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Remedial Action Plan and Site Design for Stabilization of the Inactive Uranium Mill Tailings Site at Mexican Hat, Utah

Appendix D

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100

APPENDIX D
SITE CHARACTERIZATION

TABLE OF CONTENTS

<u>Section</u>	<u>Page</u>
D.1 INTRODUCTION.	D-1
D.1.1 Site description	D-1
D.1.1.1 Location.	D-1
D.1.1.2 Physical description.	D-1
D.1.1.3 History	D-2
D.2 RADIATION DATA.	D-5
D.2.1 Background radiation data.	D-5
D.2.1.1 Background gamma exposure rates	D-6
D.2.1.2 Background radionuclide concentrations in soil.	D-6
D.2.1.3 Background radon concentrations in air.	D-6
D.2.2 On-pile contamination.	D-7
D.2.3 Off-pile contamination	D-7
D.2.4 Other radiological parameters.	D-7
D.2.4.1 Radionuclide concentrations in air samples.	D-7
D.2.4.2 Gamma radiation	D-8
D.2.4.3 Emanating fractions	D-8
D.2.4.4 Diffusion coefficients.	D-8
D.2.4.5 Ambient radon concentrations.	D-8
D.3 GEOLOGIC CHARACTERISTICS.	D-13
D.3.1 Introduction	D-13
D.3.2 Criteria and definitions	D-14
D.3.3 Scope of work.	D-16
D.3.3.1 Compilation and analysis of previous geologic work	D-16
D.3.3.2 Earthquake data compilation	D-17
D.3.3.3 Site-specific geologic data	D-17
D.3.3.4 Ground reconnaissance and mapping	D-18
D.3.3.5 Photogeologic interpretation.	D-18
D.3.3.6 Low-sun-angle aerial reconnaissance	D-19
D.3.4 Regional conditions.	D-20
D.3.4.1 Physiographic setting	D-20
D.3.4.2 Regional geology.	D-21
D.3.4.3 Regional structure.	D-23
D.3.4.4 Regional geomorphology.	D-25
D.3.4.5 Rates of denudation	D-28
D.3.4.6 Climate and vegetation.	D-29
D.3.4.7 Long-term climate variation	D-30
D.3.5 Site geology	D-32
D.3.6 Economic geology	D-33
D.3.7 Seismotectonic setting	D-34
D.3.7.1 Regional setting.	D-34
D.3.7.2 Colorado Plateau seismotectonic province.	D-35
D.3.7.3 Intermountain seismic belt.	D-36
D.3.7.4 Basin and Range province in Arizona	D-37
D.3.7.5 Rio Grande Rift	D-37

TABLE OF CONTENTS (Continued)

<u>Section</u>	<u>Page</u>
D.3 GEOLOGIC CHARACTERISTICS (Concluded)	
D.3.8 Geologic hazards analysis.	D-38
D.3.8.1 Geomorphic hazards.	D-38
D.3.8.2 Impact of natural resource development.	D-40
D.3.8.3 Seismic hazards	D-40
D.4 SITE FOUNDATION CHARACTERISTICS	D-79
D.4.1 Foundation material properties	D-79
D.4.1.1 In-situ data.	D-79
D.4.1.2 Elastic parameters.	D-79
D.4.1.3 Strength.	D-80
D.5 TAILINGS CHARACTERISTICS.	D-85
D.5.1 Design parameters of in-situ tailings.	D-85
D.5.2 Design parameters of remolded tailings and windblown contaminated material.	D-85
D.5.3 Material properties of in-situ tailings.	D-85
D.5.4 Material properties of remolded tailings and contaminated windblown material.	D-97
D.6 BORROW MATERIAL CHARACTERISTICS	D-159
D.6.1 Design parameters.	D-159
D.6.2 Erosion barrier materials.	D-159
D.6.3 Radon barrier materials.	D-160
D.6.4 Data needs	D-162
D.7 GROUNDWATER HYDROLOGY	D-171
D.7.1 Mexican Hat tailings site.	D-171
D.7.1.1 General	D-171
D.7.1.2 Unsaturated zone hydraulics	D-172
D.7.1.3 Saturated zone hydraulics	D-173
D.7.1.4 Water quality	D-182
D.7.1.5 Physical and chemical characterization of waste and contaminant transport	D-185
D.7.1.6 Groundwater use	D-190
D.7.1.7 Compliance with EPA Groundwater Standards	D-191
D.7.2 Borrow sites	D-193
D.7.2.1 Rock borrow sites	D-193
D.7.2.2 Radon borrow sites.	D-193
D.7.2.3 Flow and hydraulics	D-193
D.7.2.4 Water quality	D-193
D.7.2.5 Water use	D-193
D.8 SURFACE-WATER HYDROLOGY	D-217
D.8.1 General.	D-217
D.8.2 Drainage and flow.	D-217
D.8.3 Flooding analysis.	D-218
D.8.4 Water quality.	D-218
D.8.5 Water use.	D-219

TABLE OF CONTENTS (Concluded)

<u>Section</u>	<u>Page</u>
D.9 METEOROLOGICAL	D-223
D.9.1 Weather patterns	D-223
D.9.2 Wind	D-223
D.9.3 Temperature	D-223
D.9.4 Precipitation	D-224
D.9.5 Frost	D-224
D.9.6 Evaporation	D-224
D.10 LAND SURVEY	D-229
D.10.1 Topographic survey	D-229
D.10.2 Land survey	D-229
D.10.3 Aerial photographs	D-229
D.10.4 Ownership and easements	D-229
D.10.5 Utilities and subsurface survey	D-229
D.10.6 Drainage structures and features	D-230
D.11 MISCELLANEOUS DATA	D-233
D.11.1 Land use	D-233
D.11.2 Community services	D-233
D.11.3 Utilities	D-234
D.11.4 Transportation	D-234
D.11.5 Environmentally-sensitive issues	D-235
BIBLIOGRAPHY FOR APPENDIX D	D-239
ADDENDUM D1, Radiologic Characterization of the Mexican Hat, Utah, Uranium Mill Tailings Remedial Action Site	
ADDENDUM D2, Final Radium Concentrations and Volumes of Tailings at Mexican Hat	
ADDENDUM D3, Geologic Data	
ADDENDUM D4, Geotechnical Data	
ADDENDUM D5, Particle Size Distribution Curves	
ADDENDUM D6, Piezocone Sounding Logs	
ADDENDUM D7, Additional Radioactive Data	

LIST OF FIGURES

Figure		Page
D.1.1	Mexican Hat site.	D-3
D.1.2	Mexican Hat site designation.	D-4
D.2.1	Locations of background external gamma measurements and background soil samples	D-9
D.2.2	Exposure rate contours derived from aerial survey data taken over Mexican Hat, Utah, and surrounding areas	D-10
D.3.1	Epicentral and fault compilation.	D-52
D.3.2	Structural contour map, Kaibab Limestone, Mexican Hat region.	D-53
D.3.3	Geologic structure of the Mexican Hat region.	D-54
D.3.4	Physiographic map of the Colorado Plateau showing location of Mexican Hat site.	D-55
D.3.5	Sketch map of site region, showing past and present prevailing wind directions.	D-56
D.3.6	Generalized geologic map of the Mexican Hat region.	D-57
D.3.7	Geologic cross section.	D-58
D.3.8	Generalized stratigraphic column for Mexican Hat region	D-59
D.3.9	Drainages at the Mexican Hat, Utah, UMTRA Project site.	D-60
D.3.10	Major seismic zones of the Basin and Range - Colorado Plateau provinces	D-61
D.3.11	Map of historical and instrumentally-located earthquake epicenters of the southwestern United States, epicentral compilation limited to events of magnitude ≥ 4 and/or intensity (Im) $\geq V$	D-62
D.3.12	Graphical determination of MCE magnitude, Colorado Plateau interior and border zone.	D-63
D.3.13	Graphical determination of recurrence intervals for floating earthquake and ME, Colorado Plateau interior and border zones	D-64
D.4.1	Shear strength of soil in staged triaxial compression, HAT01-132, 27'-28'	D-81
D.5.1	Water content vs depth, HAT01-126	D-100
D.5.2	Water content vs depth, HAT01-127	D-101
D.5.3	Water content vs depth, HAT01-128	D-102
D.5.4	Water content vs depth, HAT01-129	D-103
D.5.5	Water content vs depth, HAT01-131	D-104
D.5.6	Water content vs depth, HAT01-132	D-105
D.5.7	Water content vs depth, HAT01-133	D-106
D.5.8	Water content vs depth, HAT01-130	D-107
D.5.9	Water content vs depth, HAT01-135	D-108
D.5.10	Water content vs depth, HAT01-136	D-109
D.5.11	Water content vs depth, HAT01-137	D-110
D.5.12	Water content vs depth, HAT01-138	D-111
D.5.13	Water content vs depth, HAT01-139	D-112
D.5.14	Water content vs depth, HAT01-181	D-113

LIST OF FIGURES (Continued)

Figure		Page
D.5.15	Capillary moisture curve, HAT01-126 15-17.5 ft (sample 1) . . .	D-114
D.5.16	Capillary moisture curve, HAT01-126 15-17.5 ft (sample 2) . . .	D-115
D.5.17	Capillary moisture curve, HAT01-131 10-12.5 ft (sample 1) . . .	D-116
D.5.18	Capillary moisture curve, HAT01-131 10-12.5 ft (sample 2) . . .	D-117
D.5.19	Capillary moisture curve, HAT01-134 35-37.5 ft.	D-118
D.5.20	Capillary moisture curve, HAT01-135 20-22.5 ft.	D-119
D.5.21	Capillary moisture curve, HAT01-136 20-22.5 ft.	D-120
D.5.22	Capillary moisture curve, HAT01-137 6.5-9 ft.	D-121
D.5.23	Consolidation test results undisturbed tailings, HAT01-126, 3-5.5 ft	D-122
D.5.24	Consolidation test results undisturbed tailings, HAT01-126, 22.5-25 ft	D-123
D.5.25	Consolidation test results undisturbed tailings, HAT01-131, 10.0-12.5 ft	D-124
D.5.26	Consolidation test results undisturbed tailings, HAT01-133. . .	D-125
D.5.27	Consolidation test results undisturbed tailings, HAT01-133, 8.5-11 ft.	D-126
D.5.28	Consolidation test results undisturbed tailings, HAT01-135, 30-32.5 ft	D-127
D.5.29	Consolidation test results undisturbed tailings, HAT01-135, 30-32.5 ft	D-128
D.5.30	Consolidation test results undisturbed tailings, HAT01-136, 3-5.5 ft	D-129
D.5.31	Consolidation test results undisturbed tailings, HAT01-136, 13-15 ft	D-130
D.5.32	Consolidation test results undisturbed tailings, HAT01-138, 35-37.5 ft	D-131
D.5.33	Shear strength of soil in staged triaxial compression HAT01-126 25 ft.	D-132
D.5.34	Shear strength of soil in staged triaxial compression HAT01-133 1.5-4 ft	D-134
D.5.35	Shear strength of soil in staged triaxial compression HAT01-138 35-37.5 ft	D-136
D.5.36	Triaxial shear test results HAT01-136 13-15.5 ft	D-138
D.5.37	Capillary moisture curve, HAT01-002 0-0.5 ft (test pit). . . .	D-139
D.5.38	Capillary moisture curve, HAT01-003 0.0-3.0 ft (test pit). . . .	D-140
D.5.39	Capillary moisture curve, HAT01-003 3-4 ft (test pit).	D-141
D.5.40	Capillary moisture curve HAT01-003 3-4 ft (test pit)	D-142
D.5.41	Capillary moisture curve HAT01-004 2.5-6 ft (test pit)	D-143
D.5.42	Strength of soil in staged triaxial compression.	D-144
D.6.1	Consolidation test results - radon cover borrow material. . . .	D-163
D.6.2	Triaxial shear test results	D-164
D.6.3	Shear strength of soil in staged triaxial compression	D-165
D.7.1	Locations of DOE monitor wells, abandoned boreholes, and springs at Mexican Hat site	D-195
D.7.2	Typical DOE monitor well construction	D-196
D.7.3	Locations of hydrogeologic cross sections, Mexican Hat site . .	D-197

LIST OF FIGURES (Concluded)

<u>Figure</u>	<u>Page</u>
D.7.4 Hydrogeologic cross section A-A', Mexican Hat site.	D-198
D.7.5 Hydrogeologic cross section B-B', Mexican Hat site.	D-199
D.7.6 Hydrogeologic cross section C-C', Mexican Hat site.	D-200
D.7.7 Potentiometric contour map of the upper hydrostratigraphic unit, Mexican Hat tailings site, 1985	D-201
D.7.8 Potentiometric contour map of the middle hydrostratigraphic unit, Mexican Hat site, 1985.	D-202
D.7.9 Trilinear diagram, Mexican Hat site	D-203
D.7.10 Locations of Colorado State University (CSU) borings for geochemical analyses, Mexican Hat site.	D-204
D.8.1 Major drainages and surface-water sampling locations in the area of the Mexican Hat tailings site.	D-220
D.8.2 Surface-water sampling locations on the San Juan River.	D-221
D.10.1 Topographic map for the Mexican Hat site.	D-231
D.10.2 Existing utility location plan for the Mexican Hat site	D-232
D.11.1 Mexican Hat road map.	D-236

LIST OF PLATES

Plate 1 Radiological contamination distribution map

LIST OF TABLES

<u>Table</u>		<u>Page</u>
D.2.1	Background radiation levels and background concentrations of radionuclides in surface soil near Mexican Hat, Utah	D-11
D.3.1	Existing aerial photography of the site area.	D-65
D.3.2	Probabilistic estimates of maximum acceleration, velocity, and intensity at the Mexican Hat site area from various published sources	D-66
D.3.3	Earthquakes of $M \geq 4.0$ since 1960 in the Colorado Plateau. . . .	D-67
D.3.4	Accelerations calculated for the site	D-72
D.3.5	Estimated on-site accelerations resulting from MEs in remote seismotectonic provinces, subprovinces, or domains . . .	D-73
D.3.6	Compilation of mapped faults within a 65-km radius of Mexican Hat.	D-74
D.3.7	Selected cases of induced macroearthquakes.	D-77
D.4.1	Design parameters - foundation.	D-83
D.4.2	Typical properties of unfractured sedimentary rock.	D-83
D.5.1	Design parameters--in-situ tailings	D-147
D.5.2	Design parameters--remolded tailings and windblown contaminated materials.	D-149
D.5.3	Physical characteristics (tailings)	D-150
D.5.4	Hydraulic conductivity test results (tailings) (undisturbed samples)	D-153
D.5.5	Hydraulic conductivity - indirectly determined from consolidated test results	D-154
D.5.6	Consolidation test results (tailings) (undisturbed samples) . .	D-156
D.5.7	Physical properties and mechanical behavior (remolded tailings and windblown contaminated materials).	D-158
D.6.1	Design parameters - borrow material	D-167
D.6.2	Physical characteristics - radon barrier.	D-168
D.6.3	Mechanical properties of combined, remolded, compacted soil - radon barrier.	D-169
D.6.4	Capillary moisture test results, composite sample - radon barrier	D-170
D.7.1	DOE monitor well and borehole data, Mexican Hat site.	D-205
D.7.2	Summary of aquifer characteristics, Mexican Hat site.	D-206
D.7.3	Groundwater flow rates, Mexican Hat site, 1985.	D-207
D.7.4	Volume of shallow groundwater in the area of the Mexican Hat tailings, 1985.	D-207
D.7.5	Concentrations of major and trace constituents in groundwater and surface water, upper hydrostratigraphic unit	D-208
D.7.6	Description of DOE groundwater and surface-water samples. . . .	D-210
D.7.7	Concentrations of major and trace constituents in groundwater and surface water, middle hydrostratigraphic unit	D-211

LIST OF TABLES (Continued)

<u>Table</u>		<u>Page</u>
D.7.8	Background water quality, middle hydrostratigraphic unit, Mexican Hat site.	D-213
D.7.9	Geochemical analyses of Colorado State University borings . . .	D-215
D.8.1	Radiological analyses of surface-water samples collected in the area of Mexican Hat tailings site.	D-222
D.9.1	Wind speed at Blanding, Utah.	D-225
D.9.2	Atmospheric stability distribution for the Farmington, New Mexico, airport	D-226
D.9.3	Temperature, precipitation, and pan evaporation data for Mexican Hat, Utah	D-227
D.11.1	School enrollment and capacity data	D-237
D.11.2	Average daily traffic, U.S. Highway 163	D-238

D.1 INTRODUCTION

This appendix is an assessment of the present conditions relative to the inactive uranium mill site near Mexican Hat, Utah. It consolidates available engineering, radiological, geotechnical, hydrological, meteorological, and other information pertinent to the design of the Remedial Action Plan (RAP). The objective of the data is to characterize the conditions at the mill and tailings site so that the Remedial Action Contractor (RAC) may complete final designs of the remedial action.

For ease in reading, figures and tables are grouped (figures first) at the end of each section of this appendix.

The information contained in this section is based on data collected by the Technical Assistance Contractor (TAC). During final design additional data will be collected, and it is expected that the final designs and specifications may contain small changes to the volumes contained in this section.

D.1.1 SITE DESCRIPTION

D.1.1.1 Location

The Mexican Hat site is in southeastern Utah at Halchita, 1.5 miles southwest of Mexican Hat, Utah. The site lies within the Navajo Reservation, approximately 50 miles west of Four Corners and approximately nine miles north of the Utah-Arizona border (Figure D.1.1). The site is located at 37° 07'54" north latitude, and 109° 52'30" west longitude.

D.1.1.2 Physical description

The designated mill tailings site (Figure D.1.2) consists of 235 acres and is 0.5 mile southeast of U.S. Highway 163. The site is situated in an arid desert-like environment consisting of low rolling hills and steep washes formed by drainage tributaries of the San Juan River. The San Juan River, approximately one mile from the site, is a perennial stream that drains into the Colorado River.

The site consists of two tailings piles, several of the original buildings and structures, and the concrete pad of the former mill building.

The mill yard located on the western portion of the site and the ore storage area consist of approximately 19 acres. The mill and several of the associated buildings and much of the processing equipment have been removed from the site. The former mill office building was used as a health clinic by the Utah Navajo Development Council (UNDC) until it closed in October, 1985 (Clah, 1985). The UNDC is a private non-profit organization with offices in Blanding, Utah.

Some dikes were constructed around the piles, but have eroded in several locations. Tailings which have been subject to windblown and waterborne contamination consist of 130,000 cubic yards (cy). Most of the windblown and water-eroded material is north and east of the lower pile.

0.1.1.3 History

Operations

The mill was constructed in 1957 by the Texas-Zinc Minerals Corporation who operated it until 1963, when it was sold to Atlas Corporation who operated it until it was closed in 1965. Approximately 2.2 million tons of ore were processed with an average grade of 0.28 percent uranium oxide (U_3O_8) to produce 5700 tons of U_3O_8 concentrate, which was sold to the Atomic Energy Commission. In addition to the uranium operation, a sulfuric acid plant was in operation at the site and continued to operate until 1970, after the main plant shut down in 1965.

Process

Much of the ore processed contained large quantities of copper sulfide and other sulfide minerals. Thus, the milling process included grinding the ore and treatment by froth flotation to recover the copper. The flotation concentrates and the tailings were acid-leached separately with subsequent filtration of the flotation concentrates to recover a final copper product. The filtrate was then combined with the main circuit-leached slurry obtained from the flotation tailings. A five-stage counter-current thickener was used to separate leached solids from the pregnant liquor which then went through solvent extraction. The organic solvent was stripped with an ammonium nitrate solution which was then neutralized with magnesium hydroxide to precipitate the final uranium concentrate (FBDU, 1981).

The tailings were piped to the piles after separation in a cyclone separator. Coarse sand that settled near the outside of the pile was used to build the dikes higher, while fines and slimes flowed toward the center of the pile (ORNL, 1982).

Ownership and leaseholders

The mill site leased from the Navajo Nation consisted of a 555-acre tract. When the lease expired in 1970, control of the site and all structures and materials on the site reverted to the Navajo Nation.

FIGURE D.1.1
MEXICAN HAT SITE

D.2 RADIATION DATA

This section describes the magnitude and characteristics of the radiation emitted from the Mexican Hat uranium mill tailings site. Radiological data from the vicinity of the Mexican Hat site have been collected by numerous investigators since 1961. All have contributed to an understanding of the radiological picture of the site; however, each has concentrated on a certain aspect of the site contamination and the results have not been combined to give an overall description of the extent of contamination.

The earliest radiological data collection entailed water sampling from the nearby San Juan River, when the U.S. Environmental Protection Agency (EPA) conducted a 10-year period of sampling (from 1961 to 1972) from the river upstream and downstream of the site (EPA, 1973). In May, 1968, the EPA conducted a sampling effort to measure radiation levels in air, water, and soil in the immediate vicinity of the site (Snelling, 1971). This survey was performed about three years after operations at the site were discontinued. The external gamma exposure rates around the site were further characterized by the EPA in 1974 (EPA, 1975). Also in 1974, the Atomic Energy Commission (AEC) funded a project to report on the conditions at the site (Brown, 1975).

The first attempt to map the boundaries of the contaminated regions was performed by Oak Ridge National Laboratory (ORNL, 1980) and Ford, Bacon & Davis Utah Inc. (FBDU, 1981) in March, 1976. A broad mapping of the gamma exposure rates using aerial radiological survey methods over an area of about eight square miles, including the site, was performed by EG&G in 1980 (EG&G, 1982). A program to specifically sample the tailings piles was conducted by Mountain States Research and Development in 1981 (MSRD, 1982). The data collected from that program provide the basis for estimates of the subpile contamination in this appendix. Finally, an extensive survey of the limits of contamination off the pile was performed in 1984 by Bendix Field Engineering Corporation (BFEC, 1985). Due to the comprehensive nature of this most recent work, the BFEC radiological characterization report is reproduced in its entirety as Addendum D1. The BFEC data form the primary basis for the current understanding of the distribution of contamination across the site.

Data from all major sources listed above are presented in this section, as well as other data, in an interpretation of the distribution of contamination around and beneath the Mexican Hat site. This section does not assess the health risks from this contamination, or recommend a course of remedial action. The purpose is to present the current understanding of radiological conditions associated with the inactive uranium mill tailings site.

D.2.1 BACKGROUND RADIATION DATA

The purposes of measuring background radiation near the site are to provide a reference point to which levels of contamination on the site can be compared and to assess construction impacts on the surrounding population and the environment. Measurements of background radiation near the Mexican Hat site have resulted in the following determinations.

D.2.1.1 Background gamma exposure rates

The average background external gamma exposure rate at a height of one meter above the surface is approximately 11 microR/hr. Measurements made in 1968 resulted in an average background of 30 microR/hr; however, these values were collected using Geiger tubes at the very lower limit of their detection range, and should not be considered accurate. Measurements made in 1976 (ORNL, 1980) at the locations shown in Figure D.2.1 resulted in an average background gamma exposure rate of 9 microR/hr. Table D.2.1 presents the individual measurement values at these locations.

The aerial gamma exposure survey (EG&G, 1982) indicates that the gamma exposure rate drops monotonically from the piles down to background levels in the range of 11 to 15 microR/hr. More recent measurements (BFEC, 1985) made at four locations at distances from 0.5 to six miles from the site ranged from 10.8 to 12.5 microR/hr with a mean value of 11.6 microR/hr. These measurements were made with a Pressurized Ionization Chamber (PIC) and are considered the most representative background exposure values collected to date. This is confirmed by noting the background values in Table D.2.1 for the three sites nearest the Mexican Hat site (AZ1, A26, and UT19) which average 9.7 microR/hr.

D.2.1.2 Background radionuclide concentrations in soil

Background radionuclide concentrations in soil samples collected at the same eight locations shown in Figure D.2.1 are presented in Table D.2.1. It should be noted that there is little, if any, correlation between the total activity and the gamma exposure rate. This is likely because of the highly variable spatial distribution of the radionuclides and the fact that the gamma exposure rate is a combination of the spatial average radionuclide activity and the cosmic ray flux (which varies with altitude). The average of the Ra-226 values is 0.9 pCi/g.

Samples of surface material (zero to six inches) during the most recent survey (BFEC, 1985) at four locations near the site averaged 1.1 pCi/g Ra-226. The individual data can be found in Table G-1 of Addendum D1. Considering the different locations from which the data were collected, the two sets of background Ra-226 measurements are considered to be in reasonable agreement with each other.

D.2.1.3 Background radon concentrations in air

A quarterly radon monitoring program was conducted from August 27, 1985, through September 22, 1986, at the Mexican Hat site. This program used film-type radon detectors to

measure both background and on-pile radon concentrations. The results of this monitoring program are presented in Appendix D, Addendum D7, Additional Radiological Data.

D.2.2 ON-PILE CONTAMINATION

The description of the on-pile contaminated materials and the calculation of the volume for these materials are given in Morrison-Knudsen Engineers (MK-E) Calculation No. 9-226-03, Upper Pile Tailings Excavation - Excavation Limits and Quantity Estimate and Calculation No. 9-239-05, Radon Barrier Design - Material properties for Radon Barrier Design of the Final Engineering Design.

D.2.3 OFF-PILE CONTAMINATION

The description of the off-pile contaminated materials and the calculation of the volume for these materials are given in MK-E Calculation No. 9-226-02, Contaminated Material Excavation - Windblown and Waterborne Excavation Quantities of the Final Engineering Design.

D.2.4 OTHER RADIOLOGICAL PARAMETERS

D.2.4.1 Radionuclide concentrations in air samples

Air particulate measurements were first made in 1968 (Snelling, 1971) at nine locations in and around the Mexican Hat site. Continuous 24-hour samples were collected at each location on 11 consecutive days at a rate of 140 liters per minute. Although an 11-day average is significantly shorter than an annual average, many short-term fluctuations are smoothed out. The filters were analyzed for gross alpha content and for Ra-226, Th-230, and natural uranium. Only those locations in the ore storage area and on the tailings piles showed levels above background. The most restrictive Maximum Permissible Concentration (MPC) is for Th-230 at 8.0×10^{-5} pCi/l. The highest 11-day average Th-230 value measured was 1.4×10^{-5} pCi/l or 17 percent of the MPC. This value was obtained at a location immediately downwind of the lower tailings pile.

More recent measurements (FBDU, 1981) made in 1974 confirmed the earlier data. All radionuclides except Th-230 were orders of magnitude below the 10 CFR Part 20 MPC values. The concentration of Th-230 in air particulates at a location at the center of the mill yard was measured to be 1.2×10^{-5} pCi/l (15 percent of the MPC).

It can be concluded that the Th-230 levels during remedial action construction may approach and even exceed the MPC limits.

D.2.4.2 Gamma radiation

The uncovered tailings piles at Mexican Hat produce gamma exposure rates in excess of 1500 microR/hr at some points (FBDU, 1981). The average gamma exposure rate is 380 microR/hr over the upper pile and 850 microR/hr over the lower pile. The highest gamma exposure rates are associated with the lowest portions of both piles where slimes are visible at the surface. The average exposure rate measured in the ore storage and mill yard area is 180 microR/hr.

The aerial radiological survey (EG&G, 1982) provides the most complete overall picture of the gamma fields produced by the tailings. Isopleths of gamma exposure rate from the aerial survey are presented in Figure D.2.2. This distribution is confirmed by the individual point measurements made by FBDU. Note the contribution to the fields along the large arroyo extending to the northeast of the lower tailings pile and the drainage along the trench from just north of the mill yard to the highway.

D.2.4.3 Emanating fractions

The current estimates of the emanating fractions for the contaminated materials at Mexican Hat are given in MK-E Calculation No. 9-239-05, Radon Barrier Design - Material Properties for Radon Barrier Design of the Final Engineering Design.

D.2.4.4 Diffusion coefficients

The MK-E Calculation No. 9-235-05, Radon Barrier Design - Material Properties for Radon Barrier Design of the Final Engineering Design presents the diffusion coefficients for all of the contaminated materials at the Mexican Hat site.

D.2.4.5 Ambient radon concentrations

The ambient radon concentrations at the Mexican Hat site are discussed in Appendix D, Addendum D7, Additional Radiological Data.

FIGURE D.2.1
LOCATIONS OF BACKGROUND EXTERNAL GAMMA
MEASUREMENTS AND BACKGROUND SOIL SAMPLES

FIGURE D.2.2
EXPOSURE RATE CONTOURS DERIVED FROM AERIAL SURVEY DATA
TAKEN OVER MEXICAN HAT, UTAH, AND SURROUNDING AREAS
(FROM EG & G, 1982)

Table D.2.1 Background radiation levels and background concentrations of radionuclides in surface soil near Mexican Hat, Utah

Sample point	Description of sample location	External exposure rate ^a (microR/hr)	Nuclide concentration (pCi/g)		
			Ra-226	Th-232	U-238
AZ1	In valley 9.7 km south of tailings	9	1.7	0.5	0.5
AZ2	South side of Hwy 64, approx. 0.6 km west of intersection of Hwys 89 and 64	10	0.9	1.3	0.5
AZ3	North side of Hwy 89, 2.4 km east of Glen Canyon Dam (mileage marker 548)	5	0.2	0.2	0.1
AZ4	South side of Hwy 160, approx. 0.4 km east of intersection of Hwys 160 and 89	11	2.0	1.0	0.9
AZ5	Near rest stop on Hwy 264, 11.4 km east of Tuba City, Arizona	6	0.4	0.4	0.2
AZ6	South side of Hwy 160, 3.2 km west of Kayenta, Arizona	12	0.4	0.4	0.2
UT19	12.9 km south of Mexican Hat, northeast corner of intersection of Hwy 163 and road to Monument Valley site	8	0.8	b	0.3
UT20	East side of Hwy 163, 4.8 km south of Blanding, Utah	10	1.1	b	0.4
	Average	9	0.9	0.6	0.4

^aOne meter above the ground.

^bNo analysis for this nuclide was performed.

Ref. DRNL, 1980.

Table D.2.2 Measured depths (feet) to the physical interface and 15 pCi/g interface for MSRD holes at Mexican Hat

Hole I.O.	Tails depth	Subbase depth	15 pCi/g interface depth	Hole I.O.	Tails depth	Subbase depth	15 pCi/g interface depth	Hole I.O.	Tails depth	Subbase depth	15 pCi/g interface depth	Hole I.O.	Tails depth	Subbase depth	15 pCi/g interface depth
A-1	5.1	0.0	msg	B-31	27.0	0.5	msg	D-61	11.4	1.2	msg	B-91	8.3	1.6	msg
A-2	0.0	1.0	msg	B-32	37.5	0.3	msg	C-62	34.0	0.4	msg	B-92	10.7	1.0	11.5
A-3	0.0	2.4	msg	B-33	19.0	2.4	22.0	D-63	23.2	0.9	msg	B-93	34.6	0.9	35.5
A-4	0.0	3.4	4.0	B-34	23.0	0.4	msg	D-64	46.3	0.6	msg	C-94	14.7	0.3	msg
A-5	3.9	0.0	6.0	B-35	29.4	1.6	msg	C-65	29.0	0.9	msg	C-95	20.0	0.2	msg
A-6	7.1	0.2	msg	A-36	21.6	0.6	msg	D-66	47.0	0.2	msg	C-96	12.2	1.3	msg
A-7	9.0	10.0	msg	A-37	12.6	1.9	msg	C-67	19.5	3.0	21.0	C-97	7.5	0.0	msg
A-8	12.2	1.5	13.7	A-38	9.2	5.6	msg	D-68	3.0	0.9	msg	B-98	24.5	0.5	msg
A-9	8.5	8.5	10.0	C-39	1.0	0.9	msg	C-69	4.5	3.0	6.5	B-99	10.0	1.0	11.0
A-10	5.5	1.9	7.5	C-40	4.5	0.3	msg	D-70	13.2	1.0	msg	D-100	41.0	5.2	msg
A-11	0.0	0.0	msg	C-41	9.4	1.3	msg	C-71	1.0	2.4	msg	B-101	47.0	1.9	48.0
B-12	15.0	0.3	15.5	C-42	11.6	0.3	12.0	D-72	0.3	0.9	msg	B-102	37.0	2.7	39.0
A-13	5.0	0.5	6.0	C-43	11.5	3.3	12.5	C-73	0.2	0.7	1.5	B-103	35.0	2.9	37.0
B-14	26.0	0.4	msg	C-44	22.0	1.2	23.2	D-74	0.3	1.5	msg	B-104	34.2	1.7	35.9
B-15	17.0	3.0	19.0	C-45	21.2	1.5	msg	C-75	4.5	1.7	msg	B-105	52.5	2.3	msg
B-16	33.0	2.9	35.0	C-46	9.5	0.3	msg	D-76	7.2	0.7	msg				
B-17	22.0	1.9	24.0	C-47	26.0	1.5	msg	C-77	15.0	2.4	msg				
B-18	21.3	1.3	msg	C-48	35.0	0.4	msg	D-78	18.4	0.7	msg				
B-19	24.0	1.2	msg	C-49	33.4	0.8	msg	C-79	16.8	3.0	msg				
A-20	0.0	0.4	msg	B-50	54.5	0.3	msg	D-80	18.0	0.4	msg				
B-21	12.5	0.9	14.0	B-51	39.5	2.9	43.0	C-81	25.0	1.4	26.5				
B-22	16.5	1.0	msg	B-52	41.2	0.0	msg	D-82	22.2	0.5	23.0				
B-23	17.6	1.0	19.5	B-53	18.0	0.4	msg	C-83	22.0	2.9	23.0				
B-24	27.7	0.5	msg	C-54	34.7	0.1	msg	B-84	42.6	1.4	msg				
A-25	1.0	1.0	0.0	B-55	32.4	0.7	msg	D-85	1.0	1.0	9.0				
B-26	33.8	0.5	msg	C-56	29.6	0.3	msg	B-86	30.0	1.9	msg				
A-27	0.0	0.0	msg	B-57	25.0	1.1	msg	C-87	15.1	1.1	18.0				
B-28	12.0	1.0	msg	C-58	24.0	0.2	msg	B-88	35.0	0.9	37.0				
A-29	7.3	3.0	10.0	B-59	29.0	0.3	msg	C-89	13.0	1.5	14.5				
B-30	30.0	1.4	32.0	C-60	29.2	0.5	msg	C-90	5.0	1.0	msg				

D.3 GEOLOGIC CHARACTERISTICS

D.3.1 INTRODUCTION

Detailed investigations of geologic, geomorphic, and seismic conditions at the Mexican Hat site were conducted to provide a basic site characterization and an identification of potential geologic hazards that could affect long-term site stability. Subsequent engineering studies (e.g., analyses of hydrologic and liquefaction hazards) used the data developed in these studies. The geomorphic analysis was employed in the design of effective erosion protection. Studies of the regional and local seismotectonic setting, which included a detailed search for possible capable faults within a 65-km radius of the site and review of historic earthquake data and major studies by previous investigators, provided the basis for estimation of seismic design parameters.

The scope of work performed included the following:

- o Compilation and analysis of previous published and unpublished geologic literature and mapping.
- o Review of historical and instrumental seismic data.
- o Review of site-specific UMTRA Project geologic data, including logs of exploratory boreholes advanced in the site area.
- o Photogeologic interpretation of existing LANDSAT and conventional aerial photography.
- o Low-sun-angle aerial reconnaissance of the site region.
- o Ground reconnaissance and mapping of the site region.

The characterization includes the regional geology as the first phase of the geologic studies. Detailed site geology is then correlated to the regional overview and various hazards are evaluated from the results to determine site suitability. These various hazards include:

1. Geomorphic hazards, including mitigative measures. The individual hazards assessed include:
 - o Channel erosion, aggradation, degradation, avulsion, and lateral shifting or fluvial systems.
 - o Flooding.
 - o Gully erosion and headward migration.
 - o Mass movement, including landslides, debris flows, soil creep, mud flows, rockfalls, and snow avalanches.

- o Expansive, collapsing, and dispersive soils.
 - o Soil erosion and scarp retreat.
 - o Change in erosion from climatic change, or tectonic or other change in base level.
 - o Ground subsidence, including that from mine collapse, dissolution of bedrock layers, and fluid withdrawal.
 - o Wind erosion.
2. Man-made hazards (i.e., future recovery of economic minerals, aggregates, or petroleum products in the site area).
 3. Volcanic hazards.
 4. Seismic hazards which provide the following initial design parameters:
 - o Design earthquake and resultant acceleration.
 - o On-site fault rupture potential.
 - o Potential for site damage from earthquake-induced slope failures.
 - o Hazard to the site from reservoir-induced seismicity.

This section first presents a regional overview of geologic lithology and structures, and a site review including physiography and local site geology. A discussion of the mineral resources of the site area follows. The seismotectonic setting and geologic hazards analysis are presented in subsequent sections.

This organizational framework allows a presentation of the regional lithology and structure and subsequent relation of this to local site geology, including site-specific surficial and subsurface data generated in previous UMTRA Project studies. Based on these, evaluation is accomplished for geomorphic and seismic hazards incorporating data from specific studies of these subjects.

Addendum D3 presents the National Oceanic and Atmospheric Administration (NOAA) listing of earthquakes within a 200-km (about 125-mi) radius of the site and a tabulation of faults and lineaments in the site region.

D.3.2 CRITERIA AND DEFINITIONS

This section presents definitions and criteria used to perform site hazard evaluations at UMTRA Project sites. These are presented to standardize usage throughout this section, and are pertinent to the seismic hazard evaluation because of the wide range of interpretation or usages of certain seismological terms.

Design life. As specified by the EPA promulgated standards for remedial actions at inactive uranium processing sites (10 CFR Part 192), the controls implemented at the Mexican Hat site are to be effective for up to 1000 years, to the extent reasonably achievable, and, in any case, for at least 200 years. In the case of assessing seismic hazards, the criteria established and the methodologies applied seek to ensure that the reclaimed wastes will not be damaged by earthquake ground motions or related ground rupture for up to 1000 years.

Design earthquake. The magnitude of the earthquake which produces the largest on-site peak horizontal acceleration is the magnitude of the design earthquake. This controlling earthquake could be the floating earthquake or an earthquake whose magnitude is derived from a relationship between fault rupture and/or fault length and maximum magnitude.

Capable fault. A capable fault is defined as a fault which has exhibited one or more of the following characteristics:

- o Movement at or near the ground surface at least once within the past 35,000 years or movement of a recurring nature within the past 500,000 years.
- o Macroseismicity instrumentally determined with records of sufficient precision to demonstrate a direct relationship with the fault.
- o A structural relationship to a capable fault such that movement on one fault could be reasonably expected to cause movement on the other.

This definition is essentially the one adopted by the NRC for the siting of nuclear power plants (10 CFR Part 100, Appendix A, 1975).

Acceleration. Acceleration is defined as the mean of the peaks of the two horizontal components of an accelerogram record. The exact term used is "peak horizontal acceleration." The design accelerations are determined from the constrained attenuation relationship based on distance and magnitude developed by Campbell (1981). The mean plus one standard deviation (84th percentile) values are adopted. The design value is considered a nonamplified peak horizontal acceleration in the free field.

Magnitude and intensity. Magnitude was originally defined by C. F. Richter as the base-10 logarithm of amplitude of the largest deflection observed on a torsion seismograph located 100 km from the epicenter. This local magnitude value may not be the same as the body-wave and surface-wave magnitudes derived from measurements at teleseismic distances. Unless specified otherwise, Richter magnitude values will be used in the seismic hazard evaluations.

Intensity is the index of the effects of an earthquake on the human population and man-made structures. The most commonly applied scale is the 1931 Modified Mercalli Intensity Scale, one which will be used in the studies.

