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CHEMISTRY AND METALLURGY RESEARCH FACILITY REPLACEMENT (CMRR) PROJECT
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LIST OF ACRONYMS AND ABBREVIATIONS

amsl	above mean sea level
ASR	Alkali-silica reactivity
ASTM	American Society for Testing and Materials
ATT	Advanced Terra Testing
AUX	Auxiliary Mechanical
BCR	Baseline Change Request
bgs	below ground surface (at time of measurement)
CAT	category
CAT I	Security Category I
CAT II	Security Category II
CAT I Facility	CAT I nuclear facility
CDR	conceptual design report
CHS	crosshole boring
CMR	Chemistry and Metallurgy Research Facility
CMRR	Chemistry and Metallurgy Research Facility Replacement
CSR	cyclic stress ratio
CSS	cyclic simple shear
DBE	design base earthquake
DCH	Duncan-Chang Hyperbolic
DCN	document control number
DFR	daily field report
dim	dimensionless
D_{min}	damping ratio at small strains
DMJM H+N	Daniel Mann Johnson & Mendenhall, Holmes + Narver (formerly Holmes+Narver Raytheon (HNR))
DOE	Department of Energy
DSC	deep seismic characterization
DSHA	deterministic seismic hazard analysis
El	elevation, in feet above mean sea level
FCR	field change request
FLAC	Fast Lagrangian Analysis of Continua
ft	feet
ft/sec	feet per second
G	Shear modulus
γ_r	reference strain
GDR	geotechnical data report
GER	geotechnical engineering report
gINT	geotechnical boring log and laboratory database software
G_{max}	Maximum Shear Modulus at Low-Strain
G/G_{max}	Ratio between G and G_{max}
GMF	Guaje Mountain Fault
GM-GP	silty gravel to poorly graded gravel
GSI	geological strength index
G/SIP	geotechnical and seismic investigation plan
H-B	Hoek-Brown

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HQ	H-diameter of core barrel, Q-wireline drilling method
HSA	Hollow Stem Auger
I.D.	inside diameter
INP	Integrated Nuclear Planning
KA	Kleinfelder, Inc.
Km/sec	Kilometers per second
K_o	the coefficient of earth pressure at rest
KSL	Kellogg, Brown & Root, Inc.; Shaw Environmental and Infrastructure, Inc.; and Los Alamos Technical Associates, Inc.
LAB	laboratory
LANL	Los Alamos National Laboratory
LCM	loss control materials
LIDAR	light detection and ranging
LLOB	Light Lab Office Building
M	meters
Ma	million years ago
MAIN	main laboratory building consisting of AUX, LAB and SNM
M-C	Mohr-Coulomb
NGA	next generation attenuation
NGVD	National Geodetic Vertical Datum
NMDOT	New Mexico Department of Transportation
NQA-1	Nuclear Quality Assurance Level-1 Standard
ORIGINAL BASE MAP	topographic base map
pcf	pounds per cubic foot
PF	Pajarito Fault
PF-4	Plutonium Facility at TA-55
PGA	peak ground surface acceleration
PI	plastic index
PIC	person in charge
PIDAS	Personnel Intrusion Detection System
psf	pounds per square foot
PSHA	probabilistic seismic hazard analysis
PUMP	fire pump enclosure
PVC	poly-vinyl chloride pipe
PW	drill casing size designation
QA	quality assurance
Qbo	Otowi Member
Qbt1g	Unit 1g of the Tshirege Member
Qbt1v	Unit 1v of the Tshirege Member
Qbt2	Unit 2 of the Tshirege Member
Qbt2-t	Transition layer above Qbt2
$Qbt3_L$	Lower Unit 3 of the Tshirege Member
$Qbt3_L$-t	Transition layer above $Qbt3_L$
$Qbt3_U$	Upper Unit 3 of the Tshirege Member
Qbt4	Unit 4 of the Tshirege Member
Qct	Cerro Toledo interval
RC/TS	resonant column and torsional shear

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RCF	Rendija Canyon Fault
R/H	receiving and handling level
RLUOB	Radiological Laboratory, Utility, and Office Buildings
RMR	rock mass rating
σ_{ci}	unconfined compressive strength
S&L	Sargent and Lundy, Inc.
SASW	spectral analysis of surface waves
SBT	special block test
SIC	seismically induced compaction
SM	silty sand
SNM	special nuclear materials
SOP	standard operating procedure
SP	poorly graded fine sand
SPT	standard penetration test
SSC	shallow seismic characterization
SSI	soil structure interaction
SWHS	Site Wide Hazard Study
TA	technical area
τ_{max}	asymptotic shear strength
TRUCK	truck access area
UCB	University of California, Berkeley
UC	unconfined compressive
UPDATED BASE MAP	updated topographic base map
URC	Unconfined, free-free resonant column
USCS	Unified Soil Classification System
UT	University of Texas at Austin
V_s	shear-wave velocity
V_p	compression-wave velocity
WBS	Work Breakdown Structure
WCFS	Woodward-Clyde Federal Services
WDC	Water Development Corporation
XRD	X-Ray Diffraction

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I. INTRODUCTION

A. General

This Geotechnical Engineering Report (GER) presents the results of the geotechnical and seismic investigation plan (G/SIP) performed for the proposed Chemistry and Metallurgy Research Facility Replacement (CMRR) project. The project is located at Technical Area (TA) 55 within Los Alamos National Laboratory (LANL) in Los Alamos, New Mexico. The GER is one of two companion reports prepared by Kleinfelder, Inc. (KA) under Contract No. 13568-109-02-CX, Task No. 109 with Daniel Mann Johnson Mendenhall Holmes and Narver (DMJM H+N).

The following report was originally submitted in draft form on June 30, 2006 and was designated as Rev C. LANL personnel reviewed the Rev C report and provided comments. The Rev D report was submitted on March 21, 2007 as a final draft and incorporated revisions based on the comments provided by LANL on the Rev C submission. The Rev. D report was accepted by LANL. However, this version, Rev. 0, is submitted as a QA-Valid document and includes only minor modifications from the Rev. D document.

The Rev C draft report addressed the specific facility layout and features as defined in General Arrangement (GA) drawings dated March 3, 2006. Since submission of the Rev C version of the report, some of the facility layout and features of the project have been modified; those modifications are reflected in revised GA drawings dated October 6, 2006. However, to maintain continuity with the review comments, LANL has directed that this report (Rev 0 version) and the final report be revised based on March 06 GA drawings. As such, there are certain references and discussions presented in this report that are no longer relevant based on the revised GA drawings because some elements of the project have either been entirely eliminated or modified. Resulting discrepancies that may arise due to the facility changes reflected in the revised GA drawings will be addressed during preliminary (Title I) and final (Title II) design.

This report summarizes the results of the investigations, engineering analyses and interpretation of data, and provides geotechnical recommendations for design and construction of the facility. The companion report entitled "*Geotechnical Data Report, Chemistry and Metallurgy Research Facility Replacement (CMRR) Project, Los Alamos National Laboratory, Los Alamos, New Mexico,* DCN 19435.10528.5-ALB06RP001", herein referred to as the GDR, presents the geotechnical data collected during the CMRR project and utilized in the interpretations, analyses, and recommendations presented herein.

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This report was prepared to satisfy the requirements of, and subsequent modification to, the G/SIP which was originally submitted in the document entitled, *"Geotechnical/Seismic Investigation Plan, CMRR Project, TA-55, Los Alamos National Laboratory, Los Alamos, NM, 32013-006-02, Revision 0"* dated October 8, 2002. Modifications to the G/SIP are summarized in Table I-1, G/SIP Revisions.

B. Project Description

The CMRR will replace the existing Chemistry and Metallurgy Research (CMR) Facility, located at TA-3, with a new facility located at TA-55 as shown on Figure I-1, Site Location Map. A CMRR facility-planning layout was generated by LANL to assist with development of the preliminary project scope and the conceptual design report (CDR). Based on LANL's original concept, three buildings were planned for the CMRR facility; these included a Security Category I (CAT I) building, a Security Category II (CAT II) building, and what was initially called the Light Lab Office Building (LLOB). Over the project lifetime, the facility arrangement has changed and now includes a total of two buildings. These are the CAT I Nuclear Facility (CAT I Facility) and the Radiological Laboratory, Utility, and Office Building (RLUOB). The current arrangement of the proposed buildings at the site is shown on Figure I-2, CMRR Facility Layout, which is based on DMJM H+N's Drawing No.GA1002.dwg, dated 3/16/06. Descriptions of each building and appurtenant structures are presented in the following sections of the report. Building layouts and configurations might change from those described in the following sections.

1. RLUOB

DMJM H+N completed the conceptual design for the RLUOB in 2004. LANL elected to utilize a design-build approach to advance the RLUOB through Title I/II (Preliminary/Final) design and Title III (construction). In preparation for that design-build approach, KA submitted the report entitled, *"Geotechnical Investigation, Radiological Laboratory, Utility and Office Building, Chemistry and Metallurgical Research Replacement Project, Los Alamos National Laboratory, Los Alamos, New Mexico*; Kleinfelder Project No. 19435, Contract No. 13568-109-02 CX, Task No. 109; DCN ALB04RP001 Rev. 2", dated March 4, 2005. The RLUOB geotechnical report presented the site characterization, geotechnical engineering design, and construction recommendations for that facility. The report for the RLUOB project was included by LANL in the design-build bid package.

The current configuration of the RLUOB as shown on Figure I-2 includes a main building roughly in the shape of an "L". This building has maximum plan dimensions of approximately

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175 by 305 feet (ft). At the southwest corner of the main building is a utility area, which is roughly square in shape with maximum plan dimensions of about 120 ft on a side. A tunnel will connect the RLUOB facility to the CAT I Facility as shown on Figure I-2. A separate utility duct exits the northwest corner of the utility area and crosses westerly to the south corner of the CAT I Facility as shown on Figure I-2.

The design-build contract for the RLUOB was awarded by LANL in late 2005. The RLUOB plans were is completed by others without KA participation and more refined details of the building configuration were not available to KA in the development of these plans. The KA (2005) report does not address additional interpretation, analyses, or geotechnical recommendations for the RLUOB based on new design information.

2. CAT I Facility

The CAT I Facility of CMRR includes the Main Laboratory Building (MAIN) and the adjacent Fire Pump Enclosure (PUMP). The MAIN includes three primary elements designated as laboratory (LAB), special nuclear materials (SNM), and auxiliary mechanical (AUX). The MAIN is essentially rectangular in shape with maximum plan dimensions of approximately 295 by 325 ft The MAIN contains all of the functions associated with the mission of the CMRR facility and includes three primary levels: the basement, mezzanine, and laboratory. These various levels include the following:

- Storage vaults for the SNM,
- Laboratory and machining areas,
- Mechanical equipment rooms,
- Air handling and filter equipment,
- Waste handling areas,
- Communications and security operations,
- Staff changing rooms and lavatories,
- Decontamination rooms,

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- Passenger and freight elevators and pedestrian stairwells,
- Material handling vestibules,
- Exhaust stack, and
- Access to tunnels between adjacent facilities.

