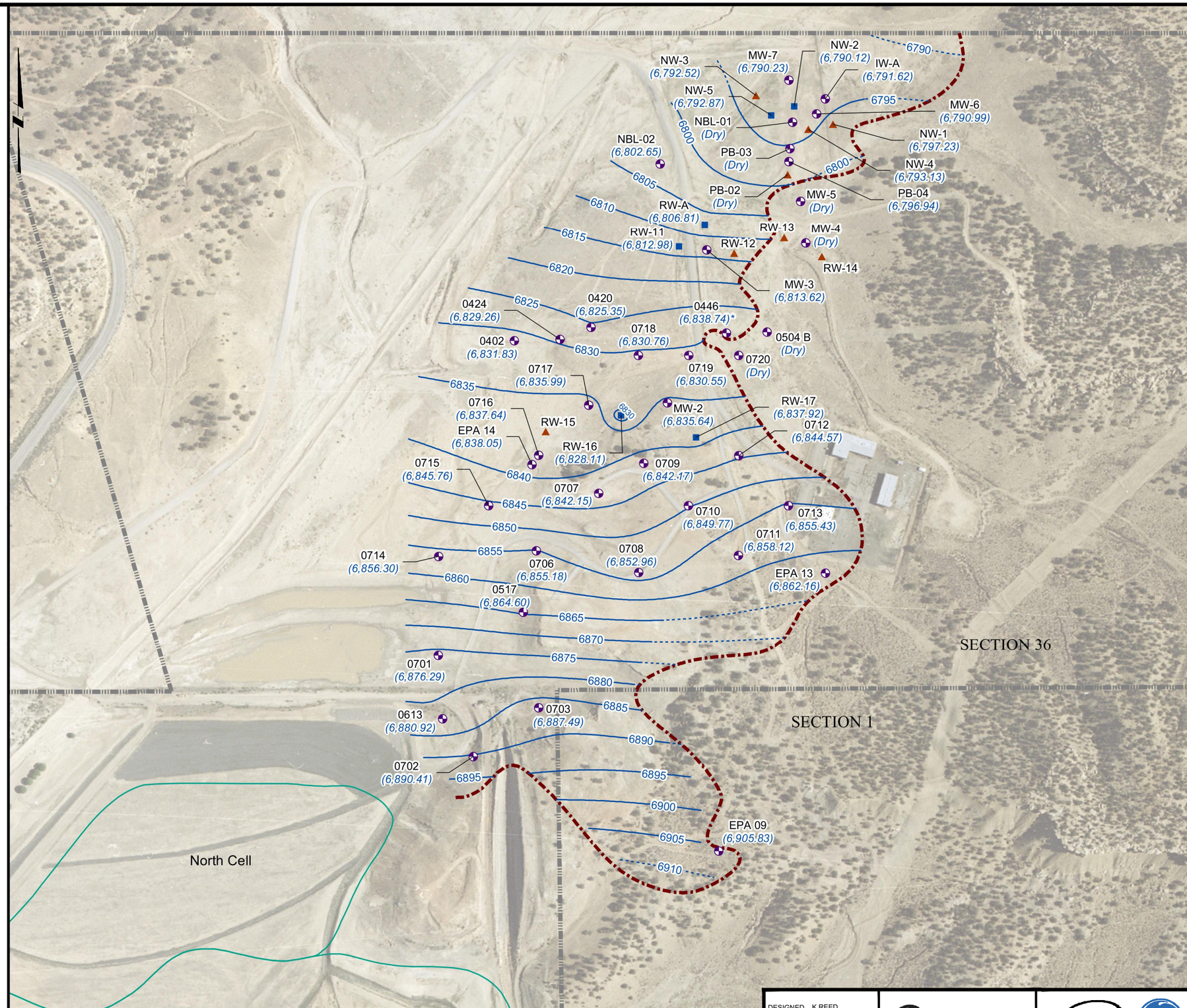
Stantec

UNITED NUCLEAR CORPORATION AND NORTHEAST CHURCH ROCK MINE
McKINLEY COUNTY, NEW MEXICO

LICENSE AMENDMENT REQUEST SUA 1475
SOUTHWEST ALLUVIUM POTENTIOMETRIC SURFACE MAP

FIGURE
4
233001048
SEPTEMBER 2018

C:\pwworkdir\d0484697\NECR FIG-4_3A SAPS.dwg LAYOUT:Figure PLOT DATE:8/31/2018 5:02 PM BY:DMORGANDO