Because pre-instrumental earthquakes are reported in intensity and more recent instrumental records are in magnitude, there may be a need to relate these values. Several equations have been proposed. Unless otherwise specified, the relationship developed by Gutenberg and Richter (1956) will be applied. This equation is as follows:

$$M = 1 + 2/3 I_0$$

where M = magnitude in the Richter scale and I_0 = Modified Mercalli (MM) Intensity in the epicenter area.

Floating earthquake (FE). An FE is an earthquake within a specified seismotectonic province not associated with a known tectonic structure. Before assigning the maximum FE magnitude, the earthquake history and tectonic character of the province are analyzed.

Geomorphic evaluation. The purpose of the geomorphic evaluation of the Mexican Hat site is to characterize the current geomorphic conditions and to assess the impact of geomorphic processes on the long-term stability of the uranium mill tailings piles. These evaluations are restricted to the assessment of natural geomorphic processes and the geomorphic effects of past land-use activities, and do not address future human activities or potential hazards related to site hydrology.

Schumm and Chorley (1983) have prepared a detailed publication presenting a theoretical discussion of geomorphic processes which may affect a tailings site. Nelson et al. (1983) present a handbook approach to specific methods for site assessment, engineering procedures for mitigation, and confidence levels for hazard predictions over periods of 200, 500, and 1000 years. The methodologies and criteria presented in those publications were used as guides for the geomorphological investigations of the Mexican Hat site.

D.3.3 SCOPE OF WORK

D.3.3.1 Compilation and analysis of previous geologic work

The initial phase of investigation for the site geologic characterization consisted of collection and review of all available pertinent data. The data include the subjects of lithology, stratigraphy, structure, seismicity, geomorphology, mineral resources, tectonics, and soils. The acquired publications are based on geological, geophysical, seismological, and agricultural studies in the site region, and include those generated in earlier UMTRA Project studies for the site.

This compilation is based, in part, on a GEOREF data search yielding an extensive bibliography on various subjects including those pertinent to this study. The size of this compilation precludes inclusion as an addendum to this appendix and is available on request from the UMTRA Project Office, Albuquerque, New Mexico.

Based on this compilation of data, a regional fault map was compiled (1:250,000 scale) for all mapped faults within a 65-km (40-mi) radius of the site, including all mapped regional faults which are near or extended outside this radius. Figure D.3.1 is derived from this large-scale compilation map. Published regional and local geologic and topographic maps were used as a base to complete this study in those subject areas.

D.3.3.2 Earthquake data compilation

For an analysis of seismic hazards to the site, an extensive data base of historical seismicity was compiled. This includes generation of an updated NOAA search for a 200-km (125-mi) radius of the site (NOAA, 1985). Seismicity from other regional studies of the site region was examined to determine completeness of the NOAA search and to include smaller magnitude events (microseismicity) which may not be included in the NOAA data base. These studies included Kirkham and Rodgers (1981) for Colorado; Arabasz et al. (1979) for Utah; DuBois et al. (1982) for Arizona; and Sanford et al. (1981) for New Mexico.

A search was also conducted for unpublished data on earthquake events in northern Arizona. The only regional seismic monitoring network in northern Arizona is run by the Geology Department of Northern Arizona University (NAU) in Flagstaff. A summary of earthquake activity from 1980 to 1985 from the NAU network (Brumbaugh, 1985a) updates the published summary of Arizona earthquakes (DuBois et al., 1982, 1981).

Regional and local seismological studies for other areas were also used to determine the floating earthquake within the site region and the Maximum Earthquakes (ME) for the seismotectonic provinces adjoining the site region. This determination allowed for an analysis of the effects of earthquakes in the Colorado Plateau and adjoining seismotectonic provinces on site seismic design parameters.

D.3.3.3 Site-specific geologic data

Subsurface geologic data for the Mexican Hat site consist of logs of boreholes drilled on and off the tailings pile in 1980, 1981, 1982, 1984, and 1985. The DOE and MSRDC/CSU boring logs are included as Addendum D4 of this appendix.

Subsurface site geology and hydrology were investigated and described by Fuhrman and Hintze (1976), FBDU (1977), and Colorado State University (CSU, 1983). Generalized site geology was mapped and described during studies by Sergeant, Hauskins, & Beckwith (SHB, 1985a,b) and Fuhrman and Hintze (1976). Potential borrow source locations and geology have

been investigated (SHB, 1984). A geomorphic hazard study for the site area was completed in 1984 (SHB, 1984).

Detailed site geology has not been mapped at a scale larger than 1:125,000. The stratigraphy and lithology of bed-rock units in the site area have been compiled from borehole logs and outcrop exposures. All geologic and borehole data were reviewed during the initial site data compilation. The site-specific geologic data were correlated with regional studies of geology and stratigraphy.

D.3.3.4 Ground reconnaissance and mapping

Ground reconnaissance and mapping for the Mexican Hat site were conducted for this study. Local geology and geomorphic hazards were evaluated on the site by the DOE between June 5 and 8, 1984; and January 6 and 7, 1986. Field notes for ground reconnaissance are presented in Addendum D3.

Ground inspection of structures suspected of being active faults, both small-scale near the site, and regional structures within 100 km (62 mi) of the site, was carried out by DOE between August 19 and 29, 1985. The compilation fault map of mapped faults and faults or lineaments detected by photogeologic studies was used as a basis for this reconnaissance. Ground studies were conducted on faults or fault systems numbered 9, 10, 13, 16, 17, 22, 23, and 24 on Figure D.3.1.

These studies resulted in an evaluation of geomorphic processes and hazards at the site and evaluation of the capability of faults in the site region that could affect seismic design parameters at the site.

D.3.3.5 Photogeologic interpretation

An evaluation of capable faulting from existing remote sensing imagery was performed for this study. This evaluation included photogeologic interpretation, using the methodology of Glass and Slemmons (1978), of 1:250,000 scale color composite LANDSAT imagery of the site region and conventional black-and-white and/or color aerial photography for a 65-km (40-mi) radius of the site. Scales of this photography varied from 1:18,000 to 1:24,000. Within the 65-km (40-mi) radius of the site, photographic coverage was complete except for one small, nearly five-by-five-square-mile area just southwest of Mexican Hat, Utah, in Townships 42S and 43S of Range 18E. Table D.3.1 is a compilation of photo numbers examined and location of the photography.

The Landsat imagery and aerial photography were inspected between August 19 and 29, 1985. A Wilde ST-4 mirror stereoscope using 3X multiplier lenses was used to examine suspect

features. Scale and quality of the photography and inspection methods utilized gave resolution limits of sharp geomorphic features of about 0.33 meter (one foot) minimum height and more subtle features of about one meter (3.3 feet) in height.

The photo interpretation study was combined with the fault map compilation (Figure D.3.1) to enable close examination of mapped faults. In addition, a search was made for unmapped faults, which were also compiled on the 1:250,000 scale fault compilation map, the base map for Figure D.3.1. Transfer was done by visual inspection.

D.3.3.6 Low-sun-angle aerial reconnaissance

Two low-sun-angle (LSA) aerial reconnaissance missions were flown over the study area. These missions had two primary objectives: (1) analysis of the geomorphic expression of mapped faults and fault systems for detection of Quaternary age (the last 1.8 million years) activity, and (2) inspection of the site region for unmapped faults or fault systems with possible Quaternary age activity.

The missions were flown in a five-passenger Cessna 206 Turbocharged single-engine high-wing aircraft. The flights were made on the evening of September 4, and the morning of September 5, 1985, with excellent weather and LSA illumination conditions. Missions were flown at altitudes of 500 to 3000 feet above the terrain for detection of small geomorphic features.

Glass and Slemmons (1978) indicate the most definitive indication of active faulting is oversteepened land surfaces (fault scarps). They also indicate that the single most effective method of detecting and delineating fault scarps is to conduct aerial reconnaissance and remote sensing using low solar irradiation angles to produce shadows or highlights on scarps. Slemmons (1977) indicates the use of LSA methods can greatly aid in delineating very subtle geomorphic features associated with active faulting.

The natural degradation of fault scarps in unconsolidated material has been described by Wallace (1977) and Bucknam and Anderson (1978) and occurs as a result of mass wasting and erosional processes. This slowly reduces the slope angle of the scarp over a period of several hundreds of years to a few million years. These scarp degradation studies, performed in the Basin and Range physiographic and structural province, are believed to be applicable to the site region because of similar erosional processes and climate. They indicate that any major surface faulting of Late Quaternary age (the last 500,000 years) should be readily detectable using LSA methods of observation.

LSA methodology has been discussed by Slemmons (1969, 1977), Clark (1971), Cluff and Slemmons (1972), and Glass and Slemmons (1978). They indicate LSA aerial reconnaissance in areas of low to moderate terrain is best conducted when the sun is between 8° and 25° in elevation above the horizon. These sun illumination angles occur in the approximate 2.5-hour time interval beginning 0.5 hour after sunrise or ending about 0.5 hour before sunset. Glass and Slemmons (1978) recommend a "multi" approach (i.e., using multiple observers, multiple times of day (morning and evening), and multiple season missions).

Missions flown for this study included multi-observers and multi-times of day. Multi-season observations are not necessary in the site region as vegetation differences with respect to seasons are negligible.

During the LSA aerial reconnaissance missions, all known faults within 15 km (9.3 mi) of the site were inspected and an intense low-altitude search for undetected faults within 10 km (6.2 mi) of the site was made. All faults of over a few miles in length within a 65-km (40-mi) radius of the site were inspected and any topographic structures which could result from faulting were inspected. All regional structures which could be capable of large or great earthquakes within 200 km of the site were also examined.

D.3.4 REGIONAL CONDITIONS

D.3.4.1 Physiographic setting

The Mexican Hat site is on the northern boundary of the Navajo Uplands section of the Colorado Plateau physiographic province (Hunt, 1967). The site is about 1.5 km (0.9 mi) south of the San Juan River, the border of the Navajo Uplands section, with the Canyonlands section to the north. This part of the Navajo Uplands section is characterized by: (1) extensive areas of gently dipping sedimentary rocks deformed by warping; (2) aridity and shortage of water; (3) large areas of bedrock overlain by thin surficial deposits; and (4) sparse vegetation and population.

The tailings are in the western flank of the valley of Gypsum Creek Wash, a north-trending ephemeral tributary of the San Juan River. The location of Gypsum Creek is governed by the axis of the Mexican Hat syncline. The tailings overlie the lowermost member (Halgaito Tongue) of the Cutler Group. The bedrock stratigraphy dips about 6° to the east, toward the axis of the Mexican Hat syncline. West of the site, highlands are formed by a doubly plunging, gently folded anticlinal structure, the Halgaito anticline. To the east, highlands are formed by the moderately dipping limbs of the Raplee anticline. The Mexican Hat syncline occupies the area between the two anticlines.

Drainages in the tailings site drain to the northeast, to Gypsum Creek, and then northward into the San Juan River. The site elevation is about 1305 m (4280 ft) and topography varies in elevation from about 1250 m (4100 ft) at the San Juan River to over 1705 m (5600 ft) in the Raplee anticline structure to the east.

D.3.4.2 Regional geology

Exposed consolidated strata in the Mexican Hat area range in age from Pennsylvanian to Permian. The oldest formation exposed is the Hermosa Group of Pennsylvanian age, which crops out along the canyon of the San Juan River and, immediately west of the site, crops out along U.S. Highway 163, southwest of the town of Mexican Hat, Utah. Unconformably overlying the Hermosa Group is the Cutler Group of Permian age which crops out east of U.S. Highway 163, underlies the site, and extends eastward to the westerly dipping beds of Raplee Ridge, east-southeast of the town of Mexican Hat, Utah (Figure D.3.1).

Rock strata older than the Hermosa Group are not exposed in the area of the tailings site. A few wells drilled for oil and gas adjacent to the area of the site have penetrated the entire Pennsylvanian and into the underlying Paleozoic section. These wells show that below the Hermosa Group is a sequence of rocks from 1450 to 1650 feet thick which includes rocks of Cambrian, Devonian, Mississippian, and Pennsylvanian age. So far as is known, none of the wells have penetrated rocks of the Ordovician or Silurian systems (O'Sullivan, 1965).

The Hermosa Group is subdivided into three formations. The lowest formation is the Pinkerton Trail and does not crop out in the San Juan Canyons. Overlying the Pinkerton Trail is the Paradox Formation which is exposed in canyons along the San Juan River northeast of Mexican Hat, Utah. The upper formation is the Honaker Trail which crops out throughout the Mexican Hat area along the canyon walls of the San Juan River.

The Hermosa Group consists dominantly of fine- to very fine-grained crystalline gray limestone. Interbedded with the limestone are minor amounts of gray, greenish-gray, and purple shale and siltstone. The shale and siltstone are in beds as much as two feet thick and commonly form slopes between limestone ledges. A few light-colored, very fine-grained sandstone beds are also in the Hermosa Group (O'Sullivan, 1965).

The lower two formations of the Hermosa Group will not be discussed herein as their stratigraphic and sedimentological significance to the site are insignificant. The Honaker Trail Formation consists of interbedded siltstones, limestones, shales, and sandstones. The thick siltstone units are pre-vaillingly reddish brown. Locally, thin, less than 1.5-foot lenticular beds or irregular masses of reddish-brown and gray

limestone are present. Chert and limestone nodules are found in thin beds in a siltstone matrix or scattered throughout the siltstone units. The siltstone units are commonly evenly bedded, but small-scale crossbedding is locally present (O'Sullivan, 1965). The limestone units are predominantly fine- to very fine-grained, crystalline, fossiliferous, and gray in color. The sandstone is generally very fine- to fine-grained, exhibiting parallel bedding with some small-scale, low-angle crossbeds present locally. These strata were deposited in a marine environment of a very shallow sea.

Disconformably overlying the Pennsylvanian-age Honaker Trail Formation is the basal member of the Permian-age Cutler Group. This disconformity has been recognized only recently (Wengerd, 1973). Regionally, past U.S. Geological Survey (USGS) reports recognized the "Rico Formation" as a transitional formation between the marine rocks of Pennsylvanian age below, grading into non-marine redbeds of Permian age above. The existence of regional unconformity indicates there are no transitional beds between Pennsylvanian and Permian age strata and therefore the Rico Formation does not exist in the Mexican Hat area. Wengerd (1973) includes all of the marine strata to the top of the Pennsylvanian sequence as Honaker Trail Formation (Baars, 1973).

The Halgaito Shale or Halgaito Tongue is the basal formation of the Cutler Group and consists of an interbedded sequence of very fine-grained silty sandstone and siltstone beds that are characteristically reddish brown. At places, the beds are micaceous and mottled very pale green or greenish gray. Calcareous, well-cemented beds alternate with softer beds to form ledges, slopes, and benches. Thin, lenticular, unfossiliferous beds of light-gray limestone are widely dispersed throughout the sequence. Beds of conglomerate occur at different levels within the Halgaito. The conglomerate consists of subrounded pebbles of siltstone and limestone generally in a silty matrix. Locally, beds of very fine- to fine-grained sandstone showing irregular bedding are present (O'Sullivan, 1965). It is probable that the Halgaito is composed mainly of continental lowland deposits that include stream channel and point bar deposits, floodplain deposits, and tidal flat accumulations. The Halgaito Member is the bedrock unit that directly underlies the tailings site and surrounding area primarily east of U.S. Highway 163 and along Monument and Gypsum Creeks. The contact between the Halgaito Member and the underlying Hermosa Group-Honaker Trail Formation is the top of the highest fossiliferous limestone bed of the Honaker Trail Formation.

Conformably overlying the Halgaito Member is the Cedar Mesa Sandstone Member of the Cutler Group. The Cedar Mesa Sandstone Member consists of two totally unlike facies--a sandstone facies present only in the area near Cedar Mesa, northwest of the site, and a gypsiferous facies present south

of the San Juan River and east of Gypsum Creek. (The sandstone facies will not be described herein as it is present only along Cedar Mesa and in remnants capping erosional features near Mexican Hat, Utah.) The gypsiferous facies consist dominantly of siltstone and shale and subordinate amounts of sandstone, gypsum, and limestone. The member is soft and weathers to a series of mounds and gentle slopes, although locally a resistant ledge may form a minor hogback. The depositional environment of the gypsiferous facies of the Cedar Mesa Sandstone was probably in a near-shore, littoral to sublittoral environment (O'Sullivan, 1965).

Rock units stratigraphically above the Cedar Mesa Sandstone Member crop out farther east of the site along Comb Ridge. These formations include the Organ Rock and DeChelly Sandstone Members of the Cutler Group, Triassic-age Moenkopi and Chinle Formations and Wingate Sandstone, Triassic-Jurassic-age Kayenta Formation and Navajo Sandstone, Jurassic Carmel Formation, Entrada Sandstone, Summerville Formation, Bluff Sandstone, and Morrison Formation. These strata crop out some distance away from the site and are of no significance to the geologic setting underlying the site and therefore are not described herein.

Sparse volcanic rock in the site area consist of Pliocene-age explosive breccias and intrusive dikes with associated small stocks. Surficial deposits consist of eolian sand occurring as unconsolidated dunes in low-lying areas. Minor terrace gravels of the San Juan River occur about 1.9 km (1.2 mi) north of the site, at the town of Mexican Hat, Utah.

D.3.4.3 Regional structure

The Mexican Hat site is in the central portion of the Colorado Plateau, a stable intracontinental subplate characterized by a thick cover of relatively flat lying sedimentary rock of Phanerozoic age overlying a complex Precambrian igneous and metamorphic core. The central, stable portion of the plateau exhibits characteristics of cratonic areas while the margins of the subplate exhibit crustal structure similar to more highly active bordering provinces. The plateau is bordered on the east, south, and west by the extensional, block-faulted regime of the Rio Grande Rift and Basin and Range Provinces.

The principal present-day structural elements of the Colorado Plateau are as follows (Hunt, 1967) (Figure D.3.2):

- o Broad west- and northwest-trending basins such as the Uinta, San Juan, and Navajo Basins.
- o Clusters of uplifts which lie between these broad basins, including the Monument, San Rafael Swell, Circle Cliffs, Kaibab, Defiance, and Zuni Uplifts.

- o Northwest-trending anticlines and faults of east-central Utah and southwestern Colorado, underlain by thick salt deposits.
- o Northward-trending fault blocks of the high plateaus in Utah and Arizona. These features represent a zone of transition between the Colorado Plateau and Basin and Range Provinces.
- o Domes and other folds related to laccolithic intrusions, most of which are in the central part of the plateau.

The modern day structural elements of the plateau are principally the result of Cenozoic deformation, but follow, at least in part, the trends of older features. Structures in the Precambrian basement have undoubtedly influenced the later tectonic history of the plateau. However, relatively little is known of the Precambrian structure. Precambrian rocks are exposed only in the Uncompahgre Uplift, Grand Canyon, Zuni Mountains, and Defiance Uplift (Hunt, 1967). These rocks consist of a complex series of metasedimentary and metavolcanic rocks cut by a series of large intrusions. During much of Paleozoic and Mesozoic time, the plateau region was occupied by a stable continental shelf area. From the Late Mississippian to Permian time, the "Ancestral Rockies" disturbance affected parts of the eastern plateau and the Rocky Mountain region further east. The Uncompahgre Uplift rose during this time, while the Paradox Basin received large volumes of Pennsylvanian and Permian redbeds and evaporite deposits (Cater, 1966, 1970). The Zuni Mountains and Defiance Upwarps probably were also uplifted at this time (Hunt, 1967). The Late Cretaceous-Eocene Laramide orogeny produced a series of monoclines and fault-bounded uplifts and basins within the plateau. From the end of the Eocene until Miocene or Pliocene time, the plateau was undergoing erosion by the ancestral Colorado River. Since late Tertiary time, the plateau has been experiencing gradual uplift at an average rate of about two mm/year (Gable and Matton, 1980). This uplift is regional in character and produces relatively little internal deformation.

The Mexican Hat site lies on the eastern edge of the Monument Uplift (Monument Upwarp of O'Sullivan and Beikman, 1963) (Figure D.3.3). South and east of the Monument Uplift is the Tyende Saddle (O'Sullivan and Beikman, 1963), separated from the uplift by the Comb Ridge Monocline, a major monocline of the Colorado Plateau (Kelley, 1955), with a length of about 153 km (95 mi). Stratigraphic units are displaced a maximum of 914 m (3000 ft) at a maximum dip angle of 45° across the Comb Ridge Monocline (Kelley, 1955).

Overprinted on these major subprovinces defined by folded uplifts are smaller amplitude and more gently folded features. Within the Monument Uplift and west of the Mexican Hat site

about 13 km (eight mi) is the Halgaito anticline, a north-south trending, doubly plunging structure about 16 km (10 mi) in length and 150 m (500 ft) in amplitude. Another structure of equivalent length but over 300 m (1000 ft) in amplitude, the Raplee anticline, is about 4.8 km (three mi) east of the site.

The major structural feature of the area is the Monument Upwarp, a broad north-south trending uplift about 35 miles wide and 100 miles long that extends from Kayenta, Arizona, to near the junction of the Colorado and Green Rivers in Utah. Smaller anticlines and synclines with north-south to northeast-southwest trends are superimposed on the major structure. The upwarp is asymmetrical, and has a steeply dipping east limb and a gently dipping west limb. Comb Ridge, which marks the eastern flank of the uplift, lies about eight miles to the east of the site.

Faulting is rare in the Mexican Hat area, and the few faults which have been mapped have offsets ranging from only a few inches to a few feet. None are mapped as offsetting strata of Quaternary age. The dominant geologic structural features probably formed during the Early Tertiary, and there has been minimal tectonic activity in the region since at least the end of Eocene time. The ages of the last episodes of volcanic intrusion are not known, but most studies of the area conclude that the latest volcanic activity occurred during the Pliocene (e.g., Williams, 1936, as quoted by O'Sullivan, 1965).

D.3.4.4 Regional geomorphology

Hunt (1967) subdivided the Colorado Plateau physiographic province into physiographic units (Figure D.3.4). The Mexican Hat site lies on the northern border of the Navajo Uplands section adjacent to the Canyonlands section, north of the San Juan River. Essentially, the Navajo Uplands section is a large depression enclosing Black Mesa and the San Juan Basin. The so-called "valley" of Monument Valley is the eroded broad summit area of the Monument Upwarp, a major feature that continues northward into Utah and the Canyonlands physiographic section. Erosion has cut below the Mesozoic rocks of the Monument Upwarp into the continental Permian redbeds and sandstones. In the site region, on the east edge of the Monument Upwarp, erosion has generally reached the lower members of the Cutler Group or the underlying Hermosa Limestone.

The site lies in northeast-trending channel tributaries to Gypsum Creek, an ephemeral stream that drains northward into the San Juan River. Gypsum Creek has eroded to the Halgaito Tongue of the Cutler Group or the underlying limestone of the Hermosa Group, but the channels at the site and the site itself lie in the Halgaito Tongue of the Cutler Group. Erosion of the Halgaito Tongue forms a dendritic drainage pattern with

very minor surface cover, except eolian sand dunes in low-lying areas.

Arroyos are formed generally in bedrock from infrequent periods of runoff, and channel incision is a very slow process in the site region. Gullying and headward erosion proceed slowly in channels cut into sandstone bedrock. Well developed gullies and more rapid erosion occur in channels incised into weakly cemented silty sandstone and deposits of alluvial and eolian materials. Alluvial-colluvial cover is generally thin and soils are very poorly developed.

The geomorphic features of the Colorado Plateau region of southeast Utah reflect mainly the processes of erosional denudation throughout the Cenozoic. The general geomorphic processes described herein are those characteristic to the physiographic subdivision of the plateau closest to the Mexican Hat tailings site--the eastern Navajo Uplands, and the eastern Monument Valley. Although the development of the ancestral and present Colorado River systems has been the subject of regional and local studies since the late 1800s, little has been published on the regional geomorphic development of the Colorado Plateau region of northern Arizona and southeastern Utah. Landform development during the Cenozoic Era has been influenced by the bedrock lithology, volcanic and tectonic processes, fluvial drainage system changes, and eolian erosion. Poorly developed soils and sparse vegetation have been attributed to the rates of differential erosion of the weakly- to moderately-resistant sedimentary formations (Stokes, 1973). Likewise, the tectonic history is only known in a general sense. Uplift of over one mile above mean sea level has occurred since the Cretaceous, but the rate and timing of uplift is not fully known. The history of climatic changes in the Colorado Plateau and Grand Canyon regions is now understood in some detail, but the effects of different climates on geomorphic processes is not well known.

The eastern part of the Navajo Uplands section curves to the east of the Mexican Hat site and lies between the Monument Valley section to the north and the Black Mesa section to the south. The southern inner border is the base of the slope leading to Black Mesa. Bedrock of the Navajo Uplands east of Mexican Hat consists of sandstones and mudstones of the Glen Canyon Group, the San Rafael Group, and the Morrison Formation, in ascending order. The Navajo Sandstone of the Glen Canyon Group is the most widespread and conspicuous component (Stokes, 1973). Much of the area is mantled by thin deposits of sand or sandy soils. Elevations range from about 1525 to 1780 m (5000 to 5835 ft). Weathering and erosion have produced a combination of mesas and "slickrock" surfaces cut by deep gorges eroded below the general level of the terraced surfaces. Drainage is mainly to the north to the San Juan River, but the sandy regolith results in many areas of no surface runoff.

West of the Mexican Hat site lies the Monument Valley subdivision of the Navajo country. The landforms of Monument Valley are developed on the Cedar Mesa Sandstone of the Cutler Group. The region is a nearly featureless, stripped surface with mesas, buttes, and spires rising abruptly from the flat plain (Barnes, 1984). Lower slopes of the buttes are formed on the Organ Rock Shale, the vertical cliffs consist of the DeChelly Sandstone, and the resistant caprocks are tan sandstones of the Shinarump Member of the Chinle Formation. The "valley" of Monument Valley is actually formed in the broad summit area of the Monument Upwarp, a major feature which continues northward into east-central Utah. Stream incision into the broadly-folded, low arch eroded the soft Organ Rock Shale leaving the mesas protected by thick caprocks of fluvial sandstones and gravels of the Shinarump Member (Barnes, 1984). Some mesas stand more than 260 m (853 ft) above the valley floors of about 1435 m (4700 ft). Surface drainage is mainly to the north in deeply incised tributaries to the San Juan River.

Northeast of Mexican Hat, Cedar Mesa and the Grand Gulch Plateau are developed on Pennsylvanian and Permian age limestones, sandstones, and siltstones of the Hermosa and Cutler Groups. This region consists of extensive, flat, stripped surfaces cut by narrow, deep gorges trending eastward toward Comb Wash and southward toward the San Juan River. Thin surficial deposits of eolian sand and alluvial sand and gravel occur sporadically on the plateau surface. The plateau terminates to the east at the Comb Ridge Monocline (Haynes et al., 1972). Elevations range from about 1515 m (5000 ft) near the San Juan River to about 1983 m (6500 ft) on upper plateaus.

Erosional processes throughout the interior Colorado Plateau are related to the development of the Colorado River, Little Colorado River, and San Juan River systems since Miocene times (Lucchitta, 1984). Prior to Arizona Basin and Range faulting, the Little Colorado River and streams in central Arizona drained to the north toward Utah. Basin and Range faulting and opening of the Gulf of California Rift about five million years ago resulted in stream capture of the ancestral Colorado River system in Colorado and Arizona, downcutting into the sedimentary rocks of the Colorado Plateau, and development of the present drainage system. The major period of the Grand Canyon development appears to have occurred between one and six million years ago. Development of canyons and river downcutting has continued throughout the period of plateau uplift and will slowly reduce the plateau region to a smooth, subdued landscape. Processes of canyon head erosion and cliff retreat in rock units of varying erosional resistance will continue to produce the mesa and butte topography currently observable in the Monument Valley area.

Wind activity has been a major factor in the development of the landscape of northeastern Arizona and southeastern Utah.

Eolian deposits of Pliocene age indicate wind directions similar to those at present (Figure D.3.5). Longitudinal dunes of Pleistocene and Holocene age also indicate a prevailing southwest wind direction. Vast areas of the Colorado Plateau are covered with dunes and thin sand sheets. Eolian erosion continues to be an important factor in the denudation of the flat plateau surfaces (Figure D.3.5).

An increase in the magnitude and rate of stream arroyo development has been observed in the semiarid Southwest during the past 100 years. Studies of gully erosion in sediment-filled valleys in New Mexico, Arizona, and Colorado indicate an increased rate of gully development beginning in the mid- to late-1800s. The cause has been attributed to either livestock overgrazing or short-term climatic change. Gully erosion seems to be less intense on the plateaus and valleys of the Colorado Plateau interior. The shallow valley fills and thin sandy soils appear to have been major factors preventing the rapid development and growth of arroyos.

D.3.4.5 Rates of denudation

In reference to the possible influence of the long-term erosional process upon reclamation design, a discussion of the rates of denudation and neotectonics of the Colorado Plateau is provided below.

The rates of denudation are the rates at which a land surface is being lowered as a result of erosional processes. These rates vary with climate and the amount of precipitation. As reported by Schumm (1963), it is obvious that no surface is lowered in a uniform manner; however, rates have been estimated, generally for a period of 1000 years.

Denudation rates for drainage basins of about 1500 square miles are 0.29 foot per 1000 years from gauging station data, and 0.51 from reservoir data with an effective precipitation (amount required to produce a known amount of runoff) of 30 inches (Langbein and Schumm, 1958). In another publication, Schumm (1963) concludes that an average rate of denudation for a 1500-square-mile area is about 0.25 foot per 1000 years. For a small drainage basin in southwestern Utah, Eardley (1966) determined a rate of 2.75 feet per 1000 years.

In 1948 and 1949, a comprehensive survey was made of the sedimentation in Lake Mead as the result of construction of Hoover Dam. For 1935 and 1948, the average rate of sedimentation was estimated at 140 million tons for the Colorado River (Gould, 1960). The mean specific weight for the sediments was 64.6 pounds per cubic foot. The drainage basin for the Colorado River above Lake Mead is about 168,000 square miles. In addition, it is assumed that the specific weight of the

uneroded soils is 90 pounds per cubic foot. Using these figures, the rate of denudation for the Colorado River drainage basin for the period 1935 through 1948 is 0.93 foot per 1000 years.

Thomas (1960) prepared a map showing source areas as inferred from surface geology of the drainage area upstream from Lake Mead. This map indicates that the Mexican Hat site is in an area that produces sandy sediments. The amount of sediments produced from a unit area in these sediments would probably be less than the amount produced from areas of silt and clay.

The rates of denudation that have been determined are from large drainage areas. For smaller drainage areas, the variability in rates is apparent. In the case of this site, an extremely small area, the importance of a rate of denudation needs to be put in perspective in relation to the site geology.

Wind is the dominant factor acting on the surficial geology. Undoubtedly over a period of 1000 years, sediment would be removed by runoff, but this would not be as important as that removed by wind.

As an example of localized variation, Moenkopi Wash near Tuba City has cut down about 300 feet since the formation of the early Wapatki and Black Point surfaces. Damon et al. (1974) indicate that the rate of denudation in the Little Colorado River since the time of these surfaces is about 3.15 feet per 1000 years. Although the Moenkopi Wash has been dissected, the surfaces remain and are easily identifiable. Thus, Moenkopi Wash indicates a large rate of denudation, whereas the above-mentioned surfaces have not been subjected to such high rates.

D.3.4.6 Climate and vegetation

Details of site meteorology are presented in Section D.9 of this appendix. Rainfall in the site region is generally less than 20.3 cm (eight in) and maximum daily temperatures in July and August exceed 100°F (USGS, 1970; Hicks, 1969). This climate directly affects vegetation, leading to sparse cover. The vegetation type is controlled by topography, with a grass-shrub zone occurring below 1675 m (5500 ft), pinon-juniper between 1675 m (5500 ft) and 2285 m (7500 ft), and pine forest above 2285 m (7500 ft) in elevation (Hicks, 1969). The Mexican Hat site, at an approximate elevation of 1305 m (4280 ft), is in the grass-shrub zone dominated by isolated stands of grass and sparse growth of shrubs, principally greasewood and sagebrush.

Wind direction determined from modern sand dune morphology is predominantly from the southwest (Figure D.3.5). Paleoclimate studies of the Chuska Sandstone indicate a predominant

regional wind direction from the southwest during the Pliocene (Hicks, 1969).

D.3.4.7 Long-term climate variation

Major climatic changes during the next 1000 years in the Mexican Hat area will probably be of lesser magnitude than the climatic shift following the last full glacial period in the southwestern United States. This period ended in Arizona about 10,000 years ago. Paleoclimatic reconstructions for the last full glacial period differ in interpretation of precipitation and temperature in comparison with the present. Most interpretations of this period propose a climate characterized by increased precipitation, cooler summer temperatures, and milder winters for the desert areas (Spaulding et al., 1983). Other paleoclimatic data suggest a period of precipitation amounts similar to today's but with much colder summers and winters (Brackenridge, 1978). Studies in Canyon de Chelly, Arizona (Betancourt and Davis, 1984), and in the Chaco Canyon region of New Mexico (Betancourt et al., 1983; Hall, 1977) indicate a late glacial climate for the Colorado Plateau characterized by cooler summers and more abundant rainfall concentrated in the winter months. Paleoeological reconstructions based on evidence from the eastern Grand Canyon and the Mogollon Rim regions also indicate glacial climatic conditions cooler and probably moister than at present. Upper elevation plateaus north of latitude 36°N probably experienced severe winters. Paleohydrologic reconstructions of the Colorado Plateau region suggest an increase of total annual precipitation of 18 to 25 cm (seven to 10 in) (Spaulding et al., 1983). Analysis of periglacial features in the San Francisco Peaks and the White Mountains of Arizona indicate mean annual mountain temperatures 5°C to 6°C (9°F to 11°F) lower than today's (Péwé et al., 1984). Colorado Plateau temperatures were probably lowered by about the same amount.

Although no paleoclimatic studies are available for the local Mexican Hat area, a generalized late glacial climatic reconstruction can be made for the Colorado Plateau interior. Based on an increase in total annual precipitation of 18 cm to 25 cm (seven to 10 in), the full glacial annual precipitation may have been as great as 33.5 cm to 40.5 cm (13 to 16 in). Using an average lowering of 5.5°C (10°F), the average annual temperature about 10,000 years ago was about 7.5°C (45°F). Summer temperatures were probably much lower than today's, while winter temperatures were only slightly lower (Spaulding et al., 1983). Annual evaporation rates for soil moisture may have been 10 to 50 percent less than current rates. It is important to note that the magnitude of climatic change differs throughout the Southwest depending on geographic location, site elevation, and local topography.

The record of climatic variation during the past 10,000 years (Holocene) in the northwestern New Mexico region of the Colorado Plateau has been inferred from soil and alluvial stratigraphy and paleoecological evidence. In Chaco Canyon, botanical evidence supports increasing annual temperatures and summer monsoon patterns by about 8300 years before the present (B.P.) (Betancourt et al., 1983). Holocene warming in the White Mountains of Arizona commenced about 10,000 years B.P. (Péwé et al., 1984). Decreasing effective precipitation and warmer temperatures allowed desert grasslands to replace conifer woodlands by about 8000 years B.P. (Baker, 1983). A major period of incision of alluvial fills in the Chaco Canyon area from about 8000 to 7000 years B.P. implies increased water runoff and possible large flood events during summer monsoons (Hall, 1977). Widespread regional erosion suggests a change from cool, moist conditions to warm, moist conditions (Knox, 1983). An increase in aridity in northwestern New Mexico from about 7000 to 2400 years B.P. correlates with the establishment of the present-day desert vegetation in the Southwest. Throughout the Southwest, the intensity of alluviation lessened from about 3000 to 2000 years B.P., indicating a slight increase in precipitation. Slightly more arid conditions existed from about 2000 to 850 years B.P., followed by increased rainfall until about 650 years B.P. (Lowe, 1980). A period of fairly stable climatic conditions similar to those of today's existed until the onset of the late 19th century channel trenching.

In northeastern Arizona, the end of the Pleistocene was characterized by the disappearance of mountain glaciers and a climatic period similar to today's. This was followed by a short period of warmer, drier conditions during the early Holocene (Péwé et al., 1984). This in turn was followed by a cool, wet time during which small mountain glaciers formed in the White Mountains. A drier and warmer climate followed, continuing to the present time.

An increase in the rate and magnitude of gully erosion began in the mid- to late-19th century throughout the Southwest. The initiation of erosion is thought to be a result of either livestock overgrazing or short-term climatic change. Previous work on the causes of increased gully erosion is summarized by Euler et al. (1979). Along the Rio Puerco, east of Grants, New Mexico, gully erosion apparently began in the 1840s. In other areas of the Southwest, gully erosion began in the late 1880s. Most investigators agree that a brief period of decreased moisture preceded gully erosion and that overgrazing initiated gully formation. Gully erosion may have developed later in response to drought without overgrazing.

Climatic variation within the last 1000 years in Arizona has probably been significantly less than the glacial-postglacial change. Interpretations of Colorado Plateau climatic conditions from tree ring studies (Stockton and Jacoby,

1976) and historical records show a high degree of variability in precipitation during the past few hundred years. Average ranges of temperature and precipitation for the past few hundred years and for the future few hundred to a thousand years probably are similar to the range of historical records, although it is recognized that extreme events beyond the range of recorded variability probably occurred in the past and can be expected to occur within the next thousand years. Extreme precipitation events may produce rapid surface-water runoff, localized flooding, and increased channel erosion and deposition.

D.3.5 SITE GEOLOGY

The Mexican Hat site lies in northeast-trending tributaries along the western flank of Gypsum Creek, a north-trending drainage which joins the San Juan River. The bedrock at the site consists of gently dipping rocks of the Halgaito Tongue, lowermost member of the Cutler Group. The overlying Cedar Mesa Member of the Cutler Group crops out about 1.6 km (one mi) east of the site in Gypsum Creek Wash. Disconformably underlying the Halgaito Tongue at the site is the Hermosa Group.

The site lies on the west limb of the Mexican Hat syncline, the structure separating the Halgaito anticline to the west and the Raplee anticline to the east (see Figure D.3.3). The axis of the north-south-trending Mexican Hat syncline lies about 4.8 km (three mi) east of the site. These two anticlines and the intervening Mexican Hat syncline are gently- to moderately-folded, minor structures on the edge of the Monument Upwarp, a regional feature of the Colorado Plateau. The eastern edge of the Monument Upwarp in the site region coincides with the eastern flank of the Raplee anticlinal structure. Figure D.3.6 is a generalized geologic map of the site region.

Based on the logs of test borings advanced on the site during this investigation, a geologic cross section through the site area was prepared (Figure D.3.7). The site is on east-southeast-tilted Halgaito Tongue redbeds. There are only minor eolian sand dune deposits as surficial cover of the bedrock. The Halgaito is 15 to 30 m (50 to 100 ft) thick at the site. The thickness of the Hermosa at the site is unknown. The Halgaito thins to the east and the Hermosa thickens to the east. Figure D.3.8 is a generalized stratigraphic column for the site area. Borehole data for the site are in Addendum D4 of this appendix.

The tailings at Mexican Hat are in two adjoining piles. The tailings rest within a small drainage which originally flowed north-eastward to a confluence with Gypsum Creek. The relatively low topography to the north and west is interrupted by incised arroyos, while an alignment of small hills flanks the lower impoundment to the south. A relatively small watershed is present above the tailings impoundments. Runoff from this area is presently around the disposal area in a diversion

channel which flanks the site to the south. The site is about 240 feet higher than the San Juan River, which is about 1.3 miles to the north.