The majority of the MAIN is below grade except for the at-grade receiving and handling level (R/H) and above-grade AUX elements which are generally limited to the south and east sides of the structure, respectively. Within and around the SNM element, the facility includes retained earth-fill berms for confinement and projectile protection. Similarly, the R/H level is protected to the south by a retained earth-fill berm. The finish floor of the MAIN will be at approximately elevation (El) 7241. At each of the proposed elevators within the MAIN, the finish floor will be at approximately El 7235. A waste sump, about 30 ft by 30 ft in plan dimension, is also proposed near the center west end of the MAIN that will have a finish floor at approximately El 7231. The basement floor of the MAIN will generally be 3 feet thick.

The CAT I Facility also includes the PUMP building. The PUMP, approximately 170 ft by 40 ft, will be located adjacent and west of the MAIN. The PUMP will contain electric and diesel fire pumps and fuel tanks, standby diesel generators and fuel tanks, and fire water tanks. The PUMP will be a heavily reinforced concrete structure with two levels. The maximum plan dimensions will be approximately 38 ft by 179 ft This building will have a finish floor elevation at approximately El 7263.

Several large reinforced concrete retaining walls will be utilized around the CAT I Facility to support grade changes and provide confinement for the earth-filled berms. A relatively large reinforced concrete retaining wall, about 20 to 25 feet high, will be situated between the R/H and PUMP areas to support grade differential. The retaining walls supporting the earth berm along the south side of the MAIN will be of similar height.

Truck-compatible access roads will be provided around the entire CAT I Facility. On the south side, the access road will be essentially flat and finished at a grade of approximately El 7277. On the east and west sides, the access roads will slope downward from north to south at grades of about 4.25 and 6.0 percent, respectively. Along the north side, the access road will slope from a high of about El 7295 near the west edge downward to the east to about El 7290.

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On the south side of the CAT I Facility, south of the access road, another retaining wall will support the outer fence of the Personal Intrusion Detection System (PIDAS).

3. Tunnels

Two pedestrian tunnels and one utility duct will be constructed as part of the CAT I Facility. One of the pedestrian tunnels will exit from the northwest corner of the MAIN at approximately El 7281. This tunnel will be approximately 8 ft wide by 9 ft high will extend westerly, and connect with an existing tunnel stub from the plutonium facility PF-41. The newly constructed portion of tunnel will be approximately 162 ft long.

The second pedestrian tunnel will connect the east side of the MAIN with the west side of the RLUOB. This tunnel will be approximately 12 ft wide by 9 ft high and have a finish floor at approximately El 7269. The top of the tunnel will be at approximately El 7279 and will support about 11 ft of soil cover. This tunnel will cross beneath the planned PIDAS fencing that separates the CAT I Facility from the RLUOB.

A utility duct will also be constructed from the southeast corner of the MAIN to the west side of the RLUOB utility area. This duct will be approximately 8 ft wide by 3 ft high and its finish floor elevation will be about El 7277 where it enters the MAIN. This duct will also cross beneath the planned PIDAS fencing.

4. Proposed Excavations

A plan showing proposed excavation limits has been developed by Sargent and Lundy, Inc. (S&L) and included herein as Figure I-3. The plan shows a nominal excavation grade of El 7238 and includes 30- to 50-ft wide access corridors between the perimeter of the MAIN and the toe of the excavation. S&L's excavation plan presumes the use of cut slopes of 1:2.75 (horizontal and vertical) with a small section of vertical excavation at the northeast corner of the site.

C. Purpose and Scope

The purpose of the G/SIP was to characterize the subsurface soil and rock conditions at the site of the proposed project. The scope of the site characterization was originally developed to provide the basis for geotechnical-related foundation and structural design as well as to provide input parameters (strain-compatible soil properties and three-component time histories) for use in soil structure interaction (SSI) analyses. Because of institutional and project delays, LANL and the Department of Energy (DOE) determined that a sequenced ten-year-cycle project, the

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Probabilistic Seismic Hazard Analysis (PSHA), would be tasked with developing the input for the SSI analyses; therefore, portions of that scope of the work were withdrawn from KA and are currently being performed by others using relevant data, provided by KA, that are also detailed in later sections of this report. The revised G/SIP scope of work was to provide:

- Subsurface profiles of geologic units based on stratigraphic and geomechanical properties,
- Static engineering properties of the intact subsurface materials and compacted structural fill materials including unit weight, unconfined compressive strength, shear strength parameters, modulus of elasticity, modulus of subgrade reaction, Poisson's ratio, earth pressure coefficients, and coefficient of friction,
- Intact rock mass properties including rock mass rating (RMR), geological strength index (GSI), rock mass moduli of elasticity, modulus of vertical subgrade reaction, and rock mass quality material constants,
- Baseline dynamic properties and profiles to be used in the PSHA to develop strain-compatible soil properties and three-component time histories,
- Evaluation of suitable foundation types and design parameters,
- Bearing capacity values for intact rock and compacted structural fill materials,
- Assessment of the potential for seismically-induced compaction of soil-like foundation materials encountered at the site,
- Estimates of tunnel loading and backfill and surcharge pressures,
- Total and differential settlement criteria and deflection estimates for spread footing and mat foundation elements,
- Recommendations for ground improvement to increase bearing capacity and to mitigate unsuitable subsurface conditions, if needed,
- Construction recommendations for site preparation and clearing, excavation, temporary excavation slopes and support, subgrade preparation, suitable fill material, fill placement and compaction, construction observation, and compliance testing, and

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- Site drainage and moisture protection recommendations to mitigate erosion and surface water infiltration into below-grade facilities.

The activities performed to develop these properties and recommendations included the following:

- Review of published and unpublished reports related to the site and/or characteristics of the subsurface materials encountered at the site,
- Subsurface exploration using drilled borings and excavated test trenches,
- Subsurface *in-situ* seismic velocity measurements using downhole, suspension, and crosshole methods,
- Near-surface resistivity measurements to provide data related to corrosion potential and electrical grounding requirements,
- Static and dynamic geotechnical laboratory testing to develop physical and engineering properties of soil and rock and to evaluate the potential for deleterious impacts to concrete foundations, and
- Engineering analyses to interpret the site characterization data and to develop foundation and geotechnical-related structural design and construction criteria.

D. Limitations

The data, interpretation, findings, and recommendations contained in this report were developed during field explorations and laboratory tests authorized and approved by LANL. It is anticipated that subsurface conditions may vary from those indicated by the information presented in this report. The nature and extent of variations may not be evident until construction occurs.

This report was prepared in accordance with generally accepted standards of practice at the time the report was written. No warranty, expressed or implied, is made. It is the client's responsibility to see that all parties to the project, including the designer, contractor, subcontractors, and other authorized users are made aware of this report in its entirety. The use of information contained in this report for design and construction-bidding purposes should be done at the user's option and risk.

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Other standards or documents referenced in this report, or otherwise relied upon by the authors of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference" as that latter term is used relative to contracts or other matters of law.

This report may be used only by the client and their designees only for the purposes stated..This report is based on the S&L design drawings of March 3, 2006. Design drawings, land or facility use, site conditions (both on- and off-site), regulations, or other factors may change over time and may affect the applicability or relevance of the information contained in this report. Therefore, additional work may be required with the passage of time.

Any third party, other than the client or their designee, who wishes to use this report, shall notify KA of such intended use. Based on the third party's intended use of this report, KA may require that additional work be performed and that an updated report be issued. Unauthorized use of this report will release KA from any liability associated with this report.

E. Organization of the Report

Following the foregoing introductory material, this report includes a site description (Chapter II) and summary of previous studies (Chapter III) as background for the subsequent chapters. Chapter IV describes the investigations, both field and laboratory, performed for the CMRR project. Geologic conditions determined from both previous studies and CMRR investigations are discussed in Chapter V. The static properties and dynamic properties of site geomaterials, determine by analyses of investigation data, are described in Chapters VI and VII, respectively. Chapter VIII addresses the geomechanical model of the site that is used as the framework for geotechnical analyses described in Chapter IX. Recommendations concerning the geotechnical aspects of the project are provided in Chapter X. References used in the development of this report are presented in Chapter XI.

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II. SITE DESCRIPTION

This section of the GER describes the basis for site elevations and coordinates, the general site description relative to regional site features, and the site features at the CAT I Facility.

A. Elevations and Horizontal Control

All elevations used in this report were provided by LANL and were measured by LANL's maintenance and operations contractors. All elevations are based on the 1929 National Geodetic Vertical Datum (NGVD). The locations of benchmarks used for vertical control are shown on Figure I-2. Horizontal control is based on the New Mexico State Plane Coordinate system. The project baseline is Northing 1,769,000 and Easting 1,625,000.

B. General

CMRR will be located within LANL approximately 26 miles northwest of Santa Fe, New Mexico (Figure I-1). It is accessible from Santa Fe by US285 to NM 502 to NM4 to Pajarito Road. The site is on Pajarito Mesa at TA-55, between the PF-4 facility and Pajarito Road (Figures I-1 and I-2). The center of CMRR will be at approximately N1769120, E1624740, New Mexico State Plane Coordinate System and 106.30W longitude, 35.86N latitude at an elevation of approximately 7295 ft above mean sea level (amsl) as shown in Figure II-1, Site Investigation Plan.

C. Existing Site Features

The CMRR CAT I facility will be constructed south and east of the existing PIDAS in an area previously used as a parking lot (Figure I-2). The parking lot was constructed in 2002 by placing fill over a south-sloping original ground surface, creating a nearly flat, gravel-surfaced lot. The fill covers the old Pajarito Road as well as buried utilities.

D. Site Plan Changes

The original field work for this report began in November of 2002. Over the lifespan of the project a number of different site facility and topographic plans have been provided and used by Kleinfelder to develop site investigation and boring location plans. The current site investigation plan incorporates the most recent topographic information provided by LANL. However, the vertical elevation data collected by LANL after drilling of these borings may not match with contour elevations where these borings are shown in X-Y space on the current site investigation plan. The apparent variances are subject to resolution by LANL. In most cases the variation is less than 0.5 feet.

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III. PREVIOUS STUDIES

A number of previous geologic, geotechnical, and seismic studies have been conducted at or near the TA-55 site. None of these other previous studies was performed under Nuclear Quality Assurance Level-1 (NQA-1) requirements standard; therefore, results of these previous studies are not incorporated in this current study. However, these previous studies were reviewed and their findings, as applicable, were used in planning the investigation and performing the analyses for the GER.

