5

5.0 FINAL RECLAMATION PLAN

Final reclamation activities will be initiated after completing interim stabilization measures and interim stabilization monitoring. Final reclamation of the South Cell will not be conducted until the NRC CAP and the EPA remedial actions, described in Section 6.0 of this report, have been completed. The site configuration following completion of final reclamation activities will be as shown on Figures 5-1 and 5-2.

Final reclamation activities will include the following:

1. Complete backfilling and grading Borrow Pit No. 2 to final contours,
2. Regrade and place the radon attenuation soil cover over the evaporation ponds,
3. Place the final 6 inches of radon attenuation soil cover and the soil/rock matrix erosion protection cover,
4. Construct surface water control channels, diversion ditches, drainage swales, Pipeline Arroyo low-flow channel, and the buried jetty, including placing riprap,
5. Revegetate disturbed areas and secure reclaimed areas.

This section of the reclamation plan describes work performed as part of final reclamation activities. It provides the technical basis for designing those activities, particularly with respect to compliance with 10 CFR 40, Appendix A. The plan meets the objectives of 10 CFR 40, Appendix A, to the extent practicable, by minimizing final slopes, containing and controlling major flood events, minimizing radon emanation from the tailings disposal area, and maximizing stability of the reclamation for the long-term.

5.1 Completion of Backfilling and Grading Borrow Pit No. 2

After dewatering in 1991, backfilling of Borrow Pit No. 2, shown on Figure 5-1, began with mill demolition debris, ore and tailings from within mill area drainages, excess material from the ore storage pad, and alternating layers of backfill soil from the site. During final reclamation, additional backfill will be placed and compacted in the borrow pit to meet the contours shown on Figure 5-1. Soil from the adjacent clean-soil stockpile will be used as required. An estimated 155,000 cy of additional fill will be required to bring Borrow Pit No. 2's surface up to the planned final grade.

5.2 Regrading Evaporation Ponds

At the completion of the seepage remediation program, described in Section 6.0 of this report, the evaporation ponds constructed in the South Cell (Figure 4-3) will be closed to allow placement of the final reclamation cover. Closure of the evaporation ponds will include the following tasks:

1. Grading the pond embankments into the ponds by spreading the embankment material in lifts and compacting the material with mechanical equipment.

2. Covering the compacted materials with 1.5 feet of radon attenuation soil cover. This soil cover will be placed using the same technical specifications used in placing the soil cover over tailings. The 6-inch soil/rock matrix will be placed over the ponds when it is placed on the remainder of the South Cell.

The hypalon liners will be buried in-place in the pond backfill. The closed ponds will be graded to provide a uniform surface that conforms to the rest of the regraded tailings cells, shown on Figure 5-1.

5.3 Final Tailings Cover Design

The final radon attenuation soil cover incorporates the previously constructed interim cover by adding 6 inches of radon attenuation soil cover to the 12 inches of cover placed during the interim stabilization phase (Figure 5-3). In addition, a 6-inch-thick soil/rock matrix erosion protection layer will be constructed using on-site soils from various net excavation areas (drainage channels) and the existing soil stockpile. Rock used in the soil/rock matrix will be derived from off-site quarries. This erosion protection layer will be installed over the 1.5-foot radon attenuation soil cover, in accordance with the Construction/Technical Specifications provided in Appendix B, and integrated as part of the final soil cover. The resulting 2-foot-thick cover meets the design requirements stipulated by 10 CFR 40, Appendix A, in that the first 1.5 feet of radon attenuation soil cover provides reasonable assurance for effectively controlling radiological hazards for 1,000 years, and that releases of Rn-222 to the atmosphere will not exceed an average release rate of 20 picoCuries per square meter per second ($pCi/m^2/sec$) throughout the effective design life of the cover. Also, the 6-inch soil/rock matrix is designed to protect the radon attenuation layer for 1,000 years.

The design procedure employed for the final reclamation cover consisted of the following basic steps, using the RADON computer model:

1. Determining the characteristics of coarse and fine tailings to identify the source term used in the RADON computer model for radon flux calculations from the tailings.

2. Determining the characteristics of soil borrow materials to calculate the bulk radon diffusion coefficient for the soil cover.

3. Identifying the required thickness of the soil cover to limit radon flux to 20 $pCi/m^2/sec$ in excess of that emitted by the cover soils.

4. Calculating the total cover thickness to account for long-term performance, infiltration, dispersion and frost heave.

The proposed design, submitted to the NRC in 1987, used soil characteristics determined from the field explorations conducted during the reclamation investigation and previous site investigations. Subsequently, the design was modified using actual field construction data obtained during the two years of interim cover placement over the North and Central Cells.

5.3.1 Tailings Characteristics

Based on characterization of ore during mill design (Kaiser, 1976), mineral composition of the ore host rocks consists of 78-79 percent quartz, 2-3 percent calcite, and 18-20 percent kaolinite and feldspars. Accordingly, the tailings are expected to approximately reflect these coarse-to-fine ratios of about 80 percent coarse tailings and 20 percent fine tailings.

The coarse tailings typically exhibit lower radon flux than the fine-grained fraction. Regrading the tailings as part of interim stabilization activities was designed to cover the limited fine-grained tailings deposits with a minimum seven-foot thickness of coarse tailings and/or other fill materials, minimizing radon flux from the tailings and reducing the ultimate required soil-cover thickness.

In the first two years of tailings regrading, the quantity of coarse tailings available in the work area was insufficient to cover the fines with seven feet of coarse tailings. Consequently other fill materials, including wind-blown tailings, affected soils, and clean soil were used to provide the additional fill necessary. The interim stabilization soil cover was placed over this fill material. The imported fill materials have much lower radium contents and lower diffusion coefficients than those modeled in the soil cover design which assumed a coarse tailings layer of seven feet thickness over the fine tailings. Use of fill material to augment the coarse tailings in the minimum 7-feet-thick layer of material over the fine tailings resulted in a layer with greater radon-attenuation properties than properties of the coarse tailings materials used for design purposes.

5.3.1.1 Coarse Tailings

A laboratory testing program was conducted to evaluate various geotechnical and radiological parameters of the coarse tailings, which affect cover design. Table 5.1 presents the results of these tests.

The long-term moisture content of the coarse tailings can be expected to approach that of natural near-surface soils with a similar particle size gradation. The long-term moisture content of the coarse tailings was determined based on NRC acceptable methodology, as described in Regulatory Guide 3.64 or equivalent.

NRC guidance (Regulatory Guide 3.64) describes three acceptable methodologies for evaluating long-term moisture contents:

1. In-situ moisture content
2. The Rawls equation
3. Laboratory testing

The use of the laboratory test methodology was discounted in conducting the evaluation as a result of the NRC's review comments regarding its representativeness.

In-situ moisture contents of 79 samples of native sandy soils on-site were compiled to estimate the long-term moisture content of the coarse tailings, as shown in Appendix D. The moisture contents of these soil samples are representative of long-term moisture contents of coarse-grained tailings. (These soils were chosen for use in establishing an average value because their texture and gradation were similar to that of the coarse tailings.) The average in-situ moisture content was found to be 10.1 percent by weight. This value was used for the long-term moisture of the coarse-grained tailings in the June 1987 proposed plan. In reviewing that design, the NRC questioned whether the value was truly representative of long-term conditions.