In addition to the truncation of the main arroyo by the lower tailings dam, three other significant secondary drainage paths extend up to the northern and western toe of the lower embankment. The heads of these arroyos contain little, if any, alluvium, and rocks of the Cutler Group are persistently exposed transverse to the drainage paths.

The site lies on gently sloping eroded bedrock. No indications of subsidence or landslide features are present. The drainage is governed by two northeast-trending ephemeral stream channels through the site (Figure D.3.9). The absence of surficial cover precludes expansive, collapsing, or dispersive soils, but the cold winter temperatures could affect frost heaving, solifluction, and slope creep in the eolian sand or the tailings under proper moisture conditions. Wind has removed some of the tailings to the northeast of the existing piles.

D.3.6 ECONOMIC GEOLOGY

Known mineral and fossil fuel deposits in the area of the tailings site are natural aggregate, crushed stone (manufactured aggregate), and oil and gas.

Natural aggregate deposits in this area are confined to a few scattered occurrences of stream and terrace alluvium. A high-level terrace alluvium occurs at several localities along the San Juan River in the Mexican Hat, Utah, area. The largest deposit of this type is on the north edge of the San Juan River and immediately west of U.S. Highway 163. This deposit caps a series of low-rounded hills and consists of up to 20 feet of gravelly material. Similar deposits of high-level terrace gravels occur elsewhere along the San Juan River (BIA, 1955).

Crushed stone sources consist predominantly of limestone which crops out at several localities in the vicinity of Mexican Hat, Utah. Several of these sites are within the San Juan River Canyon and would require the removal of thick rock overburden or underground quarrying to produce a large tonnage. The most accessible and ideal deposit of limestone occurs southwest of Mexican Hat along U.S. Highway 163. At this site, a limestone bed three to four feet thick crops out on the eroded dip-slope of the Halgaito anticline. The limestone at this site is essentially bare or covered by a thin silt and "blow" sand overburden (BIA, 1955).

Oil and gas are present in the Pennsylvanian age sedimentary bedrock of the Mexican Hat Oil Field. This oil field lies directly north of the San Juan River and extends from the town of Mexican Hat northward for a distance of about three miles (Wengerd, 1955). Production from this oil field is ongoing, and to December, 1984, the Mexican Hat Oil Field had produced 10,951 barrels of oil, 1584 million cubic feet of gas, and 673 barrels of water (Carney, 1985). Drill holes west of the

tailings piles encountered oil and gas traces within the Honaker Trail Formation at depths of 175 to 190 feet.

D.3.7 SEISMOTECTONIC SETTING

D.3.7.1 Regional setting

Seismic hazard studies in much of the southwestern United States are hampered by the lack of a reliable long-term historical record. Movements on major fault systems in the region may have recurrence intervals on the order of tens to hundreds of thousands of years, while the historical record dates back only to the middle or late 19th century. The historical record for Arizona dates back to 1776 (DuBois et al., 1982); for Utah to 1850 (Arabasz et al., 1979); for Colorado to 1870 (Kirkham and Rogers, 1981); and for New Mexico to 1849 (Sanford et al., 1981). Reliable and reasonably complete instrumental records generally date back only to the early 1960s. As a general rule, the historical record is probably reliable for moderate to large earthquakes since about 1900 to 1910, while the instrumental record is probably reliable for earthquakes of magnitude 4.5 or greater since the early 1960s (Von Hake, 1984).

In the absence of a reliable long-term historical record, probabilistic analyses of seismic risk are of limited use. Therefore, seismic risk analyses are largely based on studies of the geologic and seismotectonic setting, Cenozoic geologic history, and geomorphic evidence of late Tertiary and Quaternary fault movements. Fortunately, erosion rates are slow and vegetation is generally sparse in the arid to semiarid climates that prevail in most of the region. Long faults, which are necessary for large earthquakes, will not remain undetected if careful geologic investigations are made (Krinitzsky and Chang, 1975).

The site is near the geographic center of the Colorado Plateau physiographic and seismotectonic province. The boundaries of seismotectonic provinces in the site region, as defined for this study, are shown on Figure D.3.10. They are determined on the basis of published studies of Neogene faulting, regional seismicity trends, areas of Cenozoic igneous activity, geophysical data, and the distribution of major physiographic provinces. Also shown on Figure D.3.11 is a plot of historical and instrumentally located earthquake epicenters (for events of magnitude ≥ 4 and intensity (I_{MM}) $\geq V$) for the Colorado Plateau region. The epicentral plot is also reproduced on Figure D.3.11. These data were provided by the National Oceanic and Atmospheric Administration (NOAA), National Geophysical Data Center (NGDC). A listing of the epicentral data used in compilation of this figure is available from the UMTRA Project Office, Albuquerque, New Mexico.

The Colorado Plateau, Basin and Range, Rio Grande Rift, and Sierra Nevada Provinces appear to be part of an inter-related system that has experienced major uplift and extension during the last 20 million years (Thompson and Zoback, 1979). Within the Basin and Range and Rio Grande Rift bounding the Plateau are found geologic and geomorphic evidence of repeated surface faulting events associated with large earthquakes during Quaternary time. They have experienced some of the largest historical earthquakes in the entire United States. These regions are characterized by large volumes of Cenozoic intrusive rock, thinner crust, higher heat flow, and stress fields oriented differently than the modern stress field in the interior of the Plateau (Thompson and Zoback, 1979). The boundary of the Colorado Plateau and the Basin and Range Province on the west is marked by the Wasatch Frontal fault system in Utah, which forms a major segment of the Intermountain Seismic Belt (Smith and Sbar, 1974). The transition zone in northern and central Arizona is referred to in this study as the Arizona Border Zone. Some of the largest historical earthquakes of the Colorado Plateau have occurred in this region. The Rio Grande Rift and the eastern Colorado Plateau border zone in New Mexico and southwestern Colorado and the border zone of the Colorado Plateau and Western Mountain Provinces in western Colorado have also been the locus of elevated seismicity in historical times. To the north, the Plateau is bordered by the Wyoming Basin, a series of broad basins and uplifts that are structurally and tectonically similar to the Plateau. This transition zone is not marked by elevated seismicity.

The Mexican Hat site is within the stable interior portion of the Colorado Plateau, at distances of 100 to 200 km from the major seismic zones. The occurrences of Maximum Earthquakes (MEs) at the closest approaches to the site of the Intermountain Seismic Belt, Rio Grande Rift, and Basin and Range Provinces would not produce significant accelerations in the site area. The major features of interest to a seismic hazard study of the Mexican Hat site are: (1) possible floating earthquakes (not associated with known structures) in the immediate site area; (2) mapped faults, fault groups, and lineaments within a 65-km radius of the site; and (3) faults of the San Francisco volcanic field fault domain in north-central Arizona (Menges and Pearthree, 1983). These features all lie within the Colorado Plateau Interior, except for the San Francisco volcanic field fault domain, which overlaps the boundary of the Interior and the Arizona Border Zone.

0.3.7.2 Colorado Plateau seismotectonic province

The modern Colorado Plateau is composed of a stable interior portion bounded on the west, south, and east by more highly active border zones. For this study, the interior and border zones are defined as separate subprovinces, and the

boundary is drawn at the 40-km crustal thickness contour. The border zones lie within the physiographic boundary of the Colorado Plateau, but are characterized by elevated seismicity, thinner crust, higher heat flow, common normal faulting, and elevated levels of Tertiary and Quaternary volcanism relative to the interior. Nearly all of the larger historical earthquakes of the Plateau have occurred within the border zones.

Neogene faulting is generally rare within the interior portion of the Colorado Plateau, except for faulting associated with the Uncompahgre Uplift and the collapsed salt anticlines of the Paradox Valley. Earthquakes are rare. The historical seismicity of the interior portion has been classified by Wong et al. (1982) as very low level, having events of small to moderate magnitude with diffusely distributed epicenters. The largest instrumentally recorded earthquakes within the interior portion have fallen in the magnitude range 4.5 to 5.0.

The largest historical earthquakes recorded within the Colorado Plateau have occurred in the border zones. These include:

- o Events of estimated magnitudes 5.5 to 5.75 (M_L) at Lockett Tanks, Arizona, in 1912 and Fredonia, Arizona, in 1959 (DuBois et al., 1982). These events occurred within the Arizona Border Zone separating the Colorado Plateau from the Basin and Range Province to the south.
- o The Dulce, New Mexico, earthquake of January 23, 1956, of magnitude (M_b) 5.5 (NOAA Earthquake Data File). This event occurred in the zone of transition between the Colorado Plateau and Rio Grande Rift (Herrmann et al., 1980).
- o The earthquake of October 11, 1960, of magnitude 5.5, northeast of Ridgway, Colorado, which was strongly felt in the Ridgway-Montrose area. This event may be associated with the Ridgway fault which terminates the southeastern end of the Uncompahgre Uplift, marks the northwestern boundary of the San Juan volcanic field, and may be the boundary between the Colorado Plateau and Western Mountain provinces (Kirkham and Rogers, 1981; Sullivan et al., 1980).

Recurrence intervals have not been established for large earthquakes within the Colorado Plateau. They may be on the order of tens or hundreds of thousands of years.

0.3.7.3 Intermountain seismic belt

The following discussion is largely excerpted from Smith and Sbar (1974). The Intermountain Seismic Belt is a zone of

pronounced earthquake activity extending north from Arizona through Utah, eastern Idaho, and western Wyoming, and terminating in northwestern Montana (see Figure D.3.10). It is coincident with the boundary between the Basin and Range Province and the Colorado Plateau-Middle Rocky Mountains in central Utah, and lies roughly 200 km to the northwest of the Monument Valley site. The largest historical event of the Intermountain Region was the 1959 earthquake of magnitude 7.1 at Hebgen Lake, Montana. More than 15 events with magnitudes greater than 6.0 have been reported since the mid 1800s.

The Intermountain Seismic Belt includes the Wasatch Frontal Fault System and other major potentially active faults of northern and central Utah. This zone is highly seismic and capable of large earthquakes (up to $M=7.6$) at recurrence intervals as short as 250 years and averaging about 500 years (Swan, 1983).

D.3.7.4 Basin and Range Province in Arizona

To the south, the Colorado Plateau is bounded by the Basin and Range Province of Arizona and New Mexico. This province has moderate seismicity with no large historical events, although it is believed capable of large magnitude earthquakes (Menges and Pearthree, 1983; Pearthree et al., 1983). Recurrence intervals for this province have not been established, but are probably on the order of hundreds to thousands of years.

D.3.7.5 Rio Grande Rift

The Rio Grande Rift is a north-south-trending extensional graben feature of great length and tectonic significance. It extends from Chihuahua, Mexico, through west Texas, New Mexico, and most of central Colorado, almost to the Wyoming state line. The rift was initiated in Neogene time and has experienced continued activity through the Quaternary. It is characterized by fault scarps in young alluvium, abrupt mountain fronts that exhibit faceted spurs, deep, narrow linear valleys, Neogene basin-fill sedimentary rocks, and a bimodal suite of mafic and silicic igneous rocks.

A high percentage of all the potentially active faults in Colorado and New Mexico lie within this province. The rift has been subdivided into northern and southern subprovinces in Colorado by Kirkham and Rogers (1981) on the basis of young faulting. Well-defined evidence of repeated Late Quaternary movement is abundant on several faults in the southern subprovince, whereas such evidence is obscure in the northern subprovince.

D.3.8 GEOLOGIC HAZARDS ANALYSIS

D.3.8.1 Geomorphic hazards

The Mexican Hat site is in drainages tributary to Gypsum Creek, an ephemeral stream eroded into the Halgaito Tongue Member of the Cutler Group. The site lies on an easterly sloping erosional surface. Two drainages exit the tailings area, one from the northwest portion and one from the northeast portion of the tailings. These drainages head in the vicinity of the tailings area, and tailings material fills the upper reaches of both channels. Thin, sporadic surficial deposit cover in the site area minimizes the potential for drainage channel headward erosion and gullying within the indurated deposits.

Three major issues are related to erosion protection of the Mexican Hat site: (1) adequate surface-water diversion; (2) wind erosion protection; and (3) prevention of arroyo encroachment.

The topographic setting of the site provides some protection from the erosive force of the wind. Due to the fact that the prevailing winds come from the west, the higher topography west of the site provides this protection. Any reshaping of the site should consider this positive factor by contouring the area in such a way as to eliminate relief facing west. With the placement of an adequate erosion-resistant cover, no long-term dispersion should be experienced.

The heads of several arroyos approach the northern and eastern edges of the lower tailings pile. The arroyos are cut into moderately resistant sandstone bedrock and contain thin, discontinuous deposits of sandy alluvium and colluvium. Processes of rilling and gullying are producing headward erosion of the gullies at a slow rate. Gullies east of the lower tailings pile are actively eroding the embankment of the dirt perimeter road. Noticeable net erosion in narrow rills and gullies has occurred between site inspections in 1984 and 1986. Deeply incised gullies also approach the north and northeast sides of the lower tailings pile. Gully heads are very slowly cutting into resistant sandstone bedrock. The upper end of these washes terminates at the site perimeter road and is actively eroding the road embankment in narrow rills and gullies. Lateral and headward extension of gullies incised into sandstone bedrock appears to be proceeding at a slow rate, but may impact the tailings pile within the 1000-year design life. The site stabilization plan should protect from possible encroachment of these drainages to the edge of the tailings pile.

The tailings piles lie within the main drainage from the basin above the site to the San Juan River. Along the south side of the upper and lower tailings piles, a wide, deep water

diversion channel has been constructed adjacent to the site perimeter road. Surface runoff from the hills east of the site is currently channeled away from the tailings piles to the northeast. Sandstone blocks composing the channel berm appear to be rapidly weathering to loose, sandy material subject to wind erosion. Minor amounts of surface-water flow enter the upper tailings pile from deeply gullied areas between the diversion channel and the tailings. Diversion of surface-water runoff from the watershed above the site is a major concern. Without diversion, the site would probably receive an unacceptable amount of throughflow from the watershed.

No site hazard is present from mass movement processes of landslides, debris flows, soil creep, mudflows, or rock falls. No quantitative data exist on site-specific rates of slope erosion or scarp retreat. No hazardous soil conditions have been identified.

Subsidence caused by mine collapse or solutioning of underlying bedrock is not a hazard at the site. No evaporites are present in the local stratigraphy nor are there mines under or near the tailings. Traces of oil and gas have been found at depths of about 175 feet beneath the western edge of the site. Development of any large reservoirs of oil or gas could result in differential settling of ground surface near the site, although the possibility of subsidence is remote.

Gypsum Creek is in a relatively tectonically stable area. The effects of tectonic activity are discussed in the section on seismic hazards. The slow uplift of the Colorado Plateau region would not affect specific site design. Changes in the base level of streams or the local water level may produce changes in the process of degradation and alluviation in the site area. No quantitative data are available on the possible effects of base level or water table level changes in the Mexican Hat area.

Climatic changes would probably be of a lesser magnitude than those which occurred at the end of the Pleistocene, about 10,000 years ago. Predicted changes in the amount of precipitation, seasonality of precipitation, or temperature are not expected to produce site changes that would affect the stability of the final tailings pile design within the next 1000 years.

No hazards from volcanic activity are present in the tailings site area. No volcanic flows, cinders, or ash deposits have been recognized in the immediate vicinity of the Mexican Hat site. Small outcrops of Tertiary age explosive breccias occur about eight km (five mi) east of the site. No evidence occurs of volcanic intrusive activity younger than Pliocene in age.

D.3.8.2 Impact of natural resource development

In the site area, only three economic natural resources may occur: petroleum and natural gas, uranium, and sand and gravel.

The economic uranium occurrences in the Mexican Hat region occur in the Recapture Member of the Morrison Formation. The nearest known economic deposit is about 37 km (24 mi) northeast of the site. No known uranium deposits occur close enough to the site area to affect future mining activity and site stability.

Although traces of oil and gas may have been found in the Paleozoic strata underlying the site, there appears to be little hazard to site stability, except for drilling directly on the stabilized tailings. A remote possibility of subsidence from fluid withdrawal exists if petroleum resources were developed.

Sand and gravel exploitation possibilities are low. The quality and quantity of sand and gravel deposits and eolian sand deposits are very poor.

No hazards exist from coal mining. All known coal resources in the region occur in strata younger than that at the site.

D.3.8.3 Seismic hazards

Technical approach

The objectives of the seismic hazard analysis performed for this study are as follows:

- o Selection of the design earthquake and estimation of the on-site peak horizontal acceleration for use in subsequent engineering analyses.
- o Recognition of any potential for on-site fault rupture.
- o Recognition of any potential for earthquake-induced landsliding or subsidence due to tectonic causes.

The technical analysis performed for this study involves a critical review of all the information developed during the investigation and a step-by-step approach to estimating seismic risk.

The first step is the determination of the largest possible magnitude of floating earthquakes (FEs) in the seismotectonic province within which the site is located. The maximum FE is then assumed to occur at a radial distance of

15 km from the site, and the resulting on-site acceleration is calculated using the constrained acceleration/attenuation relationship of Campbell (1981). In this and subsequent steps of the analysis, the mean-plus-one standard deviation (84th percentile) value will be adopted. The design value is considered a nonamplified, free-field, peak horizontal acceleration.

Following this, the maximum possible on-site acceleration resulting from earthquakes occurring in each of the remote seismotectonic provinces within the region of interest is determined. A detailed analysis of individual faults within remote provinces is not performed unless they lie within a radius of 65 km of the site. A conservative approach is taken wherein the closest distance of the remote province from the site is first measured. The measurement is made using published maps and literature to delineate province boundaries. The Maximum Earthquake (ME) values for the remote provinces are estimated based on published studies and personal communications from researchers active in the areas. The ME is then assumed to occur at the closest distance of each remote province from the site, and the resulting on-site acceleration is calculated.

After completion of the first two steps in the analysis, the on-site accelerations resulting from the FE within the province containing the site and from the MEs at the closest approach of each of the remote provinces are compared. The largest value is taken as the critical acceleration during the subsequent capable fault analysis.

Based on the review of published and unpublished geologic data and the air-photo analysis, a compilation of all mapped faults and air-photo lineaments within a radial distance of 65 km of the site is prepared. The fault length/magnitude relationships of Bonilla et al. (1984) are used to determine the maximum magnitude earthquake that each structure could be capable of producing if it were determined to be a capable fault. An on-site acceleration resulting from each fault is then calculated using the acceleration/attenuation relationship of Campbell (1981). These values are then compared to the critical acceleration determined during the previous analysis. Any features potentially capable of producing a larger on-site acceleration than the critical value are subjected to a detailed field investigation to determine if they are capable faults.

The field investigation consists of analysis of the seismic record for evidence of micro- or macroseismicity associated with the fault, close inspection of the mapped fault trace on aerial photography, detailed ground reconnaissance for evidence of Late Quaternary or Holocene movements, and careful investigation of the indicated fault during the LSA aerial reconnaissance.

If any evidence is found to indicate the fault (or faults) to be capable, the calculated on-site acceleration is then recommended as the design acceleration value. The fault is designated as the controlling fault, and the ME on that fault is specified as the design earthquake.

Previous seismic zoning studies

Several probabilistic earthquake maps, which plot contours of maximum horizontal accelerations, velocities, and intensities for various return periods, have been prepared for the contiguous United States. Examples of such studies are those by Liu and DeCapua (1975), Algermissen and Perkins (1976), the Applied Technology Council (1978), and Algermissen et al. (1982). These studies were utilized to estimate the maximum value of each parameter for the site area. The resulting values are listed in Table D.3.2.

Liu and DeCapua (1975) developed 100-year contour maps of intensity and acceleration for the Rocky Mountain states. On their 100-year contour map of maximum predicted intensity, the site area lies in a north-south-trending region characterized by Modified Mercalli Intensity IV to V, which lies between the Intermountain Seismic Belt and the Rio Grande Rift. The Colorado Plateau is not clearly distinguished as a seismotectonic province. This indicates that a Modified Mercalli Intensity IV-V event will be experienced in the site area once every 100 years. Based on their 100-year contour map of peak accelerations for the Rocky Mountain states, an acceleration of 0.02 g to 0.03 g is predicted once every 100 years.

Contours of horizontal acceleration in rock having a 90 percent probability of not being exceeded in 50 years were presented for the contiguous United States by Algermissen and Perkins (1976). On their map, the stable interior portion of the Colorado Plateau is defined as a zone of maximum predicted horizontal acceleration in rock of less than 0.04 g.

The preliminary study of Algermissen and Perkins (1976) was updated by Algermissen et al. (1982), who presented probabilistic estimates of maximum acceleration and velocity in rock for periods of 10, 50, and 250 years. In comparison with the 1976 study, the 1982 study resulted in only minor modification of their estimates of peak accelerations for the Mexican Hat site region. They estimate that the maximum acceleration that will be experienced in the site region during a period of 250 years falls in the range 0.07 g to 0.10 g.

A study performed by the Applied Technology Council (1978) presented a map showing effective peak accelerations for the contiguous United States in which the geographic contours of seismic source zones and horizontal accelerations in the site area are the same as those presented by Algermissen and Perkins (1976) and Algermissen et al. (1982).

The accelerations calculated in these previous studies are uniformly lower than the maximum values calculated in this study. The discrepancy in the values determined results from the difference in approach taken for the studies. The previous studies are probabilistic analyses based on the rather brief historical record, which predict parameters for limited time periods. The approach taken in the present study is a very conservative analysis, which assumes the potential occurrence of the most extreme possible events. A great deal of attention is paid to the details of the regional and local structure in this study. The maximum capabilities of all faults in the study area are taken into account. In addition, the design life of 1000 years is the basis of the present analysis, as compared to the time periods of 10, 50, 100, and 250 years assumed in the previous studies. Extrapolation of the results of the previous studies to the 1000-year design life would result in values more consistent with the conclusions of this study.

Review of epicentral data for the Colorado Plateau

An epicentral compilation for use in derivation of seismic parameters for the Colorado Plateau was obtained for this study from the NOAA/NGDC earthquake data file. The complete listing is available on request from the UNTRA Project Office, Albuquerque, New Mexico.

In order to facilitate the computer search, and to restrict the search area as closely as possible to the actual limits of the Colorado Plateau, two search areas were specified: a circle of radius 320 km and center at 36.0°N, 110.0°W, and a rectangular block covering the area 38.2°N-40.3°N, 107.3°-112.0°W. The choice of search areas resulted in some overlap beyond the borders of the Colorado Plateau, especially in central Arizona. These data were considered to be representative of seismicity of the border zones, and were included in the analysis.

Table D.3.3 was derived from this list. It represents all instrumentally located earthquakes within the Colorado Plateau (interior and border zones) of magnitude 4.0 since January 1, 1960. The list contains 70 events. Of these, a total of 15 occurred either in the eastern Utah coal mining belt or in the oil and gas fields near Rangely, Colorado. These are considered to be artificially induced events caused by mining or oil and gas withdrawal (Smith and Sbar, 1974). They were not included in the subsequent analysis. Of the remaining 55 events, only two occurred within the stable interior portion of the Plateau as defined for this study. These include magnitude (mb) events of 4.0 in the Paradox Basin on February 3, 1970, and 4.4 near Grand Junction, Colorado, on January 30, 1975. The data indicate that only four percent of the seismicity of the Colorado Plateau occurs

in the stable interior. Those events that do occur are associated with tectonically unique structures such as the Uncompahgre Plateau and Paradox Basin.

The remaining 53 events, representing 96 percent of the data, occurred in the border zones. Twenty-three events occurred in the Rio Grande Rift border zone, and of these, 18 were associated with the swarm of events near Dulce, New Mexico, from January, 1966, to January, 1967.

Eighteen of the events are associated with the border zone of the Colorado Plateau and the Intermountain Seismic Belt in Utah and northern Arizona. Most of these events occurred along the Wasatch Frontal fault system. Seven events occurred in the border zone between the Colorado Plateau and the Western Mountain Province. These include the event of October 11, 1960, of magnitude 5.5 near Montrose, Colorado.

The remaining five events occurred in the Arizona Border Zone, and include several events of magnitude (m_b) 5.5 to 5.75 in the Flagstaff area.

Graphical determination of ME

The data were plotted on Figure D.3.12 to determine the ME value for the Colorado Plateau. Due to the scarcity of data for the Colorado Plateau interior, the data are representative of the border zones. The data show that there is no basis for any determination of the ME value for the interior from the instrumental seismic record. The historical record is also extremely limited and is probably even less reliable. The scarcity of recorded earthquakes of magnitude 5.0 and greater also limits the reliability of the ME determination for the border zones. The true ME value may lay anywhere within the range from 6.2 to 6.8. The average value of this range, magnitude 6.5, is considered to be a reasonably conservative value. This value is recommended as the ME value for the Colorado Plateau interior and border zones together. This value is also the value adopted by Kirkham and Rogers (1981) as the ME for the Colorado Plateau. The data do not permit any estimate of the recurrence interval for the ME event within the interior province. It may be on the order of tens to hundreds of thousands of years. For the interior and border zones combined, a reasonable estimate can be made on the basis of the historical record. Assuming the record for moderate to large earthquakes to be complete since about 1900, the data base covers a period of 85 years. This value represents an absolute minimum recurrence interval for the border zones. If it is assumed, conservatively, that one magnitude 6.5 earthquake occurs every 85 years within the approximately 425,000-km² area of the Colorado Plateau (interior and border zones), the probability of occurrence of a magnitude 6.5 event within any 15-km radius within the

region is 0.06×10^{-4} . The recurrence interval of an ME earthquake within any 15-km-radius area is thus 166,666 years.

A graphical determination of the recurrence interval of the ME for the entire Colorado Plateau (interior and border zones) was also done, using the data shown in Table D.3.4, and is represented in Figure D.3.13. The results indicate a recurrence probability of 0.0019 ME event per year, or a recurrence interval of 526 years.

Determination of FE magnitude

The definition of "floating earthquake" adopted for use in UMTRA Project seismic hazard evaluations is "an earthquake within a specific seismotectonic province which is not associated with a known tectonic structure." It is important to distinguish between the terms Maximum Earthquake (ME) and Floating Earthquake (FE). The ME magnitude should generally be larger than the FE magnitude, because large earthquakes are generally associated with ground breakage on known tectonic structures. The FE magnitude should never be greater than the ME.

It is generally accepted that floating earthquakes are of event slow to moderate magnitude. For example, Slemmons et al. (1982) state "The maximum magnitude for this type of earthquake in the eastern and central United States is about $M_s = 5.75$ to 6."

Krinitzsky and Chang (1975) state that "Uncertainties in the association of earthquakes with faults affect only small, magnitude 4.0 or 5.0 events. Long faults, which are necessary for large earthquakes, would not remain undetected if careful geologic investigations were made." They further state that the formation of new faults capable of causing destructive earthquakes is not a possibility that should be considered in design.

The maximum magnitude of the floating earthquake should therefore be equal to the threshold magnitude at which ground breakage will occur. It is generally assumed that all earthquakes of magnitude greater than about 6.0 to 6.2 do produce fault scarps at the ground surface in the western United States. Wallace (1978) indicates that earthquakes of magnitudes greater than about 6.0 are generally associated with ground breakage in the Basin and Range Province. The threshold magnitude is not precisely known for the Colorado Plateau, because there are no recorded seismic events associated with ground breakage. The largest recorded earthquakes in the Colorado Plateau have all fallen in the approximate magnitude range of 5.5 to 5.75. It is conceivable that this range may represent the ME value for the prevailing stress field. Thompson and Zoback (1979) state that the lack of major

faulting and/or seismicity within the Plateau interior indicates low differential (shear) stresses. This range is considerably lower than the ME value of 6.5 used for UMTRA Project studies in the Colorado Plateau. The larger value is adopted in part because of the limitations of the historical data base. The magnitude range 5.5 to 5.8 may be a reasonable value for the FE magnitude, but a more conservative value is advisable considering the lack of a long-term data base. Since all earthquakes of magnitude 6.5 should be associated with ground breakage (i.e., be associated with a known tectonic structure) and the largest historical events fall in the range 5.5 to 5.75, the FE magnitude must fall somewhere within the range 5.75 to 6.5.

A more precise estimate can be made by evaluating the threshold magnitude at which ground breakage occurs from the appropriate fault length/magnitude relationship. The relationship of Bonilla et al. (1982) for plate interiors is:

$$M_s = 6.2 + 0.729 \log L$$

where

L = mapped fault rupture length.

The threshold magnitude can be estimated by setting L equal to zero. The resulting magnitude (M_s) is equal to about 6.0. This is in general agreement with the arguments of Krinitzsky and Chang (1975), Slemmons et al. (1982), and Wallace (1978), cited above. Allowing for possible errors and variability in the determinations discussed above, the more conservative value of $M_s=6.2$ is recommended for the maximum magnitude of the FE in the Colorado Plateau.

In accordance with TAD guidelines, this event is assumed to occur at a radial distance of 15 km for the site. Using the constrained acceleration/attenuation relationship of Campbell (1981), this results in an on-site peak horizontal acceleration (mean plus one standard deviation value, or 84th percentile) of 0.21g. In the absence of known capable faults capable of producing higher accelerations at the site area, this event will be designated as the design earthquake, with a design acceleration of 0.21g.

The recurrence interval for this event cannot be reliably estimated, due to the lack of data, especially for the Colorado Plateau interior. It may be on the order of tens or hundreds of thousands of years. A graphical estimate of recurrence values (Figure D.3.13) shows an occurrence rate of 0.004 events of this magnitude per year, or one event of magnitude 6.2 every 250 years within the Colorado Plateau. This may be an extremely conservative estimate, considering the largest actual historical events and the limited data base. If it is assumed that one earthquake of magnitude 6.2 occurs randomly

every 250 years within the entire plateau (an area of approximately 425,000 km²), then the apparent probability of occurrence is approximately 1×10^{-8} events per square km per year. Using this base value, the probability of an event occurring within a specified time period and within any specified radial distance of the site can be simply derived. The probability of occurrence of the FE within the 15-km radius of any site during the 1000-year design life is 2.25×10^{-3} , or less than one percent.

Effects of MEs occurring within remote seismotectonic provinces

Remote seismotectonic provinces, subprovinces, or domains considered for this study include the following (see Figure D.3.10):

- o Arizona Border Zone of Colorado Plateau (including San Francisco volcanic field fault domain). The state of Arizona was divided into separate domains of Late Quaternary faulting, based on regional distribution and spacing of faults and estimates of recurrence intervals, by Menges and Pearthree (1983). No Quaternary faults or potentially active features were recognized in northeastern Arizona. Faults associated with the San Francisco volcanic field near Flagstaff and extending north to the Utah border are the closest identified potentially active features to the site (see Figure D.3.7). This domain appears to overlap the Interior/Border Zone transition. Rough ME estimates based on the approximate mapped lengths of faults in this domain indicate that magnitudes on the order of 6.5 may be possible. Faults in this domain are apparently the major potential sources of earthquakes which could affect the Mexican Hat site. Closest approaches of these features to the site area range from about 50 to 55 km. The occurrence of a magnitude (M_b) earthquake of 6.5 at a distance of 50 km would produce an on-site peak horizontal acceleration of less than 0.1g in the site area (Table D.3.5).
- o Intermountain Seismic Belt. The following discussion is largely excerpted from Smith and Sbar (1974). The Intermountain Seismic Belt is a zone of pronounced earthquake activity extending north from Arizona through Utah, eastern Idaho, western Wyoming, and terminating in northwestern Montana. It is coincident with the boundary between the Basin and Range Province and the Colorado Plateau-Middle Rocky Mountains in central Utah. The largest historical event of the Intermountain Region was the 1959 earthquake of magnitude 7.1 at Hebgen Lake, Montana. More than 15 events with magnitudes greater than 6.0 have been reported since the mid 1800s.

The Intermountain Seismic Belt includes the Wasatch Frontal fault system, and other major potentially active faults of northern and central Utah.

The ME value for the Intermountain Seismic Belt may be as high as 7.5 (Smith and Sbar, 1974). Though the seismic activity associated with the Intermountain Seismic Belt is irregularly distributed over a broad area, there is no evidence that seismicity associated with it has occurred within the interior of the Colorado Plateau. It therefore appears that the closest approach of potential earthquakes associated with the Intermountain Seismic Belt to the Mexican Hat site is approximately 200 km. A magnitude 7.5 earthquake occurring at the closest approach of the Intermountain Seismic Belt would produce a peak horizontal acceleration of less than 0.05g at the site (Table D.3.5).

- o Rio Grande Rift. The Rio Grande Rift is a north-south-trending extensional graben feature of great length and tectonic significance. It extends from Chihuahua, Mexico, through west Texas, New Mexico, and most of central Colorado, almost to the Wyoming state line. The rift was initiated in Neogene time and has experienced continued activity through the Quaternary. It is characterized by fault scarps in young alluvium, abrupt mountain fronts that exhibit faceted spurs, deep, narrow linear valleys, Neogene basin-fill sedimentary rocks, and a bimodal suite of mafic and silicic igneous rocks.

A high percentage of all the potentially active faults in Colorado and New Mexico lie within this province. The rift has been subdivided into northern and southern subprovinces in Colorado by Kirkham and Rogers (1981) on the basis of young faulting. Well-defined evidence of repeated Late Quaternary movement is abundant on several faults in the southern sub-province, whereas such evidence is obscure in the northern subprovince.

The closest approach of active or potentially active faults associated with the Rio Grande Rift to the Mexican Hat site area is about 275 km.

The estimated magnitude of an ME associated with the Rio Grande Rift (Kirkham and Rogers, 1981) is 6.5 to 7.5. An event of magnitude 7.5 occurring at a distance of 275 km from the site area would result in a peak horizontal acceleration of less than 0.01g, as detailed in Table D.3.5.

The Rio Grande Rift Border Zone is anomalous in that it apparently is not associated with pronounced crustal thinning

(see Figure D.3.7). However, the 1966 Dulce earthquake swarm occurred near the approximate transition. For this study, the border zone is defined arbitrarily as a 100-km-wide belt extending into the eastern margin of the Plateau from the mapped boundary of the Rio Grande Rift. The closest approach to the site is thus about 175 km. The occurrence of a magnitude (m_b) event of 6.5 at a distance of 175 km would produce an on-site peak horizontal acceleration of less than 0.01g.

Fault and epicentral compilation: capable fault analysis

A compilation of all mapped faults and earthquake epicenters within 65 km of the site is shown on Figure D.3.7 and listed in Table D.3.6. In addition to previously mapped faults, all lineaments derived from aerial photographic interpretation were compiled. All faults were observed at least once during the low-sun-angle aerial reconnaissance. Most features within 35 km of the site were field-checked during the ground reconnaissance phase of the study. Other features of regional significance outside the 35-km radius were also field-checked.

It was determined on the basis of the investigations performed that no faults lying within the 65-km radius show any evidence of Quaternary movements. In addition, no historical or instrumentally located earthquake epicenters lie within this radius. Regional features which are potentially capable of producing large earthquakes are too distant to affect design parameters at the site.

Since there are no capable faults within the 65-km radius from the site, design parameters are not affected by the capable fault analysis.

Recommended seismic design parameters

This study recommends that the floating earthquake be adopted as the design earthquake for the Mexican Hat site. The floating earthquake has a magnitude of $m_b=6.2$. At 15 km (9.3 mi) from the site, this event would result in 0.21g of peak horizontal acceleration at the site. As this event is not assigned to a specific source or source area, recurrence, focal depth, and fault mechanism cannot be determined.

The acceleration derived in this report for the floating earthquake is a free-field acceleration, and site conditions (i.e., amount of alluvial cover, possible focussing, and the like) need to be considered during subsequent engineering efforts.

Potential for on-site fault rupture

As discussed above and illustrated in Figure D.3.7, there are no indications of any capable faults in the immediate site area. All mapped faults within the 65-km radius from the tailings piles were carefully examined during the investigation and showed no indications of Quaternary movements. Bedrock is fairly well exposed over much of the site area, except in the deeper washes and areas of large dunes. The possibility that large faults have gone undetected is remote.

There is therefore no evidence of any potential for on-site fault movements during the design life of the proposed facility.

Potential liquefaction hazard

A review of published earthquake reports by Youd and Hoose (1977) indicates that shallow, saturated, Holocene fluvial, deltaic, and eolian deposits, and poorly compacted artificial sand fills have the highest susceptibility to liquefaction and subsequent ground failure. Holocene alluvial fan, alluvial plain, beach, terrace, and playa deposits were found to be less susceptible. Pleistocene sand deposits are generally even less susceptible, and glacial till, clay-rich, and pre-Pleistocene deposits are usually immune to liquefaction. The degree of sorting, the degree of compaction during sedimentation or construction, and the grain-size distribution are major factors controlling liquefaction potential. The greater the sorting and the looser the packing, the greater the liquefaction potential. Most episodes of liquefaction have developed at relatively shallow depths (probably less than 30 feet) and in areas where the water table (free or perched) was within a few meters of the ground surface.

The potential for liquefaction also depends on the degree of seismic shaking. The opportunity for ground failure in a given area is a function of the seismicity of the area and the rate of occurrence of earthquake ground motions of sufficient materials. The maximum distance from a seismic source to potentially damaging ground failures as a function of earthquake magnitude was determined by Youd and Perkins (1978). Liquefaction is not likely to be produced by earthquakes of magnitudes less than about 5.0 or at a distance greater than 75 to 150 km (47 to 93 mi) from the hypocenter.

The mapping and exploratory drilling performed for this study do not indicate the presence of natural materials susceptible to liquefaction beneath the tailings piles. The piles are located primarily on bedrock of the Halgaito Tongue of the Cutler Group. Minor amounts of unconsolidated eolian sands may underlie the pile in places, but are not saturated.

The recommended design earthquake is a magnitude 6.2 event occurring at a radial distance of 15 km from the site. Following the determination by Youd and Perkins (1978) cited above, this event should be considered capable of producing liquefaction at the site in susceptible materials.

Reservoir-induced seismicity

Published studies of reservoir-induced seismicity include reports by Carder (1945, 1970), the National Academy of Sciences (1972), Judd (1974), Milne (1976), Gupta and Rastogi (1976), Stuart-Alexander and Mark (1976), Packer et al. (1979), and Meade (1982).

A review of the previous literature by Meade (1982) indicates that the phenomenon occurs only in association with very deep reservoirs or reservoirs with very large storage capacity. Less than one percent of the world's reservoirs have been associated with macroearthquakes ($M_L \geq 3.0$) (Stuart-Alexander and Mark, 1976). Table D.3.7 lists representative cases of reservoir induced seismicity.

There are no large reservoirs in the vicinity of the Mexican Hat site at the present time. Due to the lack of abundant surface water in the site region, there is no potential for development of future large reservoirs. Reservoir induced seismicity therefore does not present a potential hazard to site stability.

The nearest large impounded body of water to the Mexican Hat site is Lake Powell, whose upper reaches are about 70 km to the west. At this distance, even a very large induced earthquake would not produce an acceleration at the site greater than the design acceleration. At 50 km, induced seismicity would have to exceed magnitude 7.8 to produce a greater acceleration than the recommended 0.21g. Furthermore, there are no known structures in the Lake Powell area which could be activated by crustal loading capable of producing an earthquake of this size.