Kleinfelder performed two other CMRR-related studies in accordance with the NQA-1 requirements. They include:

- *Geotechnical Investigation, Radiological Laboratory, Utility and Office Building, Chemistry and Metallurgical Research Replacement Project, Los Alamos National Laboratory, Los Alamos, New Mexico*; Rev.2, Contract No. 13568-109-02-CX, Task No. 109, and
- *Geotechnical Data Report, Special Block Test, Chemistry and Metallurgical Research Replacement Project, Los Alamos National Laboratory, Los Alamos, New Mexico,* Kleinfelder Project No. 19435 Rev. A, November 14, 2005.

These two other CMRR-related studies were planned and conducted in response to modifications of the G/SIP and to address specific CMRR program objectives. They contain some of the same investigations and data that have been subsequently incorporated in the GDR and this GER.

A. Topographic Maps

At the project initiation, a topographic base map (CONT02.dxf) was provided to KA to evaluate site features and conditions that would impact planning for the subsurface investigation. This original base map depicted topographic features at the site that were subsequently altered by placement of fill and construction of an at-grade parking area. A later topographical aerial survey was performed for the site, and the results of that survey were provided to KA in an updated topographic base map (NEW CONTOURS.dwg, dated 9/24/05) that is used as the basis for locating the work described in this report. The information provided includes surface topography, buildings, roads, surface and underground utilities. The topographic contours have subsequently been revised for the project site plans, based on a drawing provided by LANL

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(ACAD-2000-CMRR-topo.dwg, dated 10/9/06), because it is more accurately comparable to the completed boring spot elevations.

B. LIDAR Mapping Survey

LANL also provided to KA a series of digitally enhanced images produced from light detection and ranging (LIDAR) mapping surveys. Five separate images were provided, each with the title *"TA-55 Area With ½-Mile Radius"*. The images generally show an oblique aerial view of the TA-55 area. The LIDAR maps were evaluated to identify visible site geologic features. The LIDAR maps are not included in this report but may be obtained from LANL.

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IV. INVESTIGATION PROGRAM

The original investigation plan for CMRR was described in the G/SIP and included the following general activities:

- Field Investigation
  - Geologic Reconnaissance
  - Subsurface Investigation
  - *In-Situ* Testing

- Laboratory Testing
  - Conventional Index and Classification Test
  - Conventional Static Geomechanical Testing
  - Specialized Static Geomechanical Testing
  - Specialized Dynamic Geomechanical Testing
  - Geochemical Testing

Over the course of the project, the investigation plan has been modified several times to respond to findings, program changes, funding directives, and investigation challenges. These modifications resulted in a split-phased approach to the original investigation, the addition of several new phases of work, changes in the methods of investigation, and changes in the sequence of project deliverables. A summary of the LANL-authorized modifications to the G/SIP is presented in Table I-1.

Field investigation and laboratory testing were prime activities of the G/SIP, generating high quality (ASME NQA-1) data and related to the static and dynamic geomechanical properties of the subsurface materials. The following section of the report summarizes the various field investigation activities performed, the applicable LANL work directives, the final arrangement of field investigation phases, and how the accumulated field data were distributed among the various reports associated with the CMRR project. Table IV-1, Field Investigation Activities Performed for CMRR, provides a listing of all the field activities that were performed and is intended to help the reader of this report understand the evolution of the project. Following the Field Investigation Summary are sections detailing the field activities and the laboratory testing.

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A. Field Investigation Summary

The original purpose, objectives, and work plan for the CMRR field investigations were described in detail in the G/SIP. The original G/SIP designated a total of 51 borings and four test trenches to be performed to characterize the subsurface conditions for all originally planned building elements including the CAT I and CAT II nuclear structures as well as the LLOB. Under the G/SIP work plan, all field investigations were to be performed in a single effort covering approximately 11 months.

KA performed the geologic reconnaissance and most of the subsurface investigations during the period of November 12, 2002 to April 7, 2003. However, a number of factors including winter weather, difficult drilling, and poor sample quality hindered the fieldwork and prevented it from being completed on the original schedule. In April 2003, KA received a stand-down order, which resulted from Federal Government funding shortfalls. This stand-down order suspended all field investigation activities before the deep seismic characterization borings (DSC) could be completed. The field investigation activities completed through April 2003 are referred to as phase one.

Subsequent to the stand-down order, LANL directed KA to develop a scope of work to characterize a potentially weak layer of volcanic tuff, Lower Unit 3 of the Tshirege Member ($Qbt3_L$). This layer, $Qbt3_L$, was encountered during the phase one fieldwork, but high quality samples could not be obtained. A scope of work was developed in early 2004, and this work, called the Special Block Test (SBT) program, was initiated in May 2004. The phase-SBT program focused on collecting large block samples of $Qbt3_L$ from nearby off-site outcrops, performing geomechanical laboratory testing on smaller samples from the blocks, performing additional field surface testing at or near the CMRR site, and comparing the results to establish the basis for demonstrating uniformity of local geology. The purpose of the SBT program was to independently develop a set of geomechanical properties of the $Qbt3_L$ should future borings at the CMRR site be unsuccessful in collecting high-quality, undisturbed, testable samples of the $Qbt3_L$. The results of this phase of work (phase SBT) are presented in the referenced SBT report.

In September 2004, after initiation of the phase SBT and after revisions were made to the original G/SIP by means of the field change request (FCR) procedure, KA began the effort to complete the originally-planned borings and to employ supplemental procedures to collect samples of the $Qbt3_L$. This element of work is referred to as the phase two and includes all

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remaining field activities listed in the original G/SIP. Phase two fieldwork was substantially complete in April 2005.

In October 2005, LANL directed KA to perform additional seismic velocity measurements by means of the crosshole method. KA completed the fieldwork on January 14, 2006. This work is referred to as the phase crosshole and includes drilling and sampling of three additional borings and performing crosshole seismic velocity measurements. The results of this phase of work were presented in a report by University of Texas at Austin (UT), Geotechnical Research Center (Dr. Kenneth Stokoe, P.E.), entitled "Seismic Crosshole Testing at Los Alamos National Laboratory, CMRR Site," Report No. GR06-3, May 12, 2006 which is included in the GDR.

The four phases of geotechnical fieldwork performed for the CMRR project are summarized in Table IV-1, which includes the following information:

- Specific field investigation activities performed for overall CMRR project - geologic reconnaissance, subsurface investigations (borehole drilling, logging, and sampling; test trench excavation and logging); *in-situ* measurements; and SBT Program,
- Document reference in which the field investigation activities were either specified or revised (G/SIP, FCR, or SBT Proposal),
- Phase of work in which the field investigation activities were performed (One, Two, SBT, or Crosshole), and
- Report in which the field investigation activities identified were utilized (RLUOB, SBT, GDR/GER).

With the project changes noted in Section I.B of this report, the geotechnical investigation for the newly designated RLUOB has been submitted previously as identified in KA, 2005a. All of the data presented in the RLUOB report were generated in phase one. SBT field and laboratory work is addressed in KA, 2005b. The SBT report includes all data generated from the SBT fieldwork as well as some additional experimental laboratory test data generated from samples collected during phase two. Data collected in phase one and phase two that are relevant to the CAT I Facility are presented in the GDR and used as the basis for analyses, interpretations, and recommendations presented in this GER.

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B. Details of Field Investigation

Field investigation of the CAT I Facility included a variety of techniques including geologic reconnaissance, subsurface investigations, and *in-situ* measurements. Locations where each of the various field investigation methodologies was utilized are shown on Figures I-2 and II-1 Details of each of the methodologies performed are presented in the following sections.

1. Geologic Reconnaissance

The geological reconnaissance originally planned in the G/SIP was replaced by a general site walk-over in the company of LANL geologists. LIDAR imagery provided by LANL was examined but provided no information relevant to the G/SIP objectives. Aerial photography was not available for examination.

2. Subsurface Investigations

Subsurface investigations were performed to investigate the geotechnical conditions at the site and to facilitate *in-situ* testing. Forty-five (45) borings were drilled for the CAT I Facility and included general site borings, shallow and deep seismic characterization borings, crosshole borings, and alternate area borings. Six test trenches were excavated to characterize the near-surface geology and geologic structure within the CAT I Facility area. The following sections of the report summarize the means and methods used for subsurface investigations. Figures I-2 and II-1 show the locations of these subsurface investigations along with the ground surface elevation and plan coordinates.

a) General Site Borings

Twenty-five (25) general site borings were drilled within and around the CAT I Facility building area. General site borings within the CAT I Facility building area were drilled to a nominal depth of 100 ft below ground surface existing at the time of drilling (bgs), and those outside of this area (nearby RLUOB borings that are relevant to the CAT I Facility) were drilled to a nominal depth of 50 ft bgs. All 25 general site borings were advanced with a truck-mounted drill rig using air-rotary drilling equipment and an HQ-sized GeoBarrel wireline coring system. These borings were drilled on a regular grid pattern with spacings of approximately 100 ft for the purpose of characterizing the general subsurface conditions within and around the CAT I Facility area. Borings were visually logged by Kleinfelder geologists and engineers; LANL geologists provided assistance in lithlogic descriptions and in identifying stratigraphic contacts.

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b) Shallow Seismic Characterization Borings

Shallow seismic characterization (SSC) borings were drilled near the corners of the CAT I Facility area. These borings were drilled to a nominal depth of 150 ft bgs and, following drilling, were completed with solid-wall poly-vinyl chloride (PVC) well casing for seismic velocity measurements. Four SSC borings were drilled as part of phase-one field activities; however, boring SSC-2 caved after drilling and could not be completed with the solid-wall PVC well casing as planned. Boring SSC-2A was added and completed as part of phase-two field activities. Borings SSC-1, -2, -3, and-4 were drilled using the methods described previously for general site borings. Boring SSC-2A was advanced using mud-rotary drilling methods. The SSC borings were drilled and sampled deeper than general site borings to provide more detailed subsurface characterization and to facilitate a greater number of seismic velocity measurements of strata within the influence zone of the MAIN.

c) Deep Seismic Characterization Borings

Deep seismic characterization (DSC) borings were drilled to characterize the complete geologic column down to the “basement” bedrock level at TA-55. Three DSC borings were identified in the G/SIP but only two of these borings were drilled. Two deep borings were deemed necessary to provide corroborative characterization of the deeper portions of the geologic column for the site selected by LANL for the CMRR Facility. The third boring, DSC-3, was identified as an alternate and would have been drilled only if the original (and currently planned) site for the CMRR Facility were deemed not viable.