To address NRC's review comments, another acceptable method of evaluating long-term moisture was implemented. The Rawl's empirical equation was then used to compute the long-term moisture content of the coarse-grained tailings, conservatively assuming only a 15 percent, minus 200 fraction. The resultant long-term moisture content was determined to be 6.5 percent by weight. This value is close to the NRC default value of 6 percent. The default value of 6 percent was used to evaluate the radon barrier. The diffusion coefficient was also calculated using this more conservative long-term moisture content.

Test results presented in Table 5.1 provide dry-bulk densities obtained from in-situ coarse tailings samples during investigation activities. As indicated in the Construction/Technical Specifications (Appendix B), the surface of the graded coarse tailings are to be compacted to a minimum 95 percent of the maximum dry density, as determined by the Standard Proctor compaction method (ASTM D 698). The dry-bulk density of coarse tailings used for the cover design was 97.5 pcf, and can be obtained by track-mounted dozers and other construction equipment. This density is consistent with observed in-place densities of the coarse tailings as identified in Table 5.1.

Given the dry density and the moisture content, various other physical parameters were calculated for the coarse tailings as follows:

Dry Bulk Density at 95 Percent Compaction:	97.5 pcf, (1.56 g/cm^3)
Long-term Moisture Content (NRC default value):	6 percent
Void Ratio:	0.79
Porosity:	47 percent

The porosity value of 47 percent and the resulting void ratio of 0.79 are the upper-bound 95 percent confidence interval values. Using these parameters, a radon diffusion coefficient was computed for the coarse tailings using guideline procedures stipulated in NUREG/CR-3533 (NRC, 1984b) for radon attenuation cover design. The radon diffusion coefficient of .0360 cm^2/sec for the coarse tailings was computed using the RADON computer model. This value was used for cover design purposes and is higher (i.e., more conservative) than the majority of actual values measured in the laboratory (see Table 5.1). Use of this value produces a thicker soil cover than if the measured laboratory values were used. However, the values measured in the laboratory were typically tested at higher moisture contents than the default value of 6 percent or the calculated value of 6.5 percent. As discussed previously, the representativeness of values measured in the laboratory had been questioned by NRC. Nonetheless, the measured bulk diffusion coefficient of 0.023 cm^2/sec for the coarse tailings at a moisture content of 10.5 percent is lower than the 0.036 cm^2/sec used for cover design purposes, demonstrating that the cover design values for diffusion coefficient and moisture content are conservative.

The radium content and radon emanation coefficient were determined in the laboratory by Rodgers and Associates, Salt Lake City, Utah (United Nuclear Corporation, HSA, 1985) for the coarse tailings. These values were used in the RADON computer code to determine radon flux through the soil cover. The tested coarse tailings have an average radium content of 154 pCi/g, and a radon emanation coefficient of 0.26. This value represents the average of nine samples, as shown in Table 5.1.

5.3.1.2 Fine-Grained Tailings

The same three methods used to determine long-term moisture content for coarse-grained tailings were also used for fine-grained tailings. The use of this approach is also the result of NRC review of the design proposed in 1987. These methods included use of Equation 5 from NRC Regulatory Guide 3.64 (Rawl's equation), in-situ moisture content measurements, and laboratory testing to determine the wilting-point moisture content. Comparison of these methods substantiates the long-term moisture content selected for input to the RADON model as representative of long-term conditions. The following discussion provides the result of the comparison and identifies the parameters used to design the soil cover.

Table 5.2 and Appendix D present the in-situ moisture content of fine-grained tailings samples. The average in-situ moisture content was 38.7 percent with a 95 percent confidence interval of 29.7 percent to 47.7 percent. A value of 29.7 percent was used for comparison to the calculated long-term moisture content.

Using the empirical Rawl's equation for comparison, the volumetric long-term moisture content of the fines fraction for the fine-grained tailings was calculated at 32.6 percent. Converting this volumetric moisture content to a long-term weight-ratio moisture content yields a moisture content of 23.6 percent by weight. Appendix C presents this calculation. This value is comparable to a long-term, weight-ratio moisture content of 29.7 percent derived from in-situ data for the fine-grained tailings.

On further review, the NRC commented that although the fine-grained tailings classify as a clay, the total fines fraction is not appropriate for use in the Rawl's equation. Further, only the clay portion of the fines fraction [0.002 millimeter (mm) or less in size] should be used and, therefore, the value of 23.6 percent is not appropriate. Since the samples used in calculating long-term moisture content were not all clay-size, the use of a lower moisture content (i.e., the default value) was considered more appropriate by NRC. Therefore, the cover was designed using the NRC's 6 percent default value. The diffusion coefficient for the fine tailings was also calculated using the 6 percent long-term moisture content.

Table 5.2 also presents additional fine-grained tailings characteristics used to select parameters for cover design. The average dry-bulk density for the selected samples was found to be 86 pcf. A review of available data indicates that the dry bulk density typically ranges from as low as 74 pcf to as high as 97 pcf (Appendix C). Although the design cover thickness is not highly sensitive to the dry bulk density of the fine-grained tailings, the value of the average dry-bulk density was increased 5 percent (i.e., to 90 pcf) when used for design purposes to account for consolidation. The 48 percent porosity of fine-grained tailings was calculated using the calculated bulk density and the average specific gravity.

Geotechnical parameters of fine-grained tailings for calculating an associated bulk diffusion coefficient using the RADON computer model are summarized as follows, based on data presented in Table 5.2, and the previously described selection criteria:

Maximum Bulk Dry Density Plus 5 percent:	90 pcf (1.45 g/cm^3)
Weight Percent Moisture (NRC default value):	6 percent
Porosity:	48 percent

The bulk radon diffusion coefficient was calculated using the RADON computer model with the default value for the moisture content and calculated porosity. The diffusion coefficient was calculated to be 0.0400 cm^2/sec, which is much higher (i.e., more conservative) than the actual measured value.

The radium content and radon emanation coefficient were determined in the laboratory by Rodgers & Associates, Inc., Salt Lake City, Utah, for United Nuclear (United Nuclear Corporation, 1985) on the fine-grained tailings. These values are used in the RADON computer model to determine radon flux through the soil cover.

The fine-grained tailings were tested and found to have an average radium content of 547 pCi/g, and a radon emanation coefficient of 0.26. These values represent the average of seven samples, as shown in Table 5.2.

5.3.2 Radon Attenuation Soil Cover Characteristics

Soils used for the radon attenuation layer (i.e., 1.5 feet) of the reclamation cover will be obtained from the north area flood plain, excavation of several surface water channels, the tailings embankment and the existing soil stockpile. The geotechnical characteristics of these soils for use in cover design were defined by a field investigation program, as documented in Appendix A, a laboratory testing program, and a literature review of previous geotechnical engineering reports (Sergent, Hauskins, and Beckwith, 1979).