FIGURE D.3.1
EPICENTRAL AND FAULT COMPILATION

FIGURE D.3.2
STRUCTURAL CONTOUR MAP, KAIBAB LIMESTONE, MEXICAN HAT REGION

National Parks and Monuments

1. Dinosaur Nat. Mon.	13. Rainbow Bridge Nat. Mon.
2. Black Canyon of the Gunnison Nat. Mon.	14. Capitol Reef Nat. Mon.
3. Colorado Nat. Mon.	15. Bryce Canyon Nat. Park
4. Arches Nat. Mon.	16. Cedar Breaks Nat. Mon.
5. Canyonlands Nat. Park	17. Zion Nat. Park
6. Natural Bridges Nat. Mon.	18. Grand Canyon Nat. Park
7. Hovenweep Nat. Mon.	19. Wupatki and Sunset Crater Nat. Mon.
8. Mesa Verde Nat. Park	20. Walnut Canyon Nat. Mon.
9. Aztec Ruins Nat. Mon.	21. Petrified Forest and Painted Desert Nat. Mon.
10. Chaco Canyon Nat. Mon.	22. Montezuma Castle Nat. Mon.
11. Canyon de Chelly Nat. Mon.	23. Tusigoot Nat. Mon.
12. Navajo Nat. Mon.
(Betsakini and Kiet Seel)	

Escarpments at South End of High Plateaus

- pc Pink Cliffs
wc White Cliffs
vc Vermilion Cliffs

Other Prominent Features

- wf Waterpocket Fold
er Elk Ridge
cr Comb Ridge
mv Monument Valley
ag Agathia Peak
sr Shiprock
cb Caberton Peak

FIGURE D.3.4
PHYSIOGRAPHIC MAP OF THE COLORADO PLATEAU (AFTER HUNT, 1967)
SHOWING LOCATION OF MEXICAN HAT SITE

FIGURE D.3.5
SKETCH MAP OF SITE REGION, SHOWING PAST AND PRESENT PREVAILING WIND DIRECTIONS

LEGEND

- TI TERTIARY IGNEOUS ROCKS
- Pcd DECHELLY SANDSTONE MEMBER OF CUTLER GROUP
- Pco ORGAN ROCK TONGUE OF CUTLER GROUP
- Pcc CEDAR MESA SANDSTONE MEMBER OF CUTLER GROUP
- Pch HALGAITO TONGUE OF CUTLER GROUP
- IPh HERMOSA GROUP

FIGURE D.3.6
GENERALIZED GEOLOGIC MAP OF
THE MEXICAN HAT REGION

FIGURE D.3.7
GEOLOGIC CROSS-SECTION AA' AT MEXICAN HAT

FIGURE D.3.8
GENERALIZED STRATIGRAPHIC COLUMN
FOR MEXICAN HAT REGION

FIGURE D.3.9
DRAINAGES AT THE MEXICAN HAT, UTAH, UMTRA PROJECT SITE

REF: WONG, ET AL., 1982

FIGURE D.3.10
MAJOR SEISMIC ZONES OF THE BASIN AND
RANGE - COLORADO PLATEAU PROVINCES

FIGURE D.3.11
MAP OF HISTORICAL AND INSTRUMENTALLY-LOCATED EARTHQUAKE
EPICENTERS OF THE SOUTHWESTERN UNITED STATES (AFTER NOAA/NGDC).
EPICENTRAL COMPILATION LIMITED TO EVENTS OF MAGNITUDE > 4
AND/OR INTENSITY (IMM) > V.

FIGURE D.3.12

GRAPHICAL DETERMINATION OF ME MAGNITUDE, COLORADO PLATEAU INTERIOR AND BORDER ZONES

FIGURE D.3.13
 GRAPHICAL DETERMINATION OF RECURRENCE INTERVALS FOR FLOATING EARTHQUAKE
 AND ME, COLORADO PLATEAU INTERIOR AND BORDER ZONES

Table D.3.1 Existing aerial photography of the site area

Location	Type of photography	Approx. scale
BLM - Monticello, UT	conventional black-and-white	1:24,000
SCS - Kayenta, AZ	conventional black-and-white	1:24,000
SCS - Shiprock, AZ	conventional black-and-white	1:24,000
SCS - Monticello, UT	conventional black-and-white	1:24,000
USFS - Monticello, UT	conventional color	1:16,000

Notes: BLM - Bureau of Land Management.
 SCS - Soil Conservation Service.
 USFS - U.S. Forest Service.

Table D.3.2 Probabilistic estimates of maximum acceleration, velocity, and intensity at the Mexican Hat site area from various published sources

Source	Return period or probability	Maximum acceleration G-units	Maximum velocity cm/s	Maximum Modified Mercalli Intensity
Liu and DeCapua (1975)	100 years	0.02-0.03 ^a	--	IV-V
Algermissen and Perkins (1976)	90% probability of not being exceeded in 50 years	<0.04 ^b	--	--
Applied Technology Council (1978)	--	0.05 ^c	--	--
Algermissen et al. (1982)	90% probability of not being exceeded in 10 years	<0.04 ^b	<2	--
Algermissen et al. (1982)	90% probability of not being exceeded in 50 years	<0.04 ^b	2-3	--
Algermissen et al. (1982)	90% probability of not being exceeded in 250 years	0.07-0.10 ^b	3-4	--

^aPeak horizontal ground acceleration.

^bMaximum acceleration in rock.

^cEffective peak acceleration.

Table D.3.3 Earthquakes of M₂4.0 since 1960 in the Colorado Plateau

Date			Time			Location		Depth km	Magnitude				Intensity
YR	MO	DA	HR	MIN	SEC	LAT(N)	LONG(W)		m _b	m _s	other	local	
1960	10	11	08	05	30.5	38.3	107.6			5.5			VI
1963	06	19	08	38	41.5	37.9	112.6	33	4.2				
1963	07	07	19	20	42.4	39.5	111.9		4.9				VI
1963	07	09	20	25	21.5	40.0	111.2	33	4.1				
1963	07	10	18	32	50.6	39.9	111.4	33	4.2				V
1963	09	02	17	40	15.4	39.6	110.1	33	4.1				
1963	09	11	11	59	41.0	33.2	110.7	33	4.2				
1964	06	06	12	46	59.9	39.4	110.2	30	4.2				
1965	01	18	20	08	14.4	37.4	113.7	33	4.0				IV
1965	03	21	22	56	39.7	39.5	110.3		4.0				IV
1965	03	26	00	51	24.5	39.5	110.3		4.3				V
1966	01	23	01	56	38.0	37.0	107.0	14	5.5				VII
1966	01	23	06	14	15.6	36.9	107.2	5	4.2				V
1966	01	23	07	44	35.7	36.9	107.3	5	4.6				
1966	01	25	10	38	05.0	36.8	107.7	5	4.0				

1966-01-23 01:56:38
1966-01-23 06:14:16
1966-01-23 07:44:36
1966-01-25 10:38:05

Table D.3.3 Earthquakes of $M \geq 4.0$ since 1960 in the Colorado Plateau
(Continued)

YR	Date			Time			Location		Depth km	Magnitude				Intensity
	MO	DA	HR	MIN	SEC	EAT(N)	LONG(W)			m_b	m_s	other	local	
1966	01	23	11	01	07.1	36.9	107.2	Dulce, NM (RGR Border Zone)	5	4.3				V
1966	01	23	23	48	09.1	36.9	107.0	Dulce, NM (RGR Border Zone)	5	4.6				V
1966	01	23	19	43	19.7	36.9	107.1	Dulce, NM (RGR Border Zone)	5	4.5				V
1966	04	23	20	20	54.5	39.2	111.4	Nasatch Zone, Utah (IMSB Border Zone)	33	4.4				
1966	04	30	18	29	13.8	39.6	110.4	CO Coal Mining Belt (Mining-Induced ?)	3	4.0				V
1966	05	04	05	40	37.5	36.8	107.1	Dulce, NM (RGR Border Zone)	5	4.1				
1966	05	08	17	23	37.8	36.9	106.9	Dulce, NM (RGR Border Zone)	5	4.5				V
1966	05	08	17	50	35.6	37.0	106.8	Dulce, NM (RGR Border Zone)	5	4.2				V
1966	05	09	02	08	53.5	36.9	106.9	Dulce, NM (RGR Border Zone)	5	4.2				V
1966	05	09	02	57	23.6	37.0	106.9	Dulce, NM (RGR Border Zone)	5	4.4				V
1966	05	19	00	26	42.2	36.9	107.0	Dulce, NM (RGR Border Zone)	5	4.6				
1966	05	20	13	40	48.8	37.9	112.1	SW Utah (IMSB Border Zone)	18	4.3				
1966	06	02	21	59	11.5	36.9	107.0	Dulce, NM (RGR Border Zone)	5	5.0				VI
1966	06	21	05	24	38.2	36.9	107.1	Dulce, NM (RGR Border Zone)	5	4.2				
1966	06	04	10	29	39.3	37.0	107.0	Dulce, NM (RGR Border Zone)	5	4.1				V
1966	07	06	05	47	08.3	40.2	108.9	Wangely, CO (oil & gas withdrawal ?)	5	4.5				V
1966	09	04	09	52	34.5	38.3	107.6	SW CO (Colorado Plateau/ MMP Border Zone)	33	4.2				

Table D.3.3 Earthquakes of $M \geq 4.0$ since 1960 in the Colorado Plateau (Continued)

Date			Time			Location		Depth km	Magnitude				Intensity
YR	MO	DA	HR	MIN	SEC	LAT(N)	LONG(W)		M_b	M_s	other	local	
1966	10	03	16	03	50.8	35.8	111.6	Cocconino Co., AZ (AZ Border Zone)	33	4.4			V
1966	12	16	02	00	44.0	37.0	107.0	Dulce, NM (RGR Border Zone)	33	4.2			
1967	01	06	15	41	15.5	36.9	107.0	Dulce, NM (RGR Border Zone)	33	4.3			V
1967	01	12	03	52	06.2	39.0	107.5	SW CO (Colorado Plateau/ WMP Border Zone)	33	4.4			V
1967	01	16	09	22	45.9	37.7	107.9	Silverton CO (WMP Border Zone)	33	4.1			V
1967	02	15	03	28	03.6	40.1	109.1	Rangely, CO (Oil & gas withdrawal ?)	5	4.5			V
1967	04	04	22	53	39.5	38.3	107.7	SW CO (Colorado Plateau/ WMP Border Zone)	33	4.5			V
1967	10	25	02	41	34.4	39.4	110.3	EUI Coal Mining Belt (Mining-Induced ?)		4.0			V
1967	10	25	05	53	08.4	39.4	110.3	EUI Coal Mining Belt (Mining-Induced ?)		4.0			V
1968	01	16	09	42	54.2	39.2	112.0	Wasatch Zone, Utah (IMSB Border Zone)	33	4.0			V
1968	08	04	06	23	36.4	39.1	111.4	Wasatch Zone, Utah (IMSB Border Zone)	15	4.0			V
1968	08	29	09	31	48.1	39.5	110.2	EUI Coal Mining Belt (Mining-Induced ?)		4.2			V
1968	09	24	02	10	51.8	38.0	112.1	SW Utah (IMSB Border Zone)	33	4.0			V
1968	11	17	14	33	37.5	39.5	110.9	EUI Coal Mining Belt (Mining-Induced ?)	6	4.6			V

Table D.3.3 Earthquakes of M24.0 since 1960 in the Colorado Plateau (Continued)

Date			Time			Location		Depth km	Magnitude				Intensity
YR	MO	DA	HR	MIN	SEC	LAT(N)	LONG(W)		M _b	M _s	other	local	
1969	03	13	07	03	14.8	39.4	110.2	EUT Coal Mining Belt (Mining-Induced ?)	2	4.1			V
1969	05	23	05	24	53.6	39.0	111.0	Wasatch Zone, Utah (IMSB Border Zone)	31	4.0			V
1969	12	25	12	49	10.1	33.4	110.6	San Carlos, AZ (AZ Border Zone)	15	4.4		5.1	
1970	02	03 ^a	05	59	35.6	37.9	108.3	Paradox Basin, CO (Colorado Plateau Interior)	33	4.0			V
1970	04	14	10	40	54.2	39.7	110.0	EUT Coal Mining Belt (Mining-Induced ?)	13	4.2			
1970	04	18	10	42	11.9	37.8	111.6	SW Utah (IMSB Border Zone)	10	4.4			
1970	04	21	08	53	52.4	40.1	108.9	Rangeley, CO (Oil & gas withdrawal)	4	4.3		3.9	V
1970	04	21	15	05	47.5	40.1	108.9	Rangeley, CO (Oil & gas withdrawal)	4	4.6			V
1970	05	23	22	55	22.4	38.0	112.3	SW Utah (IMSB Border Zone)	3	4.6		4.9	
1970	10	25	07	46	42.1	39.2	111.4	Wasatch Zone, Utah (IMSB Border Zone)	5	4.3			V
1970	11	28	07	40	11.6	35.0	106.7	Albuquerque, NM (RGR Border Zone)	9	4.5			V
1971	01	04	07	39	06.7	35.0	106.7	Albuquerque, NM (RGR Border Zone)	9	4.7			
1971	01	07	20	39	52.1 ^f	39.4	107.3	Colorado Plateau (WMP Border Zone)	33	4.3		3.8	
1973	07	16	06	36	42.8	39.1	111.5	Wasatch Zone, Utah (IMSB Border Zone)	10	4.2			

Table D.3.3 Earthquakes of M₂4.0 since 1960 in the Colorado Plateau (Concluded)

Date			Time			Location		Depth km	Magnitude				Intensity
YR	MO	DA	HR	MIN	SEC	LAT(N)	LONG(W)		mb	ms	other	local	
1973	12	24	02	20	14.9	35.3	107.7	Mt. Taylor Region, NM (RGR Border Zone)	18	4.4			V
1975	01	30 ^a	14	48	40.3	39.2	108.6	Grand Junction, CO (Colorado Plateau Interior- Moncompagre Uplift)	5	4.4		3.7	
1975	10	06	15	50	46.9	39.0	111.4	Wasatch Zone, Utah (IMSB Border Zone)	005	4.2		3.2	II
1976	07	05	06	23	32.9	35.8	108.3	Crown Point, NM (RGR Border Zone)	25	5.0		4.6	VI
1976	02	04	00	04	58.1	34.6	112.5	Chino Valley, AZ (AZ Border Zone)	12	4.9	5.1	5.2	VI
1976	02	09	03	07	22.0	34.6	112.5	Chino Valley, AZ (AZ Border Zone)	10	4.6		3.3	
1977	03	05	03	00	54.7	35.9	108.2	Crown Point, NM (RGR Border Zone)	22	4.6		4.2	VI
1977	09	24	11	16	48.4	39.3	107.3	Colorado Plateau/ WMP Border Zone	005	4.0		3.0	
1980	05	24	10	03	36.3	39.9	111.9	Wasatch Zone, Utah (IMSB Border Zone)	005	5.0		4.2	V
1980	05	14	05	11	04.7	39.4	111.0	Wasatch Zone, Utah (IMSB Border Zone)	1	4.0	4.0	3.5	V

^aColorado Plateau Interior.

Ref. NOAA earthquake data file.

Table D.3.4 Accelerations calculated for the site

Source/source area	Maximum earthquake	Distance to site ^a	Expected maximum peak horizontal acceleration at the site ^b
Basin and Range (Intermountain Seismic Belt)	7.6 ^c	160 km (99 mi)	less than 0.04g
Basin and Range of New Mexico - Arizona	7.3 ^d	270 km (168 mi)	less than 0.02g
Rio Grande Rift	7.5 ^e	160 km (99 mi)	less than 0.04g
Floating Earthquake for the Colorado Plateau Interior	6.2 ^d	15 km (9.3 mi)	0.21g

^aFrom edge of stable interior portion of Colorado Plateau bordering seismotectonic province (i.e., source area).

^bCalculated from Campbell (1981), using formula for constrained ground acceleration, as a fraction of gravity (g).

^cFrom Bucknam et al. (1980).

^dThis study (Section D.3.8.3).

^eKirkham and Rogers (1981).

Table D.3.5 Estimated on-site accelerations resulting from MEs in remote seismotectonic provinces, subprovinces, or domains

Source area	ME ^a	Distance from site area	Peak horizontal acceleration expected at site (fraction of unit gravity) ^b
San Francisco volcanic field fault domain	6.5	50 km	0.06 - 0.07
Intermountain Seismic Belt	7.5	200 km	<0.018
Rio Grande Rift Border Zone	6.5	125 km	<0.01 g

^aSee test for derivation of ME values.

^bUsing acceleration/attenuation relationship of Campbell (1981).

Table D.3.6 Compilation of mapped faults within a 65-km radius of Mexican Hat

Fault/ lineament number	Source	Examined by: low-sun-angle			Comments
		Aerial recon.	Aerial recon.	Ground recon.	
1	Witkind et al., 1957	X	X		Shorter than mapped. No Late Quaternary offset.
2	This study	X	X		Not faulted - lineament from exposure by erosion of resistant stratigraphy.
3	Hackman and Wyant, 1973	X	X		No Late Quaternary offset.
4	Cooley et al., 1969	X			No Late Quaternary offset.
5	Haynes and Hackman, 1978	X			No Late Quaternary offset.
6	Haynes and Hackman, 1978	X			No Late Quaternary offset.
7	Hackman and Wyant, 1973	X	X		No Late Quaternary offset.
8	Hackman and Wyant, 1973 and this study	X	X		No Late Quaternary offset.
9	Cooley et al., 1969	X	X	X	No Late Quaternary offset.
10	Hackman and Wyant, 1973	X	X	X	No Late Quaternary offset.
11	Cooley et al., 1969 Hackman and Wyant, 1973	X	X		No Late Quaternary offset.
12	Hackman and Wyant, 1973	X	X		No Late Quaternary offset.
13	Hackman and Wyant, 1973 Haynes and Hackman, 1978 this study	X	X	X	No Late Quaternary offset.
14	This study	X	X		No Late Quaternary offset.

Table D.3.6 Compilation of mapped faults within a 65-km
radius of Mexican Hat (Continued)

Fault/ lineament number	Source	Examined by: low-sun-angle			Comments
		Aerial recon.	Aerial recon.	Ground recon.	
15	Hackman and Myant, 1973 Haynes and Hackman, 1978 Cooley et al., 1969	X	X		No Late Quaternary offset.
16a	Haynes et al., 1972	X	X	X	No Late Quaternary offset.
16b	Haynes et al., 1972	X	X		No Late Quaternary offset.
17	Haynes et al., 1972	X	X	X	No Late Quaternary offset.
18	Haynes et al., 1972	X	X		No Late Quaternary offset.
19	Cooley et al., 1969 Hackman and Myant, 1973	X	X		No Late Quaternary offset.
20	Cooley et al., 1969 Hackman and Myant, 1973	X	X		No Late Quaternary offset.
21	Cooley et al., 1969 Hackman and Myant, 1973	X	X		No Late Quaternary offset.
22	This study	X	X	X	Not a fault-dike emplacement.
23	Haynes et al., 1972 and this study	X	X	X	Excavated scarps in resistant Rico Fm.
24	This study	X	X		Excavated scarps in resistant Rico Fm.
25	This study	X	X		Possibly Quaternary displacement but probably erosion-exposed fault line. Not critical as length is only about 12 km (7.5 mi).

Table D.3.6 Compilation of mapped faults within a 65-km radius of Mexican Hat (Concluded)

Fault/ lineament number	Source	Examined by: low-sun-angle			Comments
		Aerial recon.	Aerial recon.	Ground recon.	
26	O'Sullivan & Beikman, 1963	X	X		No Late Quaternary offset.
27	O'Sullivan & Beikman, 1963 and this study	X	X		No Late Quaternary offset.
28	This study	X			No Late Quaternary offset.
29	This study		X		No Late Quaternary offset.
30	Cooley et al., 1969 Haynes and Hackman, 1973	X	X		No Late Quaternary offset.

Table D.3.7 Selected cases of induced macroearthquakes

Reservoir	Location	Depth, m	Volume $\times 10^6, m^3$	Largest earthquake
Hoover (Lake Mead)	U.S.A.	166	35,000	$M_L=5.0$
Kariba	Zambia/Rhodesia	122	175,000	$m_b=5.8$
Kremasta	Greece	120	4750	$M_S=6.3$
Koyna	India	103	2780	$M_S=6.5$
Kurobe	Japan	186	149	$M_S=4.9$
Manic 3	Canada	98	10,423	$m_b=4.3$
Hsinfengkiang	China	80	10,500	$M_S=6.1$
Nurek	U.S.S.R.	215	11,000	$M_S=4.5$

Ref. Adapted from Meade, 1982.

20

D.4 SITE FOUNDATION CHARACTERISTICS

The parameters recommended for use in the design process with regard to the foundation material are summarized in Table D.4.1. It is recommended that the surface natural soils not be considered since they are thin and of similar properties to the sand tailings. Thus, only parameters for the siltstone/sandstone are provided. Any parameters not provided should be determined by further lab testing or assumed to be some typical conservative value as found in the literature.

D.4.1 FOUNDATION MATERIAL PROPERTIES

D.4.1.1 In-situ data

In-situ data from exploratory borings indicate that competent bedrock underlies the tailings and a thin (approximately one-foot-thick) natural soil veneer. Of 120 borings conducted by MSRD (see boring log addendum at the end of this appendix) where sampling was attempted at the soil/bedrock interface, all experienced sampler refusal. Of seven borings conducted by the TAC where sampling was attempted at the soil/bedrock interface, all experienced sampler refusal. Examinations of rock cores obtained during the TAC drilling programs indicate that it is appropriate to use typical properties for the siltstone/sandstone bedrock in all analyses.

D.4.1.2 Elastic parameters

Typical values of the ratio of the modulus of deformation (E) to the unconfined compressive strength (q_u) for various sedimentary rocks, as well as typical values of unconfined compressive strength and Poisson's Ratio (Goodman, 1980), and the resulting calculated modulus of deformation are presented in Table D.4.2. However, it should be pointed out that these values are not based on tests of the actual foundation materials at the site and that the horizontal fracturing and presence of clay zones will cause the rock mass to display anisotropic (most likely orthotropic) stress-strain behavior.

Using the typical values presented in Table D.4.2 for the rock and typical values of the modulus of elasticity for clays (Duncan and Buchignani, 1976), it is possible to calculate the equivalent deformation modulus and Poisson's Ratio of the jointed rock mass using a procedure outlined by Goodman (1976). Using this method it is estimated that the modulus of deformation would be on the order of 3.5×10^4 per square inch. One of the assumptions of this method is that Poisson's ratio is 0.0. However, it is recommended that some small value (i.e., 0.001) be used to avoid computational difficulties in some theory of elasticity based calculations.

D.4.1.3 Strength

Testing of a siltstone sample from borehole HAT01-132 indicated that the strength mobilized at failure can be described by an effective friction angle (ϕ) of 51° with no cohesion (see Figure D.4.1 for a summary of test results). Typical values of ϕ for sandstones and siltstones reported by Goodman (1980) range from 27.8° to 45.2° . This would indicate that the test results are most likely not valid. It is thus recommended that an average value of 37° be used in design. Weaker clay zones are too deep to influence the stability of the pile.

STAGE NO.	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	MATERIAL DESCRIPTION
1	129.4	9.7	RED SILTSTONE
2	131.4	11.1	RED SILTSTONE
3	133.9	10.2	RED SILTSTONE

SAMPLE TYPE UNDISTURBED
RING SAMPLE

FIGURE D.4.1
SHEAR STRENGTH OF ROCK IN STAGED TRIAXIAL COMPRESSION
HAT01-132 27 - 28 FT.

SHEAR STRESS (ksf)

FIGURE D.4.1 CONT.
SHEAR STRENGTH OF ROCK IN STAGED TRIAXIAL COMPRESSION
HAT01-132 27 - 28 FT.

Table D.4.1 Design parameters - foundation

<u>Material description</u>	<u>Design parameters</u>	<u>Method of determination</u>
Siltstone/sandstone	Elastic parameters:	
$\gamma_d = \gamma_s = 150$ pcf	$E = 3.5 \times 10^4$ psi $\nu = 0.001$	Calculated from typical values
	Shear strength:	
	$\phi = 37^\circ$ $c = 0$	Average of typical values

Table D.4.2 Typical properties of unfractured sedimentary rock

<u>Rock description^a</u>	<u>Modulus ratio q_u/E</u>	<u>Unconfined compressive strength (q_u)(psi)</u>	<u>Modulus of deformation (E)(psi)</u>	<u>Poisson's ratio</u>
Berea Sandstone	261	1.07×10^4	2.79×10^6	0.38
Navajo Sandstone	183	3.10×10^4	5.68×10^6	0.46
Tensleep Sandstone	264	1.05×10^4	2.77×10^6	0.11
Hackensack Siltstone	214	1.78×10^4	3.81×10^6	0.22
Monticello Dam Greywacke	253	1.15×10^4	2.91×10^6	0.08
Average			3.59×10^6	0.25

^aRef. Goodman, 1980.

2001

D.5 TAILINGS CHARACTERISTICS

D.5.1 DESIGN PARAMETERS OF IN-SITU TAILINGS

The recommended parameters for use in the design of the tailings embankment are presented in Table D.5.1. These values are based primarily on laboratory test data but, in some cases, the laboratory results are combined with SPT or piezocone data to arrive at a realistic but conservative design value. In a very few cases, typical properties as obtained from the literature are recommended.

D.5.2 DESIGN PARAMETERS OF REMOLDED TAILINGS AND WINDBLOWN CONTAMINATED MATERIAL

The design parameters for remolded tailings and windblown materials are presented in Table D.5.2. It was assumed that the compaction characteristics of the remolded tailings and windblown materials would be similar to the in-situ sand-slime tailings.

D.5.3 MATERIAL PROPERTIES OF IN-SITU TAILINGS

Classification

A total of 52 samples had grain size-distribution determinations made via sieve analysis (ASTM C136) and of these, 28 were further analyzed using the hydrometer method (ASTM D422) to determine the grain size distribution of particles smaller than the No. 200 sieve (approximately 0.127 mm). Where possible the samples were classified from the grain size distribution and in several cases Atterberg limits (ASTM D4318) were determined to aid in classification. The results of these analyses are summarized in Table D.5.3 and grain size distribution curves are presented in Addendum D5.

The Atterberg limits test (ASTM D4318) is used to determine the water contents at which the soil behaves as a viscous liquid, as a plastic material, and as a brittle solid. The lowest water content at which the soil behaves as a liquid is called the liquid limit (W_L). At water contents below the liquid limit, the soil behaves as a plastic. The lowest water content at which the soil behaves as a plastic material is called the plastic limit (W_p). The liquid limit minus the plastic limit is defined as the Plasticity Index (PI), and is the range of water contents over which the soil reacts as a plastic material.

It has also been found that W_L , W_p , and PI can be empirically related to such things as shear strength and compressibility of clays and are thus useful in estimating mechanical properties or checking laboratory tests for reliability. It must be noted that the Atterberg limits describe the behavior of the remolded soil and may or may not indicate the behavior of the in-situ soil deposit.

Twenty Atterberg limits tests were performed and the results are presented in Table D.5.3. Only those soils which classify as slime (except for one foundation soil sample, HAT01-131 16.5 ft to 17 ft) were tested for Atterberg limits. The average slime soil has $W_L = 37$, $W_p = 22$, and $PI = 15$, and classifies as CL (medium plasticity clay) in the USCS.

One common indicator of a clay's mechanical behavior is its activity. Activity is defined as the PI divided by the percent of clay-size particles (<0.002 mm) (Lambe and Whitman, 1969). The average activity of the slimes (CL) is about 1.4. It is interesting to compare this with illite which has an average activity of 0.9, and montmorillonite with an average activity of 7.2 (Lambe and Whitman, 1969). Thus, it is likely that the slimes will behave similar to an illite (e.g., low to moderate shrinking and swelling due to changes in water content, moderate susceptibility to frost heave, and the like).

Natural moisture content, specific gravity, and unit weight

The average values determined from laboratory testing of natural moisture content, specific gravity, and unit weight have been selected for use as design parameters.

The natural water content (W_n) was determined for 85 tailings samples and the results are presented in Table D.5.3, while Figures D.5.1 through D.5.14 show how water content varied with depth. The sands had an average moisture content of 18 percent, sand-slimes were found to be at an average moisture content of 21 percent, and for slimes, $W_n = 37$ percent. It should be noted that the slimes in the upper pile had a higher average moisture content (about 54 percent) than did the slimes in the lower pile (about 34 percent). No direct measurements were made on sand tailings, but calculations indicate that the saturation is about 70 percent, while test results for undisturbed samples of sand-slime and slime indicated average saturations of 47 percent and 77 percent, respectively. (It should be noted that calculations based on average properties indicate that sand-slime tailings may have an average saturation as high as 64 percent.) The saturation level of slimes varied greatly from a low of 29 percent to a high of 100 percent (Table D.5.3).

In addition to the activity of clay (mentioned earlier), another indicator of clay behavior is the Liquidity Index. The Liquidity Index is defined as the natural water content minus the plastic limit divided by the Plasticity Index or

$$\frac{W_n - W_p}{PI} \quad (\text{Lambe and Whitman, 1969}).$$

This is a measure of how close the in-situ soil is to the liquid limit. This can be an indicator of the stability of a soil, since the soil flows at the liquid limit, and thus may be susceptible to flow movement if subjected to a change in stress. The average slime sample had a Liquidity Index slightly greater than 1.0 and thus has an in-situ water content slightly greater than the liquid limit.

The specific gravities of the tailings soils were determined and are presented in Table D.5.3. It was found that one sand sample tested had a specific gravity of 2.70 (quartz has an average specific gravity of 2.65) while both the sand-slime and the slime tailings have an average specific gravity of 2.76 (Illite has an average specific gravity of about 2.73 - Lambe and Whitman, 1969).

The in-situ dry density was determined using Shelby tube samples of slime tailings. The values for in-situ dry density as determined from laboratory testing are presented in Table D.5.3. The average in-situ dry densities of sand-slime and slime tailings were found to be about 87 pcf and 75 pcf, respectively.

SPT data can be used to estimate the relative density (D_r) and this can then be used in conjunction with values for maximum and minimum densities to calculate the in-situ density of the sand and sand-slime tailings. Laboratory tests (ASTM D4253 and D4254) of sand-slime tailings indicate an average maximum density of 100 pcf and an average minimum density of 75 pcf. Average SPT blow count for sand-slimes was approximately nine, indicating a relative density of about 52 percent, thus resulting in an average in-situ dry density of about 86 pcf. This is in very good agreement with the values determined in laboratory tests of undisturbed samples of sand-slime tailings.

Lambe and Whitman (1969) indicate values of maximum and minimum densities for silty sands (SM) of 127 pcf and 87 pcf, respectively. The average SPT blow count for the sand tailings (SM) is about 12. This results in a D_r =59 percent and a calculated in-place dry density of about 107 pcf.

Using the piezocone and lab test results, correlations between point stress and the relative density (D_r) of sands have been obtained by Robertson and Campanella (1984). These were used to estimate the relative densities of the sand and coarse-grained sand-slimes. Piezocone test results (Addendum D6) indicate that the sands had an average D_r of 58 percent while sand-slimes had an average D_r of about 48 percent. Thus, using the same maximum and minimum densities as were used with the SPT data, the in-situ dry density of sand tailings was, again, found to be about 107 pcf. The in-situ dry density of the sand-slimes was calculated to be about 85 pcf which is in agreement with both laboratory results and SPT estimates.

Thus, the dry density of the sand tailings will be taken as 107 pcf for design purposes. A dry density of 87 pcf will be used for sand-slimes for design purposes since both SPT and piezocone correlations support the lab determinations. The average value for the in-situ dry density as determined from lab tests, or 75 pcf, is recommended for use in design. However, it should be noted that, as well as differing in their in-situ moisture contents, the slimes in the upper and lower tailings pile have average dry densities that are quite different. These dry densities are 62 pcf and 81 pcf, respectively, for the upper and lower piles.

Hydraulic conductivity

The hydraulic conductivity (permeability) of the various soil layers is a necessary input in the determination of groundwater flow patterns and contributes to the analysis of settlements, slope stability, and liquefaction.

Both constant head and triaxial permeability tests were performed on a total of six Shelby tube samples and the results are presented in Table D.5.4. The one sand tailings sample tested showed a hydraulic conductivity of 1.2×10^{-2} cm/s which, although on the high side, is within the expected range for sands (Lambe and Whitman, 1969). A sand-slime sample had a hydraulic conductivity of about 3.0×10^{-6} and is about average for silty soils (Lambe and Whitman, 1969). The average hydraulic conductivity of the slime soils tested was 1.7×10^{-6} cm/s and all four samples tested produced results which were close to this average. Again, this is well within the expected range for clayey soils (Lambe and Whitman, 1969).

Although the values obtained for the hydraulic conductivity of the slime tailings are fairly consistent and are thought to be reliable, it is interesting and informative to compare the hydraulic conductivity of similar samples as calculated from consolidation tests. These calculated hydraulic conductivities are presented in Table D.5.5. Two slime samples, HAT01-126, 22.5 to 25 feet, and HAT01-136, three to 5.5 feet, had hydraulic conductivities of 6.1×10^{-7} cm/s and 1.3×10^{-7} cm/s, respectively, as determined in permeability tests. The hydraulic conductivity, as calculated from consolidation test data, averaged 9.2×10^{-8} cm/s and 7.9×10^{-8} cm/s, respectively. The calculated values are about an order of magnitude less than the permeability test values. This would be expected since the consolidation test values reflect much higher confining pressures and lower void ratios than those found in the tailings deposit and thus tend to confirm the permeability test data for these two samples.

The overall average value of hydraulic conductivity for slime samples as calculated using consolidation test data is about 1.3×10^{-7} cm/s. This is, again, about an order of magnitude less than the average of the permeability tests. As stated above, this tends to reflect the higher confining pressures used in the consolidation tests and would tend to confirm the reliability of the results of the permeability tests. Since the addition of cover and substantial tailings fills are expected to be necessary over most of the existing lower tailings pile, it is anticipated that the resulting consolidation and reduced void ratios caused by the increased overburden stresses will warrant the use of the reduced permeabilities as calculated from consolidation test data. Thus, for loadings below 500 psf, the average hydraulic conductivity of the slime tailings as determined in the permeability tests, or 1.7×10^{-6} cm/s, will be used for design. For loadings of 500 psf or more, the average values for the hydraulic conductivities at the various loadings will be used (see Table D.5.1 for these values).

Capillary moisture relationship

Capillary moisture data are useful in determining the long-term moisture content of the soil, its wilting point, and can also be used to calculate the unsaturated hydraulic conductivity.

The capillary moisture test is performed in order to determine the amount of water a soil can hold under a given pressure difference and thus indirectly measures the moisture content of the soil as a function of the capillary tensions developed within the soil matrix. Six undisturbed soil samples were tested and the results are presented in Figures D.5.15 through D.5.22.

Compressibility

In order to design an efficient, long-lasting, low-permeability radon attenuation barrier (cover) an estimate of the expected long-term settlements within the tailings embankment as a function of time and location on the embankment surface is required. This enables the designer to determine the magnitude of differential settlements and thus to evaluate and avoid both the possibility of cover-cracking and the creation of preferred drainage paths leading to possible erosional breaching of the cover.

A soil deposit, when subjected to increased stresses, will tend to reduce in volume via a reduction in its void ratio and thus cause settlement of the surface of the deposit. There are two types of settlement: immediate settlement, also referred to as elastic settlement; and long-term settlement, composed of primary or consolidation settlement and secondary or creep settlement.

Immediate elastic settlement: Elastic settlements can be calculated using elasticity theory, knowing the deformation modulus (E) and the Poisson's ratio (ν) of the soil in question.

It is possible to use the results of triaxial testing in order to estimate the value of E . Even the best "undisturbed" sample (especially sands) will show some effects due to disturbance and thus the value of E found in the lab can vary significantly from that found in the actual soil deposit. Further complicating the determination of E is the fact that E generally will vary with confining stresses. In order to avoid these problems, the data from in-situ tests (SPT and piezocone) can be used in conjunction with empirical relationships in order to estimate E in the more sandy soils.

To estimate E for silty sand soils using SPT data, Bowles (1982) suggests the use of the empirical formula $E = 6(N + 15)$ ksf. In this relationship N is the number of blows required to drive the SPT sampler 12 inches through the soil deposit. Using the average N values of 12 and nine for sands and sand-silts, respectively, results in an estimated E of 1.6×10^5 per square foot (psf) for sands and 1.4×10^5 psf for sand-silts.

Using piezocone data (Addendum D6), E can be estimated for sands using the relationship :

$$E = 3.5q_c \text{ (Schmertman, 1978).}$$

Thus, from the above-stated relationship, the average E for sand and sand-slime tailings can be estimated to be approximately 3.5×10^5 psf and 1.7×10^5 psf, respectively.

For use in design calculations, the average of the values of E as determined from SPT and piezocone data, or 2.6×10^5 psf and 1.6×10^5 psf, will be used for sands and sand-slime mixtures, respectively.

Duncan and Buchignani (1976) present a graph which allows one to obtain a value of E for clays based on PI, overconsolidation ratio, and undrained strength (S_u). Since the average PI for the slime tailings is 15, the average overconsolidation ratio about 3.0 (see following paragraphs on consolidation), and the average undrained strength about 1000 psf (see section on shear strength), then E (drained) is estimated to be about 1.7×10^5 psf.

Correlations of q_c from piezocone testing, undrained strength (S_u), and overconsolidation ratio (OCR) with the elastic properties of a clay soil have been developed and may be used to estimate elastic settlements in the slime tailings.

Thus, an estimate of E (undrained) for clays can be determined from charts (presented by Robertson and Campanella, 1984). Using these charts, a rough estimate of an average E for slime tailings would be about 7.2×10^5 psf.

Thus, for design purposes, the value of E for saturated slimes should be taken as 7.0×10^5 psf while for unsaturated conditions this value should be reduced to about 2.0×10^5 psf.

The value of Poisson's ratio can vary between about 0.1 and 0.5 for most soils. Bowles (1982) recommends 0.25 for fine sands, 0.3 to 0.35 for silts, 0.1 to 0.3 for unsaturated clays, and 0.4 to 0.5 for saturated clays. Thus, for design purposes, recommended values of Poisson's ratio are 0.25 for sand tailings, 0.35 for sand-slime tailings, 0.3 for unsaturated slime tailings, and 0.5 for saturated slime tailings.

Consolidation parameters: The most commonly used equation for calculating the consolidation settlement of a saturated soil layer is one adapted from the original differential equation developed by Terzaghi (Lambe and Whitman, 1969). This equation uses the coefficient of compressibility C_c , which is the slope of the void ratio (e) versus log of effective stress ($\log p'$) curve, to estimate consolidation settlement. It has been pointed out (Znidarcic and Schiffman, date unknown) that the simplified Terzaghi equation was developed by assuming that only infinitesimal strains will occur in response to the increased loading and ignoring the effects of the self weight of the soil, while further assuming that the coefficient of permeability

remains constant as the void ratio decreases during consolidation. These assumptions do not result in substantial differences in calculated settlements in most natural soil deposits. However, tailings may exist at relatively large void ratios and thus settlements can become a large percentage of original soil height. Thus tailings may violate the assumption of infinitesimal strains and can have such large void ratio changes as to cause permeability to decrease with decreasing void ratio.

For use in the standard or infinitesimal strain consolidation calculations, the Figures D.5.23 through D.5.32 are reproductions of the laboratory consolidation curves. Data related to the laboratory consolidation tests and calculated values of the coefficient of consolidation and the secondary compression index are found in Table D.5.6.