Two DSC borings were ultimately completed and cased to the target depths identified in the G/SIP. These include boring DSC-1B to a depth of approximately 741 ft bgs and boring DSC-2A to a depth of approximately 550 ft bgs. Both of these borings were completed during phase-two field investigation activities using an air-rotary, casing-advance drilling system referred to as TUBEX. While the predecessor DSC borings of these series (DSC-1, DSC-1A, and DSC-2) did not reach the target depth due to reasons detailed in the GDR, samples suitable for geomechanical testing were retained from all of these borings.

d) Crosshole Seismic Borings

The crosshole seismic (CHS) borings were drilled and crosshole seismic surveys performed as an alternative method of shear-wave velocity measurement after problems were experienced with the suspension logging method. *In-situ* seismic velocity measurements were made using suspension logging at all of the SSC and DSC borings that had to be completed with solid-wall

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PVC well casing to prevent borehole collapse. Evaluation of these results indicated that all the velocity data recorded for the top 150 ft of the geologic column were anomalous and unusable. Credible waveforms were not generated in the suspension logging survey measurements completed in this depth range. The primary reason is presumably the inability to perform the suspension logging in an uncased, fluid-filled borehole (the preferred method), although grout intrusion in poorly welded layers or de-coupling of the grout from the casing may have contributed to the anomalous readings.

At the direction of LANL, KA developed a scope to perform crosshole seismic velocity logging (CROSSHOLE) to improve the velocity data within 0-150 feet, the depth range of concern. As part of this new scope, additional crosshole borings (CHS) were drilled for the source holes, while previously completed borings (DSC-1, DSC-1A, SSC-2A, and SSC-4) were utilized as receiver borings. Three CHS-series borings were completed to a depth of approximately 150 ft bgs and were drilled approximately 10 ft away from adjacent existing DSC or SSC borings as shown on Figures I-2 and II-1. Borehole deviation surveys were performed to obtain actual center-to-center boring spacings. The initial CHS boring near SSC-2A encountered difficulties during drilling; therefore, the boring location was moved and the boring completed to the target depth. All CHS borings were drilled using hollow stem augers (HSA) and samples were collected using the Standard Penetration Test (SPT) methodology in accordance with the American Society for Testing and Materials (ASTM) test method D1586-99. During sampling, SPT blow counts (N values) were also measured.

e) Alternate Area Borings

Alternate area borings were drilled to provide data on alternate site locations for the CMRR if the current site was deemed unviable for the proposed facility. These borings were drilled to depths of about 50 ft bgs using the same methods of drilling described for the general site borings.

f) Test Trenches

Test trenches were excavated within the CAT I Facility area to provide information on the geologic structure of the near-surface volcanic tuff units. Four test trenches were specified in the G/SIP and each was to be configured in a three-ray spoke and hub pattern with each hub located at an SSC boring. The layout and configuration of the test trenches was modified later to accommodate the fill placed for parking lot construction discussed previously. This modified layout separated the spokes of TP-1 and TP-4, resulting in six trench locations (Figure II-1). The

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test trenches were excavated with a backhoe to a maximum depth of about 8 ft Removable shoring was utilized to protect personnel entering the trenches. The trenches were logged by an experienced engineering geologist (GDR, Appendix B).

3. *In-Situ* Measurements

In-situ measurements were made at select locations to further characterize the subsurface conditions at the site. Details of these activities are presented in the following sections.

a) Subsurface Seismic Velocity Logging

The field investigation activities included several methods of subsurface seismic velocity logging including downhole, suspension, and crosshole, and all were performed by KA subcontractors. Downhole and suspension methods were originally designated in the G/SIP for all of the DSC and SSC boreholes with PVC casing. The crosshole method was added in accordance with FCR #05-05 to resolve anomalous readings encountered in the upper 150 ft of the suspension logging. Each of these methods of logging is described in detail in the GDR, which includes the complete subcontractor report of activities and results.

b) Natural Gamma Geophysical Logging

Natural gamma logging was performed on the DSC and SSC borings by KA personnel using equipment provided by LANL. Details of these procedures and the data collected are described in the GDR. Logging was performed at various times (February 18; March 6,7, and 9; and April 8, 2005) in DSC-1B and in DSC-2A (March 31 and April 14 2005). SSC borings were logged on April 14, 2005.

Five full-depth logs and two partial logs were performed in DSC-1B. On the complete logs, each showed inflections of the gamma trace at the top of Qbt4, Qbt1g, Qct, Qbo, Guaje Pumice, and alluvial sediments above the dacite. The inflections for the Qct unit are especially pronounced. Except for one of the records from March 9, which shows a downward shift of about 25 ft in all stratum inflections compared to earlier and later logs, the DSB-1B gamma logs show consistent depths of inflections for strata contacts that match, and consequently support, the strata contacts identified in the boring log. Probably because of instrument adjustments, the gamma log of April 8, 2005 shows more distinct inflections for the $Qbt3_U/Qbt3_L$, $Qbt3_L$/Qbt2, Qbt2/Qbt1v, and sediment/dacite contacts than the logs of earlier dates. The gamma logs also show a gamma low at 635-645 ft that has no lithologic distinction on the boring log.

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The gamma log for DSC-2 was performed to a depth of 328 ft on March 18, 2003 after 8-inch casing had been installed to 125 ft and 5.5 inch casing to 250 ft The casings reduced the strength of the gamma signal enough that the record above 250 ft is not useful. Below 250 ft the record is good and clearly indicates the Qbt1g/Qct contact at approximately 308 ft, consistent with the boring log.

Gamma logs were run on DSC-2A on March 31, 2005 with 45 ft of SW casing in the borehole and subsequently on April 14, 2005 with 550 ft of PVC casing in the borehole. Although the gamma logs from both dates record distinct strata contacts, the latter record has stronger signal and provides more contrast, and in turn better distinction, between strata. All units contacts are clearly separable with the exception of $Qbt3_U/Qbt3_L$ and $Qbt3_L$/Qbt2; these are present but less well defined.

SSC-boring gamma logs were similar to each other. Except for the gamma log on SSC-2, the gamma logs were performed inside a 4-inch PVC casing. All show sharp contrast between Qbt4 (where present) and $Qbt3_{u.}$ The contrast at the $Qbt3_U/Qbt3_L$ is indistinct on these records except for SSC-1, where a sharp gamma deflection occurred at 83 ft versus the contact visually identified in the boring log at 76 ft The $Qbt3_L$/Qbt2 contact is discernible on the gamma logs for SSC-1 and SSC-2A but not for SSC-3 and SSC-4.

c) Caliper Borehole Diameter Measurements

Measurements of borehole diameter were performed in borings DSC-1B and DSC-2A by KA personnel using equipment provided by LANL. The plots of the caliper log records are included in Appendix D of the GDR. Boring DSC-1B was caliper-logged on February 18, 2005 from 395 ft to 733 ft to determine the extent of borehole enlargement due to sloughing or raveling ground. The caliper log showed that DSC-1B was enlarged by 1 to 2 inches through the Qct interval (400-460 feet) and as much as 8+ inches in the dacite section from about 700 ft to 728 ft Subsequently, a caliper log was run on this borehole on April 11, 2005 after the PVC casing was installed to check the integrity of the casing and to ascertain that the casing had sufficient inside clear diameter to allow unobstructed passage of the downhole seismic survey tool. This log showed several locations of probable grout leakage through pipe joints and an interval from 690-710 feet with readings of closely spaced hole narrowing and enlargement probably caused by grout intrusion, casing separation, or possibly sensor malfunction. On April 14, 2005 a caliper log of the PVC casing in DSC-2A indicated intact casing with two locations of grout leakage through casing joints.

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d) Downhole Video Recording

Video recording was performed over selected intervals of borings DSC-1B and DSC-2A to evaluate borehole wall conditions and to assist with identification of stratigraphic units. KA personnel using downhole video camera and recording equipment provided by LANL performed this procedure. Downhole video logging documented the condition and geology of intervals of boreholes DSC-1B (395 to 733 ft) and DSC-2 (252 to 323.5 ft). These records were especially useful in identifying the uniformity or rugosity of the borehole walls and zones of collapse or erosion caused by drilling fluids.

The video record for DSC-1B before installation of the PVC casing started at the bottom of the temporary SW casing at 395 ft The borehole was smooth and cylindrical from 395. to 414 ft, then become rougher (1-2 inch relief) to 420 ft through a lithic-rich zone. After another smooth section from 420 to 455 ft, the wall roughness increased due to lithics of 2 to 6 in. size. From that depth to 661 ft, the borehole wall was cylindrical with a roughness corresponding to the size of lithic fragments within the Qbo tuff, 1/2 to 1 inches. From 661 to 682 ft, lithics were smaller but the borehole wall shape was irregular due to soft tuff. After another short interval of cylindrical shape from 682 to 686, the borehole wall became irregular again with breakouts of larger rock fragments in the gravelly sediments from 686 to 693 ft At 693 ft, the borehole wall belled slightly as coarser lithics were encountered, then became progressively more irregular because of large gravel to cobble-size lithics dislodged from the borehole. The limit of light and field of vision of the camera lens limited the view of the horizontal extent of borehole enlargement below 689 ft to 697 ft, at the top of the dacite. Through the dacite the borehole wall was very blocky and irregular, retaining a cylindrical shape in only a few locations to 733 ft, where slough had backfilled the hole. The overbeak through the dacite appeared to be at least one borehole diameter beyond the borehole wall.

The only other video record in a deep boring was over an interval of Qbt1g in DSC-2. This video record, from 252 ft to 323.5 ft, showed borehole wall sloughing at 297, 301, 309, and 320-322 ft The latter two locations had sloughing that enlarged the borehole by at least one diameter.

DSC-1B and DSC-2A were video-logged after PVC casing installation to check for casing damage, de-coupling, or grout leaks. The video record for DSC-1B ran from 0 to 724 ft Grout leaks through casing joints were common below a depth of about 300 ft Below top of water (in

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the casing) at 604 ft, suspended sediment (probably grout) obscured the video image. However, grout leaks through joints at 660, 680, and 690 ft were visible.

Two attempts were made to video-log DSC-2A after PVC casing installation. The first attempt on April 8, 2005 encountered water in the casing (used for ballast during casing installation) at 346 ft A possible casing separation was observed at 108 ft A possible grout leak through the casing joint at 344 ft may be responsible for an obstruction at 345 ft During the second logging on April 14, 2005, water stood at 321 ft in the casing and grout leakage was noted at 433 to 434 ft (possible casing break) and at 504 ft.

e) Optical Televiewer Recording

Digital video recordings were made to obtain borehole structure data in borings SSC-1, -3, and -4 using the optical televiewer. The optical televiewer is a video device, oriented to magnetic north, which records the borehole wall condition and discontinuities in the rock, providing a means to determine strike and dip of rock structure. A subcontractor to KA performed these services. Details of these procedures and the data collected are described in the GDR. The results were used in the kinematic analysis described in Section IX.C.2 of this report.

f) Electrical Resistivity Survey

Near-surface electrical resistivity measurements were made using the Wenner 4-probe methodology in accordance with ASTM G57-95a. KA personnel surveyed nine resistivity lines around the CMRR Facility area. The survey was made to characterize the corrosion characteristics of the soil and rock of the site, as further discussed in Section V.B.4. Details of these procedures and the data collected are described in the GDR.