As described in Section 3.0 and indicated by the laboratory data in Appendix A, representative borrow soils contain approximately 40 percent sand and 60 percent silts and clays. Representative soils with this approximate gradation were tested to identify anticipated properties of the soil cover summarized as follows:

Maximum Bulk Dry Density (Average value):	113.7 pcf
Bulk Dry Density at 95 percent Compaction:	108.0 pcf (1.73 g/cm^3)
Natural Moisture Content:	13.4 percent
Void Ratio:	0.65
Long-term Moisture Content:	12.9 percent
Porosity:	33.0 percent
Specific Gravity:	2.63
Degree of Saturation:	54.0 percent

The cover soil bulk dry density was determined by considering only test results from materials that satisfy the specification's requirements. Figure 3-2 shows acceptable gradation ranges for the soil cover material as approved by NRC. Appendix C presents the calculation of dry-bulk density.

5.3.2.1 Diffusion Coefficient

The diffusion coefficient has the greatest impact on the cover thickness. In refinements to the soil-cover design since submitting the proposed plan in 1987, the diffusion coefficient was calculated using higher in-place densities derived from actual interim cover testing (Appendix C), and lower porosities compared to the more conservative parameters estimated in the original design. The original estimated in-place density and porosity were 99.0 pcf and 0.39, respectively. The revised in-place density and porosity using actual soil data from the interim stabilization cover already placed are 108.0 (95 percent of maximum dry density) and 0.33, respectively. This revision results in a revised, lower diffusion coefficient of 0.0036 cm^2/sec compared to an original diffusion coefficient of 0.0093 cm^2/sec (Appendix C). Table 5.3 summarizes the values used in the final soil-cover design as approved by NRC. The lower diffusion coefficient reduces the amount of radon that can emanate through the soil, which significantly decreases the thickness of the soil cover needed.

5.3.2.2 Soil Cover Long-Term Moisture

The long-term moisture content of the soil-cover material was evaluated using three of the NRC-accepted methodologies presented in NRC Regulatory Guide 3.64 (i.e., the Rawl's equation, in-situ moisture contents, and laboratory-derived, long-term moisture contents). Use of the Rawl's empirical equation produces a long-term, weight-ratio moisture content of 17.9 percent when the average fines fraction from the acceptable soil-cover grain-size envelope is used (Figure 3-2). NRC comments in reviewing the proposed plan indicated that, even though the soil-cover material generally classifies as a clay, the total fines fraction is not appropriate for input into the Rawl's equation. Accordingly, only the clay portion of the fines fraction (0.002 mm or less in size) should be used and, therefore, the value of 17.9 percent was not considered appropriate by the NRC.

Laboratory testing (Method ASTM-D3152) of a representative soil sample to determine the long-term moisture content of the soil cover produced a value of 13.6 percent by weight. Appendix C provides the laboratory test data. The tested sample had a fines fraction of 65 percent. Compared to the allowable soil-cover gradation presented on Figure 3-2, the particle-size distribution of this sample indicates it is acceptable for use in the soil cover. In subsequent review, the NRC commented on the representativeness of the tested soil sample due to the 65 percent fines content, which is near the upper 95 percent confidence limit of fines in the acceptable soil-cover grain-size envelope. Only one long-term moisture content, as determined by laboratory procedures, was available and the NRC questioned its representativeness. In addition, the NRC commented that the Rawl's equation using the fines fraction as input calculates an inappropriate long-term moisture content value. NRC further indicated that this value is not acceptable alone for input to the RADON model.

Evaluation of in-situ moisture content measurements from 119 representative borrow soil samples identified a long-term moisture content of 13.4 percent by weight. This value is the average moisture content of the samples tested, as provided in Appendix D. Samples were obtained from available borrow soils within Pipeline Arroyo, the tailings embankment and the soil stockpile located east of the tailings impoundment.

NRC commented on the representativeness of these soils, and indicated that only samples meeting the acceptable soil cover grain-size envelope and obtained from a depth between 120 centimeters (cm) and 500 cm should be used to evaluate long-term moisture based on observed moisture contents. Evaluation of in-situ moisture content measurements was refined to include only the data meeting these criteria. Forty-seven soil samples were identified as meeting the gradation and depth requirements, and are located in potential borrow areas. The average in-situ moisture content of these samples was 12.9 percent. Appendix D presents this calculation.

The 12.9 percent value was used in designing the radon attenuation layer, and was obtained from the average of the above-mentioned 47 samples for which in-situ moisture data are available. This value was derived using an NRC-accepted methodology, and NRC approved its use as the long-term moisture content. The soil-cover diffusion coefficient was also calculated using this 12.9 percent long-term moisture.

5.3.3 Radon Attenuation Soil Cover Calculation

The required soil-cover thickness for the radon attenuation layer was determined using the RADON computer model for the tailings regrading plan, and tailings and soils characteristics described previously. The radon attenuation layer of the final soil cover, illustrated on Figure 5-3, was designed to limit the long-term radon flux to 20.0 pCi/m^2/sec. Table 5.3 lists the input parameters used for the RADON computation. The radon attenuation soil cover portion of the tailings cover has been reduced to a total thickness of 1.5 feet to be constructed over regraded tailings (Appendix C) as approved by NRC. The radon attenuation layer will be overlain by a 6-inch soil/rock matrix, which will serve as an erosion control layer, as described below. The soil/rock matrix will aid in further reducing radon emanation, but has not been included in the analyses.

The areas within the tailings disposal area with the highest radon source (i.e., the fine-grained tailings areas) will be covered with a minimum of 7 feet of coarse-grained tailings, or other fill material, before placing the soil cover. This layer essentially attenuates all the radon emanating from the fine-grained tailings. The soil cover will, therefore, only be required to attenuate radon emanating for the lower radon source coarse-grained tailings. Placing a minimum 7-foot-thick coarse-grained tailings layer, or other fill material, over the fine-grained tailings enables a thinner, single, radon-attenuating soil layer to be designed, instead of having two soil-cover thicknesses (i.e., one for attenuating radon from coarse-grained tailings and one for the higher radon source, fine-grained tailings).

In addition, the minimum 7-foot-thick layer of material placed over the fine-grained tailings will in actuality consist of a combination of coarse-grained tailings overlain by clean borrow soil. This was the case when regrading the North and Central cells during the initial phases of interim stabilization. Typically, clean borrow soil has been required in excess of the regraded coarse tailings to attain the specified design grades. This excess adds additional conservatism to the cover design, in that the thickness of clean soils over the regraded tailings is typically thicker than the design thickness of 1.5 feet. This conservatism has not been accounted for in the RADON model.

In summary, the final radon attenuation soil cover has been designed using:

1. Conservative radon-source parameter values for the fine- and coarse-grained tailings.

2. A minimum 7-foot-thick layer of coarse-grained tailings over the fine-grained tailings. In many areas, however, this layer will actually consist of a combination of coarse-grained tailings and other fill material. As such, radon source parameter values will be significantly less than has been assumed for the design purposes.