Consolidation tests indicate that the average overconsolidation ratio (OCR) for slime tailings is about 3.5. In order to help determine whether the slimes are normally consolidated or overconsolidated, the undrained strength (S_u) of the slimes based on piezocone data was estimated to be about 1100 psf. If this value for S_u is compared to that for the average normally consolidated clay (200 to 400 psf), it is evident that the piezocone data support the contention that the slime tailings are overconsolidated.

Robertson and Campanella (1984) present a chart from which the OCR can be estimated using the approximate current overburden stress, the S_u of the soil, and the ratio of S_u (for normally consolidated soil) to the current overburden stress. Using this method, the average OCR for the slime tailings was estimated to be 3.0.

This overconsolidation may result from capillary tensions induced by the evaporation of moisture from the soil during the deposition process. However, this concept cannot account for samples at all depths showing the effects of overconsolidation when it is probable that, during the active milling period, only a few layers of tailings dried out before more were placed over them. Another possibility, which would require research beyond the scope of this document, is that some chemical process occurring at all depths within the tailings is capable of inducing overconsolidation. Still another possibility is that salts or other chemicals within the tailings have caused a moderate amount of cementation to occur thus causing the soil to act as if it were overconsolidated.

Whatever the reason, the majority of the applicable data point to the conclusion that the tailings will behave as if they are overconsolidated and therefore an OCR of three to 3.5 should be used in design calculations.

Total consolidation settlement is calculated from consolidation theory using consolidation test data. The slope of the strain vs $\log p$ curve is called the compression index (C_{cc}). At stresses below the preconsolidation pressure (P_p), C_{cc} is smaller than for stresses greater than the preconsolidation pressure and is referred to as the rebound compression index or C_{rc} . (It should be noted that the

epsilon (e) subscript indicates that the indices are calculated with reference to the strain vs log p plot and that the same indices (C_c and C_{rc}) are referenced to a void ratio vs log p plot are related by:

$$C_{ce} = \frac{C_c}{1 + e_0} \text{ and } C_{rc} = \frac{C_r}{1 + e_0}$$

where

e_0 is the void ratio at the start of loading).

It is recommended that the expected strains due to loading of sand tailings be calculated directly from the consolidation curve for sample HAT01-135 (30 to 32.5 feet). This sample, although classifying as a sand-slime, was relatively low in fines content (36 percent passing #200) and had a relatively high dry density (96.8 pcf) and thus can be expected to behave very similarly to the sand tailings.

The compression indices recommended for sand-slimes are $C_{cc} = 0.12$ and $C_{rc} = 0.010$. These values were determined from the consolidation curve presented for HAT01-136 (three to 5.5 feet).

In calculating the total consolidation settlement in the slime tailings, it is recommended that $C_{cc} = 0.19$ and $C_{rc} = 0.019$ be used. These values are averages of the values as determined from the consolidation curves for slime tailings.

Schmertman (1978) has suggested that the quantity S_u/σ_v' (see shear strength section) could also be used to estimate the coefficient of compressibility (C_{cc}) given the OCR. Thus, the slime tailings with an OCR of about three should have a C_{cc} of about 0.15 using the tables prepared by Schmertman.

Lambe and Whitman (1969) have developed a correlation between W_L and C_c . This relationship is

$$C_c = 0.009 (W_L - 10).$$

Note that the average W_L of the samples tested was 40, using the above relationship results in a calculated value for C_{cc} of 0.13. Typical values reported by Lambe and Whitman indicate that C_{cc} may vary from 0.13 to 0.21.

However, these typical values are based on tests on natural soils. Vick (1983) indicates that tailings slimes will have a C_c in the range from 0.20 to 0.30 which is equivalent to a C_{cc} of approximately 0.10 to 0.15.

Thus the values of C_{cc} and C_{rc} chosen for use in the design calculations, although on the high side for tailings, are well within the range of values reported in the literature.

Table D.5.6 gives a summary of the various time rate settlement parameters obtained through consolidation tests performed in the laboratory on six samples of slime tailings and a sand-slime sample.

The coefficient of consolidation (C_v) is used to estimate the percent of total consolidation settlement that will occur over a given time period following application of the load to the soil deposit. The value to be used in design calculations must take into account the load applied to the soil since C_v will change with loading. Thus, in order to estimate the values of C_v for slimes tailings for use in design, the laboratory determined values of C_v for each load increment from one to 32 ksf were averaged for all slime samples for which C_v data were available. (The results are presented in Table D.5.1.)

Various analytical and empirical solutions have been proposed which allow an estimation of the coefficient of consolidation from piezocone pore pressure decay data. However, the rate of pore pressure decay is a function of permeability and permeability is reduced in unsaturated soils. This makes the determination of both permeability and C_v for the predominately partially saturated slimes prone to large uncertainties and thus will not be attempted here.

Lambe and Whitman (1969) state that, for soils with $W_L = 30$ percent, the value of C_v can vary from 0.014 to 0.330 in^2/min but that the expected value would be about 0.047 in^2/min . The average laboratory values for the various loadings varied from 0.032 to 0.050 in^2/min . These values of C_v are near the low end of the above stated range but are in very good agreement with the expected average value.

The values of C_v for sample HAT01-136 (three to 5.5 feet) should be used for the design calculations of settlements of sand-slime tailings. These values of C_v vary between 0.012 and 0.027 in^2/min and thus are on the low end of the range quoted above.

In the case of sand tailings, the use of C_v in calculations is not recommended. Due to the high permeability of the sand tailings, consolidation settlement will occur very rapidly and thus can be considered to occur immediately upon loading.

No parameters relating to the finite strain analysis are provided in this document. The average void ratio of the slime tailings was calculated to be 1.18 using the average unit weight and specific gravity from laboratory tests (Table D.5.1). This void ratio is in the typical range for natural silty clays under the overburden stresses in the tailings pile (Lambe and Whitman, 1969) and in the low range for copper tailings which are generally thought to be similar to uranium tailings (Vick, 1983). Thus the slime tailings at Mexican Hat, which have been consolidating since production stopped in 1965, are in a dense enough state to preclude the need for a finite strain analysis. The consolidation strains should thus be well within the range for which conventional (infinitesimal strain) analysis is valid.

Strength

The shear strengths of the various soils in the tailings pile are a key aspect of slope stability analysis. In order to determine the

shear strengths of the tailings, two types of laboratory tests were used: (1) the unconsolidated-undrained or "Quick" (Q) test; and (2) the consolidated-undrained test with pore pressure measurements or "Rapid" (R) test. In addition to the lab testing, empirical correlations between SPT, piezocone data, and shear strength parameters were used.

The Q test is generally performed only on cohesive fine-grained, low permeability soils that are expected to show intermediate to long-term increases in pore pressures in response to loading. Since this test was performed on undisturbed soil samples which were not initially saturated, the shear stress (τ) vs normal effective stress (σ') relationship is highly nonlinear. This nonlinearity is maintained until the stresses become high enough to drive all the air in the voids into the water thus causing the soil to become 100 percent saturated. Thus, a "Q" triaxial test on an initially unsaturated sample will yield a value for both the friction angle (ϕ) and cohesion (C).

When a sample which is completely saturated remains undrained during loading (Q test), the normal effective stress remains zero since changes in stress are reflected only in increased pore pressures. If effective normal stresses go to zero in the Mohr-Coulomb equation then the only shear strength available to the soil is cohesion and, in a saturated clay sample, this value remains the same for all confining pressures near the in-situ confining pressure. This value, known as the saturated-undrained (S_u) strength, tends to increase linearly with depth in normally consolidated deposits.

The R triaxial shear test is performed on a soil sample that is nearly 100 percent saturated. If the sample is not saturated as it comes from the field, it is saturated in the testing apparatus by forcing water, under pressure, through the sample until nearly 100 percent saturation is reached. In this test both total stress (c, ϕ) and effective stress (c, ϕ) parameters can be obtained.

A summary of the Q and R triaxial testing results is presented in Figures D.5.33 through D.5.36.

Since triaxial tests are not generally performed on sand tailings due to sampling problems, it is necessary to use N and q_c values from SPT and piezocone testing, respectively, to estimate ϕ . Using the correlations developed by Peck et al. (1974) with the N value for sands (12) yields an estimated average ϕ of about 30°. This same correlation used to determine ϕ for the sand-silts is not as reliable due to the higher fines content. However, it may be useful as a guide and was thus determined to be about 29° using an average N = nine.

An R triaxial test performed on an undisturbed sample of very silty sand-silt tailings resulted in a ϕ of 36.5° and a cohesion (c) of 300 psf.

Robertson and Campanella (1984) present charts for determining the ϕ for sands from piezocone data. Using those charts and average q_c and overburden stresses, ϕ was determined to be about 32° for the

sand tailings and 31° for sand-slime tailings. Vick (1983) indicates that the typical value of ϕ for copper sands varies between 33° and 37°.

An average value of $\phi = 31^\circ$ is recommended for sand tailings. This value is somewhat conservative given the data presented by Vick but is a very reasonable value as demonstrated by piezocone and SPT correlations and will result in a conservative design.

Since most of the sand-slime tailings are silty to very silty sands, it is recommended that ϕ for these soils be taken as 29°. This is lower than the range for tailings sands indicated by Vick but is a reasonable and conservative value as indicated by both the SPT and piezocone tests. The above-mentioned triaxial test on a recompacted specimen is probably more representative of slime tailings (see the following paragraphs).

Since both sands and sand-slime tailings have relatively high permeabilities and thus will not allow the buildup of significant pore pressures, it is recommended that ϕ be used in all analyses (drained and undrained conditions).

In the case of slime tailings, one "Q" test on an unsaturated, relatively low plasticity slime (CL) resulted in a $\phi = 11^\circ$, $C = 600$ psf failure envelope (Figure D-5.36). This would indicate that the true undrained strength of this sample is greater than 600 psf. Since the sample tested at the highest confining stress is probably very nearly saturated, the radius of the Mohr's circle for this pressure would indicate that S_u is probably closer to 3.5 ksf.

Vick (1983) presents data indicating that, in copper slimes at similar densities to the Mexican Hat slimes, ϕ can vary from 14° to 24° while C may vary between zero and 400 psf.

SPT (Addendum D4) and piezocone (Addendum D6) data can be used to estimate the S_u of a clay via empirical correlations. Data presented by Terzaghi and Peck (1967) indicate that a clay soil with an average SPT $N = 7$ will have an S_u of about one ksf.

Research has determined that the undrained strength of clays can be estimated from piezocone data using the empirical equation:

$$S_u = \frac{q_c}{N_c} \quad (\text{Robertson and Campanella, 1984})$$

where N_c is a constant that varies with the cone geometry and soil. Wissa and Fuleihan (1983) recommend an average value of $N_c = 17$ be used.

Using the above empirical relation, it was found that the estimated average undrained strength of the slime tailings is 1100 psf.

Lab tests and data presented by Vick indicate that, for design purposes, the undrained strength of in-situ unsaturated slimes tailings can be modeled using $\phi = 11^\circ$ and $C = 400$ psf.

In the case of saturated slimes, the undrained strength (S_u) recommended for design is 1000 psf, the lower of the values presented above. Data presented in Lambe and Whitman (1969) indicate that S_u should vary between about 600 psf and 1300 psf for the overburden pressures found within the slimes areas of the tailings pile and is thus in good agreement with the value chosen for use in design.

Two undisturbed slimes samples were tested in the R triaxial test. These test results showed that ϕ varies between 35.1° and 39.1° and C varies from 400 to 500 psf. A correlation between plasticity index (PI) and ϕ presented by Lambe and Whitman (1969) indicates that the slimes tailings, with an average PI of 15, should have a ϕ of about 33° . Vick (1983) states that copper slimes typically have a ϕ that varies between 33° and 37° .

Thus, it appears that the test indicating a $\phi = 39.1^\circ$ is probably atypical and that the other test, indicating a $\phi = 35.1^\circ$, is probably more typical. Therefore it is recommended that a $\phi = 35^\circ$ be used for the slimes tailings in design calculations and that $C = 0$ be used in order to be conservative.

The above-discussed shear strength parameters were determined using tests and empirical data based on static loads. It must be recognized that dynamic loading, such as in an earthquake, can result in severe losses of shear strength in some soils (Seed, 1982).

One indicator of this condition is sensitivity. The sensitivity of a clay is defined as the ratio of undisturbed strength to remolded strength and indicates the reduction in strength that might be expected under shock, earthquake loading, or other circumstances which may cause sudden rearrangement of the soil structure. Sensitivity (S_t) can be estimated using piezocone data (Addendum D6) in conjunction with the approximate correlation

$$S_t = \frac{10}{FR} \quad (\text{Schmertman, 1978})$$

where

FR = friction ratio.

The sensitivity of the slime tailings was found to be about 2.8, thus indicating an insensitive soil (Bowles, 1982).

This would lead to the conclusion that, at least in the case of the slime tailings, no significant loss in strength should occur due to dynamic loading. However, no parameters will be recommended here due to the fact that the dynamic strength of a soil deposit depends not only on the soil itself but also upon the nature and duration of the

dynamic loads applied. Thus, the shear strength parameters used in design should reflect the response of the soil to the expected ground motions generated in the design earthquake (see Section D.3).

D.5.4 MATERIAL PROPERTIES OF REMOLDED TAILINGS AND CONTAMINATED WINDBLOWN MATERIAL

Relative density

Samples from two test pits were tested to establish their maximum and minimum densities for use in relative density calculations. The results are presented in Table D.5.7.

Moisture-density relationships

In order to determine the compaction characteristics of the remolded tailings and contaminated windblown material, the standard Proctor compaction test (ASTM D698) was performed on a number of tailings samples excavated from test pits on the tailings pile.

The results of these compaction tests on various samples of sand-slime and slime tailings are presented in Table D.5.7. Also presented in Table D.5.7 are the physical characteristics of these samples including grain size distribution (grain size distribution curves are presented in Addendum D4), natural water content, Atterberg limits, specific gravity of solids, and USCS classification.

Hydraulic conductivity and capillary moisture relations

Data presented by the U.S. Navy (1982) and Sowers (1979) indicate that silty sands and silts can be expected to have hydraulic conductivities in the range of 10^{-2} to 10^{-6} cm/s. Since the recompacted tailings are expected to have a large percentage of silt size particles, it is recommended that a hydraulic conductivity of 10^{-5} cm/s be used in design.

Portions of seven of the bulk samples were recompacted to approximately 90 percent of standard Proctor maximum density in metal rings and capillary moisture tests were performed on these specimens. The results of these tests are presented in Figures D.5.37 through D.5.41.

Compressibility

No testing was done to determine the compressibility of the remolded sand-slime tailings. Thus, a discussion of the available data in the literature is presented below.

In order to determine the compressibility of the recompacted tailings, it will be assumed that the tailings will be a relatively

homogeneous mixture when compacted. Thus, it is assumed that the material will exhibit the properties and behavior of a very silty sand (SM) or, possibly, a silt (ML) and thus could be classified as a sand-slime.

As presented in Table D.5.2 this material, if placed at 90 percent of standard Proctor maximum density (as determined by averaging the test data for remolded sand-slimes), will be at a dry density of 95 pcf at a moisture content of about 15 percent.

Elastic settlement

Due to the relatively dense state of the recompacted tailings, elastic settlement is expected to be a minor component of total settlement. However, in order to get an order of magnitude estimate of elastic settlement it is recommended that the elastic parameters used for the in-situ sand-slime tailings be used in design calculations (see Tables D.5.1 and D.5.2).

Consolidation settlement

Data presented by the U.S. Navy (1982) indicate that, at the stress levels expected in the tailings embankment, the value of the compression index (C_c) for a silt soil should be about 0.008. Winterkorn and Fang (1975) indicate that a silt soil compacted at optimum moisture to a density of 109.7 pcf should have a C_c of less than 0.03. However, the above-quoted data from Winterkorn and Fang were obtained under loads of 37.5 psi and greater and thus probably represent the extreme upper bound value. They also present data showing that C_c can vary between 0.003 and 0.065 with the average value being about 0.02.

The above-quoted values for C_c probably represent the extreme range of values (0.008 to 0.065) and thus the average value (0.02), is probably a good estimate for the sand-slime tailings, and corresponds to a C_{cc} of about 0.011. This is a very small value when compared to the values for the in-situ tailings. However, it should be noted that compaction tends to "overconsolidate" the soil and thus it would be reasonable to expect that future consolidation would be occurring over a range of stresses for which the rebound compression index is appropriate. In light of this consideration, the average value would seem to be reasonable for use in design. Therefore, a C_{rc} of 0.011 is recommended for design calculations while C_{cx} is assumed to be irrelevant and thus no value is recommended.

Time rate of consolidation settlement

It is assumed that the recompacted tailings will consolidate "immediately" (prior to completion of construction). Thus, no value for C_v is recommended. Since correlations exist which relate secondary settlement to immediate elastic settlement (Schmertman, 1978), no values of C_α will be recommended for use in design.

Shear strength

An R test was performed on a specimen recompacted from a composite sample of sand-slime (obtained by combining test pit samples HAT01-001 [0.0' - 1.5'] and HAT01-002 [3.0' - 4.5']). This sample was recompacted to a dry density of 90.2 pcf (D_r = 68 percent) in order to approximate the in-place densities of the sand and sand-slime (SM) tailings. These tailings are in such a loose state in-situ that the laboratory could not successfully remold a specimen to the average in-situ density. Thus the test results (ϕ = 31.7°, c = 0 in Figure D.5.42) probably indicate the upper bound in-situ strength of the sand and sand-slime tailings as well as giving a good indication of the expected strength of the recompacted tailings.

As an indication of the lower bound strength of the in-situ sand-slime tailings, a similar in-situ material was found to have an average ϕ of 29° at an average density of 87 pcf.

Information presented by the U.S. Navy (1982), Lambe and Whitman (1969), and Winterkorn and Fang (1975) indicates that ϕ for recompacted silts and silty sands should vary between 30° and 34°. Therefore, based on the test data, it is recommended that an average value of 32° be used in design.

The above-recommended value should also be used in both short-term and dynamic analyses. Since saturation will not occur within the recompacted tailings, no significant pore pressure buildup is expected upon loading. Thus, under rapid static loading (construction) or dynamic loading (earthquake), no loss in strength is anticipated.

FIGURE D.5.1
WATER CONTENT VS DEPTH
HAT-01-126

FIGURE D.5.2
WATER CONTENT VS DEPTH
HAT01-127

FIGURE D.5.3
WATER CONTENT VS DEPTH
HAT01-128

**FIGURE D.5.4
WATER CONTENT VS DEPTH
HAT01-129**

**FIGURE D.5.5
WATER CONTENT VS DEPTH
HAT01-131**

FIGURE D.5.7
WATER CONTENT VS DEPTH
HAT 01-133

D-107

FIGURE D.5.8
WATER CONTENT vs. DEPTH
HAT01-130

**FIGURE D.5.9
WATER CONTENT VS DEPTH
HAT01-135**

FIGURE D.5.10
WATER CONTENT VS DEPTH
HAT 01-136

FIGURE D.5.11
WATER CONTENT VS DEPTH
HAT01-137

**FIGURE D.5.13
WATER CONTENT VS DEPTH
HAT01-139**

FIGURE D.5.14
WATER CONTENT VS DEPTH
HAT01-181

FIGURE D.5.15
CAPILLARY MOISTURE CURVE
HAT-126 15-17.5 FT. (SAMPLE 1)

FIGURE D.5.16
CAPILLARY MOISTURE CURVE
HAT01-126 15 - 17.5 FT (SAMPLE 2)

FIGURE D.5.17
CAPILLARY MOISTURE CURVE
HAT01-131 10 - 12.5 (SAMPLE 1)

FIGURE D.5.18
CAPILLARY MOISTURE CURVE
HAT01-131 10 - 12.5 FT. (SAMPLE 2)

FIGURE D.5.19
CAPILLARY MOISTURE CURVE
HAT01-134 35 - 37.5 FT.

FIGURE D.5.20
CAPILLARY MOISTURE CURVE
HAT01-135 20 - 22.5 FT.

FIGURE D.5.21
CAPILLARY MOISTURE CURVE
HAT01-136 20 - 22.5 FT.

FIGURE D.5.22
CAPILLARY MOISTURE CURVE
HAT01-1376.5 - 9 FT.

LEGEND						
SAMPLE	DEPTH INTERVAL (FT.)	DRY DENSITY PCF LB./CU. FT.		MOISTURE CONTENT (%)		UNIFIED SOIL CLASSIFICATION
		INITIAL	FINAL	INITIAL	FINAL	
HAT01-126	3.0-3.5	81.7	84.2	38.4	29.0	CL

TESTED SUBMERGED

FIGURE D.5.23
CONSOLIDATION TEST RESULTS UNDISTURBED TAILINGS
HAT01-126 3 - 5.5 FT.

LEGEND					
SAMPLE	DEPTH INTERVAL (FT.)	DRY DENSITY PCF LBS./CU. FT.		MOISTURE CONTENT (%)	
		INITIAL	FINAL	INITIAL	FINAL
HAT01-126	22.5-25.0	67.4	77.1	46.3	43.7
					UNIFIED SOIL CLASSIFICATION
					ML

TESTED SUBMERGED

FIGURE D.5.24
CONSOLIDATION TEST RESULTS UNDISTURBED TAILINGS
HAT01-126 22.5 - 25 FT.

LEGEND						
SAMPLE	DEPTH INTERVAL (FT.)	DRY DENSITY PCF LBS./CU. FT.		MOISTURE CONTENT (%)		UNIFIED SOIL CLASSIFICATION
		INITIAL	FINAL	INITIAL	FINAL	
HAT 01-131	10.0-12.5	84.0	92.5	28.2	28.4	CL

TESTED SUBMERGED

FIGURE D.5.25
CONSOLIDATION TEST RESULTS UNDISTURBED TAILINGS
HAT01-131 10.0 - 12.5 FT.

LEGEND						
SAMPLE	DEPTH INTERVAL (FT.)	DRY DENSITY PCF LBS./CU. FT.		MOISTURE CONTENT (%)		UNIFIED SOIL CLASSIFICATION
		INITIAL	FINAL	INITIAL	FINAL	
HAT 01-133	8.5-11.0	86.7	92.9	8.0	31.2	(CL)

TESTED SUBMERGED

FIGURE D.5.26
CONSOLIDATION TEST RESULTS UNDISTURBED TAILINGS
HAT01-133

LEGEND					
SAMPLE	DEPTH INTERVAL (FT.)	DRY DENSITY PCF LBS./CU. FT.		MOISTURE CONTENT (%)	
		INITIAL	FINAL	INITIAL	FINAL
HAT 01-133	8.5-11.0	86.8	93.8	25.1	31.1
					UNIFIED SOIL CLASSIFICATION
					(ML)

TESTED AT INSITU
MOISTURE TO 1 KSF
THEN SUBMERGED

FIGURE D.5.27
CONSOLIDATION TEST RESULTS UNDISTURBED TAILINGS
HAT01-133 8.5 - 11 FT.

LEGEND						
SAMPLE	DEPTH INTERVAL (FT.)	DRY DENSITY PCF LBS./CU. FT.		MOISTURE CONTENT (%)		UNIFIED SOIL CLASSIFICATION
		INITIAL	FINAL	INITIAL	FINAL	
HAT01-135	30.0-32.5	85.6	92.5	12.6	27.2	SM

TESTED SUBMERGED

TESTED SUBMERGED

FIGURE D.5.28
CONSOLIDATION TEST RESULTS UNDISTURBED TAILINGS
HAT01-135 30 - 32.5 FT.

LEGEND						
SAMPLE	DEPTH INTERVAL (FT.)	DRY DENSITY PCF LBS./CU. FT.		MOISTURE CONTENT (%)		UNIFIED SOIL CLASSIFICATION
		INITIAL	FINAL	INITIAL	FINAL	
HAT01-135	30.0-32.5	96.8	101.0	11.5	22.6	SM

TESTED AT INSITU MOISTURE
TO 2 KSF THEN SUBMERGED

FIGURE D.5.29
CONSOLIDATION TEST RESULTS UNDISTURBED TAILINGS
HAT01-135 30 - 32.5 FT.

LEGEND					
SAMPLE	DEPTH INTERVAL (FT.)	DRY DENSITY PCF LBS./CU. FT.		MOISTURE CONTENT (%)	
		INITIAL	FINAL	INITIAL	FINAL
HAT01-138	3.0-5.5	77.8	88.5	35.2	32.7
					CL

TESTED SUBMERGED

FIGURE D.5.30
CONSOLIDATION TEST RESULTS UNDISTURBED TAILINGS
HAT01-138 3 - 5.5 FT.

LEGEND						
SAMPLE	DEPTH INTERVAL (FT.)	DRY DENSITY PCF LBS./CU. FT.		MOISTURE CONTENT (%)		UNIFIED SOIL CLASSIFICATION
		INITIAL	FINAL	INITIAL	FINAL	
HAT01-136	13.0-16.0	74.9	81.3	48.8	30.5	CH

TESTED AT INSITU
MOISTURE CONTENT

FIGURE D.5.31
CONSOLIDATION TEST RESULTS UNDISTURBED TAILINGS
HAT01-136 13 - 15 FT.

LEGEND					
SAMPLE	DEPTH INTERVAL (FT.)	DRY DENSITY PCF LBS./CU. FT.		MOISTURE CONTENT (%)	
		INITIAL	FINAL	INITIAL	FINAL
HAT 01-138	35.0-37.5	85.3	98.4	35.4	27.9
					CL

TESTED AT INSITU
MOISTURE CONTENT

FIGURE D.5.32
CONSOLIDATION TEST RESULTS UNDISTURBED TAILINGS
HAT01-138 35 - 37.5 FT.

STAGE NO.	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	SOIL DESCRIPTION
1	87.5	33.3	ML (SLIME)
2	88.9	34.3	
3	91.8	32.1	

SAMPLE TYPE SHELBY TUBE

FIGURE D.5.33
SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION
HAT01-126 25 FT.

FIGURE D.5.33 (CONT.)
SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION
HAT01-126 25 FT.

STAGE NO.	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	SOIL DESCRIPTION
1	82.0	16.9	SAND TAILS
2	85.0	34.5	SAND TAILS
3	87.2	32.7	SAND TAILS

SAMPLE TYPE UNDISTURBED
SHELBY TUBE

FIGURE D.5.34
SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION
HAT01-133 1.5 - 4 FT.

FIGURE D.5.34 (CONT.)
SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION
HAT01-133 15 - 4 FT.

STAGE NO.	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	SOIL DESCRIPTION
1	83.7	37.0	CL (SLIME)
2	88.1	37.1	
3	85.5	34.8	

SAMPLE TYPE UNDISTURBED
 SHELBY TUBE

FIGURE D.5.35
SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION
HAT01-138 35 - 37.5 FT.

FIGURE D.5.35 (CONT.)
SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION
HAT01-138 35 - 37.5 FT.

TEST NUMBER	1	2	3	4
LOCATION	1-23			
SAMPLE NO.	008			
DEPTH FT.	13-15.5			
WEIGHT-INCH	5.532	5.785	5.566	
DIAMETER-INCH	2.872	2.667	2.848	
WATER CONTENT-%	31.0	42.8	32.3	
DRY DENSITY-pcf	65.3	77.3	67.3	
$\sigma_1 \sigma_3$ -ksf	1.53	1.82	3.83	
σ_3 -ksf	1.01	3.02	6.05	
σ_1 -ksf	2.54	4.84	9.88	

TYPE OF SPECIMEN UNDISTURBED SHELBY TUBE

SOIL DESCRIPTION CLAY; TAILINGS

TYPE OF TEST UNCONSOLIDATED-UNDRAINED

UNSATURATED

FIGURE D.5.36

TRIAXIAL SHEAR TEST RESULTS
HAT01-136 13 - 15.5 FT.

FIGURE D.5.37
CAPILLARY MOISTURE CURVE
HAT01-002 0 - 0.5 FT. (TEST PIT)

FIGURE D.5.38
CAPILLARY MOISTURE CURVE
HAT01-003 0 - 3 FT. (TEST PIT)

FIGURE D.5.39
CAPILLARY MOISTURE CURVE
HAT01-003 3 - 4 FT. (TEST PIT)

FIGURE D.5.40
CAPILLARY MOISTURE CURVE
HAT01-004 2.5-6 FT. (TEST PIT)

FIGURE D.5.41
CAPILLARY MOISTURE CURVE
HAT01-004 6-7.5 FT. (TEST PIT)

STAGE NO.	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	SOIL DESCRIPTION
1	90.2	16.3	TAILINGS SAND
2	95.8	27.1	" "
3	98.4	25.3	" "

LOCATION NO.
TP-1 & TP-5

DEPTH (FT.)
COMBINED

SAMPLE TYPE
REMOLO

FIGURE D.5.42
SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION

DEVIATOR STRESS
 $\sigma_1 - \sigma_3$ (ksf)

PRINCIPAL STRESS RATIO
 σ_1 / σ_3

SHEAR STRESS (ksf)

NET PORE PRESSURE RATIO
 u_p (ksf)

FIGURE D.5.42 (CONT.)
SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION

Table D.5.1 Design parameters--in-situ tailings

Material description	Design parameters	Method of determination
Sand (SM):	<u>Hydraulic conductivity:</u>	
$\gamma_d = 107$ pcf $w = 9\%$ $\rho = 2.7$ $PI = NP$	Saturated $k = 1.2 \times 10^{-2}$ cm/s	Lab test
	<u>Compressibility:</u>	
	<u>Elastic parameters:</u>	
	$E = 2.6 \times 10^5$ psf	Average of SPT and piezocone correlations
	$\nu = 0.25$	Typical value from literature
	<u>Consolidation parameters:</u>	
	$C_c =$ (Use laboratory curves in Figures D.5.28 and D.5.29 directly) (HAT01-135 30'-32.5')	Lab tests
	$OCR = 3.0-3.5$	Lab tests and piezocone correlations
	$C_v = C_\alpha = 0$	
	<u>Shear strength:</u>	
	$\phi = \bar{\phi} = 31^\circ$	Average SPT and piezocone correlations
	$c = \bar{c} = 0$	
Sand-slime (SM, ML):	<u>Hydraulic conductivity:</u>	Lab tests
$\gamma_d = 87$ pcf $w = 17\%$ $\rho = 2.8$ $PI = NP$	Saturated $k = 2.6 \times 10^{-6}$ cm/s	Lab tests

Table D.5.1 Design parameters--in-situ tailings (continued)

Material description	Design parameters	Method of determination
Sand-slime (Concl.)	<u>Compressibility:</u>	
	<u>Elastic parameters:</u>	
	$E = 1.6 \times 10^5$ psf	Average of SPT and piezocone correlations From literature
	$\nu = 0.35$	
	<u>Consolidation parameters:</u>	
	$C_c = 0.12$	Lab test
	$C_r = 0.010$	Lab test
	OCR = 3.0-3.5	Lab tests and piezocone correlations
	$C_v =$ See load increment vs. C_v	
	$C_\alpha =$ and C_α data in Table D.5.6 for HAT01-136 3'-5.5'	
Slime (ML, CL): $\gamma_d = 75$ pcf $w = 37\%$ $p = 2.8$ $w_L = 37\%$ $w_p = 22\%$ $PI = 15\%$	<u>Shear strength:</u>	
	$\phi = \bar{\phi} = 29^\circ$	SPT correlation
	$c = \bar{c} = 0.$	
	<u>Hydraulic conductivity:</u>	
	Stress (psf)	k (cm/s)
	500	1.7×10^{-6}
	1000	1.5×10^{-7}
	2000	2.3×10^{-7}
	8000	7.6×10^{-8}
	<u>Compressibility:</u>	
	<u>Elastic parameters:</u>	
	$E = 7.0 \times 10^5$ psf	Piezocone correlation From literature
	$\nu = 0.5$ saturated	
	$E = 2.0 \times 10^5$ psf	Correlation with index properties From literature
	$\nu = 0.3$ unsaturated	

Table D.5.1 Design parameters--in-situ tailings (Concluded)

Material description	Design parameters	Method of determination																					
Slime (Concl.)	<u>Consolidation parameters:</u>																						
	$C_c = 0.19$	Lab tests																					
	$C_r = 0.019$	Lab tests																					
	$OCR = 3.0-3.5$	Lab tests and piezocone correlations																					
	<table> <tr> <th>Load (ksf)</th><th>C_v (in²/min)</th><th>C_α</th></tr> <tr> <td>1</td><td>0.032</td><td>0.0009</td></tr> <tr> <td>2</td><td>0.047</td><td>0.0019</td></tr> <tr> <td>4</td><td>0.045</td><td>0.0032</td></tr> <tr> <td>8</td><td>0.040</td><td>0.0059</td></tr> <tr> <td>16</td><td>0.050</td><td>0.0068</td></tr> <tr> <td>32</td><td>0.034</td><td>0.0090</td></tr> </table>	Load (ksf)	C_v (in ² /min)	C_α	1	0.032	0.0009	2	0.047	0.0019	4	0.045	0.0032	8	0.040	0.0059	16	0.050	0.0068	32	0.034	0.0090	Lab tests
Load (ksf)	C_v (in ² /min)	C_α																					
1	0.032	0.0009																					
2	0.047	0.0019																					
4	0.045	0.0032																					
8	0.040	0.0059																					
16	0.050	0.0068																					
32	0.034	0.0090																					
	<u>Shear strength:</u>																						
	$\bar{\phi} = 35^\circ$	Lab tests and literature																					
	$\bar{c} = 0$																						
	$\phi = 11^\circ$	unconsolidated																					
	$c = 400$ psf	unsaturated																					
		undrained																					
	$S_u = 1000$ psf	saturated																					
		undrained																					
		SPT and piezocone correlations																					

Table D.5.2 Design parameters--remolded tailings and windblown contaminated materials

Material description	Design parameters	Method of determination
Remolded tailings (SM) and windblown materials (SM/ML):	<u>Hydraulic conductivity:</u>	
Standard Proctor - Maximum density = 106 pcf Optimum moisture content = 16.6%	Saturated $k = 1 \times 10^{-5}$ cm/s	From literature
$\gamma_d = 95$ pcf (90% Proctor) $w = 14.6\%$ (-2% OMC) PI = NP	<u>Compressibility:</u> C_{cc} = not determined $C_{re} = 0.011$ Preconsol. pressure = 2 ksf C_v, C_α = not determined	From literature Assumed
	<u>Shear strength:</u>	
	$\phi = \bar{\phi} = 32^\circ$	Lab test
	$c = \bar{c} = 0$	

Table D.5.3 Physical characteristics (tailings)

Horing number	Depth (ft)	Percent Passing			Description of tailings	In-situ water content	In-situ saturation (%)	Liquid limit	Plastic limit	Plasticity index	USCS class	Activity	Liquidity index	Specific gravity	In-situ dry unit weight (pcf)
		No. 4	No. 200	.002 mm											
D-150	HA101-126	3-5.5	100	97	10	Slime	38.4	98.0	37	20	17	CL	1.70	1.08	81.7
		7-8.5					24.4								
		15-17.5	100	83	12	Slime			34	24	10	CL	0.83		52.1
		19-20.5	100	95		Slime	39.4								
		20.5-22					29.7								
		22.5-25	100	85	12	Slime	46.3	83.0	37	30	7	ML	0.58	2.33	67.4
		28-29.5					42.2								
	HA101-127	0-2.5	100	89	18	Slime	33.3	29.0				ML		3.241	43.0
		5.5-7					9.8								
		10-11.5					9.5								
		14.5-16	100	79		Slime	31.0		28	20	8	CL	1.38		
		16-18.5	100	88	12	Slime	71.7	100.0	36	24	12	CL-ML	1.00	2.819	58.5
		19-20.5	100	58		Sand-slime	19.3					ML			
		20.5-22					10.3								
		25-26.5					48.0								
	HA101-128	0-1.5	100	90	15	Slime	40.5					ML			
		3-4					67.8								50.3
		10.5-12					40.2								
	HA101-129	3-4					34.8								65.7
		7-8.5	100	87	8	Slime						ML			
		12.5-14					77.5								
	HA101-130	3-4					51.4								56.4
		4-5.5	100	61	9	Sand-slime	49.8								
		10-11.5					203.4								
		12-13.5					33.2								
	HA101-131	10-12.5	100	93	9	Slime	29.2	78.0	36	21	15	CL	1.67	0.55	2.727
		15.5-16.5					30.7								84
	HA101-132	1.5-4	100	92	12	Slime	29.0		35	20	15	CL	1.25	0.60	
		7-8.5	100	74		Slime	22.5								
		11.5-14	100	68		Sand-slime	16.2								
		20-21.5					19.8								
		24.5-26.5	100	74	11	Slime	40.9		25	19	6	CL-ML	0.55	3.65	

Table D.5.3 Physical characteristics (tailings) (Continued)

Boring number	Depth (ft)	Percent Passing			Description of tailings	In-situ water content	In-situ saturation (%)	Liquid limit	Plastic limit	Plasticity index	USCS class	Activity	Liquidity index	Specific gravity	In-situ dry unit weight (pcf)
		No. 4	No. 200	.002 mm											
HAT01-133	1.5-4	100	62	16	Sand-slime	35.6	93.0							2.682	84.2
	8.5-11	100	76	11	Slime									2.738	86.7
	14-15.5					21.3									
	15.5-17	100	60		Sand-slime	14.7									
	21.5-22.5					24.9									
	27-28.5	100	96	12	Slime	40.3		47	25	22	CL	1.83	0.70		
	35.5-37					29.3									
HAT01-134	1.5-3	100	60		Sand-slime	13.3									
	4.5-6	100	51		Sand-slime	8.1									
	9-11.5	100	65	13	Sand-slime	12.3	36.0				ML			2.726	87.9
	14.5-16					12.4									
	22-23.5	100	58		Sand-slime										
	28-29.5					21.7									
	35-37.5	100	61		Sand-slime	10.2									88.1
	42-43.5	100	54		Sand-slime	12.5									
HAT01-135	0-1.5					43.6									
	3-5.5	100	93		Slime	27.2	63.5				NL				80.2
	8.5-10	100	38		Sand-slime	9.0									
	13-14.5					6.0									
	20-22.5	100	89	12	Slime	35.5	83.0	33	21	12	CL	1.00	1.21		88.5
	30-32.5	100	36	9	Sand-slime	12.6	36.0				SM				85.8
	35.5-37					33.0									
HAT01-136	3-5.5	100	99	12	Slime	35.2	81.0	42	23	19	CL	1.58	0.64	2.723	77.8
	7-8.5					40.2									
	10-11.5					33.7								2.728	
	13-15.5	100	96	9	Slime	48.8	100.0	51	24	27	CH	3.00	0.92	2.731	74.9
	20-22.5	100	96	12	Slime	38.8		40	23	17	CL	1.42	0.93	2.785	81.5
HAT01-137	1.5-3	100	52		Sand-slime	11.0									
	6.5-9	100	98	14	Slime			43	21	22	CL	1.57			
	10.5-12	100	62		Sand-slime	16.9									
	16.5-18	100	97	14	Slime	41.4		43	21	22	CL	1.57	0.93		

Table D.5.3 Physical characteristics (tailings) (Concluded)

Boring number	Depth (ft)	Percent Passing			Description of tailings	In-situ water content	In-situ saturation (%)	Liquid limit	Plastic limit	Plasticity index	USCS class	Activity	Liquidity index	Specific gravity	In-situ dry unit weight (pcf)
		No. 4	No. 200	.002 mm											
HAT01-138	5.5-7					27.0									
	10-11.5	100	91	14	Silts	39.8		38	21	17	CL	1.21	1.11	2.778	
	12-14.5					35.9									
	14.5-16					39.6									
	20.5-22	100	70		Silts	20.1		25	19	6	CL-ML		0.18		
	23.5-26					30.4									
	29-30.5					38.1									
	32-33.5					37.9									
HAT01-139	35-37.5	100	88	12	Silts	35.4	54.0	37	23	14	CL	1.17	0.89	2.756	85.3
	4.5-6	100	85		Silts	20.4		28	20	8	CL		0.05		
	7.5-9					18.5									
	10.5-12	100	65		Sand-silts	17.1					ML				
	13.5-15	100	90	13	Silts	32.1					CL				
	19.5-22	100	62	18	Sand-silts	9.7					ML				
	25-26.9					33.7									
HAT01-181	0-1.5	100	66		Sand-silts	14.1									
	4.5-7	100	67		Sand-silts	14.2	39.8				ML				85.8
	8.5-10	100	52		Sand-silts	12.6									
	14.5-17	100	53		Sand-silts	9.8	30.5				ML				90.2
	20-21.5	100	29		Sand	8.6					SM				
	24.5-26	100	30	10	Sand	8.5					SM			2.703	
	27.5-29					25.7									
AVERAGES:															
	Sand (N-2)	100	30	10		8.6								2.703	
	Sand-silts (N-21)	100	58	13		16.1	47.1							2.760	87.0
	Silts (N-27)	100	89	12		37.0	77.0	37	22	15	CL	1.37	1.07	2.757	75.1

Table D.5.4 Hydraulic conductivity test results (tailings)
(undisturbed samples)

Boring number	Sample interval (ft)	USCS class	Tailings type	Dry unit weight (pcf)	Water content (percent)		Saturation (percent)		Hydraulic conductivity (cm/s)	Type of test
					Initial	Final	Initial	Final		
HAT01-126	15-17.5	CL	Silts	52.1	84.7	81.4	100	100	1.8×10^{-6}	T
	22.5-25	ML	Silts	90.0	21.9	31.5	66.4	99.5	6.1×10^{-7}	T
HAT01-127	16-18.5	CL-ML	Silts	58.5	71.7	71.9	100	100	4.2×10^{-6}	C
HAT01-133	1.5-4	---	Sand	84.2	35.6	33.6	93	87.8	1.18×10^{-2}	C
HAT01-134	9-11.5	ML	Sand- silts	87.8	12.3	29.9	36	87	2.6×10^{-6}	C
HAT01-136	3-5.5	CL	Silts	75.3	49.9	42.8	100	100	1.3×10^{-7}	T

T = Triaxial permeability test.