C. Laboratory Testing

Laboratory testing was performed on samples recovered from three of the four phases of the overall CMRR geotechnical investigation. No testing was performed on samples recovered from the phase crosshole work. Specific details of the testing performed for the RLUOB and the SBT are contained in the KA reports on those investigations; no summary or detailed discussion of laboratory testing results for those two phases is contained in the GDR or this report.

Detailed results of the laboratory testing performed for the phase one and phase two field investigations are presented in the GDR; a summary of these results is presented herein. Laboratory testing, performed to develop both geomechanical and geochemical properties, was categorized as:

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- Conventional Index and Classification Testing (Atterberg limits, gradation, unit weight, moisture content, specific gravity),
- Conventional static geomechanical testing (triaxial compressive strength, UC strength, direct shear frictional properties),
- Specialized static geomechanical testing (stress-path controlled triaxial compressive strength),
- Specialized dynamic geomechanical testing (resonant column torsional shear, cyclic simple shear),
- Geochemical testing (alkali-silica reactivity), and
- Petrographic Analyses.

1. Conventional Index and Classification Testing

Index property and material classification tests were performed on selected samples of the various geologic units recovered during phase-one and phase-two site investigation activities. Laboratory testing included particle-size analysis, Atterberg limits determinations, bulk density, moisture content, specific gravity, and moisture density relationships. The complete results of these tests are presented in the GDR. A discussion of compaction characteristics is presented in Section VI of this report.

2. Conventional Static Geomechanical Testing

A representative number of samples recovered from the borings indicated on Figures I-2 and II-1 were subjected to conventional static geomechanical testing including triaxial compression, confined compression, unconfined compression, and direct shear. The majority of these tests were performed on undisturbed samples to evaluate the *in-situ* strength properties; however, several remolded specimens were tested to evaluate their strength properties when used as compacted structural fill materials during construction. The complete results of these tests are presented in the GDR.

3. Specialized Static Geomechanical Testing

Select undisturbed samples were subjected to stress-path controlled triaxial compression tests. These tests were performed using parameters developed by numerical modeling based on a

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simplified building excavation/construction model. The results of the numerical modeling were used to establish unloading and loading stress paths parameters for the triaxial testing. The complete results of these tests are presented in the GDR and are summarized in Section VI of this report.

4. Specialized Dynamic Geomechanical Testing

Dynamic geomechanical testing was performed to evaluate the potential for seismically-induced compaction and to develop the non-linear relationship of shear modulus and material damping versus shearing strain for ground response and SSI modeling.

a) Cyclic Simple Shear

Fourteen cyclic simple shear (CSS) tests were performed on undisturbed samples of the $Qbt3_L$. These tests were performed at cyclic stress ratios (CSR) expected for the site based on the design basis earthquake (DBE). The results of the tests provide estimates of vertical strain (settlement) and to evaluate the potential for and magnitude of seismically-induced compaction (SIC) that could be expected in $Qbt3_L$ under cyclic shear stress due to the DBE. Large strain shear modulus values were also obtained from the CSS tests. The results of these tests are presented in a report prepared by Dr. Michael Riemer of the University of California, Berkeley, presented as Appendix K of the GDR.

b) Resonant Column / Torsional Shear

Resonant column /torsional shear (RC/TS) tests were performed at the University of Texas at Austin (UT) on samples recovered from the borings to evaluate the dynamic linear and non-linear properties of the layers above the base rock at the CMRR Facility. The results are included in Appendix J of the GDR and are discussed in Section VII of this report.

5. Geochemical Testing

Alkali-silica reactivity (ASR) geochemical testing was performed in phase two to evaluate the potential for an adverse reaction between *in-situ* materials mixed with cementitious materials. The results of these tests are presented in Appendix I of the GDR.

6. Petrographic Analyses

Separately from but in coordination with Kleinfelder's phase SBT investigations, LANL EES-9 geologists performed petrographic analyses of 19 samples of Qbt3 tuff obtained from the TA – 61 borrow pit, local outcrops, and core samples from CMRR test borings (Lewis et al, 2005 and

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2007). These studies found that both the matrix and pumice in the 19 samples are vapor-phase altered, overprinted by devitrification and vapor phase crystallization. All 19 samples analyzed by LANL are composed of the same minerals with no significant variation except for relative abundances. Petrographic analysis of polished thin sections shows devitrified nonwelded tuff with 20-36% recognizable voids. X-Ray Diffraction (XRD) measured 19.2 ± 0.6 % quartz, 19.0 ± 0.5 % tridymite/cristobalite, 24.9 ± 0.4 % alkali feldspar, and 38.4 ± 0.8 % anorthoclase (imaged by XRD as plagioclase) in the 19 samples. The ratio of cristobalite/tridymite changes with depth from about 1/19 in $Qbt3_U$ to 8/11 in $Qbt3_L$. Lewis et al (2007) found that vapor-phase minerals did not fill pores to any measurable extent in lower Qbt3. These analyses demonstrate that the Qbt3 tuff is essentially identical in all sampled locations. They conclude that the primary controls on the compressive strength of tuff are the packing of the groundmass shards (i.e., porosity), shard alignment, density of pore-filling material, and the extent to which shards are welded at their points of contact.

The analyses of Lewis et al are useful for understanding the structure of the Qbt3 tuff at a microscopic level and how this structure influences the geomechanical behavior of the tuff. These results supported Kleinfelder's interpretation of its own field and laboratory data, but they were not used in calculations or numerical modeling performed by Kleinfelder.

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V. GEOLOGIC CONDITIONS

A. Regional Conditions

LANL is located on the Pajarito Plateau on the east flank of the Jemez Mountains and along the western boundary of the Rio Grande Rift (Gardner et al, 1999). The Jemez Mountains are the result of volcanic activity centered around the Valles-Toledo caldera complex (Gardner et al, 1993). Although the region has experienced volcanic activity for several million years, the activity culminated with two major eruptions of 1.61 million years (Ma) ago and 1.22 Ma (Izett and Obradovich, 1994). These eruptions ejected the ash that deposited as the Bandelier Tuff and built up the Pajarito Plateau (Broxton and Reneau, 1995). Subsequent volcanic activity occurred until as recently as 50,000 years ago but has been largely limited to the caldera; none of this activity appears to have impacted the Pajarito Plateau in the vicinity of CMRR (Keller, 1968).

The Bandelier Tuff consists of two members of ash-flow tuff, the lower Otowi and upper Tshirege Member formed by the first and second major eruptions, respectively. These are separated by the Cerro Toledo interval consisting of a variety of pyroclastic and volcaniclastic materials from silt to boulder size (Broxton and Reneau, 1995).

The Rio Grande Rift is the result of east-west extension, a characteristic it shares with the rest of the Basin and Range tectonic province. The Rift also experiences frequent microseisms and occasionally larger events – seven earthquakes have been felt in Los Alamos (magnitude up to 4). Larger earthquakes have been recorded with epicenters farther south along the Rift (Gardner et al, 1999).

B. Site Conditions

1. Site Stratigraphy

The geologic formations that are relevant to CMRR are those that will influence seismic ground response and foundation performance. Seismic ground response is affected by the relatively high shear wave velocity of the mafic volcanic rocks (Cerro del Rio basalt and Tschicoma dacite) and the much lower shear wave velocities of the overlying Bandelier Tuff. Accordingly, geologic units below the mafic volcanics are not addressed in this report. The term “geomaterial” used subsequently in this document refers to rock and soil material from the mafic volcanics upward to ground surface. Based on test borings and references, the stratigraphic section of geomaterials at the CMRR site is, from lowest to highest:

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Cerro del Rio basalt and Tschicoma dacitic lava – extrusive mafic volcanic rocks originating from vents east (Cerro del Rio basalt) and west (Tschicoma dacite) of the CMRR site. At least three distinct flows were identified in the 41 ft of dacite penetrated at the bottom of boring DSC-1B, but the total thickness is probably several hundred feet. The upper boundary was encountered at about 697.5 ft depth or El 6597.5 ft The sampled portion of this unit is heavily fractured and vesicular. Based on our review of video logging of DSC-1B, this boring possibly intersected a lava tube near the top of the dacite.

Alluvial sediments – These sediments consist of thinly bedded to unstratified fine sand and silt, possibly a finer-grained interval of the Puye Conglomerate or the Old Alluvium described by Griggs, 1964. This stratum, first encountered at a depth of 679.5 ft (El 6615.5 ft), is about 18 ft thick in boring DSC-1B.

Guaje Pumice – Consisting of unlithified white rhyolitic lump pumice and lapilli, the Guaje forms the base of the Bandelier Tuff. It is about 14.5 ft thick in DSC-1B and has a top elevation of 6630 ft amsl.

Otowi Member (Qbo) – This ash-flow tuff consists of relatively homogeneous, non-welded to moderately welded, light gray to salmon-colored pumice lapilli and ash, about 272 ft thick under CMRR. Visually, it appears to be unstratified and relatively uniform throughout its thickness. The top of the Otowi in DSC-1B is at a depth of 392.5 ft, or El 6902.5 ft amsl, based on visual logging.

Cerro Toledo interval (Qct) – Lithologically the most variable of all the Bandelier Tuff units, the Cerro Toledo consists of volcaniclastic sediments with some ash-fall and pumice-fall deposits of silt size to boulder size (Broxton and Reneau, 1995). Generally also highly variable in thickness, it is about 62 ft and 66 ft thick in DSC-1B and DSC-2A, respectively. The top of the Cerro Toledo in DSC-1B is at about El 6964 ft.

Tsankawi Pumice of Tshirege Member – The Tsankawi is primarily rhyolitic pumice lapilli from 8 inches to 3.3 ft thick (Broxton and Reneau, 1995). It was not identified in DSC-1B and was approximately 1.0 ft thick in DSC-2A.

Unit 1g of the Tshirege Member (Qbt1g) – This unit is white to light gray to light orange, vitreous, poorly welded to non-welded ash-flow tuff with abundant volcanic glass (Broxton and

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Reneau, 1995). The thickness is about 70 ft under the CMRR site, where the top of this unit was encountered at about El 7035 ft.