3. Actual soil-cover radon attenuation properties achieved during placement of the interim stabilization cover in the North and Central Cells.

Use of interim soil-cover radon attenuation properties, which is more effective at attenuating radon than those conservatively assumed in the original design, results in a soil cover design thickness of 1.5 feet (Appendix C). This soil cover, along with the soil/rock matrix for erosion protection, will provide a final soil cover that meets the requirements set forth in Appendix A of 10 CFR 40 (Figure 5-3).

The acceptable soil types for use in the soil-cover construction will classify as silty clay (CL), clayey sand (SC), silt (ML), or silty sand (SM), in accordance with the Unified Soil Classification System (USCS). Figure 3-2 shows the gradation limits of the mixed soil to be placed in the soil cover, as approved by the NRC.

5.3.4 Soil Cover Quality Control Program

The quality control program originally outlined in the specifications for the 1987 proposed plan represented the design engineer's minimum acceptable program to confirm that the construction meets the design intent. United Nuclear has been conducting interim reclamation of the tailings impoundment and performing quality control monitoring, at the direction of the NRC, since 1989. A minimum of one foot of radon attenuation soil cover has been placed over regraded tailings in the North and Central Cells. In placing that cover, United Nuclear has conducted a quality control program that exceeds the minimum acceptable program stipulated by the original design specifications, and more closely complies with the guidance provided by NRC's Staff Technical Position Paper (STP) on Testing and Inspection Plans.

The NRC has inspected this work on two occasions and has found the field quality control program to be in compliance with the NRC requirements. However, NRC indicated greater quality control monitoring in the field than that proposed in 1987 would be required for the approved, thinner soil cover. United Nuclear has agreed to an expanded quality assurance/quality control (QA/QC) program for reclamation activities conducted after March 1, 1991. The QA/QC program has been revised to reflect NRC guidance in the STP Testing guidance.

Table 5.4 summarizes the required testing frequencies in the Field Testing and Inspection Plan (FTIP) for the soil cover. United Nuclear may petition the NRC for a license amendment for reducing required QA/QC activities to those originally proposed, if testing and inspection activities consistently meet QA/QC criteria during reclamation.

Accordingly, the quality control program described in more detail here, reflects the quality control procedures implemented in the field beginning 1991. With the exception of field moisture/density testing, the quality control program described below is consistent with the program that has been conducted to-date in the field, and that was approved during the NRC's inspections. The tests presented here will be implemented for all future reclamation activities, and will be used to verify the criteria used in the design of the radon barrier. The quality control data are expected to be adequate to meet this verification requirement.

5.3.4.1 Soil Cover Materials

In reviewing the proposed design, NRC requested procedures for determining acceptability of borrow material used as soil cover, should borrow sources differ from those identified in this plan. It is not expected that other sources of borrow material will be required other than those identified in this plan. However, if other borrow sources are required, gradation and classification tests will be performed to ensure that these materials meet project specifications. Samples will be obtained by means of borehole drilling and sampling, or by test pit excavation, and sampling and analyses for gradation and classification. The on-site quality assurance engineer will review and accept or reject the test results before placement of imported fill as soil cover material. In addition, to address NRC review comments, a graphic presentation of allowable soil-cover soil types provides a reliable method to evaluate gradation results before soil is placed in the soil cover. This method ensures the soil placed in the soil cover is consistent with the soil-cover design (Figure 3-2).

5.3.4.2 Standard Proctor Compaction Testing (ASTM D 698)

The frequency of Standard Proctor laboratory tests to confirm the moisture versus density relationship of the soil, will be a minimum of one compaction test for every 15 field moisture/density (i.e., compaction verification) tests performed as recommended by NRC's Testing STP. NRC's Testing STP frequency is one Standard Proctor test for every 10 to 15 field moisture/density tests performed.

The actual frequency for Standard Proctor compaction tests, during the 1989 and 1990 interim stabilization activities at the Church Rock site was one test for every four field moisture/density tests performed, exceeding NRC's Testing STP guidance. Fifteen tests were conducted during the placement of approximately 125,000 cy of soil cover, or one standard Proctor compaction test for every 8,000 cy of material placed. The actual compaction testing frequencies during initial interim stabilization activities satisfied the NRC during their inspections.

5.3.4.3 One-Point Proctor Testing

NRC's Testing STP specifies that a one-point Proctor test be performed for every five field moisture/density tests. The Standard Proctor testing frequency employed by United Nuclear in 1989 and 1990 of one full Standard Proctor test for every four field moisture/density tests exceeds the total recommended frequency of testing for both one-point and Standard Proctor compaction tests. One-point tests will be conducted at the minimum frequency of one test for every five field moisture/density tests conducted during future reclamation activities.

5.3.4.4 Field Moisture/Density Verification Test

NRC's Testing STP recommends that one moisture/density (compaction verification) test be completed for every 500 cy of fill placed, and that a minimum of two moisture/density tests be conducted each day. United Nuclear has been conducting field moisture/density testing at the frequency of one test for every 2,000 cy of fill placed and performing at least two tests daily. This frequency exceeds the design engineer's minimum specification requirements included in the 1987 proposed plan and is in accordance with Bureau of Reclamation (BUREC) dam construction standards.

Use of the BUREC standard for tailings cover material is appropriate at the Church Rock site because the relative risks posed by failure of a soil cover, compared to the risks associated with dam failure, are much smaller. That is, construction specifications accepted for structures used to impound water are considered sufficiently conservative for use in the constructing covers for tailings sands.

Additional support for use of this BUREC standard arises from the FTIPs for constructing the original embankments used to retain the tailings. Tailings retention embankment construction standards for many tailings disposal sites, including Church Rock, employed testing frequencies similar to the BUREC standard, rather than NRC's Testing STP guidance. These tailings embankments will remain part of the impoundment reclamation, having a 1,000-year design life. Therefore, United Nuclear did not consider it necessary to use more conservative testing frequencies for the reclamation cover than those used for constructing the original retention embankments.

Thus, this field moisture/density testing frequency to confirm that soil for earthen embankments has been sufficiently compacted will also be used at the Church Rock site, as approved by NRC. This standard is one test for each 2,000 cy of fill or a minimum of two tests for each day of fill placed in excess of 150 cy. At this frequency, over 300 field density and moisture verification tests would be completed once the final soil cover is in place.

Due to NRC's review comments regarding the thinner soil cover as described above, the field moisture/density testing frequency will be increased to an average of one test per 500 cy of soil cover placed for the project. During initial interim stabilization activities, a total of 62 moisture/density tests were taken in the interim cover for 125,000 cy of soil cover material placed, which is equivalent to a testing frequency of one test per 2,000 cy, which meets the BUREC frequency. It will be possible to attain the higher testing frequency of one test per 500 cy, based on an average of all tests taken for the project. The higher frequency will be accomplished when the interim soil cover is conditioned (i.e., moisture-adjusted and compacted) immediately before placement of the final soil cover. Additional in-place density tests will be completed at this time to meet required project frequencies, ensuring the material placed meets the requirements modeled in the design.