C = Constant head test.

Table D.5.5 Hydraulic conductivity - indirectly determined from consolidation test results

Sample number	Depth interval (ft)	USCS class	Load increment (psf)	Calculated hydraulic conductivity ^a (cm/s)
HAT01-126	3-5.5	CL	250	2.1×10^{-6}
			1000	6.9×10^{-8}
			2000	4.0×10^{-7}
			4000	1.3×10^{-7}
			8000	9.9×10^{-8}
			16000	4.7×10^{-8}
HAT01-126	22.5-25	ML	250	4.2×10^{-9}
			500	6.2×10^{-8}
			1000	2.9×10^{-8}
			2000	8.0×10^{-8}
			4000	1.3×10^{-7}
			8000	1.6×10^{-7}
HAT01-131	10-12.5	CL	250	1.9×10^{-7}
			500	1.6×10^{-7}
			1000	1.6×10^{-7}
			2000	8.6×10^{-8}
			4000	6.6×10^{-8}
			8000	4.9×10^{-8}
			16000	1.8×10^{-8}
			32000	1.6×10^{-8}
HAT01-136	3-5.5	CL	250	1.3×10^{-7}
			500	3.1×10^{-7}
			1000	7.5×10^{-8}
			2000	4.8×10^{-8}
			4000	1.6×10^{-8}
			8000	3.1×10^{-8}
			16000	1.3×10^{-8}
			32000	1.1×10^{-8}
HAT01-136	13-15	CL	500	3.8×10^{-7}
			1000	4.2×10^{-7}
			2000	6.7×10^{-7}
			4000	3.0×10^{-7}
			8000	7.1×10^{-8}
			16000	9.4×10^{-8}
			32000	3.8×10^{-8}

Table D.5.5 Hydraulic conductivity - indirectly determined from consolidation test results (Concluded)

Sample number	Depth interval (ft)	USCS class	Load increment (psf)	Calculated hydraulic conductivity ^a (cm/s)
HAT01-138	35-37.5	CL	125	1.9×10^{-7}
			250	1.4×10^{-7}
			500	2.2×10^{-7}
			1000	7.6×10^{-8}
			2000	9.1×10^{-8}
			4000	1.0×10^{-7}
			8000	4.5×10^{-8}
			16000	4.6×10^{-8}
			32000	3.2×10^{-8}

^aRef. Lambe and Whitman, 1969.

Table D.5.6 Consolidation test results (tailings)
(undisturbed samples)

Boring number ^a	Sample interval (ft)	USCS class	Tailings type	Preconsolidation pressure (KSF)	In-situ vertical stress (KSF)	Over-consolidation ratio	Initial void ratio (e_0)	Load increment (KSF)	Coefficient of consolidation (in^2/min)	Rate of secondary compression (C_α) ^b
HA101-126	3-5.5	CL	S1lime	1.3	0.3	4.3	1.052	0.25	0.021	0.0010
								1.00	0.019	0.0013
								2.00	0.087	0.0028
								4.00	0.042	0.0040
								8.00	0.040	0.0044
								16.00	0.036	0.0060
HA101-126	22.5-25	MH	S1lime	4.8	2.3	2.1	1.531	0.25	0.002	0
								0.50	0.019	0.0004
								1.00	0.010	0.0004
								2.00	0.031	0.0016
								4.00	0.054	0.0026
								8.00	0.060	0.0055
HA101-131	10-12.5	CL	S1lime	4.4	1.1	4.0	1.017	0.25	0.090	0.0006
								0.50	0.050	0.0002
								1.00	0.075	0.0009
								2.00	0.039	0.0020
								4.00	0.038	0.0038
								8.00	0.037	0.0060
HA101-131	10-12.5	CL	S1lime	4.4	1.1	4.0	1.017	16.00	0.025	0.0120
								32.00	0.032	0.0080
HA101-133 ^c	8.5-11	ML	S1lime	3.3	1.0	3.3	1.14	-----Not calculated-----		
HA101-136	3-5.5	(SM/ML) (Sand-slime)		2.0	0.3	6.7	1.175	0.25	0.021	0.0015
								0.50	0.027	0.0024
								1.00	0.020	0.0044
								2.00	0.013	0.0105
								4.00	0.012	0.0056
								8.00	0.016	0.0085
HA101-136	3-5.5	(SM/ML) (Sand-slime)		2.0	0.3	6.7	1.175	16.00	0.015	0.0085
								32.00	0.022	0.0055

Table D.5.6 Consolidation test results (tailings)
(undisturbed samples) (Concluded)

Boring number ^a	Sample interval (ft)	USCS class	Tailings type	Preconsolidation pressure (ksf)	In-situ vertical stress (ksf)	Over-consolidation ratio	Initial void ratio (e_0)	Load increment (ksf)	Coefficient of consolidation (in^2/min)	Rate of secondary compression (C_α) ^b
HAT01-136 ^c	13-15	CH	Slime	2.8	1.0	2.8	1.268	0.50	0.052	0.0011
								1.00	0.062	0.0014
								2.00	0.150	0.0052
								4.00	0.072	0.0010
								8.00	0.024	0.0090
								16.00	0.059	0.0095
								32.00	0.042	0.0100
HAT01-139 ^c	35-37.5	CL	Slime	5.6	3.8	1.5	1.013	0.125	0.021	0.0009
								0.25	0.047	0.0010
								0.50	0.084	0.0011
								1.00	0.024	0.0011
								2.00	0.032	0.0011
								4.00	0.045	0.0025
								8.00	0.021	0.0075
								16.00	0.033	0.0020
								32.00	0.036	0.0100

^aTested submerged throughout test unless otherwise noted.

^bThe coefficient of secondary compression referenced to stress can be calculated by multiplying the values referenced to strain by $(1 + e_0)$.

^cTested at in-situ moisture content to 1 ksf then submerged.

^dNot available.

^eTested at in-situ moisture content throughout test.

() indicates that classification was determined based on micrologs (visual classifications) of Shelby tubes at time of testing.

Table D.5.7 Physical properties and mechanical behavior (remolded tailings and windblown contaminated materials)

Test pit number	Depth (ft)	Percent passing			Description of tailings	In-situ water content	Liquid limit	Plastic limit	Plasticity Index	USCS class	Activity	Specific gravity	Moisture-density results			Relative Density	
													Max dry density (pcf)	Optimum moisture (%)	Sat. at opt. moist. and max. den. (pcf)		
		No. 4	No. 200	.002 mm												Min density (pcf)	Max density (pcf)
HAT01-001	0-1.5	100	47	12	Sand-slime	6.4				SM		2.681	99.8	17.7	70.1	77.5	97.5
	0-1.5-R	100	51	22	Sand-slime	8.1						2.740	100.6	16.5	70.1	77.5	97.5
HAT01-002	0-0.5	100	63	11	Sand-slime (windblown)	7.5				ML		2.708	116.7	12.2	74.6		
HAT01-003	0-3	100	100	14	Slime		54	22	32	CH	2.29	2.794	91.9	26.4	82.1		
	1-2-R	100	100	21	Slime	46.8						2.790	95.1	25.9			
	3-4-R	100	60	10	Sand-slime	17.7						2.850	93.8	23.6			
	3-4	100	75	11	Slime	18.8				ML		2.773	96.3	23.3	81.0		
HAT01-004	2.5-6	100	96	14	Slime	19.1				ML		3.400	58.0	59.8	76.4		
	2.5-6-R	93	63	17	Sand-slime	7.6						2.780	124.6	10.0			
	6-7.5	100	61	10	Sand-slime	16.2				ML		2.749	103.5	20.1	84.0		
	6-7.5-R	100	56	15	Sand-slime	19.9						2.880	99.1	21.8			
HAT01-005	3-4.5	100	42	9	Sand-slime	12.1				SM		2.716	106.3	16.7	76.2	73.0	101.5
	3-4.5-R	100	33.5	17	Sand-slime	9.3						2.790	105.2	11.1			

R Indicates tests performed by a second laboratory on bulk samples collected from the same depths at the same pit and time.

D.6 BORROW MATERIAL CHARACTERISTICS

D.6.1 DESIGN PARAMETERS

The design parameters for both rock and radon barrier materials are presented in Table D.6.1.

The rock material is assumed to have an infinitely high permeability and to be relatively incompressible and thus these properties are not given in the table.

D.6.2 EROSION BARRIER MATERIALS

To date two rock materials have been tested for suitability for use as erosion protection. The tests used are:

1. Los Angeles abrasion (ASTM C535).
2. Sulfate soundness (ASTM C88).
3. Slake durability (International Society for Rock Mechanics).
4. Specific gravity and absorption of aggregate (ASTM C127).

A bulk sample of siltstone was obtained from a location immediately adjacent to the Mexican Hat tailings pile (see Figure 3.10 in the text). The material had a total loss of 72.3 percent in the Los Angeles abrasion test and a slake durability of 88.3 percent. This indicates that this material is very susceptible to weathering and was thus judged unsuitable for use. Therefore this material will not be discussed further in this report.

A bulk sample of limestone was obtained from the borrow area shown on Figure 3.10 in the text (Alhambra Rock borrow area). Borrow area B contains the same type of rock and is described in the final design. This material had a total loss of 53.8 percent in the Los Angeles abrasion test, a percent loss of 32.2 in the sulfate soundness test, a slake durability of 97.9 percent, and an absorption of 3.6 percent. This material is therefore judged marginally suitable for use in erosion protection. The engineering properties of the crushed limestone are discussed below.

Long-term durability of rock borrow materials

The long-term durability of the limestone rock is expected to be moderate to poor based on criteria established in the Technical Approach Document (DOE, 1985c). More testing must be done on the limestone before a final determination can be made of its suitability for use as erosion protection materials. Preliminary analyses indicate that the limestone rock may have to be oversized by 20 percent or more.

Classification

The dark gray limestone can be described as a dense, relatively hard, fine-grained nonfossiliferous limestone. It is found at or

within a few feet of the surface throughout the area indicated on Figure 3.10 in the text. The limestone bed varies in thickness from about five feet to over 30 feet, is generally only slightly weathered in outcrops, and is therefore likely to be relatively unweathered in-situ.

Specific gravity and dry unit weight

The crushed limestone had an apparent specific gravity of 2.64 and a bulk specific gravity of 2.41.

For design calculations it is recommended that a unit weight of 110 pcf be assumed for the compacted crushed limestone.

Hydraulic conductivity

The permeability of the crushed limestone is infinitely high in relation to the other embankment materials and therefore no specific value will be recommended for use in design.

Compressibility

For all practical purposes, the compacted crushed limestone is essentially incompressible and it is therefore recommended that all compressibility parameters be assumed to be equal to zero in the analysis of settlements.

Shear strength

Using the empirical relations for shear strength as outlined in Hoek and Bray (1981), it was determined that a good estimate for the shear strength of compacted crushed limestone is $\phi=42^\circ$, $c=0$. These values can be used in all analyses, both static and dynamic.

D.6.3 RADON BARRIER MATERIALS

The T2 radon cover borrow area is just north of the town of Mexican Hat (see Figure 3.10 in the text). The borrow area lies in a small narrow erosional valley in alluvial and colluvial deposits. The following paragraphs describe the engineering properties of the soils found in the borrow area. This borrow area would be used only if borrow area A does not contain a sufficient volume of material. The final design contains information on borrow area A.

Classification

These soils classified as GP, GM, SP, SP-SM, and SM by visual classification and laboratory determinations (see Table D.6.2 and test pit logs in Addendum D4). Most of these samples had a significant

gravel fraction. The composite sample prepared for use in the testing program is described as a silty, gravelly sand and classified as an SM in the USCS. A total of four samples as well as a composite sample had grain size analyses performed (see Table D.6.2 and Addendum D5).

Moisture content and specific gravity

All samples had in-situ moisture contents determined and the results are presented in Table D.6.2. The average moisture content was found to be 3.6 percent moisture by weight.

Two combined samples, one with seven percent bentonite and one with 15 percent bentonite, both had specific gravities of 2.69 (see Table D.6.2).

Hydraulic conductivity and capillary moisture relation

A total of three permeability (hydraulic conductivity) tests were performed on samples of the composite material recompacted to approximately 90 percent of standard Proctor maximum density (see Table D.6.3 for the results of moisture density tests). The permeability tests indicate that a decrease in permeability of about four orders of magnitude can be attained by the addition of seven percent bentonite (see Table D.6.3). Any further addition of bentonite does not seem to reduce the hydraulic conductivity below the 10^{-9} cm/s attained at seven percent bentonite.

Tests indicate that the unamended soil will have a hydraulic conductivity of 3×10^{-5} cm/s and that the soil, when amended with seven percent or more by dry weight of bentonite, will have a hydraulic conductivity of 1×10^{-9} cm/s. These values are therefore recommended for use in design.

Capillary moisture tests (ASTM D2325 and D3152) were also performed and the results are summarized in Table D.6.4.

Cover erosion potential

The composite soil sample with seven percent bentonite added was tested for erodibility via the crumb test (STP623), the double hydrometer test (ASTM D4221), and the pinhole test (STP623). The crumb test and the pinhole test both resulted in nondispersive ratings (ND and ND2, respectively) and the double hydrometer showed the soil to have 5.7 percent dispersion. These ratings all indicate a soil with a relatively low potential for erosion.

Compressibility

A consolidation test was performed on a composite soil sample with 15 percent bentonite added and recompacted to approximately 89 percent

of standard Proctor maximum density. The resulting laboratory consolidation curve is reproduced in Figure D.6.1.

The values of C_{cc} and C_{rc} obtained from this curve are 0.140 and 0.016, respectively. Typical values for the expected compression of recompacted soils presented by the U.S. Navy (1982) would result in an estimated compression index of about 0.011. The consolidation curve indicates a preconsolidation pressure of about two ksf and thus, since the pressures in the cover are well below this, the rebound compression index will be appropriate for estimating settlements. The value reported as typical is in reasonable agreement with C_{rc} as determined in the consolidation test. Therefore, it is recommended that the test results C_{cc} and C_{rc} be used in design with a preconsolidation pressure due to compaction of two ksf.

Strength

The strength of the radon barrier soil was tested by both Q and R triaxial shear tests. Composite soil samples were mixed with 15 percent bentonite and remolded into test specimens at a density of about 89 percent of standard Proctor maximum density. The results of these tests are summarized in Figures D.6.2 and D.6.3.

These tests indicate that in unsaturated-undrained shear (Q test) a ϕ of about 9° and a C of about 0.6 ksf is mobilized by the soil at failure. These results indicate that the saturated undrained strength (S_u) of this soil is a minimum of 600 psf and that the radius of the Mohr's circle for the highest confining stress, or about 2000 psf, is probably very close to the actual S_u .

For drained conditions (R test), the tests indicate that a $\phi = 26$ and a C = 0.0 will be mobilized at failure. Data presented by the U.S. Navy (1982) indicate that ϕ should be about 31° for a recompacted SC and about 34° for a GC. This indicates that the friction angle tends to increase with the addition of the gravels. The soil being used for radon cover will be about 30 percent gravel but the gravel fraction was removed prior to triaxial testing. Also the cover will have a maximum of seven percent bentonite instead of 15 percent. Thus it is expected that these factors will increase the undrained shear strength of the soil.

Therefore, it is recommended that: $\phi=31^\circ$, C=0 be used for drained conditions; $\phi=9^\circ$, C=600 psf be used for unsaturated-undrained conditions; and for saturated undrained conditions $S_u=2000$ psf.

D.6.4 DATA NEEDS

It is recommended that the average tensile strain at cracking be determined for the clayey material used at the surface of the radon cover.

The bedrock configuration can then be used in conjunction with the tensile strain data to check the proposed pile design for safety with regard to cover cracking.

LEGEND						
SAMPLE	HAT 01	DEPTH INTERVAL (FT.)	DRY DENSITY PCF LBS./CU. FT.		MOISTURE CONTENT (%)	
			INITIAL	FINAL	INITIAL	FINAL
15% BENTONITE		COMPOSITE *	103.5	114.3	15.3	19.7

* COMPOSED OF THE FOLLOWING SAMPLES: HAT 01-501 (1'-2' & 7'-9'), HAT 01-502 (1'-2'), HAT 01-507 (5'-6'), HAT 01-508 (0.5'-1.5') AND HAT 01-510 (0.5'-1.5')

FIGURE D.6.1
CONSOLIDATION TEST RESULTS - RADON BORROW MATERIAL

TEST NUMBER	1	2	3	4
LOCATION SAMPLE NO. DEPTH FT.	Composite with 15% Bentonite			
WEIGHT-INCH	3.962	3.949	3.964	
DIAMETER-INCH	1.880	1.882	1.873	
WATER CONTENT-%	15.0	15.4	15.2	
DRY DENSITY-pcf	103.7	103.7	104.1	
σ_1 σ_3 -ksf	2.24	2.52	4.13	
σ_3 -ksf	1.01	3.02	6.05	
σ_1 -ksf	9.25	5.54	10.18	

TYPE OF SPECIMEN Remolded

SOIL DESCRIPTION Composite* with
15% Bentonite

TYPE OF TEST Unsaturated,
Unconsolidated,
Undrained

TAN ϕ _____

ϕ _____

COHESION-ksf _____

* COMPOSED OF THE FOLLOWING SAMPLES: HAT 01-501 (1'-2' & 7'-9'),
HAT 01-502 (1'-2'), HAT 01-507 (5'-6'), HAT 01-508 (0.5'-1.5') AND
HAT 01-510 (0.5'-1.5')

FIGURE D.6.2
TRIAXIAL SHEAR TEST RESULTS

STAGE NO.	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	SOIL DESCRIPTION
1	103.8	15.4	CLAYEY SAND
2	107.2	24.4	WITH 15%
3	119.2	18.6	BENTONITE

LOCATION NO.
COMPOSITE *

DEPTH (FT.)
15% BENT.

* COMPOSED OF THE FOLLOWING SAMPLES: HAT01-501 (1'-2' & 7'-9'), HAT01-502 (1'-2'), HAT01-507 (5'-6'), HAT01-508 (0.5'-1.5') AND HAT01-510 (0.5'-1.5')

FIGURE D.6.3
SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION

SHEAR STRESS (ksf)

FIGURE D.6.3 (CONT.)
SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION

Table D.6.1 Design parameters--borrow material

Material description	Design parameters	Method of determination
<u>Rock armor</u>	<u>Shear strength:</u>	
Crushed limestone $\gamma_d = 110$ pcf $w = 3.6\%$ $\rho = 2.64$	$\bar{\phi} = 42^\circ$ $\bar{c} = 0.0$	Empirical relation
<u>Radon barrier</u>	<u>Hydraulic conductivity (saturated):</u>	
Gravelly sand (SM)	3×10^{-5} (no bentonite) 1×10^{-9} (7% bentonite added)	Lab tests
Standard Proctor (7% bentonite added) Maximum dry density = 121.9 pcf Optimum moisture content = 10.7% $\gamma_d = 116$ pcf (95% Proctor) $w = 12.7\%$ (+2% OMC) PI = NP	<u>Dispersivity (7% bentonite added):</u> Non-dispersive <u>Compressibility (parameters given are for soil with and without bentonite):</u>	Lab tests
	<u>Elastic parameters</u> $E = 1 \times 10^6$ psf $\nu = 0.3$	
Standard Proctor (no bentonite) Maximum dry density = 125.7 pcf Optimum moisture content = 8.5% $\gamma_d = 119.4$ (95% Proctor) $w = 6.5\%$ (2% OMC) PI = NP	<u>Consolidation parameters:</u> $C_{cc} = 0.140$ $C_{rc} = 0.016$ $C_v, C_\alpha =$ not calculated Preconsolidation pressure (due to compaction) = 2000 psf	Lab tests
	<u>Shear strength (parameters given are for soils both with and without bentonite)</u> $\bar{\phi} = 31^\circ$ $\bar{c} = 0.0$ $\phi = 9^\circ$ $c = 600$ psf $S_u = 2000$ psf	From literature unconsolidated unsaturated undrained Lab tests

Table D.6.2 Physical characteristics - radon barrier

Test pit number	Depth (ft)	Percent passing			In-situ water content	Atterburg limits			USCS class	Specific gravity
		1 1/2"	No. 4	No. 200		Liquid limit	Plastic limit	Plasticity index		
HAT01-501	1-2	100	60	12	3.3	--	--	NP	SP-SM	
	7-9	98	61	8	1.4				(SP-SM)	
HAT01-502	1-2(A)				2.2					
	1-2(B)				3.4					
	7-8				1.6					
HAT01-503	0-1				3.5					
	2-3				1.8					
HAT01-504	0-1				2.8					
HAT01-505	1-2				3.1					
	6-7				1.5					
HAT01-506	1-2				2.4					
HAT01-507	0.5-1.5	100	96	24	3.6				(SM)	
	5-6	98	60	10	1.2				(SP-SM)	
	8-9(A)				17.4					
	8-9(B)				1.8					
HAT01-508	0.5-1.5				4.1					
	8-9				2.0					
HAT01-509	0.5-1.5				3.8					
	6-7				6.8					
HAT01-510	0.5-1.5				3.3					
<u>Combined sample^a</u>										
(-501	1'-2'									
-501	7'-9'									
-502	1'-2'(A&B)									
-507	5'-6'									
-510	0.5-1.5)									
0% bentonite		91	68	21	6	2.7			(SM)	
7% bentonite										2.69
15% bentonite						105	14	91	SC	2.69

() USCS classification based only on grain size.

^aSamples were combined in an effort to simulate mixing during handling of borrow soil then mixed thoroughly with varying percentages (by weight) of Wyoming processed bentonite.

Table D.6.3 Mechanical properties of combined, remolded, compacted soil - radon barrier

Sample description ^a	Moisture - density results			Permeability test results							Total pres. head (ft)	Perm. (cm/s)
	Maximum dry density (pcf)	Optimum moisture (%)	Saturation at opt. moist. and max. dry density (%)	Moisture (%)		Dry density (pcf)		Sat. (%)				
				Initial	Final	Initial	Final	Initial	Final			
										% Proctor		
0% bentonite added	125.7	8.5		13.4	17.0	114.2/92%	114.2	76.0	96.4	4.0	3.3 x 10 ⁻⁵	
7% bentonite added	121.9	10.7	76.4	16.0	17.7	109.9/90%	114.2	80.7	99.9	10.2	4.4 x 10 ⁻⁹	
15% bentonite added	116.5	12.0	73.1	14.7	20.7	104.2/89%	108.3	64.1	100.0	23.1	1.2 x 10 ⁻⁹	

^aSamples were combined in an effort to simulate mixing during handling of borrow soil then mixed thoroughly with varying percentages (by weight) of Wyoming processed bentonite.

Table D.6.4 Capillary moisture test results, composite sample - radon barrier

Sample description ^a	Average remolded moisture content (%)	Average remolded dry density (pcf)	Percent compaction (%)	Water retention (%)									
				Pressure (bars)									
				0.1	0.3	0.5	0.7	1.0	2.0	4.0	7.0	10.0	15.0
0% bentonite added	12.3	114.3	91	13.4	7.8	6.9	6.6	6.4	6.0	5.6	3.7	3.0	2.5
7% bentonite added	14.9	110.5	91	28.9	21.8	18.1	17.7	17.2	16.3	15.6	12.6	11.6	10.9
15% bentonite added	15.1	103.2	89	56.2	48.4	35.5	34.0	32.6	31.1	29.5	24.5	23.0	21.0

^aSamples were combined in an effort to simulate mixing during handling of borrow soil, then mixed thoroughly with varying percentages (by weight) of Wyoming processed bentonite.

D.7 GROUNDWATER HYDROLOGY

D.7.1 MEXICAN HAT TAILINGS

D.7.1.1 General

Hydrogeologic data including borehole logs, well completions records, groundwater levels, aquifer test data, and water-quality analyses were collected at the Mexican Hat tailings site by the U.S. Department of Energy (DOE) during fall 1984 to fall 1985. All field and laboratory procedures and calculations were performed according to the DOE's Standard Operating Procedures Manual (DOE, 1985a).

Seven two-inch diameter and three four-inch diameter polyvinylchloride (PVC) monitor wells were installed at the site, ranging from 12 feet to 200 feet in depth. One well (907) was constructed using inflatable packers, set at depths of 107 feet and 110 feet below land surface (bls), and a 0.75-inch riser pipe. Nine additional boreholes were drilled to depths of approximately three to 235 feet below the land surface but were unsuitable for the installation of monitor wells. Lithologic samples and geophysical logs were obtained from five of these additional boreholes prior to abandonment. Each of these five abandoned boreholes was cemented from its total depth to slightly above the land surface. The remaining four boreholes were drilled three feet into alluvium as exploration holes; the alluvium was unsaturated and the boreholes were abandoned. The locations of all monitor wells, abandoned boreholes, and seeps used in the investigation are shown on Figure D.7.1.

Following installation and development of the wells, slug withdrawal tests were performed to estimate the hydraulic characteristics of the saturated rocks near the completed intervals of the wells. The completed wells were surveyed and groundwater levels measured to determine hydraulic gradients and groundwater flow directions. Information concerning the Mexican Hat monitor wells is presented in Table D.7.1, and typical construction details are shown in Figure D.7.2.

Stratigraphy

Investigations conducted determined the presence of groundwater below the area of the Mexican Hat tailings site. This groundwater occurs in three distinct subsurface environments, the upper, middle, and lower hydrostratigraphic units.

The Mexican Hat tailings site is situated on outcrops of the Halgaito Shale Member of the Cutler Group and underlying this rock unit is the Honaker Trail Formation (more than 300 feet of alternating siltstone and sandstone). At the site, these strata are inclined six to eight degrees east-southeast, dipping toward the axis of the Mexican Hat syncline.

The Halgaito Shale Member is Permian in age and is comprised of an erratic sequence of thinly to thickly bedded, dark reddish-brown siltstones, silty sandstones, and silty shales with thin, discontinuous beds of gray, silty limestone. Minor amounts of thin, discontinuous clay layers were observed by DOE in several of the boreholes. Joints are generally closely spaced (approximately one inch or less between faces), strike east, and have approximately vertical dips.

The Halgaito Shale Member outcrops in the area of the site where soil or alluvial deposits do not occur. The Halgaito Shale is reported to be up to 400 feet thick in the region (Witkind and Thaden, 1963) and 50 to 100 feet thick in the area of the site (FBDU, 1981). The contact between the Halgaito Shale and the underlying Honaker Trail Formation is gradational and indistinct.

The Honaker Trail Formation of the Hermosa Group is Pennsylvanian in age. It is visible in an outcrop approximately one mile west of the site. The Honaker Trail Formation is reported as a varied sequence of gray to white, fossiliferous limestone interbedded with gray, massive sandstone, thinly-bedded red sandstone, minor red shale, and mudstone (Witkind and Thaden, 1963); and reported to be not less than 300 feet thick in the area of the site (FBDU, 1981).

On the basis of the reported stratigraphy in the area of the site, the shallow wells are completed within the Halgaito Shale Member (upper hydrostratigraphic unit), and the deeper wells are completed within the upper Honaker Trail formation (middle hydrostratigraphic unit). The deepest boreholes, which encountered hydrocarbons and hydrogen sulfide gas, are interpreted as bottoming lower within the Honaker Trail Formation (lower hydrostratigraphic unit). The locations of hydrogeologic cross sections are shown in Figure D.7.3; the cross sections are presented in Figures D.7.4, D.7.5, and D.7.6. The surface topography shown on the cross sections was developed from the USGS 15-minute topographic map of the Mexican Hat Quadrangle. Subsurface lithologies were drawn from borehole logs and visual inspection of recovered core samples.

D.7.1.2 Unsaturated zone hydraulics

Following mill closure, the tailings probably drained relatively slowly and continue to do so as evidenced by the fine-grained nature of the tailings, perched water below the tailings, and the high degree of saturation reported for samples taken from geotechnical borings within the tailings (DOE, 1986). Five tensiometers and six lysimeters were installed within the lower tailings pile in order to further characterize unsaturated flow conditions. However, due to

improper installation, equipment malfunction, or unknown causes, no pore water samples were obtained from the lysimeters, and the tensionmeter data are regarded as suspect.

As mentioned in Section D.9, Meteorology, pan evaporation at the tailings site is approximately 14 times greater than precipitation (pan evaporation is 84.41 inches annually with an average precipitation of 6.09 inches). This large evaporation rate would tend to minimize infiltration and contaminant migration. The large ratio of evaporation to precipitation and the length of time since tailings drainage began would indicate that the tailings presently contribute relatively minor amounts of contaminants to the upper hydrostratigraphic unit.

D.7.1.3 Saturated zone hydraulics

Very little published quantitative data are available concerning hydraulic characteristics for either the Halgaito Shale Member or the Honaker Trail Formation in the vicinity of the Mexican Hat tailings site. Groundwater was presumed absent beneath the site by Snelling (1971), Fuhrman and Hintze (1976), and FBOU (1981). A regional hydrogeologic study of the Navajo and Hopi Reservations (Cooley et al., 1969) reported no quantitative data on conditions within the Mexican Hat area due to a lack of producing water wells in the area. Woodward-Clyde Consultants (WCC, 1982) prepared a comprehensive report on the hydrogeologic characteristics for an area north of the Mexican Hat tailings site. Included in this report are analyses of drill stem tests conducted within the upper Honaker Trail Formation. A value for hydraulic conductivity of 2.1×10^{-6} feet per day was calculated for an interval within the upper Honaker Trail at a depth of approximately 1100 feet b1s (WCC, 1982). This extremely low value illustrates the confining nature of parts of the Honaker Trail Formation. Such a confined zone was encountered by the deeper boreholes drilled at the Mexican Hat site.

No published quantitative data are available for the Halgaito Shale Member. Cooley et al. (1969) stated that "the rock outcrops in the area have low permeability and carry little or no water." The Honaker Trail Formation and Halgaito Shale are not considered potentially potable aquifers with significant water yielding capabilities in the area of the tailings site. This is evident from hydrogeologic exploration by previous investigators and the absence of producing water wells in the area of the tailings site (Denny, 1985; Little, 1985; May, 1985).

The hydraulic characteristics of the three hydrostratigraphic units are discussed below. A summary of the hydraulic characteristics of the upper and middle hydrostratigraphic units is presented in Table D.7.3.

Upper hydrostratigraphic unit

Groundwater in the upper hydrostratigraphic unit occurs under unconfined conditions within the Halgaito Shale Member. This groundwater system occurs over a limited area and has formed mainly as a result of seepage from the tailings piles during milling operations.

Relatively shallow groundwater underlies at least a part of the area of the tailings site within the upper hydrostratigraphic unit. The upper hydrostratigraphic unit was penetrated by wells 905, 906, 910, 911, 912, 934, and possibly 935. Well 935 is affected by grout contamination due to an unsatisfactory seal above the well screen and is therefore omitted from the following analyses.

A potentiometric contour map of the upper hydrostratigraphic unit is presented in Figure D.7.7. This contour map was developed using water level data and the surveyed elevations of the monitor wells. The calculated average hydraulic gradient within this unit is approximately 0.04. Calculated average linear velocities of groundwater in the upper hydrostratigraphic unit, using a conservative estimate of porosity equal to 0.08 and the calculated hydraulic conductivity values presented in Table D.7.2, range between 0.01 foot per day (ft/day) and 0.38 ft/day with a mean value of 0.04 ft/day.

Locally within the upper hydrostratigraphic unit, groundwater flow directions are altered by the presence of a groundwater divide. This divide is centered below the tailings piles and is a relic of drainage from the tailings during uranium extraction. This divide will dissipate with time. No discharges of groundwater have been identified that could have originated from the existing tailings site.

Inspection of the groundwater levels within wells 911 and 912 illustrates the downward vertical hydraulic gradient and the transient nature of groundwater occurrence in the upper hydrostratigraphic unit. The two wells are approximately 120 feet apart, and well 912 is screened approximately 20 feet above the screened interval in well 911. The water level in well 912 was nearly 20 feet higher than the water level in well 911 as measured in mid April, 1985 establishing a downward hydraulic gradient of approximately one. A unit hydraulic gradient downward is to be expected between a perched water table and the underlying true water table. The perched water table intercepted by well 912 has remained relatively stable while the water level within well 911 dropped nearly six feet during April, 1985. By June, 1985, well 911 was dry through the total depth of the well and remained dry through late October, 1985. As measured in April, June, and July, 1985, the water level within well 912 fluctuated less than one foot; in the same period, the water level within well 911 dropped more than 10 feet.

The sporadic occurrence of groundwater in the upper hydrostratigraphic unit is further indicated by water level measurements in wells 906 and 910. Well 906 is approximately 12 feet deep and screened from approximately five feet to 10 feet bls. This well intercepted a shallow, perched water table approximately 7.5 feet bls in April, 1985. This perched water table rose 0.01 foot by June, 1985, and well 906 was dry in July, 1985. Well 910 intercepted a water table at approximately 168 feet bls; the relatively great depth to this water table is the result of well 910 being located topographically and stratigraphically higher than wells 906, 911, and 912. The position of the water table within well 910 has remained relatively constant at approximately 168 feet bls over the period from April, 1985, through July, 1985.

The results of a slug test performed within well 912 and a series of packer tests within well 911 indicate varying values of hydraulic conductivity within the shallow, perched groundwater system (Table D.7.2).

The slug test data from well 912 were analyzed by the Bouwer-Rice method. The Hvorslev method of analysis (Freeze and Cherry, 1979) was also attempted. This method utilizes a graphical solution to calculate values of hydraulic conductivity. The plotted data should comprise a straight line; however, the plotted slug test data from well 912 form a segmented curve and the results of Hvorslev's method were judged to be invalid. All calculations and parameter values are on file in the UMTRA Project Office, Albuquerque, New Mexico.

Assumptions inherent in the analysis of the slug test data included:

- o The aquifer is composed of homogeneous isotropic layers.
- o The radius of the well is small in comparison with the extent of the aquifer.
- o The removal of the slug and development of initial residual drawdown was instantaneous.
- o The time to the final residual drawdown measurement is large.
- o The influence of the filter pack was negligible.

The Bouwer-Rice method considers the quantity (R_e/r_w) where R_e is the effective radius over which the drawdown is dissipated and r_w is the radius of disturbance (well radius plus radius of gravel pack.) Due to the stratified nature of the aquifers, R_e was calculated for partially penetrating

wells. The equation which yields a value of hydraulic conductivity, K , is as follows (Bouwer, 1978):

$$K = \frac{r_c^2 \ln(R_e/r_w)}{2L} \frac{1}{t} \ln \frac{Y_0}{Y_t}$$

where

- K = hydraulic conductivity (ft/day).
- r_c = the inside radius of the well casing (ft).
- R_e = effective radius over which drawdown is dissipated (ft).
- r_w = well radius plus radius of gravel pack (ft).
- L = the length of the screened interval (ft).
- t = time (min).
- Y_0 = water level at first reading minus initial water level at $t = 0$ (ft).
- Y_t = residual drawdown at time t (ft).

Due to the inherent assumptions in the methods and the assessment of the slug test data collected, the most confidence was placed upon the Bouwer-Rice method of analysis (Table 0.7.2). Results produced an average hydraulic conductivity of 0.02 ft/day in tested intervals between 58 and 78 feet bls.

Five packer tests conducted within borehole 911 provided semi-quantitative values of hydraulic conductivity within a 51-foot vertical zone of siltstone bedrock underlying the site. Tests were run using single and double packers set in an open, cleaned borehole.

The equation for determining the hydraulic conductivity, K , is as follows (DOI, 1981):

$$K = \frac{Q}{C_u r H}$$

where

- K = hydraulic conductivity under a unit gradient (ft/sec).
- Q = steady flow into the well (cfs).
- C_u = conductivity coefficient for unsaturated material.
- r = radius of test hole (ft).
- H = effective head (pressure head plus elevation head minus friction loss).

Calculations and parameter values are on file in the UMTRA Project Office, Albuquerque, New Mexico.

Computed values of hydraulic conductivity for discrete intervals over a 51-foot vertical section are presented in Table 0.7.2. A general decrease in hydraulic conductivity

values with depth is apparent. The smallest value of hydraulic conductivity occurs in the interval from 94.5 feet to 104.5 feet within borehole 911. This is interpreted as representing the transition from a locally fractured, relatively permeable medium to a well consolidated, relatively impermeable medium. The average hydraulic conductivity for the interval between 53.5 and 104.5 feet bls was calculated to be 0.29 ft/day.

The groundwater flow rate through the upper hydrostratigraphic unit can be estimated using Darcy's Law (Davis and DeWiest, 1966) as follows:

$$Q = KA \frac{(h_1 - h_2)}{dl}$$

where,

Q = flow rate (cubic ft/yr).

K = hydraulic conductivity (ft/yr).

A = saturated cross sectional area perpendicular to groundwater flow (sq ft).

h_1, h_2 = hydraulic head at two points (ft).

dl = length of the flow path between points at which the head is given (ft).