Z:\databases\6209_GE_CR\GISDATA\arcmap_doc\2017_annual_report\f36_z3_piezo.mxd; January 8, 2018

Legend

- Monitoring Well
- Non-Pumping Well
- Pumping Well
- Cell Boundary
- Property Boundary
- Approximate Eastern Boundary of Zero Saturation
- Groundwater Elevation Contour
- Inferred Groundwater Elevation Contour
- *(6905.83)* Measured Groundwater Elevation

Notes:

1. Groundwater elevation values are displayed in feet above mean sea level.
2. Well names are displayed with black text.
3. Aerial photo taken on August 1, 1996.
4. * The measured Well 0446 groundwater elevation is suspect and not deemed to be representative of actual conditions. The estimated actual groundwater elevation at this location is approximately 6831 feet amsl.

0 200 400 800
Feet

Zone 3 Potentiometric Surface Map, October 2017

United Nuclear Corporation Church Rock Site, Church Rock, New Mexico

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Member of the Hatch Group

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UNC

UNITED NUCLEAR CORPORATION AND NORTHEAST CHURCH ROCK MINE
McKINLEY COUNTY, NEW MEXICO

LICENSE AMENDMENT REQUEST SUA 1475
ZONE 3 POTENTIOMETRIC SURFACE MAP

FIGURE
5
233001048
SEPTEMBER 2018

C:\pwworkdir\d0484697\NECR FIG-5_36 ZPSM.dwg LAYOUT:Figure PLOT DATE:8/31/2018 5:08 PM BY:CMORGANDO

Z:\databases\6209_GE_CR\GISDATA\arcmap_doc\2017_annual_report\f46_z1_piezo.mxd; January 8, 2018

Legend

- Zone 1 Monitoring Well
- Groundwater Elevation Contour
- Inferred Groundwater Elevation Contour
- Cell Boundary
- Property Boundary

Notes:

1. Groundwater elevation values are displayed in feet above mean sea level.
2. Well names are displayed with black text.
3. Groundwater elevations are shown with blue text and enclosed in parentheses.
4. Aerial photo taken on August 1, 1996.