5.3.4.5 Nuclear Density Gauge Correlation

As stipulated in NRC's Testing STP, moisture/density compaction verification testing performed by nuclear density gauge methods was correlated to one in-situ field density Sand Cone test (ASTM D 1556) and one oven-dry moisture test for every 10 nuclear density gauge tests performed. During initial interim stabilization activities, 20 Sand Cone tests were conducted on the interim soil cover in the North and Central Cells. Sand Cone tests were used exclusively to verify moisture/density compaction of the North Cell interim cover, when a good correlation between the nuclear gauge and Sand Cone tests could not be obtained. Future moisture/density verification testing will meet the frequency requirement of NRC's Testing STP.

Based on performance to-date for interim construction activities, test results obtained by the nuclear densimeter are erratic. The as-built report for the North Cell Interim Reclamation Activities documented the erratic nature of the nuclear densimeter testing. Therefore, only the Sand Cone method of in-place density determinations was used in the 1990 construction activities, and will be used in all future construction.

5.3.4.6 Gradation and Soil Classification

As described previously in Section 3.0, a detailed evaluation of expected soil types and gradations was performed as part of the original design. This evaluation was based on over 50 gradation analyses obtained from test pits and borings installed within the proposed borrow areas shown on Figure 3-1. The test data, summarized in Tables 3.1 through 3.3 and Figure 3-2, demonstrate the relative uniformity of grain size distribution of these soil samples. In addition, further uniformity of the soil cover will be achieved by the mixing of soils as the material is removed from the proposed borrow areas, spread, and compacted as the soil cover. Therefore, the soil-cover characteristics should be relatively uniform.

The expected uniformity has been confirmed by a total of 19 particle-size gradation and soil classification tests completed as part of interim reclamation quality control on borrow soils placed in the soil cover during interim stabilization activities. The tests represent a testing frequency of one test per 6,500 cy of soil placed. The actual field test results show that both geotechnical exploration data and initial soil-cover field testing demonstrate consistent soil grain-size distribution and soil-cover characteristics (as expected during design). As shown in Tables 3.1 through 3.3 and illustrated on Figure 3-2, the gradations of the material placed in the North and Central Cells for interim soil-cover construction are uniform and very similar to those samples in the proposed borrow areas. Due to this low variability in grain-size distribution and soil classifications during initial cover placement, a higher testing frequency is not necessarily warranted.

However, in review, the NRC expressed concern that a thinner soil-cover design would be more sensitive to material gradation fluctuations and the resultant radon attenuating characteristics. To address this comment, the gradation and Atterberg testing frequency will be increased to an average of one test per 1,000 cy of soil-cover material placed for the project. Soil-cover testing before 1991 was at the lower frequency of one test per 6,500 cy as specified in the 1987 proposed plan. It will be possible to attain the higher testing frequency of one test per 1,000 cy, based on an average of all tests taken for the project, and will be accomplished when the interim soil cover is conditioned (i.e., moisture adjusted and compacted) immediately before final soil-cover placement. At the same time, additional gradation and Atterberg tests, as well as in-place density tests as identified above, will be conducted to meet required project frequencies, which will ensure that the material placed meets the requirements of that modeled in the design.

5.3.4.7 Interim Stabilization QA/QC Summary

As a part of construction QA/QC during interim stabilization soil-cover construction in the North and Central Cells, representative tests of actual soil parameters were taken, including 15 standard Proctor compaction tests, 28 one-point Proctor tests, 62 field moisture/density tests, 19 grain-size distribution analyses, and 7 permeability tests. As described above, the representative in-situ soil cover properties were then used to calculate radon attenuation parameters (i.e., porosity and diffusion coefficient) for the average soil-cover material. The final reclamation soil-cover design has been adjusted on the basis of these actual soil parameters.

5.4 Long-Term Stability of the Radon Attenuation Soil Cover

The radon attenuation soil cover is designed to reduce radon emissions from the tailings to less than 20 $pCi/m^2/sec$, in accordance with NRC requirements. To provide long-term stability of the radon attenuation soil cover for the 1,000-year design period, several measures have been included in this plan, which incorporates the following features:

1. The grading plan was developed with the surface having nominal slopes to provide runoff yet minimize erosional forces.

2. A soil/rock matrix layer was designed to further minimize erosion of the cover.

3. The final cover was also designed to manage surface water runoff using surface water control channels, including branch swales, drainage channels and diversion ditches.

5.4.1 Soil/Rock Matrix Design

The radon attenuation soil cover will be protected from wind and water erosion by the soil/rock matrix erosion-protection cover shown on Figure 4-6. This layer will also allow a higher and more stable long-term moisture content, compared to a vegetated cover. This design meets the requirements for erosion protection of the tailings cover while maintaining adequate slopes to promote surface water runoff and reduce infiltration into the tailings. A similar soil/rock matrix design was approved by the NRC for use at Anaconda's Bluewater facility in New Mexico.

Previous studies of tailings impoundments (Mayer et al., 1981) indicate that rock covers increase the soil moisture content below the rock cover by decreasing the effective evaporative zone and reducing the overland flow velocity of runoff, compared to a cover consisting of compacted soil only. By decreasing evaporation, the soil and rock cover reduces the gradient that draws moisture from the soil cover to the atmosphere. Thus, through time, annual precipitation at the site will provide a long-term moisture content closer to the higher field-capacity moisture content rather than the wilting-point moisture content.

The CSU method described in NUREG 4651 (NRC, 1987) was used to size the rock in the soil/rock matrix. Based on flow depths and velocities computed using the unit-width method described in NUREG 4620 (NRC, 1986), a D_{50} of 1.5 inches for the rock mulch provides erosion protection.

The soil/rock matrix will be constructed by placing a 3-inch-thick rock mulch layer 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 will be forced into the rock mulch voids by driving construction equipment over the soil. The 4- to 6-inch-thick soil lift will be placed over the rock to maintain an approximate thickness of three inches above the rock layer. This overall 6-inch-thick soil/rock matrix will provide long-term erosion protection. Soil compaction will densify the rock layer by tightly wedging the stones. The soil will fill the void spaces, stabilizing the rock, and decreasing the effective evaporative zone depth, which provides more stable and higher long-term moisture content of the radon barrier. The use of soil compacted into rock has been shown to increase the stability of rock protection (NUREG 4651). The soil/rock matrix is sufficiently fine-grained that is does not require filter material. The soil/rock matrix will also protect the soil cover from wind erosion. Appendix E provides the calculations used to size the soil/rock matrix.

Frost heave and the potential effect of decreasing radon attenuation of the soil cover were also considered. For a material to be frost susceptible, there must be a source of water, close enough to the frost line to supply capillary water from a saturated soil layer. The potential for capillary action depends on the effective pore diameter. The coarse tailings located below the cover classify as poorly graded sands and are considered relatively free draining. These characteristics are indicative of relatively large pore diameters, indicating these materials have a low potential for capillary action. Since this material below the cover material will not support capillary action, the ability to transport water to the frost line by capillary action does not exist. Therefore, soil cover susceptibility to frost heave is low.