Unit 1v of the Tshirege Member (Qbt1v) – This poorly to moderately welded ash-flow tuff is orange-brown to pinkish-white/light gray, devitrified, with abundant pumice. In outcrops the lower portion is a columnar-jointed colonnade, and the upper is weaker and bench-forming (Broxton and Reneau, 1995). The total thickness at CMRR is 45 to 53 ft The top of Qbt1v is at El 7088 ft amsl in DSC-1B.

Unit 2 of the Tshirege Member (Qbt2) – This unit is light pink-tan to reddish or purplish gray, moderately to densely welded ash-flow tuff with well-developed fractures (Broxton and Reneau, 1995). It is the most welded of all the Bandelier Tuff units. The total thickness at CMRR is 76 to 79 ft, and the top of the unit is about 128 ft deep at El7167 ft in DSC-1B.

Lower Unit 3 of the Tshirege Member ($Qbt3_L$*)* – This white to light gray, non-welded to poorly welded ash-flow tuff is visually distinguishable from tuffs above and below it based on its sparse (<5%) pumice and softness (Broxton and Reneau, 1995). Although these authors do not assign this material to a separate unit, they note that it may be further subdivided according to regional characteristics. KA has treated it as a distinct unit based on its contrasting properties relative to the other units in proximity to the CMRR foundations. As inferred from logged changes in welding and hardness, as well as measured variation in static and geophysical properties, the thickness of the transition of $Qbt3_L$ to Unit 2 at its base is 7-8 feet. The transition to the overlying Upper Unit 3 ($Qbt3_U$) varies from about 5 to 10 feet. The total thickness of $Qbt3_L$ at CMRR, roughly between the middles of the transition zones, is about 50 ft This unit has a relatively high porosity and exhibits geomechanical properties similar to dense sand. Characterization of $Qbt3_L$ was a major concern the of phase-two field investigation because of its thickness, proximity to CMRR foundation, and uncertain geomechanical characteristics.

Upper Unit 3 of the Tshirege Member ($Qbt3_U$) – This tuff is light gray, moderately to partially welded ash-flow tuff with 10%-30% pumice, distinguishing it from $Qbt3_L$. The degree of welding and fracturing both increase from the bottom to the top of this unit (Broxton and Reneau, 1995). We encountered notably lower welding and hardness in roughly the upper 10 feet of this unit, compared to the rest of this stratum. The quantity and inclination of fracturing within this unit is expected to influence its stability in excavated slopes, affecting adjacent ground support and the

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practical inclination of excavation slopes. The depth to the top of the stratigraphic $Qbt3_U$ contact is 17.5 ft (El 7277.5 ft) in DSC-1B.

Unit 4 of the Tshirege Member (Qbt4) – This unit is pinkish to orange-gray, nonwelded to moderately welded, pumice-poor, crystal-poor ash-flow tuff with a basal pyroclastic surge deposit up to 0.5 ft thick (Broxton and Reneau, 1995). The total thickness at CMRR varies from zero along the south side of the site (where the unit has been eroded) to 23 ft (Boring 4-A). The remaining thickness of Qbt4 at DSC-1B is 17.5 ft.

Fill – Man-made fill as well as some residual soil, the product of weathering of the underlying tuff, covers the CMRR site to varying depths. The most recent fill was placed to construct a temporary parking lot in the CAT I Facility location during September 2002. Locally-derived fill consists of sand and silt with some clay. Imported fill includes tuffaceous soils as well as gravelly soils.

Cross sections have been prepared based on the stratigraphic information collected from the borings. A legend describing the terminology used for the test pits, boring logs, and cross sections contained in this report as well as in the GDR, Appendix C, is presented in Figure V-1. The locations of the cross sections with respect to CMRR are shown on Figure II-1 and are summarized below:

- Figure V-2 Cross Section A-A’
- Figure V-3 Cross Section B-B’
- Figure V-4 Cross Section C-C’ and D-D’
- Figure V-5 Cross Section E-E’ and F-F’
- Figure V-6 Cross Section G-G’ and H-H’
- Figure V-7 Cross Section I-I’ and J-J’

These cross sections represent the stratigraphic conditions encountered in the borings relative to the proposed building construction. The boundaries between layers are represented by straight-line interpretations and may not be representative of the actual conditions between

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boring locations. The proposed excavation limits that have been prepared by S&L are shown in these cross-sections for reference.

2. Geologic Structure

The CMRR site is located on Pajarito Mesa, part of the Pajarito Plateau. The plateau consists of the tuffs of the Bandelier Tuff, dipping approximately 7 degrees southeast (Gardner et al, 1998), away from the volcanic vents from which the ash originated.

The Pajarito Plateau is separated from the Valles-Toledo Caldera Complex, the large volcanic mountains to the west, by the Pajarito Fault (PF) System (Gardner et al, 1998). This fault system consists of the PF, the Rendija Canyon Fault (RCF), and the Guaje Mountain Fault (GMF), which together locally define the west boundary of the Rio Grande Rift (Gardner et al, 1999). This fault system displays evidence of Holocene (last 11,000 years) displacement and is considered to be potentially seismogenic (Gardner et al, 1999). The PF trends roughly north-northeast and lies approximately three miles west of the CMRR site. The RCF and GMF are north-south trending faults that are clearly delineated through the town of Los Alamos, north of the CMRR site, but appear to lose definition to the south. Gardner et al, 1995, identified an apparent fault about 1500 ft west of TA-55 with about two feet of displacement (down to the southeast) in the Qbt3/Qbt 4 contact, but this fault was not observed in the Qbt2/Qbt3 contact. Faults farther west (away from the CMRR site) are more abundant, trend north to northeast, dip nearly vertically, and have up to 8 ft of offset (Gardner et al, 1998).

Recent fracture mapping by LANL (Wohletz, 2004) in the north wall of Mortandad Canyon north of the CMRR site documented fracture clusters that were interpreted as southward extensions of the RCF and GMF, passing south-southwest along the west boundary of TA-55 and through TA-63 to the west and east of CMRR, respectively. This interpretation is consistent with studies by Vaniman and Wohletz, 1990, which found fracture clusters in Unit 3/Unit 4 along the TA-48/TA-55 boundary north and south of Pajarito Road. From these studies, it appears that the structural disturbance manifested as dense fracturing, lying 800-1000 ft west of the west edge of CMRR, is the southern extension of the RCF zone with net vertical displacement (down drop to the east) of 11.3 meters (m) (37 ft).

Mapping of fractures (Wohletz, 2004) in Unit 3 in Mortandad Canyon north and northeast of TA-55 documented two dominant, conjugate fracture sets with 1) mean strikes of N22W with steep dips to the south and 2) mean strikes of N82E with dips to mostly north and dips not as steep.

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The N82E set is the more prominent of the two. This conjugate fracture orientation suggests a principal horizontal stress in the N15E direction. A minor set striking N50W was also documented.

Wohletz (2004) documented that fracture density in Unit 3 averages 20 fractures/100 ft linear ft along the Mortandad Canyon outcrop adjacent to TA-55. However, the average fracture density is 100/1000 linear ft over the extension trace of RCF and 60/1000 linear ft over the extension of GMF. Fracture density, aperture, and variability in orientation increase within 600 and 2000 ft of the fault traces, respectively.

As part of its site investigations, KA excavated test trenches within the CMRR site. Most of the trenches were excavated into fill or Qbt4; two were excavated into $Qbt3_U$. Fractures mapped in test trenches are plotted on Figures B-2 through B-7 in Appendix B of the GDR. These figures show some clustering of very steep (81-90°) fractures dipping toward the northeast (Set A) and some lesser clustering of 71-80° fractures toward the northwest (Set B). Otherwise, fracture directions appear to be randomly distributed. These two sets do not match up with the conjugate clusters mapped by Wohletz, but some of the other, apparently random fractures mapped in the CMRR studies could belong to the clusters identified by Wohletz.

3. Groundwater

No groundwater was encountered to maximum depth of drilling (741 ft bgs). From the LANL Environmental Restoration Program deep well data, the regional groundwater table is more than 1000 ft bgs at TA-55 (B. Bockisch, 2005). Therefore, the vadose zone extends into the dacite flows below 700 ft depth; local intervals of saturation may occur above that level, especially along fractures or in the sediments of the Qct.

4. Resistivity of Site Materials

The results of the resistivity survey are presented in Appendix G of the GDR. A soil electrical resistivity survey was performed at nine separate locations within the CMRR site as shown in Appendix Figure G-1 in the GDR. In uniform soil without changes in moisture content with depth, resistivity should increase with increasing spacing of pins. Changes in soil types, moisture content, or buried conductive materials will alter the measured resistivity at pin spacings corresponding to the depth(s) where such changes occur. To minimize the effects of buried pipes, cables and other conductive materials, the survey lines were laid out at substantial offset from buried utilities or to run perpendicularly away from or across them. Known buried

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utility locations were identified from utility drawings provided to KA by LANL through DMJM H+N. In the area of the CAT I Facility, LANL placed fill over a generally south-sloping ground surface to create the existing parking lot surface. Both buried utilities and sloping soil-rock contacts can affect resistivity readings. Other sources of anomalies include:

- equipment out of calibration, and
- incomplete connections between pins and cables.

The resistivity values, measured by the Wenner 4-pin method, on several lines include some apparently anomalous numbers. Specifically, the results in question appear to be:

- R-2, 6.3 to 25.0 ft spacing – anomalously low resistivity (ρ),
- R-7, 6.3 ft (high ρ), 12.5 ft (low ρ),
- R-10, 6.3 (low ρ),
- The field log for R-2 identifies a buried cable crossing (evident from the linear pavement patch) under this line that was not shown on the utility drawing provided by LANL. The anomalously low ρ appears to be attributable to this feature,
- Line R-7 extended from the parking lot into the roadway of Pecos Drive. A sewer line crosses under the west 50 ft of this line that probably affected the recorded resistivity,
- Under the R-10 line, a LANL “labnet” communication cable and at least two telephone lines cross perpendicularly to the line direction. These utilities probably contributed to the low ρ measurement at 6.3 ft spacing, and
- The surveys of R-13 and R-14 produced the lowest resistivities of all lines. One sewer line crosses below R-14 and could have affected this survey. No apparent reason for the low ρ readings of R-13 has been identified.

Equipment/operator sources of error cannot be ruled out. However, the KA operator who performed the testing, is experienced in the operation of the apparatus, and checked the connections and calibration before each survey.

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Soil resistance in the southwest soils can be expected to be in the range of 5,000 – 25,000 ohm cm, making them mildly to moderately corrosive (http://corrosionsource.com/technicallibrary/corrdoctors/index.htm). It is reasonable, therefore, to discount the anomalously low resistivity readings as erroneous because of buried conductors, transient soil moisture, or operator/equipment error.

Because of the amount of conducting materials in the subsurface at the CMRR, re-surveys are unlikely to produce significantly better results. However, LANL is planning excavation of the RLUOB and CAT I areas in the near future. At that time, a re-survey is likely to encounter less interference and produce better results.