Groundwater flow within the upper hydrostratigraphic unit is localized within discrete flow paths, as shown by the erratic occurrences of groundwater in the area. A conservative estimate of groundwater flow may be calculated by assuming that groundwater occurs within the upper unit under the entire area shown by the potentiometric contours of Figure D.7.7. Figure D.7.6 shows a vertical distance of approximately 40 feet between the groundwater level within well 910 and the top of the middle hydrostratigraphic unit; this 40-foot interval is assumed to represent the maximum, saturated thickness of the upper unit. The length of the groundwater equipotential contour at an elevation of 4150 feet, approximately 3000 feet, conservatively represents a section of the upper hydrostratigraphic unit oriented perpendicular to groundwater flow. An estimate of the hydraulic gradient is calculated using the measured groundwater elevations and horizontal distance between well 911 and seep 923.

The groundwater flow rate varies as indicated by the disappearance of the water table intercepted by well 911. Flow rate calculations for water levels measured within the upper hydrostratigraphic unit in 1985 are summarized in Table D.7.3. These calculations represent a probable upper limit on groundwater flow rate within the upper hydrostratigraphic unit in the area of the tailings.

The volume of groundwater within the upper hydrostratigraphic unit in the area of the tailings can be approximated using the equation:

$$V = (A)(b)(n)$$

where

V = volume (cubic ft).

A = horizontal area (sq ft).

b = average thickness of the saturated zone (ft).

n = total porosity.

The assumptions in this calculation include those for the flow rate calculations. In addition, values ranging between 0.12 and 0.35 for total porosity were assumed to represent the range of lithologies present beneath the tailings and are representative for materials ranging in composition from siltstone to fine grained sandstone (Todd, 1980; Freeze and Cherry, 1979). Volume calculations are summarized in Table D.7.4.

Middle hydrostratigraphic unit

Groundwater in the middle hydrostratigraphic unit occurs under confined conditions and underlies a large area of the tailings site. Overlying and confining this unit are approximately 40 feet of low-permeability, well consolidated siltstones. The stratigraphic position of the middle unit is difficult to place due to the gradational and indistinct boundary between the Halcito Shale and the Honaker Trail Formation. However, inspection of stratigraphic and structural relationships presented by Cooley et al. (1969), Baars (1973), and FBDU (1981) make it reasonable to place the middle hydrostratigraphic unit within the upper Honaker Trail Formation.

The middle hydrostratigraphic unit was intercepted by wells 907, 908, 909, and 930, and boreholes 931, 932, 933, and 936. This system is apparently contiguous over a large area below the Mexico Hat tailings site and occurs from a depth of approximately 112 to 160 feet bls. Because wells 907, 908, 909, and 930 are screened within the same geohydrologic unit, the range in depth bls is due to the varying topography and the generally northeast dipping structure in the area. A potentiometric contour map of the middle hydrostratigraphic unit is shown in Figure D.7.8.

The results of slug tests conducted within wells 908, 909, and 930 illustrate the local variability in hydraulic conductivities between the screened intervals of these wells. The inherent assumptions in the analyses of the slug test data are listed in the previous discussion concerning slug test data analyses for the upper hydrostratigraphic unit. The slug test data for the middle hydrostratigraphic unit were analyzed by two different methods applicable to confined groundwater conditions: the Skibitske method and the Ferris-Knowles

method. The Cooper-Bredehoeft-Papadopoulos method of analysis was attempted; however, the plotted data points were too erratic to obtain a justifiable solution.

The Skibitzke method (Bentall, 1963) is the solution of the equation:

$$K = qt/4\pi sL$$

where

K = hydraulic conductivity (ft/min).

q = slug volume (ft³).

t = time from test initiation until residual drawdown measurement (min).

s = residual drawdown (ft).

L = length (ft) of contributing interval.

Results of this method were considered valid when the time of recovery, t, was greater than 20 minutes.

The Ferris-Knowles method is the solution of the equation (Bentall, 1963):

$$K = \frac{q(1/t)}{4\pi sL}$$

where

K = hydraulic conductivity (ft/min).

q = slug volume (ft³).

1/t = reciprocal of time from test initiation until residual drawdown measurement (1/min).

s = residual drawdown (ft).

L = length of contributing interval (ft).

The values of 1/t and s are taken from a straight line fit through the data points. To be valid, this straight line fit must pass through the origin.

Although storage values may be obtained by use of this analytical method, these values were not calculated due to the questionable reliability of the calculated storage coefficient (Cooper et al., 1967).

Calculated values of hydraulic conductivities, the tested intervals, and the method used in the calculations are presented in Table D.7.2. Calculations and parameter values

are on file in the UMTRA Project Office, Albuquerque, New Mexico.

The Ferris-Knowles method of analysis provided the tightest range of hydraulic conductivity values for slug tests conducted within the middle hydrostratigraphic unit (see Table D.7.2). The average value of hydraulic conductivity calculated by this method is 0.15 ft/day.

A hydraulic gradient of 0.04 was determined from a potentiometric contour map based upon measured groundwater levels (Figure D.7.8). Groundwater velocity may be calculated approximately using the equation:

$$q_s = \frac{Kh}{n_e}$$

where

- q_s = seepage velocity (ft/day).
- K = hydraulic conductivity (ft/day).
- h = hydraulic gradient (ft/ft).
- n_e = effective porosity.

Calculated groundwater velocity within the middle hydrostratigraphic unit, using an average hydraulic conductivity value of 0.15 ft/day, a hydraulic gradient of 0.04, and a conservatively assumed value for porosity of 10 percent, is approximately 0.06 ft/day (22 ft/yr).

Lower hydrostratigraphic unit

The lower hydrostratigraphic unit contains groundwater and naturally occurring hydrocarbon compounds under confined conditions. This unit occurs over a large area of the region and is contained within the Monaker Trail Formation. The regional presence of this lower hydrostratigraphic unit is suggested by the occurrence of the Mexican Hat oil field immediately north of the tailings site where the upper Monaker Trail is an oil-producing horizon (Wengerd, 1973).

The lower hydrostratigraphic unit was penetrated by boreholes 908, 909, and 910. Inspection of the core samples retrieved from boreholes 908, 909, and 910 revealed sulphur and traces of oil coating fractures and vugs at depths between 125 feet and 185 feet bls. Hydrogen sulfide (H_2S) gas was encountered at a depth of approximately 215 feet bls in borehole 910. The occurrence of sulphur and H_2S in a petroleum producing environment is a widely recognized fact (Levinson, 1974). The Mexican Hat tailings site is located on the periphery of the Mexican Hat Oil Field which has some production from the same stratigraphic interval as the lower hydrostratigraphic unit as reported by Wengerd (1973). Thus, the

presence of sulphur and H_2S is not unexpected in the rocks below the tailings. Due to the danger to human life involved in drilling through strata charged with H_2S and the concomitant natural degradation of groundwater quality in this hydrocarbon-bearing zone, boreholes 908 and 909 were cemented to a level substantially above the lower hydrostratigraphic unit, then the lowermost portion of each hole was sealed with bentonite before being completed as monitor wells. Borehole 910 was grouted through the total depth of the borehole and abandoned. Subsequently, a new borehole 910 was located approximately 100 feet south of the abandoned borehole and completed as well 910.

As a result of the hazardous nature of this lowermost, deep confined groundwater system, limited amounts of data were gathered, and only a qualitative assessment of the hydrologic regime was made. The presence of oil indicates that the overlying rock is so low in permeability as to trap the oil, indicating minimal hydraulic communication between this zone and the overlying hydrostratigraphic units. This lower hydrostratigraphic unit discharges in several small springs and seeps in the floor of Gypsum Creek, east of the tailings site. The unit also gives rise to occasional oil seeps along the banks of the San Juan River. Thus, the river is incised to an elevation at least equivalent to this hydrocarbon-bearing zone, and any overlying hydrogeologic units cannot be hydraulically connected across the river because of topographic incision of the units by the river.

Vertical hydraulic gradients

Mean values of measured water levels in the upper hydrostratigraphic unit for the period April to June, 1985, were approximately 47 feet below mean values of measured water levels in the middle hydrostratigraphic unit over the same period. The approximate average depths to the upper and middle hydrostratigraphic units are 100 feet and 140 feet bls, respectively. Thus, an upward vertical hydraulic gradient of approximately 1.2 ft/ft exists between the middle and upper hydrostratigraphic units. This upward gradient would preclude the migration of contaminants into the deep, confined groundwater system.

In summary, substantial evidence at the Mexican Hat tailings indicates the following characteristics of the saturated zone:

- o The upper hydrostratigraphic unit at present contains at least one perched water table that occurs sporadically within the unit. However, it is highly likely that no groundwater existed in this unit prior to milling.

- o Groundwater of the upper hydrostratigraphic unit is laterally limited in extent. Boreholes 930, 931, 932, and 936, which were drilled by the air-rotary method, did not encounter groundwater above the level of the middle hydrostratigraphic unit.
- o A well-consolidated siltstone interval of low permeability, approximately 40 feet thick, separates the upper and middle stratigraphic units and acts as a confining stratum for the underlying middle hydrostratigraphic unit. This extremely low-permeability stratum inhibits downward seepage of tailings contamination.
- o The gradient between the middle and upper hydrostratigraphic units is upward.
- o The lower hydrostratigraphic unit is not considered a potentially exploitable water resource due to the naturally poor quality caused by hydrocarbons and H_2S .
- o The occurrence of mobile hydrocarbons and H_2S in the lower hydrostratigraphic unit indicates the presence of an extensive, extremely low-permeability stratum at the top of this unit. Therefore, there is no downward pathway for the migration of contaminants into this unit.

D.7.1.4 Water quality

This section discusses the water quality characteristics of the upper and middle hydrostratigraphic units at the Mexican Hat tailings site. As discussed in Section D.7.1.2 of this appendix, the lower hydrostratigraphic unit was found to contain naturally occurring, potentially hazardous accumulations of H_2S . This unit was therefore omitted from any further data collection and is not discussed herein.

Results of chemical analyses and location descriptions of water-quality samples collected from the upper hydrostratigraphic unit are presented in Tables D.7.5 and D.7.6, respectively. Groundwater within the upper hydrostratigraphic unit was only encountered downgradient of the Malchita sewage lagoons and tailings piles, illustrating the limited extent of saturated conditions within this unit. Likewise, the lack of any shallow water wells upgradient of the tailings pile, even though exploratory drillings have been made, indicates a lack of groundwater within the upper hydrostratigraphic unit upgradient of the tailings. Thus, a characterization of background groundwater quality within the upper hydrostratigraphic unit was not justified.

The location descriptions and results of chemical analyses for samples collected within the middle hydrostratigraphic unit are presented in Tables D.7.6 and D.7.7, respectively. From these data, background groundwater quality was determined for the middle hydrostratigraphic unit.

A second-phase drilling program was initiated in the fall of 1985 to verify the extent of groundwater in the vicinity of the tailings site. Seven boreholes were drilled by air-rotary techniques peripherally to the tailings site. Monitor well 930 was developed within the middle hydrostratigraphic unit and wells 934 and 935 were developed within the upper hydrostratigraphic unit. The four remaining boreholes encountered no groundwater within the upper hydrostratigraphic unit and were completed in either the middle or lower hydrostratigraphic units. These four boreholes were subsequently abandoned and cemented to total depths after geophysical logging.

Background water quality

Upper hydrostratigraphic unit. Background groundwater quality is difficult to establish for the upper hydrostratigraphic unit at the Mexican Hat tailings site because the only groundwater sampled within this unit is affected by tailings seepage. This interpretation is justifiable because:

- o Boreholes 909, 930, and 933, drilled upgradient of the tailings, encountered no groundwater within the upper hydrostratigraphic unit. The lack of upgradient groundwater precludes the establishment of background water quality for the upper hydrostratigraphic unit.
- o Wells 910, 911, and 912, completed within the upper hydrostratigraphic unit, are downgradient of the tailings, and contain groundwater contaminated by tailings seepage.
- o No seeps have been located by previous investigators or the DOE in the upper hydrostratigraphic unit within Gypsum Creek. Two seeps reported within Halgaito Wash and upstream of the tailings by GECR (1982) have been confirmed as being stagnant, standing water (Bush, 1985).

DOE has proposed that the background water quality of the upper unit be defined as the existing groundwater quality found with this unit immediately underlying the tailings. Chemical analyses of groundwater in the upper unit at the tailings site show several constituents with concentrations above the proposed EPA groundwater standards. The proposed EPA standards are marginally exceeded by chromium, fluoride, mercury, nitrate, and gross alpha

particle activity, while concentrations of uranium greatly exceed the proposed standard. The sporadic occurrence of water within the upper unit precludes the definition of a contaminant plume, and no off-site migration of tailings-related groundwater contamination has been detected.

Middle hydrostratigraphic unit. Background groundwater quality was established for the middle hydrostratigraphic unit in the area of the tailings site. This was accomplished using water-quality analyses for samples collected from wells 907, 908, 909, and 930. The justification for using water-quality analyses from these four wells to establish background groundwater quality for this unit is as follows:

- o Wells 907, 908, 909, and 930 are completed within the middle hydrostratigraphic unit.
- o The techniques employed in completing these wells effectively isolate the wells from the upper and lower hydrostratigraphic units.
- o As detailed in Section D.7.1.2, the upward hydraulic gradient between the middle and upper hydrostratigraphic units places these wells hydraulically upgradient of the tailings.

Background concentrations of chemical constituents within groundwater of the middle hydrostratigraphic unit are listed in Table D.7.8. The mean value for concentrations of TDS within groundwater of this unit (4864 mg/l) classifies this unit as a Class B aquifer which contains water resources not currently usable for human consumption (NRC, 1985).

Lower hydrostratigraphic unit. Background groundwater quality was not established for the lower hydrostratigraphic unit due to the hazards involved in sampling groundwater containing naturally occurring concentrations of hydrogen sulfide (H_2S) and hydrocarbons. Although a quantitative assessment of the chemical characteristics for groundwater in this zone was not determined, it is evident that groundwater of the lower hydrostratigraphic unit is of extremely poor quality.

Water-quality classification. A trilinear diagram on which the water-quality analyses of the DOE samples are plotted is shown in Figure D.7.9. All samples are similar in major ion chemistries, except samples 938 and 939 which represent water withdrawn from the San Juan River and sample 935, which is apparently affected by grout contamination (pH = 12.28).

An indication of the poor water quality in the area of the Mexican Hat tailings site is evident by the following:

- o There are no records of any producing water wells in the area (Little, 1985).
- o The area has been drilled for groundwater exploration, but potable water was not found (Denny, 1985).
- o Neither the Halgaito Shale Member nor the Honaker Trail Formation is considered to be an aquifer in this area (Hintze and Fuhrman, 1976).

Based on water-quality definitions published by Todd (1980) and the data plotted on the trilinear diagram, the groundwater of the upper and middle hydrostratigraphic units in the area of the Mexican Hat tailings site may be classified as brackish, moderately hard to hard, calcium-sulfate type groundwater.

D.7.1.5 Physical and chemical characterization of waste and contaminant transport

The physical and chemical characterization of waste and the determination of the extent of contaminant transport have several purposes as outlined below.

- o Identification of the potential contaminants.
- o Determination of contaminants that have migrated from the tailings.
- o Identification of the extent and relative concentration of contamination (e.g., the contaminant plume).
- o Determination of the rate of contaminant migration.
- o Determination of the extent to which contaminated groundwater is discharging to hydraulically connected surface water.
- o Separation of the contamination due to mill tailings from other contaminant sources.

Geochemistry of waste

The geochemical setting of the potentially affected hydrogeologic environment has been studied in detail. Pertinent findings of these studies are discussed below.

Studies conducted by ORNL (1980), FBDU (1981), and GECR (1982) indicate that radium and other tailings contaminant concentrations in the subsoil and foundation bedrock reach background levels within three feet of the tailings-subsoil

interface. Chemical analyses of subsoil materials from three test borings reported by CSU (1983b) revealed that a zone characterized by low pH, high concentrations of radionuclides, heavy metals, toxic non-metals, calcium, and sulfate exists directly below the tailings-subsoil interface. In these locations, the low pH and elevated contaminants are present only in the upper one foot of subsoil or bedrock.

Foundation samples were collected at several depths from three CSU boreholes and analyzed for chemical and radio-chemical constituents (CSU, 1983b). The locations of the holes are shown in Figure D.7.10, and the results of the analyses are presented in Table D.7.9. Samples taken from holes MH 200, MH 205, and MH 208 were retrieved using an NX-core sampler because they were drilled in the siltstone bedrock. These samples were not analyzed for content of radionuclides due to sample contamination from different elevations in the boreholes. The analyses of the cores indicate that only shallow penetration of the acid tailings solution has occurred.

The results of the foundation material analyses, with the exception of MH-205, show that immediately beneath the tailings-subsoil interface is a zone characterized by elevated concentrations of metals, toxic non-metals, calcium, and sulfate. In MH-200, the acid front of the tailings seepage had penetrated the slightly weathered red siltstone bedrock to a depth of approximately 4.5 feet beneath the interface. At this depth, conductivity as well as concentrations of metals and toxic non-metals decrease and the pH level and calcium carbonate content of the soil increase significantly. A similar effect is seen in MH-208 except that the acid front moved less than a foot into the foundation material. High concentrations of calcium and sulfate in the solid samples correspond with the depth to which the acid front moved and are the result of gypsum precipitation. Gypsum ($\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$) is produced by the neutralization of sulfuric acid seepage by calcium carbonate soil. Located near the edge of the pile, MH 205 appears to be unaffected by seepage penetration. The high sulfate concentration in this borehole at the 4.5-foot depth is most likely a natural gypsum deposit in the siltstone.

It is interesting to note that the one water sample reported by ORNL (1980) that had elevated levels of radium was collected from standing water in an arroyo that drains from the tailings pile area. Taken together with the evidence of limited seepage penetration into the hard foundation bedrock, this suggests that during the active life of the impoundment, seepage may have drained off over the bedrock surface, perhaps along this pre-existing drainage. If, in fact, this was the case and acid seepage was concentrated along a narrow drainage path, it may have advanced a significant lateral distance away from the pile before being neutralized. The radium sampled

in the arroyo downstream of the pile may result from such a condition although no data are available to confirm this.

The maximum value of net infiltration (i.e., infiltration that percolates downward to the groundwater) may be estimated from published results of net infiltration at the Shiprock tailings site (DOE, 1985b). At the Mexican Hat tailings site, precipitation is less and evaporation is greater relative to the Shiprock tailings site. The calculated value of net infiltration at the Shiprock tailings (0.04 inch per year) must therefore be greater than at the Mexican Hat site. Thus, it can be concluded that there is little if any infiltration presently passing through the Mexican Hat tailings to the groundwater, and the installation of a low-permeability cover would further reduce infiltration and likewise reduce the potential for contaminant migration from the Mexican Hat tailings into the groundwater of the upper hydrostratigraphic unit.

Extent of contamination

It was concluded in Section D.7.1.4 that groundwater affected by contamination due to tailings seepage is restricted to the upper hydrostratigraphic unit in the immediate area of the Mexican Hat tailings site. Due to the erratic occurrence of perched groundwater in this unit, a contaminant plume could not be characterized. Groundwater which had been contaminated by tailings seepage was detected in wells 911 and 912. Contamination due to tailings seepage has not been detected below the upper hydrostratigraphic unit or within the San Juan River.

Discharge of plume to surface water

Geochemical investigations conducted in the area of the Mexican Hat tailings site have included attempts to detect tailings seepage contamination entering the San Juan River; discharge of contaminated tailings seepage into the river was not detected in any of these investigations (GECR, 1982; EPA, 1973). Water-quality data from the San Juan River at Mexican Hat, Utah, are available from the U.S. Geological Survey for a period of record from 1928 to 1985 (USGS, 1985). These data failed to show any measurable changes in San Juan River water quality attributable to the potential influx of contaminated tailings seepage. The data indicate that the relatively large flow of the San Juan River dilutes and disperses all contaminants which may be originating from the tailings. This dilution and dispersion is so effective that there is no measurable water-quality impact from the discharge of groundwater which may have some component of tailings constituents dissolved within it.

Several springs or seeps have been identified within the channel of Gypsum Creek, east of the tailings site. These surface discharges of groundwater are the result of artesian flow within the Honaker Trail Formation; the surface flows disappear into either alluvium or bedrock a short distance downstream of the individual springs or seeps. Elevated concentrations of uranium have been identified in the flowing water within Gypsum Creek. No discharges of groundwater have been identified which could have originated from the existing Mexican Hat tailings site.

Contaminant sources other than mill tailings

The most likely source of contamination of the upper hydrostratigraphic unit other than the mill tailings is seepage from the Malchita sewage lagoons. A water-quality analysis performed on a sample from the active lagoon showed concentrations of inorganic chemical constituents above the levels of the upper hydrostratigraphic unit (BECR, 1982). These constituents were chromium, nickel, and uranium with concentrations of 0.025 mg/l, 0.110 mg/l and 0.0168 mg/l, respectively. However, maximum concentrations of these constituents only occur in tailings contaminated groundwater.

Geochemical effects on contaminant migration

Hydrodynamic and geochemical processes control the movement of contaminants in seepage from uranium mill tailings piles into the adjacent soil and aquifer environments. Advection and dispersion are the primary hydrodynamic processes that affect contaminant migration. Advection is the process by which dissolved contaminants are physically transported with the pore water. Dispersion spreads the contaminant plume in three dimensions along the flow path, thereby lowering the concentration of contaminants but affecting a larger volume of the soil or aquifer. As used here, hydrodynamic dispersion includes mechanical mixing and molecular diffusion. These processes are described in detail by Freeze and Cherry (1979).

At the Mexican Hat site, potential evaporation exceeds 80 inches per year while the annual rainfall is only six inches; therefore, it is expected that there is little ongoing recharge to the piles and the current seepage rate from the piles is very low. The majority of the contamination present in the soil and groundwater is most likely due to drainage from the piles during the active milling (1957 to 1965) when large amounts of water were used to slurry the tailings onto the piles.

The primary geochemical process affecting contaminants in seepage from acid mill tailings piles similar to those at Mexican Hat is neutralization of the solution by reaction with carbonate minerals in the sediment. The rise in pH associated with neutralization produces a condition that promotes the mass transfer of many contaminants from the solution phase to the solid phase. The dissolved concentration of these contaminants decreases as they precipitate as solids or are adsorbed onto the surface of existing solids. These geochemical processes remove most radionuclides (radium and thorium), metals (aluminum, iron, manganese, chromium, zinc, cadmium, and copper), and toxic non-metals (arsenic) from solution. In general, arsenic, uranium, selenium, and molybdenum may not be removed from solution to below contaminant levels because they form relatively mobile anionic species in the oxidizing, alkaline environment characteristic of the neutralization zone in the sediment. However, the rate of movement of these three elements through the aquifer or soil will be retarded compared to that of the average linear velocity of the groundwater as a result of adsorption processes.

The concentration of major cations (calcium, magnesium, sodium, and potassium) and major anions (sulfate, bicarbonate, and chlorine) moving through the neutralization zone into the downgradient flow environment will be affected by ion exchange between the water and mineral surfaces; however, charge balance must be maintained in the solution, therefore, the TDS level will not be significantly affected by ion exchange processes. The actual movement of tailings constituents in seepage from the Mexican Hat piles will be controlled by the specific geochemical conditions of the site. These features are discussed with the observed distribution of contaminants in the vicinity of the tailings piles.

The Mexican Hat tailings are situated on outcrops of the Halgaito Shale Member tongue of the Cutler Group which overlies the Honaker Trail Formation. These units are several hundred feet thick in this area of the Navajo Reservation and have been reported as unsaturated (FBDU, 1981). Also, changes in water chemistry along the flow path reflect hydrodynamic and geochemical processes affecting water composition.

The two contaminated wells (911 and 912) directly beneath the tailings pile provide a good indication of which contaminants are relatively mobile in the seepage from the piles. Table D.7.5 shows that uranium exceeded the EPA groundwater standards consistently, and chromium exceeded the standard in one well at one sampling time. The water chemistry data for samples collected from below the tailings pile suggest that these contaminants are mobile in the upper hydrostratigraphic unit.

Water sampling by ORNL (1980) revealed an elevated Ra-226 level of 16 pCi/l in a sample taken from a stagnant pond located in a dry wash northeast of the piles. Their report states that the sampling point is 600 meters (1968 feet) from the piles, but their location map shows it to be only 400 feet from the northeast edge of the piles. The source of radium may be seepage from the piles or leaching of windblown tailings from the piles. There is no doubt part of the reason for the high level of radium in the pond is evaporative concentration of an originally more dilute solution. It is not expected that radium is mobile in this groundwater environment. As described in the following paragraph, radium is normally removed from solution and concentrated in the solid phase.

Soil chemical analyses of samples taken beneath the tailings pile also provide an indication of the mobility of some of the contaminants from the pile (Shepard and Brown, 1983). The soil pH values show that the acid front extends no farther than 4.5 feet (1.4 meters) beneath the pile into the underlying bedrock. In the chemically affected zone, iron, aluminum, calcium, magnesium, and sulfate are concentrated relative to the sediment below this zone. Iron and aluminum have probably been precipitated as oxyhydroxides or hydroxy-sulfate minerals whereas gypsum ($\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$) formation accounts for calcium and sulfate. The high solubility of gypsum produces the large amount of sulfate in the water sample from the wells beneath the piles. Magnesium may coprecipitate with these elements in the newly formed minerals. The extent of Ra-226 migration into the bedrock beneath the pile was determined by ORNL (1980) and FBDU (1981). They show the Ra-226 concentration decreasing from approximately 300 pCi/g (soil) to less than 5 pCi/g (soil) over a three-foot soil interval from the tailings-bedrock interface. Radium may be incorporated in the gypsum formed in this neutralization zone.

In summary, it appears (from soil chemical analyses at the Mexican Hat site) that many of the potential groundwater contaminants from the tailings pile are immobilized in the neutralization zone beneath the pile.

0.7.1.6 Groundwater use

There is no recorded utilization of groundwater in the Mexican Hat area. Process water for milling operations was obtained from the San Juan River. A search of the Navajo Tribal Utility Authority (NTUA) records produced no evidence of any producing water wells in the area (Little, 1985). A long-time, local resident of the community of Halchita reported that the area had been drilled in attempts to develop groundwater resources, but "potable water was not found. Prior to

construction of the present water supply system, the local residents obtained their water from windmills or wells located in Monument Valley, Arizona, or Bluff, Utah, or directly out of the San Juan River" (Genny, 1985).

Domestic water for the community of Mexican Hat is supplied by a converted oil exploration well between the community and the San Juan River. The water is not treated (Ball, 1985; May, 1985). No other information is available for this water supply system, but it is believed that the water is withdrawn from strata directly recharged by the San Juan River.

D.7.1.7 Compliance with EPA groundwater standards

Proposed action

The conceptual design incorporates many components that would minimize infiltration and leachate generation. These components, along with natural features at the site, would promote compliance with the proposed EPA standards and are as follows:

- o Emplacement of a cover system consisting of a filter layer, an erosion protection barrier, and a radon/infiltration barrier to reduce infiltration and promote surface runoff and evaporation.
- o Minimization of tailings seepage by the use of a low hydraulic conductivity radon/infiltration barrier to reduce infiltration.
- o Natural geochemical attenuation of contaminants in the tailings seepage by adsorption and precipitation reactions within the Halgaito Shale Member of the Cutler Formation fractured bedrock beneath and downgradient of the disposal site.
- o Strong, upward, vertical hydraulic gradients in the saturated bedrock downgradient of the disposal site to inhibit downward migration of contamination.

Compliance

Of the 14 constituents for which there are proposed MCL's, only uranium and gross alpha activity exceed the MCL's below the existing tailings piles and these are the only constituents predicted to exceed the MCL's at the toe of the disposal site.

Seepage rates were predicted by conservatively assuming a saturated, low-conductivity radon/infiltration barrier and a unit hydraulic gradient. Dispersion of uranium and gross alpha

activity were not modeled because of the extremely heterogeneous nature of the underlying bedrock. Three boreholes drilled downgradient of the tailings and through the upper hydrostratigraphic unit failed to detect groundwater in this unit: two monitor wells located downgradient of the tailings and within the upper unit have detected no contaminant migration within this unit. Visual inspections of the farthest downgradient location of the upper hydrostratigraphic unit (i.e., the western bank of Gypsum Creek) have likewise failed to detect tailings-related contamination.

Seepage impacts were analyzed for the upper hydrostratigraphic unit. The strong, vertically upward hydraulic gradients that exist between the upper and lower-lying units, as well as the thick, relatively impermeable strata which separate the upper and middle unit, would restrict the movement of any tailings seepage into the middle or lower hydrostratigraphic units (see Section B.2.6 Appendix B).

Alternate concentration limits

In the event compliance with the proposed standards for uranium and gross alpha activity cannot be achieved with the proposed conceptual design, alternate concentration limits (ACLs) may also be considered. However, the DOE believes that the use of supplemental standards is more appropriate than ACL's at the Mexican Hat site.

Supplemental standards

The DOE believes that the upper hydrostratigraphic unit is not an aquifer, as per the criteria proposed by the NRC (NRC, 1988), and therefore the application of supplemental standards is appropriate for the Mexican Hat site (see Section B.2.6, Appendix B). Additionally, no naturally occurring groundwater exists within the upper unit in the area of the site, the off-site migration of tailings-related contaminants has not been detected, and the present and projected water needs of the area are supplied by withdrawals from the San Juan River.

No action

Under existing conditions, the volume of recharge to the tailings pile has dropped to only that produced during natural precipitation and infiltration. Both the quantity and rate of seepage from the tailings piles has decreased significantly from the period of active milling.

If there is no remedial action, it may be expected that continued leaching of contaminants from the tailings piles would continue at a decreasing rate, and the discharge of tailings leachate would ultimately cease.

Aquifer restoration

The DOE believes that the upper hydrostratigraphic unit of the Mexican Hat site does not constitute an aquifer and that no aquifer restoration would be required.

D.7.2 BORROW SITES

D.7.2.1 Rock borrow sites

The Shadow Mountain borrow site is approximately 160 miles southwest of the tailings site in rugged terrain at the base of an extinct volcano. Residual basalt is at the surface in the area. Large rock for the erosion protection (sizes C and D) will be quarried basalt at a depth of approximately twenty feet beneath a zone of hard, glassy cinder and vesicular basalt. This site has been used previously as a rock quarry.

The Holiday Pit in Bluff, Utah, approximately 30 miles northeast of the tailings pile, will be the source of the smaller rock (sizes A and B) needed for erosion protection. The site is an existing commercial gravel pit, mining terrace deposits of the San Juan River.

D.7.2.2 Radon borrow site

The RB borrow site is approximately 5 miles south of the Mexican Hat tailings site within an arroyo that drains eastward to Haighton Wash. In the area of the borrow site, the drainage area is relatively large and of a gentle gradient.

D.7.2.3 Flow and hydraulics

Groundwater has not been encountered at any of the borrow sites.

D.7.2.4 Water quality

The Shadow Mountain borrow site is approximately 150 feet north of an ephemeral stream that drains the area southwest of the borrow site towards Moenkopi Wash. Due to the ephemeral nature of surface water flows at the Shadow Mountain and RB borrow sites, no water quality data are available for these locations.

D.7.2.5 Water use

No groundwater studies have been performed in the Shadow Mountain area. Surficial deposits at the site consist of

residual basalt underlain by the Shinarump Member of the Chinle Formation. The shinarump yields small amounts of poor-quality water in some areas (DOE, 1986).

There are no records of any producing water wells in the area of the RB borrow site and since there are no dwellings, livestock, or agricultural activities in the immediate vicinity area, water use is nonexistent in these areas.

FIGURE D.7.2
TYPICAL DOE MONITOR WELL CONSTRUCTION

FIGURE D.7.4
HYDROGEOLOGIC CROSS-SECTION A-A'
MEXICAN HAT SITE

FIGURE D.7.6
HYDROGEOLOGIC CROSS-SECTION C-C'
MEXICAN HAT SITE

FIGURE D.7.9
TRILINEAR DIAGRAM, MEXICAN HAT SITE

Table D.7.1 DOE monitor well and borehole data, Mexican Hat site

Well number	Well diameter (in)	Total depth (ft)	Surface elevation ^a (ft)	Screened interval	
				Top of screen ^a (elevation in ft)	Length (ft)
905	2.0	12	4294.6	4291.1	5
906	2.0	12	4220.8	4216.1	5
907	0.75	125	4220.5	4114.5	3 ^b
908	2.0	163	4294.6	4148.6	15
909	2.0	157	4349.8	4219.8	20
910	2.0	182	4353.2	4213.2	40 ^c
911	2.0	105	4294.4	4202.4	10
912	2.0	84	4294.4	4236.9	20
930	4.0	116	4311.7	4211.7	10
934	4.0	200	4289.3	4104.3	10
935	4.0	187	4230.6	4055.6	10
901 ^d		3			
902 ^d		3			
903 ^d		3			
904 ^d		3			
910 ^d		235			
931 ^d		177			
932 ^d		193			
933 ^d		175			
936 ^d		142			

^aElevations measured as feet above mean sea level.

^bWell 907 is not screened; a double packer was installed, and the borehole has a three-foot open interval.

^cWell 910 has five-foot sections of screen alternating with five-foot blank sections over a total length of 40 feet.

^dAbandoned borehole.

Table D.7.2 Summary of aquifer characteristics, Mexican Hat site

Well number	Test method ^a	Tested interval ^b	Hydraulic conductivity (ft/day) ^c	Average linear velocity (ft/day) ^d
908	Sk	145 - 160	0.72	0.14 - 0.36
	F-K	145 - 160	0.09	0.02 - 0.05
909	Sk	130 - 150	0.40	0.08 - 0.20
	F-K	130 - 150	0.17	0.03 - 0.09
911	P	53.5 - 64.5	0.76	0.15 - 0.38
	P	64.5 - 74.5	0.26	0.05 - 0.13
	P	74.5 - 84.5	0.19	0.04 - 0.10
	P	84.5 - 94.5	0.18	0.03 - 0.09
	P	94.5 - 104.5	0.05	0.01 - 0.03
912	B-R	58 - 78	0.02	0.004 - 0.01
930	SK	104 - 114	0.07	0.01 - 0.04
	F-K	104 - 114	0.19	0.04 - 0.10

^aSk is Skibitske method; F-K is Ferris-Knowles method; P is Packer Test method; B-R is Bouwer-Rice method.

^bTested interval in feet below the land surface.

^cFor wells 908 and 909, hydraulic conductivities are the average of two tests performed in April and July, 1985.

^dAverage linear velocity calculated for silt (porosity = 0.08) and fine sand (porosity = 0.21); hydraulic gradient = 0.04.

Table D.7.3 Groundwater flow rates, Mexican Hat site, 1985

	Maximum ^a	Minimum ^b	Average ^c
K (ft/day)	0.76	0.05	0.29
A (sq ft)	120,000	120,000	120,000
h ₁ (ft)	4198	4198	4198
h ₂ (ft)	4066	4066	4066
dI (ft)	4200	4200	4200
Q (cu ft/yr)	1.0×10^6	6.6×10^4	3.8×10^5
Q (l/yr)	2.8×10^7	1.9×10^6	1.1×10^7

^aK from Packer test in borehole 911 from 53.5 feet to 64.5 feet bls.

^bK from Packer test in borehole 911 from 94.5 feet to 84.5 feet bls.

^cK from average of Packer tests in borehole 911 from 53.5 ft to 104.5 ft bls.

Table D.7.4 Volume of shallow groundwater in the area of the Mexican Hat tailings, 1985

Area (ac)	Saturated zone average thickness (ft) ^a	Total porosity ^b	Volume (cu ft)	Volume (liters)
293	40	0.05	2.6×10^7	7.4×10^8
293	40	0.35	1.8×10^8	5.1×10^9

^aEstimated maximum thickness of the saturated zone present in the upper hydrostratigraphic unit.

^bRange of values selected from Todd (1980) and Freeze and Cherry (1979) as representative of siltstone (total porosity = 0.35) and fine-grained sandstone (total porosity = 0.05 to 0.30).