Shrinkage and its potential effect on radon attenuation is also governed by capillary action. When a saturated soil dries, a meniscus develops in each void at the soil surface. Tension develops in the soil water and a matching compression develops within the soil. Since the long-term moisture of the soil cover will remain fairly stable and the material below the cover soil will not support capillary action, shrinkage effects on the cover soil will not be significant.

5.4.2 Soil/Rock Matrix Quality Control Program

The construction of the soil/rock matrix cover will be verified by construction control, staking, and probing, as described in the specifications (Appendix B). A standard procedure will be used to verify the thickness of rock mulch placed, by measuring the rock at locations on a rectangular grid intersecting at 100-foot intervals across the tailings cover. The soil will be placed over the rock mulch only after the rock mulch thickness has been tested and documented as acceptable. The maximum and minimum thickness, and adequacy of soil intrusion into the rock mulch will be verified using the same procedure identified for measuring the rock mulch layer. The extent soil is present throughout the rock mulch layer will also be measured. The measurement procedures are described in detail in the specifications (Appendix B).

5.4.3 Tailings Cover Branch Swales

Branch swales have been added to the tailings cover design to collect surface water runoff while minimizing erosion (Figures 5-1 and 5-2). The location of the swales was chosen to provide the optimal combination of hydraulic characteristics (overland slope and slope length) that allow the use of a soil/rock matrix cover to protect the radon attenuation soil cover from erosion. The swales will consist of shallow, trapezoidal ditches with 3H:1V sideslopes (Figure 5-3). The bottom and sideslopes of the swales will be protected with riprap. Table 5.5 summarizes the design characteristics of the swales.

Hydraulic analyses of the flows generated within the swales was conducted to determine the minimum swales depth. The peak discharge of the runoff generated by the

Probable Maximum Precipitation (PMP) event on the impoundments was calculated using the Soil Conservation Service (SCS) Technical Release No. 55 (TR-55) method. The depth of these peak discharges in the swales was calculated using Manning's equation. A freeboard of at least .5 foot was added to the flow depth in the swales to determine the minimum swales depth. This design approach follows the guidelines set forth in the NRC's August 1990 Staff Technical Position (STP), "Design of Erosion Protection Covers for Stabilization of Uranium Mill Tailings Sites" and provides for stable water conveyance structures that will promote drainage and reduce infiltration into the tailings. Table 5.5 summarizes the hydraulic analyses for the swales. Appendix E details the calculations used to complete the hydraulic analyses.

The Safety Factors method of riprap design was used to size the riprap that will be installed in the swales. Table 5.5 summarizes the riprap sizes for the swales. In many cases, the same rock used for the soil/rock matrix is adequate for the swales. Appendix E shows the detailed calculations used to determine the riprap sizes.

The minimum riprap layer thickness is two times the D_{50} size. The D_{50} of the riprap in Branch Swales H and I is 3.0 inches. The D_{50} of the remaining Branch Swales is 1.5 inches. A 6-inch-thick bedding layer will be placed under the riprap layer in Branch Swales H and I. The D_{50} of the lower 3 inches of this material in Branch Swales H and I is 0.02 inch. The D_{50} of the upper 3 inches of this layer is 0.35 inch. The remaining Branch Swales will require a single 3-inch-thick bedding layer with a D_{50} of 0.02 inch. Tables 5.6 and 5.7 provide the gradation limits of the riprap and bedding layer materials for the Branch Swales, respectively.

5.4.4 South Cell Drainage Channel

The South Cell Drainage Channel, shown on Figure 5-1, is designed to remove runoff from the South Cell tailings area to enhance the long-term stability of the tailings cover.

The South Cell Drainage Channel will be excavated into bedrock along part of the upper section from approximately Station 4+50 to Station 9+00, as shown on Figure 5-1. Figure 5-4 shows typical sections. Riprap will be placed within the upper reach at all locations not protected by competent bedrock (i.e., from Station 0+00 to approximately Station 4+50). It will not be necessary to riprap the lower reach, from Station 9+00 to the confluence with Pipeline Arroyo, because this reach is isolated from the tailings by the 450-foot-wide bedrock section.

The South Cell Drainage Channel is trapezoidal in shape with a 10-foot bottom width and 3 horizontal to 1 vertical (3H:1V) side slopes. Approximately 450 feet from the beginning of the channel, it will bend to the west. At this bend, the side slope will gradually change to a 2H:1V slope as bedrock is encountered. The channel in this upper reach will have a gradient of 0.0244 foot per foot (ft/ft).

In determining the flow rate and riprap requirements used in the hydraulic design for the South Cell Drainage Channel, the following values of design parameters were used:

1. A runoff curve number of 80 was used for the soil/rock matrix, which will be placed on the catchment area draining to the channel.

2. The Probable Maximum Precipitation (PMP) amount of 8.43 inches was determined from Hydrometeorologic Report No. 29.

3. The Probable Maximum Flood (PMF) hydrograph was determined using the tabular method of hydrograph generation described in SCS TR-55.

4. The size of riprap to be placed was determined using the Safety Factors method as described in NUREG/CR-4651.

5. The material size and gradation of the bedding layer under the riprap were determined by the methods provided in NUREG/CR-4620.

The calculated PMF for the South Cell Drainage Channel has a peak discharge of 694 cubic feet per second (cfs). This peak discharge produces a peak flow depth of 3.5 feet in the upper reach.

The riprap for the unprotected portions of the upper reach will have a D_{50} of 15 inches, a D_{50} of 19 inches, and a minimum thickness of 23 inches. A 6-inch-thick bedding layer will be placed under all riprapped sections of the upper reach. The 6-inch bedding layer will consist of a lower 3-inch sand layer with a D_{50} of 0.02 inch and an upper 3-inch-thick gravel layer with a D_{50} of 0.35 inch. Tables 5.6 and 5.7 provide the gradation limits for the riprap and the bedding layer materials for this channel, respectively.

5.4.5 North Cell Drainage Channel

The North Cell Drainage Channel, shown on Figure 5-2, will remove runoff from the North Cell tailings area to enhance the long-term stability of the soil cover by collecting flows from Branch Swales B through G. The North Cell Drainage Channel will intercept the North Diversion Ditch extension at a location north of the tailings disposal area as shown on Figure 5-2. The North Diversion Ditch extension transports the runoff into Pipeline Arroyo.

The upper reach of the North Cell Drainage Channel will be trapezoidal in shape with a 10-foot bottom width and 3H:1V side slopes (Figure 5-3). This reach will have a channel gradient of 0.020 ft/ft.

The lower reach of the channel will also be trapezoidal in shape with a 10-foot bottom width and 3H:1V side slopes. The lower reach will have a channel gradient of 0.0126 ft/ft.

The same methods of PMF hydrograph determination and riprap and bedding layer design were used for this channel as were used for the South Cell Drainage Channel. The parameters included a PMP of 8.43 inches and a runoff curve number of 80.