5. Geochemistry – Potential Reactivity

Ten potential reactivity tests were performed on subsurface materials sampled in the borings. Samples tested were recovered from depths of 22 to 124 ft bgs with seven of the samples originating from the DSC-1 borehole cluster and the other three samples from boring SSC-2A. Laboratory testing by ASTM C289 indicates that all of the samples tested fall within the range of “aggregates considered deleterious” for use in Portland cement concrete. The dissolved silica measured in the test ranged from 165 to 435 millimoles per liter with an average of 327.6. The reduction in Alkalinity was typically low and ranged from 2 to 33 millimoles per liter with an average of 17.1. Details of testing are presented in Appendix I of the GDR.

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VI. STATIC ENGINEERING PROPERTIES

Trends were noted with respect to the geologic profile of the CMRR site during reduction and interpretation of the accumulated data. These trends in data and geologic profile were grouped by their general category into static and dynamic properties. The static properties were those generally required for conventional geotechnical engineering design criteria and the dynamic properties were those generally required for site response modeling associated with seismic shaking. This section (Section VI) of the report presents a summary of the results and the interpretation associated with the static properties of the subsurface materials. Section VII of the report presents a summary of the results and interpretation associated with the dynamic properties of the subsurface materials. Further analysis, review, and interpretation of these trends were performed which resulted in the development of a fully integrated geomechanical model as presented in Section VIII of this report. The geomechanical model generally follows the geologic profile of the site but differences, based on geomechanical properties including both static and dynamic, are identified.

The proposed CAT I Facility will consist of a large, heavy facility that will extend to a significant depth below existing grade, and be, for the most part, a buried structure. Design, construction, and normal service life performance of this facility requires that geotechnical recommendations be developed that are based on satisfactory characterization of the subsurface conditions. Extensive subsurface characterization and laboratory testing data has been accumulated for this facility as described in previous sections of this report as well as in the GDR. The following section presents the compilation, interpretation, and analysis of these data with respect to the development of the static engineering properties.

This section presents the static engineering properties of *in-situ* $Qbt3_U$, $Qbt3_L$ (plus upper and lower transition layers) and Qbt2, and of remolded Qbt4 and $Qbt3_U$ material that may be used for structural fill. Statistical analyses of static properties, including standard deviation and coefficient of variation, were performed (Calculation 13658-109-XCAL-008) to assist in selection of static design values. Static engineering properties include geotechnical properties under static foundation or lateral earth loading conditions as well as unit weight, moisture content, and index property relations. The static engineering properties and design parameters discussed in this section include:

- Index properties, such as plasticity index and grain-size distribution,

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- In-situ moist unit weight, specific gravity, and void ratio,
- Unconfined and triaxial compressive strength,
- Rock Mass Rating (RMR) and Geological Strength Index (GSI),
- Shear strength of tuff, in terms of Mohr-Coulomb (c, ϕ) and Hoek-Brown parameters (for rock-like tuff units),
- Shear strength of tuff along fractures, in terms of Mohr-Coulomb (c, ϕR) parameters,
- Intact and rock mass modulus of elasticity,
- Compressional characteristics,
- Moisture-density relationships of excavated tuff used as structural fill, and
- Shear strength of excavated tuff used as structural fill (in terms of Mohr-Coulomb (c, ϕ) parameters).

Geotechnical characterization of the geomaterial in the CMRR area was based initially on visual inspection and field characterization of samples obtained while drilling and subsequently refined based upon laboratory test data. The static engineering profile discussed herein is limited to those subsurface layers that are expected to be influenced by the proposed construction of the CAT I Facility. Based on the data gathered to date, the static performance of the CAT I Facility will be most greatly influenced by Units $Qbt3_U$ and $Qbt3_L$ and the transition zone between them. These two units either entirely surround or underlie the proposed facility. Below $Qbt3_L$ is unit Qbt2, which is a well-indurated (moderately to strongly welded) volcanic tuff. This unit lies more than 70 ft below the planned CMRR mat foundation, and is expected to be negligibly influenced by the CAT I Facility; however, its engineering properties are discussed in this section of the report.

The characterization of dynamic properties at the CAT I Facility is discussed in Section VII of this report. A fully integrated geomechanical model that incorporates both static and dynamic engineering properties is presented in Section VIII of this report. Design parameters that are influenced by foundation dimensions and loading, such as modulus of subgrade reaction and bearing capacity, are addressed in Section IX of this report.

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A. Static Engineering Properties Profile

All boreholes were logged and geologic unit names provided in general accordance with the geologic nomenclature of Broxton and Reneau (1995), with the inclusion of Unit $Qbt3_L$, the poorly welded layer of Qbt3. Cross sections A-A' through J-J', presented in Section V, illustrate the unadjusted field data using this nomenclature. While progressive changes in welding, hardness, and measured RQD were apparent in the field, the laboratory testing, which include compressive strength, elastic moduli, and unit weight, also exhibited gradual changes with respect to elevation within a particular unit and its transition to another unit. Due to the tangible difference of engineering properties between the more soil-like $Qbt3_L$ and the soft rock layers above and below it, transitional layers $Qbt3_L$-t and Qbt2-t have been identified as separate units based on static properties.

Based on the results of the static engineering laboratory tests, as well as the variation of logged field data the following units were used for the static profile in order from top to bottom:

- Qbt4
- Qbt3U
- Qbt3L-t
- Qbt3L
- Qbt2-t
- Qbt2

The geologic units discussed in this section are limited to those units that are expected to be influenced by the static loads and construction of the CAT I Facility.

B. Material Classification

1. Fill/Residual Soil

Soil encountered at the CMRR site consists of residual soil derived from weathering of the underlying tuff and from fill generated by mechanically excavating and disaggregating natural tuff materials. As indicated on the grain-size distribution plots and summary of index properties

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presented in Appendix I of the GDR, this tuffaceous soil is generally classified as silty sand (SM) or poorly graded fine sand (SP) according to the United Soil Classification System (USCS). If the more rock-like fragments did not break down into smaller fragments, the material would have classified as silty gravel to poorly graded gravel (GM-GP).

These materials are typically non-plastic because the parent material (tuff) contains no clay in the intact unweathered matrix. However, clays have developed in locations where the feldspathic minerals in the tuffs are extensively weathered. Clays were also observed as infilling in tuff fractures.

2. Tuff

The various units of interest relative to static properties (Qbt4, $Qbt3_U$, $Qbt3_L$.t, $Qbt3_L$, Qbt2-t, and Qbt2) are often distinguishable from each other based on visual examination of color, pumice content, degree of welding and hardness estimated as part of the characterization of the rock cores in the field program. Differences between the static engineering properties of the tuff units are also evident in the laboratory test results.

The majority of grain-size distribution tests (sieve analyses) were performed on samples collected from the upper Bandelier Tuff units. Additionally, sieve analyses were performed for laboratory dynamic test samples extending from the upper units to the Otowi Tuff unit (Qbo). For samples tested in 2005-2006, a mechanical washing vessel was used as a uniform means of disaggregating samples of variable welding. These samples generally classify as SM or SP materials, with more indurated units remaining intact and classifying as gravel (GP or GW). A comparison of grain size distribution for eight different units, including alluvial units such as the Cerro Toledo (Qct) and the Basaltic Sediments layer, is presented as Figure VI-1.

The poorly welded tuff layers, including $Qbt3_L$, Qbt1v Qbt1g, and Qbo disaggregate readily to sand-sized particles. About 10 to 20 percent of the tuff passes through the No. 200 sieve, indicating some silt-sized or finer material. The gradational characteristics of $Qbt3_L$ are very similar between the CMRR site and the other sites where sieve analyses were completed, which include the TA-61 borrow pit and the Mortandad Canyon site (SBT report, KA, 2005).

C. Density and Moisture

Laboratory index test results indicate that specific gravities of mineral constituents in each tuff unit are low, averaging 2.50 to 2.56 (Figure VI-2), reflecting the relative scarcity of mafic

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minerals and the abundance of silica. Correspondingly, the bulk densities of tuff units are low, and the apparent porosities of the tested tuff samples are generally high; this is consistent with the ashfall/ashflow origin of the tuffs. The distribution of *in-situ* moist unit weight data with respect to sample elevation is presented on Figure VI-3. Each static profile unit has a distinct range of *in-situ* moist unit weights, which generally coincides with the observed variation in sample welding.

In-situ moisture contents were also typically low, often less than 2 percent for units Qbt4 and $Qbt3_U$ (Figure VI-4). However, higher *in-situ* moisture contents of up to 10 percent were also encountered and possibly indicate either increased weathering (and altered material) or merely increased moisture along or near zones of fracturing. A summary of average density and moisture properties is presented in Table VI-1.

D. Strength Properties

Strength properties were evaluated based on the results of static laboratory strength testing including direct shear, unconfined compression, and triaxial compression tests. The results of the static laboratory strength tests were consistent with the observed trends in field geomechanical logging and geophysical testing, with respect to sample elevation. A summary of average strength properties is presented in Table VI-2. The results are discussed in the following subsections.

1. Strength Properties From Compression Tests

As shown in Figure VI-5, each static profile unit has a distinctive range of unconfined compressive strength (σ_{ci}) values, which vary with degree of welding. There is also a strong relationship between σ_{ci} and the properties of unit weight, porosity, and void ratio. Low *in-situ* moist unit weight (or high porosity) corresponds to low strength parameters that are typical of tuffs but are relatively low with respect to most common rock types. The porosity-versus-unconfined compressive strength relationships for the static profile units are presented in Figure VI-6 and illustrate the trend of higher σ_{ci} values for units with lower porosity. Figure VI-7 illustrates the linear relationship of porosity to the log of triaxial compressive strength. The average values presented in Table VI-2 also illustrate the variation of σ_{ci} between static profile units.

Units Qbt4, $Qbt3_U$, and Qbt2 are classified as “very weak” rock based on criteria established by Brown, ISRM (1981). Transitional units $Qbt3_L$-t and Qbt2-t are classified as “extremely weak” to

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"very weak" rock. Unit $Qbt3_L$ exhibits average unconfined compressive strength below the lower threshold of 36 psi for "extremely weak" rock, making it more appropriate to classify its strength on the soil scale. Based on 1) SPT N values from CHS borings, which ranged from about 30 to 40 blows per foot within $Qbt3_L$, and 2) the grain-size distribution tests on these materials, this unit is characterized as a medium-dense to dense sand.