0 250 500 1,000
Feet

Zone 1 Potentiometric Surface Map, October 2017

United Nuclear Corporation Church Rock Site, Church Rock, New Mexico

HATCH CHESTER
Member of the Hatch Group

DESIGNED K REED
CHECKED S DOWNEY
APPROVED J CUMBERS

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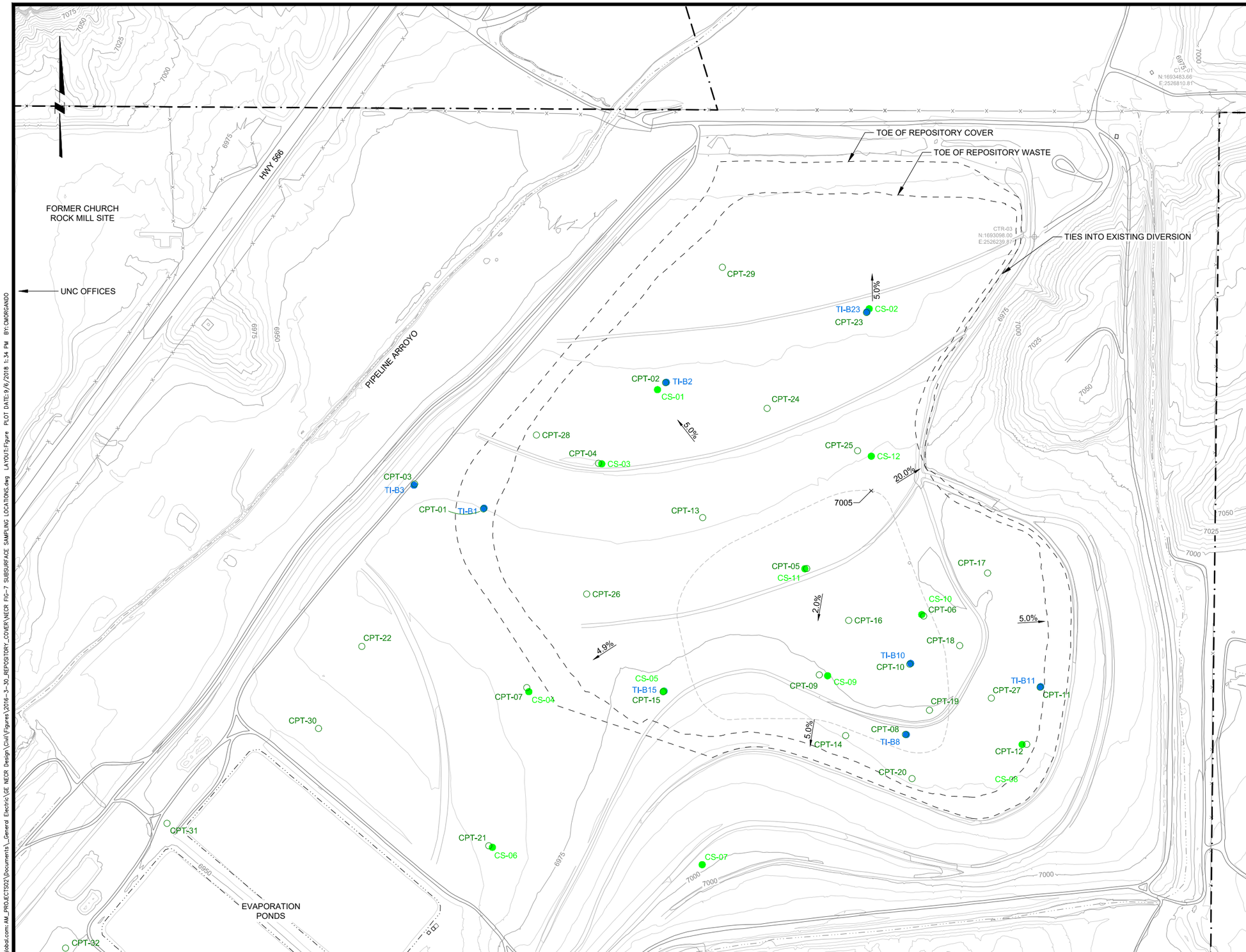
UNITED NUCLEAR CORPORATION AND NORTHEAST CHURCH ROCK MINE
McKINLEY COUNTY, NEW MEXICO

LICENSE AMENDMENT REQUEST SUA 1475
ZONE 1 POTENTIOMETRIC SURFACE MAP

FIGURE
6
233001048
SEPTEMBER 2018

C:\pwworkdir\d0484697\NECR FIG-6_46 ZPSM.dwg LAYOUT:Figure PLOT DATE:9/31/2018 5:14 PM BY:DMORGANDO



LEGEND:

- 7200 — EXISTING GROUND SURFACE CONTOUR & ELEVATION, FEET
- 7200 — PROPOSED SURFACE CONTOUR & ELEVATION, FEET
- EXISTING ROAD
- EXISTING DRAINAGE
- FENCE
- BOUNDARY OF REPOSITORY
- TI-B## — BOREHOLE SAMPLING LOCATION (MWH PDS, 2014)
- CPT-## — CONE PENETROMETER TEST LOCATION (MWH PDS, 2014)
- CS-## — COVER TEST PIT SAMPLING LOCATION (MWH PDS, 2014)
- UNC PROPERTY BOUNDARY