The PMF for the upper reach of the North Cell Drainage Channel has a peak discharge of 359 cfs. The peak discharge produces a peak flow depth of 2.5 feet in the upper reach. This depth is sufficiently lower than the 3.5-foot channel depth allowing PMF flows to pass safely.

The riprap for the upper reach will have a D_{50} of 9 inches, a D_{50} of 15 inches, and a minimum thickness of 15 inches. The 6 inches of bedding layer material will consist of a lower 3-inch sand layer with a D_{50} of 0.02 inch and an upper 3-inch-thick gravel layer with a D_{50} of 0.35 inch. Tables 5.6 and 5.7 provide the gradation limits for the riprap and the bedding layer material, respectively.

The PMF for the lower reach of the North Cell Drainage Channel has a peak discharge of 562 cfs. The peak discharge produces a peak flow depth of 3.5 feet in this reach. This depth is sufficiently lower than the 4.0-foot depth of the channel allowing PMF flows to pass without overflowing the channel bank.

The calculations for riprap size for the lower reach indicate a D_{50} of 8 inches is appropriate. Since this size is close to the size needed for the upper reach, the riprap and bedding layer design for the upper reach will also be used for the lower reach. Tables 5.6 and 5.7 provide the gradation limits for the riprap and the bedding layer material for the North Cell Drainage Channel, respectively.

5.5 Diversion Ditches

Figures 5-1 and 5-2 depict the South and North Diversion Ditches that intercept precipitation runoff from the drainage basins to the south, east, and north of the tailings disposal area. These ditches were constructed by United Nuclear during facility operation. They continue to be useful in reclamation because they enhance the long-term stability of the tailings disposal area by preventing runoff from flowing onto the tailings disposal area. Minor modifications to the North Diversion Ditch are planned during reclamation, as addressed below.

The ability of these ditches to convey the PMF from the contributing drainage basins without over-topping the banks was investigated as part of the reclamation design. The investigation determined the PMP amounts for each drainage basin, calculated the peak discharge of the PMF hydrograph, and evaluated the ditches' characteristics (cross-sectional shapes, channel gradient, and roughness coefficient). The flow depth of the PMF peak discharge was determined at several cross sections to evaluate the ability of the ditches to convey the PMF.

The one-hour PMP amount from a thunderstorm event was determined using Hydrometeorologic Report 49 (NOAA, 1984). The PMF hydrograph and peak flow rate was estimated using the SCS Triangular Unit Hydrograph Method (Bureau of Reclamation, 1977). Appendix E provides the work sheets for these calculations. The cross sections and channel slopes were determined from 1985 topographic mapping at a scale of 1:2400. A roughness coefficient of 0.025 was chosen for these earth channels. Manning's equation was used to calculate the normal flow depth at the various sections.

5.5.1 South Diversion Ditch

The South Diversion Ditch intercepts runoff from Drainage Basin C, delineated on Figure 5-5, and routes the runoff to a natural ephemeral drainage that discharges into Pipeline Arroyo approximately 3,000 feet downstream from the nickpoint. The South Diversion Ditch is generally trapezoidal in shape with a 15-foot bottom width and 2H:1V side slopes. It is generally 8-feet deep and has an average gradient of 0.003 ft/ft.

The PMF for Drainage Basin C has a peak discharge of 1,370 cfs. This peak discharge produces a peak flow depth of 4.85 feet in most sections of the channel. This depth is considerably lower than the 8-foot channel depth.

Previous evaluations (Science Applications, Inc., 1981) identified two limiting sections (Sections R-R' and S-S') on the South Diversion Ditch. Figure 5-1 shows the locations of these sections, and Figure 5-6 shows their shapes. The channel gradient at these sections is 0.002 ft/ft. Although the locations are upgradient from much of the inflow to the South Diversion Ditch, they are able to pass the full PMF peak discharge. The peak flow depth at Section R-R' is 6.4 feet and 4.25 feet at Section S-S'. At Section R-R', a freeboard of 1.1 feet exists above the peak-flow height, while freeboard at Section S-S' is 1.5 feet. Thus, the South Diversion Ditch is fully able to convey the PMF with no additional channel modification.

5.5.1.1 South Diversion Ditch Confluence Stability

In reviewing the proposed plan, the NRC expressed concern that inflow from the tributaries to the North and South Diversion Ditches may damage the diversion ditch banks at the tributary confluence with the ditch, and allow flows to pass over the bank to the reclaimed tailing impoundments. NRC commented that an occurrence could potentially allow a release of tailings. To respond to this comment, the stability of the confluences of tributaries and the South Diversion Ditch was evaluated. The evaluation indicated none of the tributaries to the South Diversion Ditch provide a large proportion of flow to the ditch. Therefore, overtopping at the confluence of a tributary and the ditch is not a concern.

5.5.2 North Diversion Ditch

The North Diversion Ditch intercepts runoff from drainage basins A1, A2, and B, delineated on Figure 5-5, and routes this runoff to the alluvial floodplain north of the tailings disposal area.

The North Diversion Ditch does not have a consistent cross-section configuration. Five channel sections (T-T, U-U, V-V, W-W, and X-X) are illustrated on Figure 5-6. Side slopes range from 1.1 to 7.6H:1V, bottom widths range from 12 to 22 feet, and channel depths range from 7 to 35 feet. The channel gradient is relatively uniform at approximately 0.0075 ft/ft.

The PMF peak discharge in the North Diversion Ditch is 1,081 cfs, with the additional contribution of Drainage Basin A2, it is 2,265 cfs, and with the additional contribution of Drainage Basin B, it is 5,850 cfs. Table 5.8 lists the contributing drainage areas and peak discharges at each section. The table also identifies characteristics for these sections of the North Diversion Ditch including the average side slope, the bottom width, channel slope, normal depth of the peak flow, and the average velocity of this flow.

Figure 5-6 shows the PMF peak discharge is contained within the North Diversion Ditch at all five sections. While some channel and bank erosion, and local scour will occur as a result of the high PMF flow velocities, the distance between the diversion ditch and the tailings disposal area is a minimum of 200 feet. Therefore, it is unlikely the potential for scour would be large enough to result in the release of tailings. However, in response to NRC comments, riprap will be installed on the two inside curves of the North Diversion Ditch, as described below.

During review and comment, the NRC expressed the concern that two sharp curves on the North Diversion Ditch may show increased amounts of erosion that could allow flood flows to pass onto the tailings area (Figure 5-1). Accordingly, cross sections were developed, hydraulic calculations were completed, and riprap was designed for these two curves. Cross Sections AA-AA' and BB-BB' on Figure 5-3 show these curves in the existing North Diversion Ditch will contain the PMF without over-topping the embankment. In addition, the calculated flow velocities were moderate and the flow was subcritical at both locations (Froude number less than 1.0). Thus, the hydraulic calculations indicate a minimal potential for the existing North Diversion Ditch to migrate into the tailings pile. Therefore, it will not be necessary to place riprap into other existing portions of the North Diversion Ditch.