Table D.7.5 Concentrations of major and trace constituents in groundwater and surface water, upper hydrostratigraphic unit^a

Constituent	Unit of measure ^b	Location number: Date:	910 07/30/85	911 04/13/85	912 04/13/85	912 07/29/85	922 04/10/85
Alkalinity	mg/l CaCO ₃		274.000	294.000	887.000	790.000	224.000
Aluminum	mg/l		0.200	<0.100	<0.100	<0.400	<0.100
Ammonium	mg/l		--	12.000	38.000	3.800	<0.180
Antimony	mg/l		0.011	<0.003	<0.003	0.005	<0.003
Arsenic	mg/l		<0.010	<0.010	<0.010	<0.010	<0.010
Barium	mg/l		<0.100	<0.100	<0.100	<0.100	<0.100
Boron	mg/l		<0.100	1.300	0.700	0.100	1.300
Bromide	mg/l		--	<2.000	<2.000	--	<2.000
Cadmium	mg/l		<0.001	<0.001	<0.001	<0.001	<0.001
Calcium	mg/l		180.000	575.000	610.000	530.000	620.000
Chloride	mg/l		170.000	100.000	88.000	81.000	190.000
Chromium	mg/l		<0.050	<0.050	<0.050	0.080*	<0.010
Cobalt	mg/l		<0.050	<0.050	<0.050	<0.050	<0.050
Conductance	micromho/cm		2400.000	4500.000	4500.000	6550.000	5000.000
Copper	mg/l		0.040	0.020	0.020	0.050	<0.020
Cyanide	mg/l		--	<0.010	<0.010	<0.010	<0.010
Fluoride	mg/l		0.700	0.400	0.600	0.600	0.600
Gross alpha	pCi/l		--	330.000	510.000	620.000	120.000
Gross beta	pCi/l		--	180.000	220.000	220.000	90.000
Iron	mg/l		0.060	0.090	0.210	0.060	0.140
Lead	mg/l		0.030	<0.010	<0.010	0.020	<0.010
Magnesium	mg/l		86.900	348.000	423.000	430.000	305.000
Manganese	mg/l		0.150**	0.380**	3.270**	2.540**	0.040
Mercury	mg/l		<0.000	<0.000	<0.000	<0.000	<0.000
Molybdenum	mg/l		0.010	<0.010	<0.010	0.150	<0.010
Nickel	mg/l		0.110	0.080	0.160	0.200	<0.040
Nitrate	mg/l		--	8.100	8.000	<1.000	1.800
Nitrite	mg/l		--	0.000	0.000	<0.100	0.000
Organic carbon	mg/l		--	--	--	140.000	146.000
Lead-210	pCi/l		--	1.100	1.300	1.100	1.100
pH	SU		7.360	7.740	6.690	6.150**	8.330
Phosphate	mg/l		<0.100	0.200	0.200	<0.100	0.100
Polonium-210	pCi/l		--	0.000	0.400	0.000	0.200
Potassium	mg/l		7.800	28.300	23.600	26.800	18.600
Radium-226	pCi/l		--	1.100	0.400	0.300	0.100
Radium-228	pCi/l		--	0.500	0.600	0.000	0.000
Selenium	mg/l		<0.005	<0.005	<0.005	<0.005	<0.005
Silica	mg/l		--	26.000	32.000	30.000	13.000
Silver	mg/l		<0.010	<0.010	<0.010	<0.010	<0.010
Sodium	mg/l		249.000	454.000	340.000	390.000	770.000
Strontium	mg/l		<0.100	<0.100	<0.100	<0.100	<0.100
Sulfate	mg/l		947.000**	3170.000**	3040.000**	3100.000**	3670.000**
Sulfide	mg/l		--	<0.100	<0.100	<0.100	<0.100
Temperature	degrees centigrade		27.000	22.000	23.000	21.000	27.000
Thorium-230	pCi/l		--	7.200	1.300	0.200	0.000
Tin	mg/l		<0.005	<0.005	<0.005	<0.005	<0.005
Total solids	mg/l		1870.000**	1960.000**	5250.000**	5390.000**	6120.000**
Uranium	mg/l		0.033***	0.602***	0.737***	0.776***	0.295***
Vanadium	mg/l		0.400	<0.010	<0.010	0.400	<0.010
Zinc	mg/l		0.178	0.050	0.060	0.028	<0.010

Table 0.7.5 Concentrations of major and trace constituents in groundwater and surface water, upper hydrostratigraphic unit (Concluded)^a

Constituent	Unit of measured ^b	Location number: Date:	922	923	924	934	935
			07/28/85	07/28/85	07/29/85	11/01/85	11/01/85
Alkalinity	mg/l CaCO ₃		310.000	277.000	218.000	139.000	2834.000
Aluminum	mg/l		0.300	0.300	0.200	0.200	1.100
Ammonium	mg/l		0.500	<0.100	<0.100	<0.100	0.400
Antimony	mg/l		0.007	0.006	0.005	<0.003	<0.003
Arsenic	mg/l		<0.010	<0.010	<0.010	<0.010	<0.010
Barium	mg/l		<0.100	0.300	<0.100	0.200	0.400
Boron	mg/l		0.200	0.100	0.300	0.500	0.300
Bromide	mg/l		--	--	--	--	--
Cadmium	mg/l		<0.001	<0.001	<0.001	<0.001	<0.001
Calcium	mg/l		503.000	520.000	456.000	105.000	99.900
Chloride	mg/l		180.000	150.000	170.000	360.000**	100.000
Chromium	mg/l		0.080*	0.080*	0.060	0.040	0.210*
Cobalt	mg/l		<0.050	<0.050	<0.050	0.070	0.110
Conductance	micromho/cm		7000.000	7000.000	4000.000	2400.000	9000.000
Copper	mg/l		0.040	0.040	0.040	0.030	0.050
Cyanide	mg/l		<0.010	<0.010	<0.010	<0.010	<0.010
Fluoride	mg/l		0.400	0.600	0.500	9.200	1.300
Gross alpha	pCi/l		140.000	17.000	90.000	--	--
Gross beta	pCi/l		60.000	15.000	57.000	--	--
Iron	mg/l		0.090	0.260	0.070	<0.030	<0.030
Lead	mg/l		<0.010	<0.010	<0.010	<0.010	<0.010
Magnesium	mg/l		310.000	240.000	200.000	26.100	0.004
Manganese	mg/l		0.160**	0.480**	0.310**	0.060**	0.030
Mercury	mg/l		<0.0002	<0.0002	<0.0002	0.0024	<0.0002
Molybdenum	mg/l		0.140	0.130	0.230	0.030	0.050
Nickel	mg/l		0.160	0.110	0.100	0.060	0.100
Nitrate	mg/l		13.100	<1.000	18.000	62.000*	90.000*
Nitrite	mg/l		<0.100	<0.100	<0.100	<0.010	<0.100
Organic carbon	mg/l		42.000	38.000	33.000	45.000	72.000
Lead-210	pCi/l		0.400	0.000	0.000	1.000	3.200
pH	SU		7.910	7.230	7.830	7.400	12.280
Phosphate	mg/l		<0.100	<0.100	<0.100	<0.100	0.100
Potassium	mg/l		19.900	19.500	14.600	18.400	370.000
Radium-226	pCi/l		0.000	0.200	0.200	--	--
Radium-228	pCi/l		0.000	0.600	0.000	--	--
Selenium	mg/l		<0.005	<0.005	<0.005	0.007	<0.005
Silica	mg/l		20.000	20.000	11.000	11.000	11.000
Silver	mg/l		<0.010	<0.010	<0.010	<0.010	<0.010
Sodium	mg/l		790.000	840.000	522.000	438.000	1370.000
Strontium	mg/l		<0.100	<0.100	<0.100	0.100	0.700
Sulfate	mg/l		3640.000**	3570.000**	2580.000**	722.000**	721.000**
Sulfide	mg/l		<0.100	<0.100	<0.100	<0.100	<0.100
Temperature	degrees centigrade		24.000	24.000	21.000	17.000	15.000
Thorium-230	pCi/l		0.300	0.200	0.200	--	--
Tin	mg/l		<0.005	<0.005	<0.005	<0.005	<0.005
Total solids	mg/l		6160.000**	5750.000**	4380.000**	1840.000**	4250.000**
Uranium	mg/l		0.212***	0.011	0.229***	0.007	<0.0003
Vanadium	mg/l		0.400	0.400	0.400	0.120	0.360
Zinc	mg/l		0.015	0.026	0.010	0.027	0.014

Table D.7.6 Descriptions of DOE groundwater and surface-water samples

DOE sample number	Hydrostratigraphic unit	Description of sample location
907	Middle	Well at head of dry wash leading from tailings.
908	Middle	Well at head of dry wash north of tailings.
909	Middle	Well approximately 1800 feet southwest of tailings.
910	Upper	Well approximately 200 feet south of tailings.
911	Upper	Well within lower tailings.
912	Upper	Well within lower tailings.
922	Upper	Seep in Gypsum Creek approximately 2200 feet northeast of tailings.
923	Upper	Seep in Gypsum Creek approximately 2500 feet northeast of tailings.
924	Upper	Seep in Gypsum Creek below confluence of dry wash leading from tailings, approximately 3600 feet northeast of tailings.
930	Middle	Well across U.S. Highway 163 approximately 1300 feet from tailings.
934	Upper	Well approximately 800 feet east of tailings.
935	Upper	Well approximately 800 feet northeast of tailings

Table D.7.7 Concentrations of major and trace constituents in groundwater and surface water, middle hydrostratigraphic unit^a

Constituent	Unit of measure ^b	Location number: Date:	907	907	908	908	909
			04/11/85	07/26/85	04/12/85	07/28/85	04/10/85
Alkalinity	mg/l CaCO ₃		126.000	125.000	92.000	98.000	140.000
Aluminum	mg/l		0.100	0.400	<0.100	0.300	<0.100
Ammonium	mg/l		0.100	<0.100	0.100	<0.100	<0.100
Antimony	mg/l		<0.003	0.004	<0.003	0.007	<0.003
Arsenic	mg/l		<0.010	<0.010	<0.010	<0.010	<0.010
Barium	mg/l		<0.100	0.100	<0.100	0.200	<0.100
Boron	mg/l		1.300	0.200	4.700	0.400	1.000
Bromide	mg/l		<2.000	—	<2.000	—	<2.000
Cadmium	mg/l		<0.001	<0.001	<0.001	<0.001	<0.001
Calcium	mg/l		600.000	440.000	570.000	410.000	445.000
Chloride	mg/l		230.000	240.000	230.000	220.000	110.000
Chromium	mg/l		<0.010	0.040	<0.030	0.070	<0.030
Cobalt	mg/l		<0.050	<0.050	<0.050	<0.050	<0.050
Conductance	micromho/cm		5000.000	5000.000	6000.000	5500.000	3000.000
Copper	mg/l		0.050	0.040	<0.020	0.050	<0.020
Cyanide	mg/l		<0.010	<0.010	<0.010	<0.010	<0.010
Fluoride	mg/l		1.400	1.200	1.500	1.400	1.400
Gross alpha	pCi/l		0.000	5.000	0.000	18.000	60.000
Gross beta	pCi/l		20.000	12.000	20.000	1.000	120.000
Iron	mg/l		0.150	0.200	0.170	0.170	0.130
Lead	mg/l		<0.010	<0.010	<0.010	0.030	<0.010
Magnesium	mg/l		183.000	200.000	163.000	180.000	165.000
Manganese	mg/l		0.050	0.030	0.050	0.060**	0.020
Mercury	mg/l		<0.000	<0.000	0.000	<0.000	<0.000
Molybdenum	mg/l		<0.010	0.100	<0.010	0.120	<0.010
Nickel	mg/l		<0.040	0.110	<0.040	0.110	<0.040
Nitrate	mg/l		<0.100	<1.000	0.600	<1.000	0.800
Nitrite	mg/l		0.000	<0.100	0.000	<0.100	0.000
Organic carbon	mg/l		25.000	18.000	22.000	16.000	31.000
Lead-210	pCi/l		1.400	1.200	0.500	0.400	0.000
pH	SV		7.400	7.290	7.840	7.650	7.080
Phosphate	mg/l		<0.100	<0.100	0.200	<0.100	0.100
Polonium-210	pCi/l		0.300	0.000	0.200	0.000	0.000
Potassium	mg/l		9.510	10.500	10.000	11.100	7.800
Radium-226	pCi/l		3.100	3.500	1.000	0.900	0.000
Radium-228	pCi/l		0.600	0.700	0.500	0.600	0.000
Selenium	mg/l		<0.005	<0.005	<0.005	<0.005	<0.005
Silica	mg/l		13.000	12.000	12.000	12.000	15.000
Silver	mg/l		<0.010	<0.010	<0.010	<0.010	<0.010
Sodium	mg/l		988.000	7030.000	1210.000	1320.000	465.000
Strontium	mg/l		<0.100	<0.100	<0.100	<0.100	<0.100
Sulfate	mg/l		3600.000**	3580.000**	3960.000**	4090.000**	2380.000
Sulfide	mg/l		<0.100	<0.100	<0.100	<0.100	<0.100
Temperature	degrees centigrade		18.000	18.000	19.500	18.000	17.000
Thorium-230	pCi/l		0.200	0.200	0.000	0.300	0.100
Tin	mg/l		<0.005	<0.005	<0.005	<0.005	<0.005
Total solids	mg/l		5870.000**	5630.000**	6320.000**	6550.000**	3880.000**
Uranium	mg/l		0.002	0.001	0.002	0.004	0.051***
Vanadium	mg/l		<0.010	0.600	<0.010	0.500	<0.010
Zinc	mg/l		<0.010	0.188	<0.010	0.005	<0.010

Table D.7.7 Concentrations of major and trace constituents in groundwater and surface water, middle hydrostratigraphic unit (Concluded)^a

Constituent	Unit of measure ^b	Location number: Date:	909 07/27/85	930 10/30/85
Alkalinity CaCO ₃	mg/l		133	59
Aluminum	mg/l		0.3	0.2
Ammonium	mg/l		<0.1	0.3
Antimony	mg/l		0.006	<0.003
Arsenic	mg/l		<0.01	<0.01
Barium	mg/l		0.1	0.2
Boron	mg/l		0.1	0.5
Bromide	mg/l		--	--
Cadmium	mg/l		<0.001	<0.001
Calcium	mg/l		320	153
Chloride	mg/l		99	38
Chromium	mg/l		0.06	0.03
Cobalt	mg/l		<0.05	0.07
Conductance	micromhos/cm		3250	2000
Copper	mg/l		0.04	<0.02
Cyanide	mg/l		<0.01	<0.01
Fluoride	mg/l		1.4	1.4
Gross alpha	pCi/l		41	--
Gross beta	pCi/l		20	--
Iron	mg/l		<0.03	<0.03
Lead	mg/l		<0.01	<0.01
Magnesium	mg/l		190	46.6
Manganese	mg/l		<0.01	0.01
Mercury	mg/l		<0.0002	<0.0002
Molybdenum	mg/l		0.20	0.04
Nickel	mg/l		0.11	0.06
Nitrate	mg/l		4	0.4
Nitrite	mg/l		<0.1	<0.1
Organic carbon	mg/l		22	32
Lead-210	pCi/l		--	--
pH	SU		7.05	10.24**
Phosphate	mg/l		<0.1	0.1
Polonium-210	pCi/l		0.0	--
Potassium	mg/l		8.51	5.64
Radium-226	pCi/l		0.1	0.4
Radium-228	pCi/l		0.4	0.5
Selenium	mg/l		<0.005	<0.005
Silica	mg/l		13	22
Silver	mg/l		<0.01	<0.01
Sodium	mg/l		470	340
Strontium	mg/l		<0.1	0.4
Sulfate	mg/l		2230**	1190**
Sulfide	mg/l		<0.1	<0.1
Temperature	degrees centigrade		17	16
Thorium-230	pCi/l		0.3	0.0
Tin	mg/l		<0.005	<0.005
Total solids	mg/l		3730**	1870**
Uranium	mg/l		0.045***	0.001
Vanadium	mg/l		0.49	0.12
Zinc	mg/l		<0.005	<0.005

^aSamples not analyzed for certain constituents are indicated by --.

^bmg/l CaCO₃ - milligrams per liter as calcium carbonate; mg/l - milligrams per liter; micromhos/cm - micromhos per centimeter; pCi/l picocuries per liter; SU - standard units.

*Concentration exceeds Primary Drinking Water Standards.

**Concentration exceeds Secondary Drinking Water Standards.

***Concentration exceeds Health Advisory Level.

Table D.7.8 Background groundwater quality, middle hydrostratigraphic unit at Mexican Hat tailings site

Constituent	Observed concentration range ^a	Number of analyses	Mean	Two standard deviations	Calculated concentration range ^a
Alkalinity	59-140	7	110	58	52-168
Aluminum	<0.1-0.4	6	0.2	0.2	<0.1-0.4
Ammonium	<0.1-0.1	6	0.1	0.0	<0.1
Antimony	<0.003-0.007	6	0.004	0.004	<0.003-0.008
Arsenic	<0.01	7	0.01	0.0	<0.01
Barium	<0.1-0.2	7	0.1	0.01	<0.1-0.2
Boron	1.0-4.7	6	2.4	3.4	<0.1-5.8
Cadmium	<0.001	7	0.001	0.0	<0.001
Calcium	153-600	7	420	302	118-722
Chloride	38-240	7	167	164	3-331
Chromium	<0.01-0.07	7	0.04	0.04	<0.01-0.08
Cobalt	<0.05-0.07	7	0.05	0.02	<0.05-0.07
Copper	<0.02-0.05	7	0.03	0.03	<0.02-0.06
Cyanide	<0.01	6	0.01	0.0	<0.01
Electrical conductivity	2000-6000	7	4250	2986	1264-7236
Fluoride	1.2-1.5	7	1.4	0.2	1.2-1.6
Iron	<0.03-0.2	7	0.1	0.1	<0.03-0.2
Lead	<0.01-0.04	7	0.01	0.02	<0.01-0.02
Magnesium	47-200	7	161	104	47-265
Manganese	<0.01-0.06	7	0.03	0.04	<0.01-0.07
Mercury	<0.0002-0.0006	7	0.0003	0.0003	<0.0002-0.0006
Molybdenum	<0.01-0.2	7	0.07	0.2	<0.01-0.27
Nickel	<0.04-0.13	7	0.07	0.08	<0.04-0.15
Nitrate	<0.1-4	6	1.3	2.8	<0.1-4.1
Nitrite	<0.1	7	0.1	0.0	<0.1
Total organic carbon	16-31	6	22	11	11-33
pH	7.05-10.24	7	7.79	2.23	5.56-10.24
Phosphate	<0.1-0.2	6	0.1	0.08	<0.1-0.2
Potassium	5.6-11.1	7	9.0	3.8	5.2-12.8
Selenium	<0.005	7	0.005	0.0	<0.005
Silica	11-15	6	13	3	10-16
Silver	<0.01	7	0.01	0.0	<0.01
Sodium	340-1320	7	832	796	36-1628
Strontium	<0.1-0.4	7	0.1	0.2	<0.1-0.4
Sulfate	1190-4090	7	3004	2170	834-5174
Sulfide	<0.1	6	0.1	0.0	<0.1
Tin	<0.005	7	0.005	0.0	<0.005
TDS	1870-6550	7	4864	3476	1388-8340
Uranium	<0.003-0.05	7	0.016	0.045	<0.0003-0.05
Vanadium	<0.01-0.6	7	0.3	0.5	<0.01-0.8
Zinc	<0.005-0.188	7	0.03	0.14	<0.005-0.188

Table D.7.8 Background groundwater quality, middle hydrostratigraphic unit at Mexican Hat tailings site (Concluded)

Constituent	Concentration range mg/l	No. of analyses	Mean	Two standard deviations (25b)	Background concentration range (mg/l)
Gross alpha ^b	0-60 ± 28	6	21	50	0-71 ± 28
Gross beta ^b	1-120 ± 17	6	32	88	0-120 ± 17
Lead-210 ^b	0-2.3 ± 1.1	7	0.9	1.6	0-2.5 ± 1.1
Polonium-210 ^b	0-0.3 ± 0.7	7	0.07	0.3	0-0.4 ± 0.7
Radium-226 ^b	0-3.5 ± 0.4	7	1.3	2.9	0-4.2 ± 0.4
Radium-228 ^b	0-0.7 ± 1.0	7	0.5	0.4	0-0.9 ± 1.0
Thorium-230 ^b	0-0.3 ± 0.6	7	0.2	0.3	0-0.5 ± 0.6
Temp, degrees	16-19.5	7	17.6	2.2	15.4-19.8

^aConcentrations in milligrams per liter (mg/l) unless otherwise noted.

^bpCi/g = picocuries per liter.

Table D.7.9 Geochemical analyses of Colorado State University borings

Boring number ^a	Sample depth (ft)	pH	Electrical conductivity (μhos/cm)	Constituent ^b										CaCO ₃ equivalent (%)
				SO ₄	Fe	Ca	Mg	Na	Cl	Al	Mn	As	Se	
MH-200	2.5	8.1	1.8	3650	158	2660	782	323	130	812	252	4	3	10.4
MH-200	3.0	8.0	1.8	3140	130	2500	507	118	60	778	224	4	2	21.8
MH-200	3.5	8.1	2.0	4240	128	4260	473	161	60	675	183	4	4	19.3
MH-200	4.0	8.0	1.9	3870	110	3380	378	69	<20	781	159	4	4	21.8
MH-200	4.8	8.2	0.2	200	118	222	29	30	272	288	390	3	5	36.0
MH-200	6.0	8.6	0.1	111	84	131	21	16	<20	363	296	2	4	17.5
MH-200	10.0	8.6	0.2	270	115	217	53	41	<20	336	191	3	4	36.0
MH-200	15.0	8.6	0.1	67	106	114	30	49	<20	348	138	3	4	49.8
MH-200	19.0	8.5	0.1	134	82	155	35	29	<20	248	224	2	3	18.7
MH-205	5.5	8.5	0.2	173	112	167	65	35	<20	278	222	3	5	41.0
MH-205	6.0	8.3	0.8	160	215	1460	186	50	<20	325	610	2	3	29.6
MH-205	7.0	8.5	0.2	190	144	149	55	81	<20	306	228	3	2	36.4
MH-205	10.0	7.9	1.7	3050	96	2110	541	268	<20	931	194	2	<1	18.0
MH-205	12.5	8.1	0.2	300	78	105	60	29	30	609	89	1	1	11.5
MH-205	17.0	8.4	0.1	150	67	131	44	35	20	276	237	1	<1	12.8
MH-205	22.0	8.3	0.2	170	38	53	23	300	30	671	45	1	3	6.1
MH-205	25.5	8.6	0.1	155	87	130	37	36	20	277	193	13	<1	18.4
MH-208	27.0	6.4	2.0	5070	430	5470	177	96	<20	549	72	13	<1	0.84
MH-208	27.5	8.0	0.2	348	304	340	47	22	<20	253	271	4	6	34.4
MH-208	30.2	8.5	0.1	90	130	131	26	24	<20	263	231	4	6	32.0
MH-208	36.0	8.4	0.1	89	93	116	32	31	<20	260	208	2	3	31.4
MH-208	38.0	8.1	0.2	370	74	289	91	72	<20	845	128	2	<1	9.8
MH-208	43.0	8.6	0.1	67	119	111	32	35	<20	342	202	2	2	29.8
MH-208	49.0	8.1	0.2	120	52	124	47	98	<20	671	103	3	<1	18.0

^aBoring numbers currently do not have DOE identification numbers; CSU boring numbers represent NX rock core samples.

^bAnalysis done by solute extraction; values are expressed in milligrams per gram of dry soil; SO₄ - sulfate; Fe - iron; Ca - calcium; Mg - magnesium; As - arsenic; Se - selenium.

2000

D.8 SURFACE-WATER HYDROLOGY

D.8.1 GENERAL

The nearest occurrences of perennial surface water in the area of the tailings site are three sewage lagoons at the northwest corner of the site. One of these sewage lagoons contains approximately 475,000 gallons of untreated liquid wastes; the other two lagoons are available to receive overflow from the active sewage lagoon. Recharge to these lagoons is from the sewage discharge system serving the community of Halchita. The impounded wastes recharge the upper hydrostratigraphic unit by infiltration through the unlined pond base; no surface discharge from the pond is evident. Overflow from the pond drains northwestward toward the San Juan River via a small, unnamed arroyo.

The San Juan River represents the largest surface-water occurrence in the area. The river is approximately one air mile north of the tailings site and 240 feet lower in elevation.

The arroyos in the area of the site, Gypsum Creek being the largest, contain ephemeral occurrences of surface water. Several seeps have been reported at various times within these drainages, but, in general, surface water in the arroyos is confined to periods immediately following precipitation events.

A planned diversion dam will divert surface flows from the stabilized tailings pile into the existing arroyos, and these diverted flows will discharge into the San Juan River.

D.8.2 DRAINAGE AND FLOW

Gypsum Creek, an ephemeral stream, is located approximately 2000 feet east of the tailings pile. Due to the ephemeral nature of flows within Gypsum Creek, no records of flow within the creek exist. This creek is separated from the tailings site by a low ridge line and is incised approximately 100 feet below the elevation of the tailings site. The effects of flows within Gypsum Creek, in terms of affecting the tailings site, are considered negligible.

The site is drained by a network of arroyos which ultimately discharge into the San Juan River (Figure D.8.1). The tailings site is located approximately one mile south of the San Juan River, a large, perennial-flowing tributary of the Colorado River.

These networks of arroyos are similarly incised, ephemeral, and have no record of measured discharges. Due to the relatively small size of the upstream watershed, slightly greater than 360 acres, and the relatively deeply incised nature of the arroyos, the effects of flows within these arroyos upon the tailings site is likewise considered negligible.

20

D.8.3 FLOODING ANALYSIS

The Mexican Hat tailings site is approximately one air mile south of the San Juan River and 240 feet above the riverbed. The potential for floodwaters reaching the pile is considered negligible; therefore, a flood analysis is not required.

D.8.4 WATER QUALITY

Tailings site

The arroyos that drain the tailings site are normally dry with surface-water flows limited to those brief times immediately following rainstorms which occur sporadically within the drainage basin. Any surface water remaining in the arroyos is subject to evaporative concentration. Consequently, surface-water quality analyses would be misleading because these analyses would exhibit elevated concentrations of chemical constituents which are not representative of surface-water chemistry within the arroyos during times of flow. There are no records of surface-water quality analyses within the arroyos during times of flow.

Perennial flowing surface water is present within the San Juan River located approximately one mile north of the tailings site. The river receives sporadic discharges from the arroyos draining the area of the tailings site, at the confluence of Gypsum Creek and the San Juan River (see Figure D.8.1).

Water-quality analyses for samples withdrawn from the San Juan River, both upstream and downstream of the tailings site, are available from several sources. The U.S. Geological Survey (USGS) has collected water-quality data continuously since 1927 at a sampling station located on the south bank of the river, downstream from the confluence of the river and Gypsum Creek (Figure D.8.1). The USGS (1985) data indicate that the flow-weighted total dissolved solids (TDS) concentration averaged approximately 360 milligrams per liter over the 59-year period of record. The dominant ions are normally calcium and sulfate, except during spring runoff when bicarbonate and calcium dominate. The pH value of the water generally ranges from 7.5 to 8. The river is generally free of chemical constituents with concentrations in excess of EPA drinking water standards, although occasionally iron, manganese, and sulfate do occur in concentrations marginally to substantially above EPA drinking water standards. Due to the relatively large flow, and subsequent diluting effect within the river, it is unlikely that the sporadic discharges from the arroyos draining the area of the tailings have any measurable impact upon the chemical quality of the San Juan River.

Radiological water-quality data have been reported for samples collected with the San Juan River, both upstream and downstream of the tailings site. These data are found in reports compiled by the EPA (1973), Geochemistry and Environmental Chemistry Research (GECR, 1982),

and the DOE (see Section D.2). Sample locations are shown in Figure D.8.1 and water-quality analyses are reported in Table D.8.1. These data indicate that Ra-226 and total uranium concentrations measured upstream of the tailings site do not vary significantly from those concentrations measured downstream of the site.

Borrow sites

Due to the ephemeral nature of surface-water flows within the arroyos draining the areas of the borrow sites, no water-quality data are available for these locations. Even if available, water-quality analyses of water within the arroyos would not be indicative of stream-water chemistry, because of evaporative effects, unless the water-quality samples could be collected during periods of stream flow.

D.8.5 WATER USE

The Halchita community obtains all of its water from the San Juan River. This is accomplished by use of a Navajo Tribal Utility Authority (NTUA) pumping station and water treatment plant located on the south bank of the San Juan River below the point where Gypsum Creek empties into the river. The treated water is pumped, via an aqueduct, into three holding tanks located in the hills to the east of Halchita. The average production rate at the pumping station for the three-year period of May, 1982, to May, 1985, was approximately 9.74 million gallons per year.

FIGURE D.8.1

**MAJOR DRAINAGES AND SURFACE-WATER
SAMPLING LOCATIONS IN THE AREA
OF THE MEXICAN HAT TAILINGS SITE**

D.9 METEOROLOGICAL DATA

Meteorological data are essential in order to estimate the length of the construction season, plan construction dust and runoff controls, design long-term erosion control and proper cover for the stabilized tailings, and determine any special protection required for personnel or equipment.

D.9.1 WEATHER PATTERNS

The climate at the Mexican Hat site is of the semiarid desert type, with light precipitation, low relative humidity, and large ranges in daily and annual temperatures. The winters are cold but not usually severe, with Fahrenheit temperatures of zero degrees or below occurring only every second or third year. The summers are characterized by hot, dry weather with high temperatures in the 90s and low 100s (°F) (FBD and SNL, 1983). Wind directions depend on the local pressure gradient and terrain features, and average wind velocities are generally less than 10 miles per hour (mph). Thunderstorms with their accompanying lightning are relatively common, especially during the warmer months of the year, and winds associated with storm fronts or severe thunderstorms may occasionally exceed 60 mph but normally are between 30 and 40 mph. More severe weather events such as hailstorms and tornadoes are less frequent (Stevens et al., 1983).

D.9.2 WIND

A one-year wind record from the White Mesa Project at Blanding, Utah, shows an overall average wind speed of 9.2 mph (all directions) with the most frequent direction being from the south (Table D.9.1) (NRC, 1979). Blanding is approximately 35 air miles northeast of the site. Wind measurements at the tailings site in July and August, 1976, showed the wind blowing predominantly from the southwest at an average velocity of five mph with gusts up to 40 mph (FBD and SNL, 1983).

Atmospheric stability data for the Farmington, New Mexico, airport are provided in Table D.9.2. This station is approximately 90 air miles west-southwest of Mexican Hat; however, it is the closest station for which these data are available.

D.9.3 TEMPERATURE

Temperature data for the Mexican Hat site are provided in Table D.9.3. The average annual maximum and minimum temperatures are 71.9°F and 40.7°F, respectively. The average monthly maximum temperatures range from 43.4°F in January to 98.4°F in July, and the average monthly minimum temperatures range from 18.9°F to 65.2°F in the same months, respectively. Diurnal temperature ranges are wide, commonly 30°F or more (Stevens et al., 1983).

Table D.B.1 Radiological analyses of surface-water samples
collected in the area of Mexican Hat tailings site

Constituent	EPA1	EPA2	Location identification				
			938	939	954	955	961
Uranium, mg/l	--	--	0.0009	0.0050	0.0014	0.0017	0.0024
Ra-226, pCi/g	0.13	0.17	0.4 ± 0.2	0.2 ± 0.2	--	--	--

D.9.4 PRECIPITATION

Precipitation data for the Mexican Hat site are provided in Table D.9.3. The average annual precipitation is 6.09 inches, more than half of which occurs as rain during July through January. Approximately 60 percent of the annual precipitation occurs between the average freeze-free (32°F) dates. Snowfall averages 3.3 inches annually, and all of this occurs during November through February (Stevens et al., 1983). The 10-year 24-hour precipitation frequency value for the Mexican Hat area is 1.7 inches (Miller et al., 1973).

D.9.5 FROST

The average freeze-free (32°F) period at the Mexican Hat site is April 19 through October 23 or 190 days (Stevens et al., 1983). Data on the depth to the frost line at the Mexican Hat site are not available.

D.9.6 EVAPORATION

Estimated pan evaporation rates for the Mexican Hat site are provided in Table D.9.3. The estimated annual evaporation rate is 84.41 inches, most of which would occur during April through October (Stevens et al., 1983). Pan evaporation rates were measured at Mexican Hat during April through November of 1957 through 1976. The average monthly rates were as follows (Crowell, 1985a):

April	9.07 inches	August	12.68 inches
May	12.35 inches	September	9.48 inches
June	14.49 inches	October	5.49 inches
July	15.22 inches	November	2.34 inches

Reportedly, 75 percent of the evaporation at Mexican Hat occurs during April through October (DOC, 1968).

Based on the period 1946 to 1955, the mean annual lake evaporation at Mexican Hat is approximately 50 inches (DOC, 1968).

Table D.9.1 Wind speed at Blanding, Utah^a

Direction	Average speed (miles per hour)	Frequency ^b (percent)
N	8.5	5.9
NNE	8.7	7.4
NE	8.1	9.1
ENE	6.2	4.2
E	5.7	3.5
ESE	7.6	3.8
SE	8.0	5.7
SSE	5.8	5.1
S	9.4	9.6
SSW	11.9	7.9
SW	10.8	8.5
WSW	11.8	4.1
W	11.0	5.0
WNW	9.5	4.0
NW	10.9	9.4
NNW	<u>9.6</u>	7.1
All	9.2	

^aData for the one-year period of March 1, 1977, to February 28, 1978, at Blanding, Utah, adapted from NRC, 1979.

^bDoes not total 100 percent because of rounding.

Table D.9.2 Atmospheric stability distribution for the
Farmington, New Mexico, airport

Stability class	Stability description	Frequency (percent)
A	Extremely unstable	4.68
B	Moderately unstable	8.74
C	Slightly unstable	12.96
D	Neutral	15.87
E	Slightly stable	12.73
F	Moderately stable	45.08

Ref. DOE, 1984b.

Table D.9.3 Temperature, precipitation, and pan evaporation data for Mexican Hat, Utah^a

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual means or totals
Normal maximum temperature ^b	43.4	52.9	61.2	71.1	81.6	93.0	98.4	95.2	87.6	74.4	57.8	45.3	71.9
Normal minimum temperature ^b	18.9	24.8	30.1	38.4	48.1	57.2	65.2	63.1	52.8	40.1	28.7	20.0	40.1
Normal monthly precipitation ^c	0.50	0.43	0.38	0.31	0.35	0.19	0.66	0.65	0.54	0.96	0.51	0.61	6.09
Average monthly snow ^c	1.7	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.4	1.0	3.3
Estimated pan evaporation ^c	0.35	1.10	3.11	7.15	11.71	14.51	15.55	12.85	9.34	5.69	2.16	0.89	84.41

^aStation 5582, elevation 4120 feet above sea level; period of record 1951-1980.

^bDegrees Fahrenheit (°F).

^cInches.

Ref. Stevens et al., 1983.

D.10 LAND SURVEY

D.10.1 TOPOGRAPHIC SURVEY

The Mexican Hat site was surveyed by aerial photography by Olympus Aerial Surveys, Inc., on August 11, 1982. The resulting topographic map is shown in Figure D.10.1. Ground surveys were conducted by Western Design Consultants on August 11, 1982. No alterations to the site have occurred since the map was made. Elevations of the site are shown on two-foot intervals and are tied to datum point north-east corner, Section 7, T42S, R19E, Salt Lake base and meridian. The topographic map extends slightly beyond the site boundary.

D.10.2 LAND SURVEY

A boundary survey was conducted by Western Design Consultants of Salt Lake City, Utah, during July, 1982, and a map of the site boundary prepared on July 8, 1982. This map can be viewed at the DOE UMTRA Project Office, Albuquerque, New Mexico

D.10.3 AERIAL PHOTOGRAPHS

The following aerial photographs are available:

<u>Date</u>	<u>Scale</u>	<u>Elevation</u>
June, 1974	1" = 1215 feet	15,000 feet
June, 1974	1" = 435 feet	3000 feet
May, 1980	oblique	(view to the north)

D.10.4 OWNERSHIP AND EASEMENTS

The entire property is owned by the Navajo Nation with rights to the use of the site belonging to Utah Navajo Development Corporation (UNDC), Blanding, Utah. There are no known easements associated with the property. Questions concerning use and access should be directed to the Navajo Nation, Division of Resources, Window Rock, Arizona.

D.10.5 UTILITIES AND SUBSURFACE SURVEY

A subsurface survey of the site to locate water lines and buried utilities was made. Figure D.10.2 shows the location of sewer, water, telephone and power lines, and the septic tank on the site.

D.10.6 DRAINAGE STRUCTURES AND FEATURES

The tailings site occurs within the San Juan River drainage system. An unknown wash collects runoff from the site and flows into Gypsum Creek, a tributary of the San Juan River. The surface drainage characteristics for the site are shown in Figure D.3.9. Flow in the unknown wash and Gypsum Creek occurs only in response to excessive precipitation. The surface water of the tailings site is discussed in more detail in Section D.8, Surface-Water Hydrology, of this appendix.

FIGURE D.10.1
TOPOGRAPHIC MAP FOR THE MEXICAN HAT SITE

D.11. MISCELLANEOUS DATA

D.11.1 LAND USE

The Mexican Hat site is on the Navajo Reservation, and the land around the site is used for housing and low-density livestock grazing. The Navajo housing development of Halchita is just southwest of the tailings site, and the Bureau of Indian Affairs (BIA) recommends a stocking rate of 150 to 200 acres per animal unit per year (Roth, 1985). The Utah Navajo Development Corporation (UNDC) has the rights to use of the tailings site itself (FBOU, 1981). The Halchita Health Clinic was housed in the former mill office building within the designated tailings site boundary until it closed in October, 1985 (Clah, 1985).

The town of Mexican Hat is on the north bank of the San Juan River, off of the Navajo Reservation, approximately two road miles northeast of the tailings site. The non-Indian land in and around Mexican Hat is used primarily for residential, commercial, and livestock grazing purposes. Both the Indian and non-Indian lands around Mexican Hat contain numerous scenic attractions including Monument Valley Navajo Tribal Park, Goosenecks State Park, and Natural Bridges National Monument.

D.11.2 COMMUNITY SERVICES

Ambulance service is provided by the volunteer fire department in Mexican Hat. The 27-bed Monument Valley Hospital is approximately 26 road miles southwest of Mexican Hat, and there is a 36-bed hospital in Monticello, approximately 70 road miles northeast of Mexican Hat. Both of these hospitals are underused (FBD and SNL, 1983).

The Halchita area is regularly patrolled by the Navajo Tribal Police, which has 31 officers stationed in Kayenta, Arizona, approximately 45 road miles southwest of Mexican Hat. Police protection in the Mexican Hat area is provided by the San Juan County Sheriff's Office and the Utah Highway Patrol. Two sheriff's deputies are stationed at Bluff, 22 road miles northeast of Mexican Hat, and highway patrol officers who police U.S. Highway 163 are stationed at Blanding, 48 road miles northeast of Mexican Hat (FBD and SNL, 1983).

Fire protection in the area is provided by a volunteer fire department in Mexican Hat. Fire protection is limited because the equipment is antiquated. The Halchita housing development has fire hydrants but no fire fighting equipment or personnel (FBD and SNL, 1983).

There are elementary schools in Halchita and Bluff. The closest high schools are in Gouldings and Blanding. Student enrollment and capacity data for these schools are provided in Table D.11.1.

There are no sanitary landfill facilities in San Juan County because there are not adequate funds and equipment for maintenance. There are open dump facilities at Mexican Hat, Bluff, and Monument Valley; however, there is no detailed information on the locations and capacities of these dumps (Montague, 1985).

D.11.3 UTILITIES

Domestic water is supplied to Halchita from a complete water treatment facility on the San Juan River just north of Halchita. The facility is operated by the Navajo Tribal Utility Authority (NTUA) and has a production capacity of 0.43 million gallons per day with a storage capacity (three tanks northeast of Halchita) of 125,000 gallons (Ball, 1985; May, 1985). Between 1982 and 1985, the annual output of the treatment facility has ranged from approximately eight to 11 million gallons (Nelson, 1985). Domestic water for Mexican Hat is supplied from a well between the town and the San Juan River. The well is a converted oil exploration well, and the water is not treated. No other information is available for this water supply system (Ball, 1985).

The Halchita sewer system is operated by the NTUA and consists of two lagoons located within the designated tailings site boundary. Currently, only one lagoon is utilized at less than full capacity. A new sewer system was installed in Mexican Hat in 1984. Reportedly, the two-cell lagoon can accommodate 100 sewer connections (FBD and SNL, 1983) but only 40 connections are presently in service (Ball, 1985).

Telephone service in Halchita is provided by the Navajo Communications Company Inc. in St. Michael's, Arizona. Continental Telephone Company of the West in Moab, Utah, provides the same service in Mexican Hat. There is no natural gas service in Halchita and Mexican Hat, but bottled propane service is available from Dooxol Propane in Kayenta, Arizona. Electricity in Halchita is provided by the NTUA in Kayenta, and there is a NTUA substation at the designated tailings site. Electricity is provided to Mexican Hat by Utah Power and Light Company in Blanding.

D.11.4 TRANSPORTATION

Bluff, Blanding, Montezuma Creek, and Monument Valley have airports capable of serving small aircraft, and there is a dirt landing strip in Mexican Hat. Skywest, a commuter airline, serves Blanding. There is no railroad service in San Juan County. Monticello has Continental Trailways commercial bus service (FBD and SNL, 1983).

U.S. Highway 163 courses north-south through San Juan County connecting Monticello, Blanding, Bluff, Mexican Hat, Halchita, and Monument Valley (Figure D.11.1). Average daily traffic data for this

paved, two-lane highway are provided in Table D.11.2. Indian Service Route 6440 joins U.S. Highway 163 at Halchita and courses southeast into Arizona. There are no traffic data for this dirt road.

D.11.5 ENVIRONMENTALLY-SENSITIVE ISSUES

The area around the Mexican Hat site contains numerous unusual rock formations (e.g., Monument Valley Navajo Tribal Park), natural rock bridges and arches (e.g., Natural Bridges National Monument), the winding, deeply incised San Juan River (e.g., Goosenecks State Park), and numerous Indian ruins (e.g., Hovenweep National Monument). Tourism is therefore an important sector of the local economy. An archaeological survey of the designated tailings site and surrounding area did not locate any significant cultural resources, and archaeological clearance has been recommended for the area (CASA, 1983).

FIGURE D.11.1
MEXICAN HAT ROAD MAP

Table D.11.1 School enrollment and capacity data

School	Student enrollment (October 1, 1984)	Student capacity (January 1, 1985)
Mexican Hat Elementary, Hatchita	257	201
Bluff Elementary, Bluff	110	196
Monument Valley High, Gouldings	275	396
San Juan High, Blanding	535	679

Ref. Heltman and Wiltsey, 1985.

Table D.11.2 Average daily traffic, U.S. Highway 163

	1983	1984
Blanding	1825	not available
Bluff	935	950
Mexican Hat	750	760
Halchita intersection	725	735
Arizona-Utah border	700	710

Ref. UDOT, 1983; Hanshaw, 1985.

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