0 150 300
SCALE IN FEET

DESIGNED J CUMBERS
CHECKED K REED
APPROVED C STRACHAN

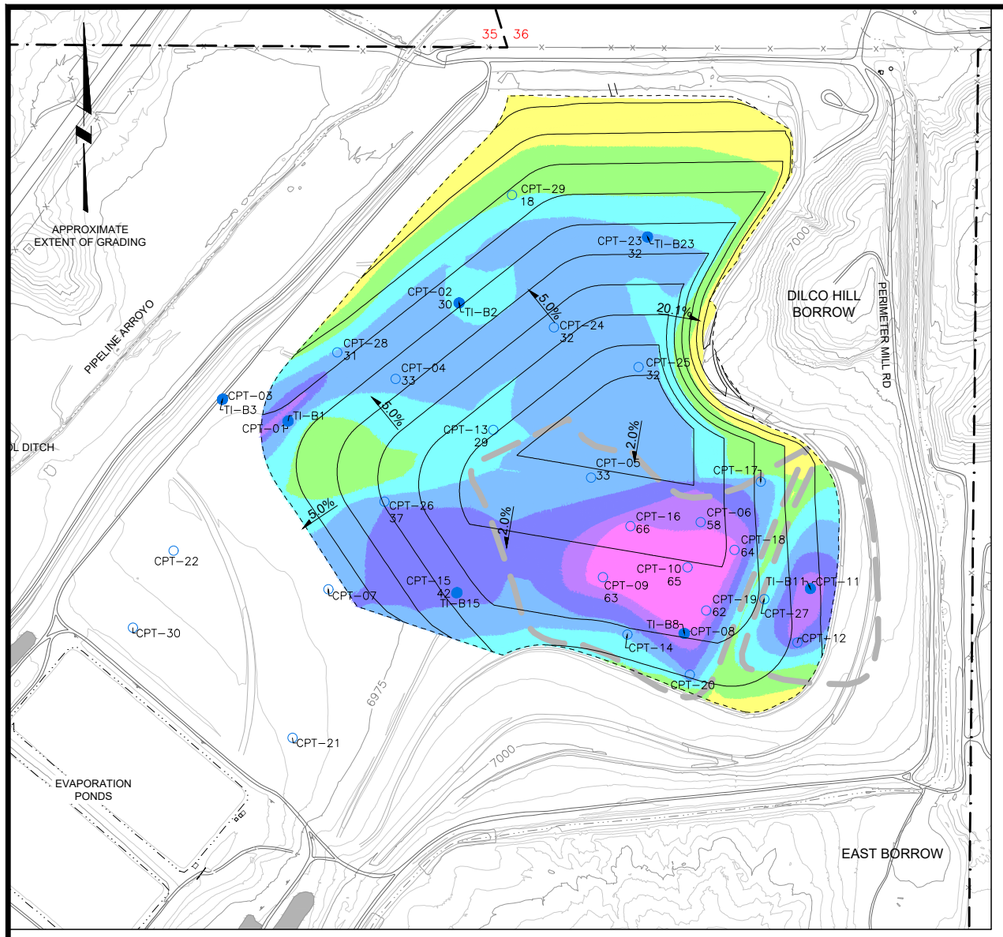
Stantec

UNITED NUCLEAR CORPORATION AND NORTHEAST CHURCH ROCK MINE
McKINLEY COUNTY, NEW MEXICO

LICENSE AMENDMENT REQUEST SUA 1475
SUBSURFACE SAMPLING LOCATIONS

FIGURE
7
233001048
SEPTEMBER 2018

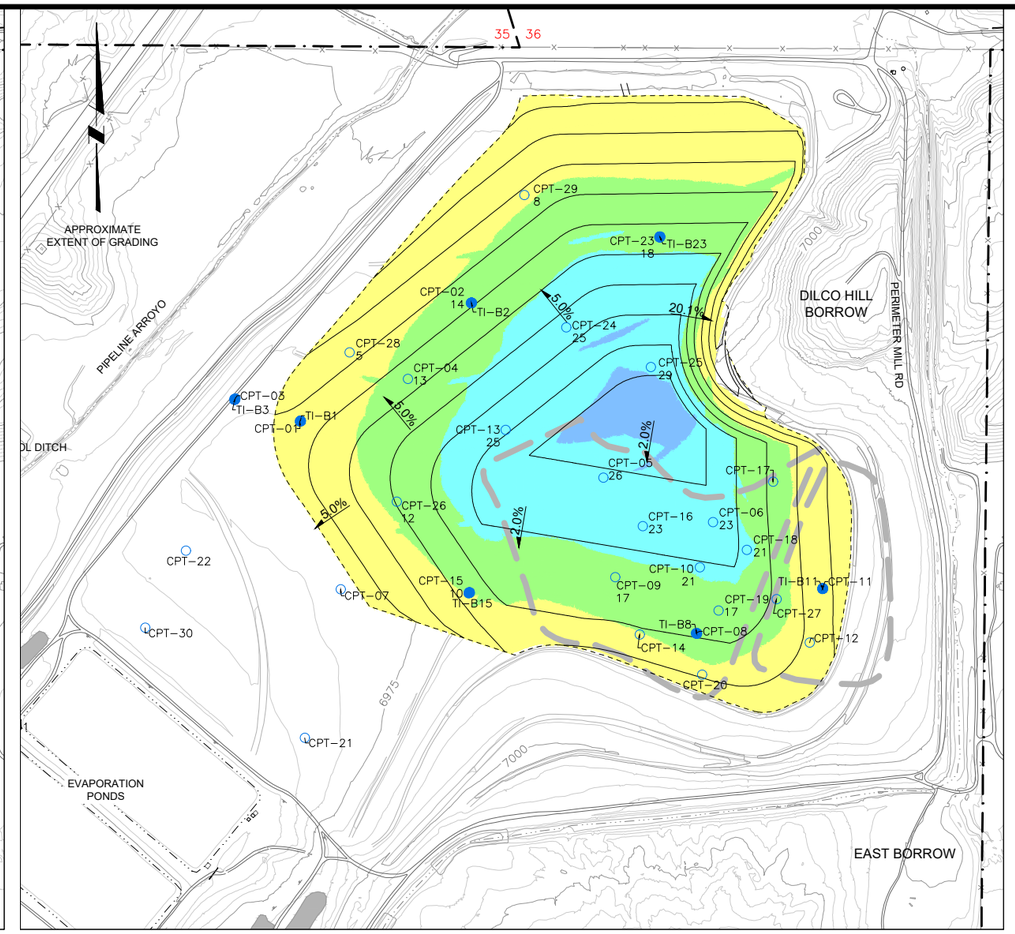
pw:\\amerpwint03.mwlglobal.com:AM_PROJECTS\22\Documents_General Electric\GE NEOR Design\Civil\Figures\2016-3-30_REPOSITORY_COVER\NEOR FIG-7 SUBSURFACE SAMPLING LOCATIONS.dwg LAYOUT:Figure PLOT DATE:9/6/2018 1:34 PM BY:OMORGANDO



PLAN - FINISHED GROUND vs BASE OF TAILINGS ISOPACH

0 250' 500'

CONTOUR INTERVAL = 5 FT



PLAN - FINISHED GROUND vs EXISTING GROUND

0 250' 500'

CONTOUR INTERVAL = 5 FT

LEGEND

- 6980 — EXISTING GROUND SURFACE CONTOUR ELEVATION, FT
- 6980 — FINISHED GROUND SURFACE CONTOUR, FT
- EXISTING DRAINAGE
- EXISTING FENCE
- EXISTING ROAD
- UNC PROPERTY BOUNDARY
- BOUNDARY OF REPOSITORY
- CPT-XX 4 — PRE-DESIGN STUDIES (MWH, 2014); REPOSITORY FILL THICKNESS WITH OR WITHOUT EXISTING TAILINGS
- FORMER BORROW PIT BOUNDARY (AERIAL PHOTO 11/4/80)
- TI-B## — BOREHOLE SAMPLING LOCATION (MWH PDS, 2014)

ISOPACH DEPTH VALUES

MIN ELEV (ft)	MAX ELEV (ft)	
0	10	
10	20	
20	30	
30	40	
40	50	
50	60	
60	70	
70	80	