Riprap was sized for the curved channel locations at sections AA-AA' and BB-BB' using criteria provided in NUREG-4620 and by the 1987 Maynard method. The riprap sized for the curve at AA-AA' will be used at BB-BB' as well, because its larger size meets more stringent flow criteria, providing a greater safety factor. The riprap will have a D_{50} of 6 inches and a thickness of 10 inches. The riprap layer will be underlain by a 6-inch-thick filter blanket. Tables 5.6 and 5.7 present the gradation requirements for the riprap and bedding material, respectively. Appendix E shows the calculations for the riprap sizing. Requirements for the rock used in riprap are discussed in more detail in Section 5.6.

5.5.2.1 North Diversion Ditch Confluence Stability

In reviewing the proposed plan, the NRC also expressed concern that inflow from the tributaries to the North and South Diversion Ditches may damage the diversion ditch banks at the tributary confluence with the ditch, and allow flows to pass over the bank to the reclaimed tailing impoundments. NRC's concern was that this occurrence could potentially allow a release of tailings. Therefore, the stability of the confluences of tributaries and the existing North Diversion Ditch was evaluated. Damage to the ditch banks could potentially occur when the flows from the tributary are generally perpendicular to the ditch bank and impinge directly on the far bank. Such flows could potentially scour the far bank of the ditch by removing bank material. The tributary flows will have to be relatively large or relatively fast compared to the flows in the diversion ditch to have sufficient energy to damage the ditch bank.

The stability evaluation was conducted by first comparing total head (velocity head plus elevation head) of the PMF in the diversion ditch and in the tributary at three critical confluences. The second part of the evaluation considered the geometry of the confluences with respect to the confluence angle and extent of material that will have to be scoured to allow the flows to pass over the diversion ditch channel bank.

Three critical locations are shown on Figure 5-1. These three confluences (A1, A2, and B) involve tributaries to the North Diversion Ditch that provide a large proportion of the total flow of the diversion ditches at discrete locations.

Figure 5-7 provides a schematic of the tributaries and related cross sections on the existing North Diversion Ditch. Table 5.9 summarizes of PMF flow characteristics for each of these locations. The PMF peak discharges were determined using the SCS TR-55 method. Manning's equation was used to determine the flow depth and discharge in the existing North Diversion Ditch and in the tributaries. Appendix E provides these calculations.

Table 5.9 also provides a comparison of total head at all the locations. The total head for the flows in tributaries A1, A2, and B are all less than those of the North Diversion

Ditch cross sections (K, L, and M), located immediately upstream of the confluences. Thus, while the tributary flow will cause turbulence at the confluence, the flow will not be able to impinge directly on the ditch banks.

The geometry of the three confluences was evaluated to identify the confluence angle and the amount of material in the far ditch bank. Figure 5-1 shows Tributaries A2 and B entering the existing North Diversion Ditch at shallow confluence angles of approximately 45 degrees. Thus, their flows will not impinge directly on the far ditch bank. Tributary A1 enters the existing North Diversion Ditch at about a 90-degree angle. This angle of tributary flow will cause more turbulence at the confluence than that caused by a shallower angle.

Figure 5-7 shows cross sections of the North Diversion Ditch at the confluences. The cross sections show the extent of material that would have to be scoured before flows left the diversion ditch and passed across the tailings. These scour distances are 90 feet at confluence A1, 170 feet at confluence A2, and 150 feet at confluence B. These distances are all sufficiently large that channel scour will not allow flows to pass over the tailings. In addition, as channel scour removes material from the far bank during a particular flow event, the channel will become wider, protecting itself from future scour. Finally, the tailings at these locations are at least 500 feet away from the locations of these confluences and the regraded surface of the tailings cover is sloped down towards the ditch making the potential release of tailings unlikely.

At confluence A1, the total head of the tributary flow is considerably less than that of the ditch flow. Furthermore, channel scour will have to remove 90 feet of material before flows could pass onto the tailings. At confluence A2, the total head of the tributary flow is less than that of the diversion ditch and the confluence angle is about 45 degrees. Thus, the tributary flows will not impinge directly on the ditch bank. Furthermore, about 170 feet of material will have to be scoured before flows could pass over the tailings.

At tributary B, the total head of the tributary flow is also less than that of the ditch and the confluence angle is about 45 degrees. Thus, the tributary flows will not impinge directly on the diversion ditch bank. Furthermore, about 150 feet of material will have to be scoured before flows could pass over the tailings.

The above evaluation indicates that, while scour and erosion will occur at the confluences, the total head differential, confluence angles, and amount of ditch material to be removed will not allow the release of tailings.

5.5.2.2 North Diversion Ditch Outlet

The outlet of the existing North Diversion Ditch will be extended during final reclamation to intersect with the North Cell Drainage Channel and on to intersect with the Pipeline Arroyo, as shown on Figure 5-2. The new ditch extension will consist of a short section cut into the underlying rock with a steep gradient. The ditch will then be continued to the Pipeline Arroyo in a curved and then straight section with a shallow gradient (Figure 5-2).

The extension of the north diversion ditch will contain the PMF, except for that portion of the ditch constructed within the floodplain north of the North Cell tailings disposal area. This section is designed to contain the 100-year flood. Designing the lower section of the ditch to contain the PMF is impractical because, as shown on Figure 5-2, the PMF boundary for the Pipeline Arroyo extends to the eastern edge of the floodplain. Therefore, if a PMF event occurred, water in the Pipeline Arroyo floodplain will inundate the area and the North Diversion Ditch extension.

5.5.2.3 North Diversion Ditch Slope Stability

When reviewing the proposed plan, NRC expressed concern that the steep slope of the North Diversion Ditch bank between Stations 41+00 and 50+00 (Figures 5-1 and 5-2) may not be stable. To address NRC comments, an evaluation of this bank was conducted. Based on a review of data contained in "Geology of the Church Rock Area," Science Applications, Inc., and visual observation of the cut-face of the North Diversion Ditch in the northeast portion of the site (Station 41+00 to Station 50+00), the excavation is geologically stable.

The channel cut is through the Dilco Coal member of the Crevasse Canyon formation. Typically, the bedding planes of the Dilco member in this channel excavation area trend from southeast to northwest and dip to the northeast at relatively small 3 to 5 degree angles. Visual observations of the channel excavation verify these conclusions. This small 3 to 5 degree angle is significantly lower than the internal angle of friction of 45 degrees (Perloff and Baron, 1976) for this material. Therefore, little probability of bedding planes slippage due to shear failure exists.

Discontinuities and saturation of the formation could potentially reduce shear resistance and cause failure. The only observable discontinuities in the Dilco Coal member typically consist of lower-strength materials (i.e., shale and siltstone) interbedded with higher strength materials (sandstone). These discontinuities alone are not likely to lead to instability because of the high cohesion and friction angles between these layers. Saturation of these layers and resultant strength losses are also highly unlikely. The water table is approximately 140 feet below the excavation, making saturation from this source unlikely. The low precipitation (12-14 inches/year), small infiltration area uphill of the excavation (4.9 acres), and impermeable nature of the member make infiltration and percolation through the bedding planes unlikely. Therefore, shear strength losses due to saturation of the interbedded layers are essentially nonexistent.