DESIGNED K REED
CHECKED S DOWNEY
APPROVED J CUMBERS

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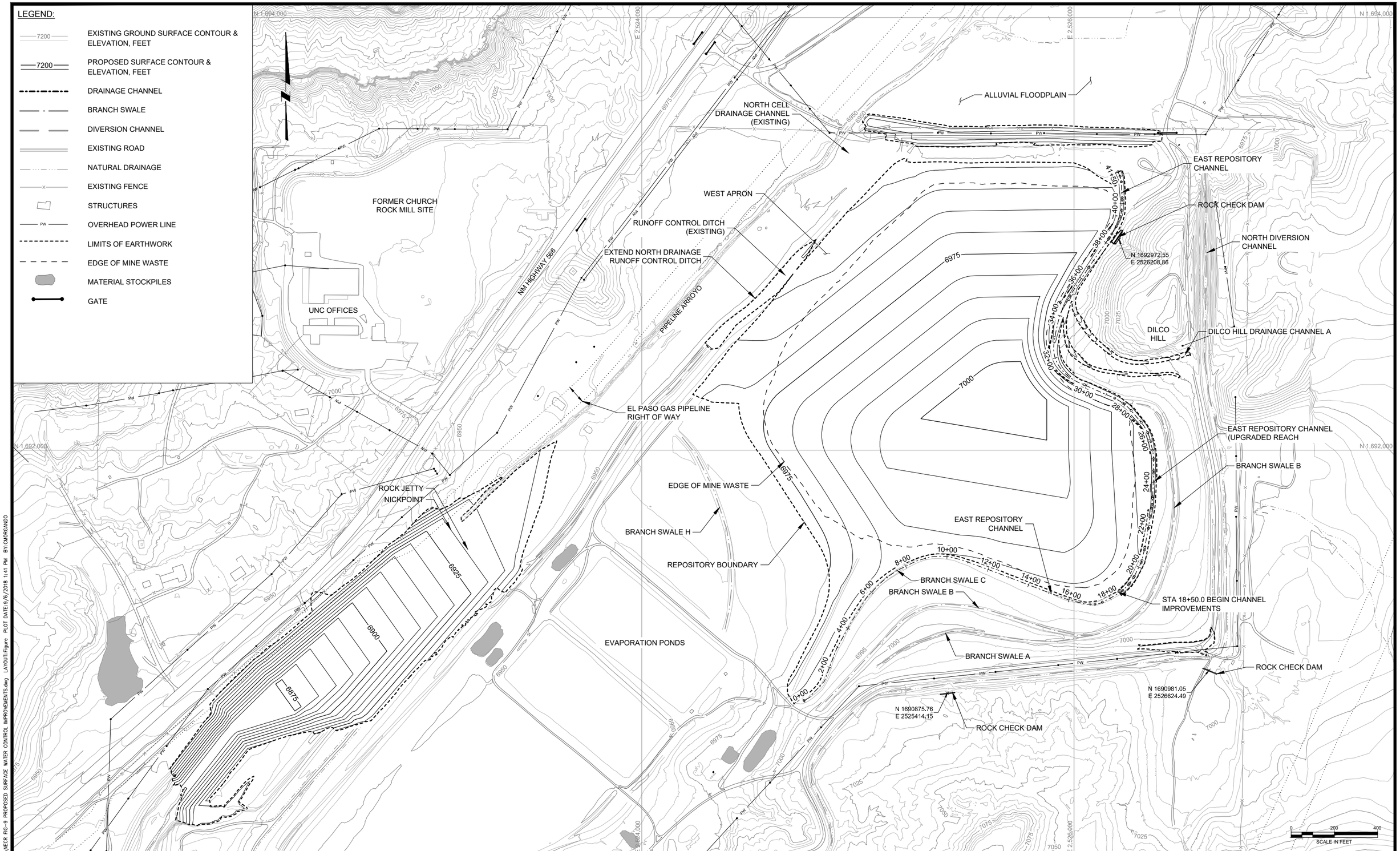
UNITED NUCLEAR CORPORATION AND NORTHEAST CHURCH ROCK MINE
McKINLEY COUNTY, NEW MEXICO

LICENSE AMENDMENT REQUEST SUA 1475
MILL SITE REPOSITORY AREA PROPOSED FILL

FIGURE
8
233001048
SEPTEMBER 2018

LEGEND:

- EXISTING GROUND SURFACE CONTOUR & ELEVATION, FEET (7200)
- PROPOSED SURFACE CONTOUR & ELEVATION, FEET (7200)
- DRAINAGE CHANNEL
- BRANCH SWALE
- DIVERSION CHANNEL
- EXISTING ROAD
- NATURAL DRAINAGE
- EXISTING FENCE
- STRUCTURES
- OVERHEAD POWER LINE
- LIMITS OF EARTHWORK
- EDGE OF MINE WASTE
- MATERIAL STOCKPILES
- GATE



DESIGNED K REED	Stantec	UNITED NUCLEAR CORPORATION AND NORTHEAST CHURCH ROCK MINE McKINLEY COUNTY, NEW MEXICO LICENSE AMENDMENT REQUEST SUA 1475 MILL SITE REPOSITORY AREA PROPOSED STORMWATER CONTROLS	FIGURE 9
CHECKED S DOWNEY			233001048
APPROVED J CUMBERS			SEPTEMBER 2018

C:\pwworkdir\d0484697\NEW FIG-9 PROPOSED SURFACE WATER CONTROL IMPROVEMENTS.dwg LAYOUT:Figure PLOT DATE:9/6/2018 1:41 PM BY:CMORGANDO