The very low unconfined compressive strength of $Qbt3_L$ is consistent with its poor welding and high porosity. The rapid descent of the stress-versus-strain curves following a low stress, low strain failure are indicative of this fragile matrix; presumably the brittle failure occurs as the weak edge-to-edge particle bonds are broken. More importantly, the test results indicate that the compressive strength increases substantially under confined loading (Figure VI-8), similar to uncemented, granular soils. While $Qbt3_U$ and Qbt2 also exhibit increases in compressive strength under confinement, the percentage increases for these two units are not as significant as the more soil-like $Qbt3_L$.

Graphical representations of the peak triaxial Mohr-Coulomb (M-C) failure envelopes were completed for each tested static profile unit. The variation of intact shear strength between units of different welding may be observed through comparison of Figures VI-9 through VI-11. These linear M-C curves reflect peak failure envelopes through intact material. While these peak-versus-residual values are similar for $Qbt3_{L,}$ which does not readily support fractures within its matrix, the frictional values should be reduced significantly for Units $Qbt3_U$ and Qbt2 to reflect the overall rock mass properties. The development of Hoek-Brown rock mass parameters and rock mass failure envelopes for these units, as well as the rock mass M-C envelopes, is discussed later in this section.

2. Strength Properties From Direct Shear Tests

a) Direct Shear on Rock-Like Material

Direct shear tests was performed to evaluate the various units' shear strength characteristics; specifically those anticipated along pre-existing fracture planes. For the more rock-like units, the laboratory tests were completed by first creating a near-horizontal fracture plane in shear through a single casting of a rock sample. Following the measurement of the peak and residual shear strength along this test-generated fracture plane, residual strength values were also measured at two increasingly higher, normal pressures to produce an envelope of residual shear strength.

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Least squares fit lines were drawn through the composite data of all samples within each static profile unit. There is a high coefficient of determination (R^2) of the data, as illustrated in Figures VI-12 and VI-13 for $Qbt3_U$ and Qbt2, respectively. The highest average residual friction angle, ϕ_r was in $Qbt3_U$; somewhat lower values were measured in Qbt2 although there was significantly increased apparent cohesion measured in this more strongly welded unit.

b) Direct Shear on Soil-Like Material

For $Qbt3_L$, the more soil-like material, tests were performed on three separate samples at three increasingly higher normal pressures to produce a series of peak and residual (or ultimate) points for each test series. These tests allowed the comparison of both peak and residual failure envelopes, which were very similar for this unit (Figure VI-14). Both the peak and residual friction angle are about 33 degrees, as indicated by the least squares fit with a high R^2 shown on this figure. Despite the relatively low density of $Qbt3_L$, these moderate frictional values are likely indicative of the high angularity of the tuff constituents within this static profile unit.

E. Static Deformation Properties

Static deformation properties, including elastic modulus (or Young's Modulus) of intact core, E_i, and Poisson's Ratio, ν, were evaluated by triaxial compression tests on all tested units. For rock-like samples, Poisson's Ratio was measured by radial strain gauges during triaxial compression testing. This was not possible for soil-like samples, for which Poisson's Ratio was estimated based on wave propagation relationship of Vs and Vp from in-situ geophysical tests. The compressibility of Unit $Qbt3_L$ due to static loads was also evaluated by a series of confined compression tests. The compression and recompression indices, as estimated from the confined compression tests, are measures of the slope of the virgin compression and recompression curves, respectively. The hydro-collapse potential is the percentage of vertical settlement due to inundation at the approximate total static pressure on the subgrade after construction. A summary of average static deformation properties is presented in Table VI-3.

1. Triaxial and Unconfined Compression Tests

Figure VI-15 illustrates the variation of E_i with respect to sample elevation and static profile unit and indicates that the stiffness of intact samples of $Qbt3_U$ and Qbt2 is about 1.5 to 2.5 orders of magnitude higher than that of $Qbt3_L$. However, when applied to engineering analysis and design, the modulus values of the more welded tuff layers have been reduced to account for rock mass discontinuities as discussed in the following subsection.

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Similar to the effect of confining pressure on the compressive strength, the stiffness (E_i) of $Qbt3_L$ increases with the confining pressure of the test, whereas stiffness of units $Qbt3_U$ and Qbt2 are less affected by confinement (Figure VI-16). The distribution of static Poisson's Ratio measurements with respect to elevation is presented in Figure VI-17. Lateral strain measurements and corresponding Poisson's Ratio estimates were feasible only in the triaxial rock frame. Thus, no tests results are available for static profile units $Qbt3_L$ and Qbt2-t. The results of Poisson's Ratio estimated from small-strain geophysical tests are discussed in Section VII of the report.

2. Confined Compression Tests

The compression characteristics of the $Qbt3_L$ at the CMRR site were evaluated to support a quantitative evaluation of potential settlement due to static load on the MAIN base mat. This evaluation was based on results from confined compression tests. The tests were conducted on samples taken from four separate core runs, each from geologic unit $Qbt3_L$. Strength test data discussed previously indicated that modulus values of $Qbt3_U$ and Qbt2 were over an order of magnitude higher than the modulus values of $Qbt3_L$. As such, the $Qbt3_U$ and Qbt2 are relatively incompressible with respect to the MAIN static load ranges. The confined compression test results were also used to develop initial deformation parameters for numerical modeling to develop stress-path testing parameters for stress path-controlled triaxial testing, as discussed in a later section of this report.

Two methods of testing, both based on ASTM D-2435, were employed to evaluate the one-dimensional vertical compression characteristics of the test samples. In the first method, the sample was loaded to increasingly higher vertical pressure and then unloaded with no water added to the sample. This method differs from the ASTM standard, which specifies that the sample be saturated before testing; however, saturation of $Qbt3_L$ due to a rise in the groundwater table is highly unlikely due to its presumed depth at the site. Therefore, performing the tests at the *in-situ* moisture content of the material should give more realistic compressive results.

The second series of tests was performed at the samples' *in-situ* moisture contents up to a loading pressure of about 6.6 ksf, whereupon the samples were inundated to evaluate the effect of hydro-collapse of this unit due to surface water infiltration. The load cycle was then completed to develop compressibility curves under inundated conditions. The percent compression of the samples due to inundation ranged from 1.2 to 3.1 percent, which is

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considered to be of moderate severity (Jennings and Knight, 1975); this amount is not insignificant considering the thickness of this layer and its proximity to the basemat. Due to the small amount of measured compression and the compliance issues of the testing apparatus, the small-sized sample within the loading device, a portion of this collapse could be attributed to sample disturbance.

F. Rock Mass Properties

Laboratory testing of the welded tuff units measures the resistance to shearing stresses through *intact* rock. However, the *rock mass* properties of these units are more appropriate for use in analysis. Rock-mass property values of the tuff are lower than their intact values due to the presence and weathering of discontinuities observed within most of the borings. The effect of these discontinuities can be evaluated using both the Rock Mass Rating and Geological Strength Index systems.

1. Rock Mass Ratings and Geological Strength Index

Rock mass rating (RMR) and geological strength index (GSI) values were estimated for the more rock-like materials encountered within the zone of influence of the proposed CAT I Facility excavation and foundation area. These values are based on geomechanical data collected from the borings drilled within the CAT I Facility building area. These ratings were developed as a means to characterize the mass strength of rock with discontinuities. The spatial variability of these data was analyzed for anomalies that may affect construction excavation support systems and building foundation design.

The RMR system was originally developed by Bieniawski (1976, 1989) as an empirical means of characterizing the rock mass properties on a scale of 0 to 100. This system was comprised of the sum of individual ratings for five basic classification properties:

- Intact rock strength
- RQD
- Joint spacing
- Joint condition
- Groundwater

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Hoek and Brown (1988) and others applied this system to rock mass failure. The RMR and Hoek-Brown failure envelope was modified by the creation of GSI by Hoek et al (1992), which better approximated the failure mechanism in the field, particularly for weak rock masses. If the 1989 version of Bieniawski, RMR classification is used, GSI is estimated from:

$$GSI = RMR - 5$$

RQD measurements are not appropriate for Unit $Qbt3_L$, which is soil-like and generally does not support open fractures. Consequently, RMR/GSI values were not developed for this unit. For the other units, the mean GSI values are summarized in Table VI-4. These values were used to develop rock mass strength failure envelopes, as discussed in the subsequent section.

Contour plots of average GSI values, extrapolated from the CMRR borings, are presented as Figures VI-18 and VI-19 for static profile units Qbt4 (where not completely eroded) and $Qbt3_U$ to illustrate their spatial variability across the proposed building footprint. GSI data were contoured over increments of 5. Each static profile unit exhibited relatively uniform GSI values within the CAT I Facility area. However, GSI values were slightly lower along the eastern part of the footprint area for Qbt4 and $Qbt3_U$.

2. Hoek-Brown and Modified Mohr-Coulomb Relationships

Raw triaxial and unconfined compression laboratory test data were utilized to develop Hoek-Brown (H-B) failure criterion and modified Mohr-Coulomb (M-C) failure envelopes for the rock mass shear strength relationships of units $Qbt3_U$, Qbt2, $Qbt3_L$-t, and Qbt2-t. The generalized H-B failure criterion is an empirically based, non-linear failure envelope for jointed rock masses. The strength reduction criterion may be correlated to a modified M-C relationship, where the apparent cohesion and friction angle are also reduced for discontinuities of the intact rock. A graphical representation of the curvilinear H-B and M-C failure envelopes is presented in Figure VI-20. The criteria were not applied to unit $Qbt3_L$ due to the soil-like nature of this stratum.

GSI values were used to adjust the H-B failure criterion envelope and account for rock mass discontinuities. In addition, the H-B material constant and disturbance factor, D, were used to further modify the appropriate rock-mass failure envelope. The disturbance factor, D, characterizes the disturbance of the rock mass due to factors such as blasting, excavation, and stress relief. Typical values of D are provided by Hoek and Diederichs (2006) and may vary from 0 for undisturbed rock masses to 1 for very disturbed rock masses. Failure envelopes for

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units within the projected zone of the MAIN excavation were calculated for both undisturbed (D=0) and disturbed (D=0.7) rock mass conditions to account for mechanical excavation disturbance, as well as D=1.0 to evaluate the trend of reduced strength due to higher disturbance. For the units below the base of the excavation, only the D=0 case was calculated. Figure VI-21 displays equivalent M-C failure envelopes for unit $Qbt3_U$ at various disturbance factors.

As shown in Figure VI-21, the failure envelope has been applied over a range of normal pressures extending to about one-half of the unconfined compressive strength. Because the range of normal pressures extended well beyond the range of interest for CMRR construction, the modified M-C relationship was replotted over the applicable range of normal pressures anticipated for subsequent stability analyses. M-C relationships of $Qbt3_U$ and $Qbt3_L$-t were plotted for normal stress ranges up to a maximum excavation depth of 60 ft utilizing the unit weights of the geomechanical units for an estimated maximum normal pressure of between 0 to 40 psi. For Qbt2-t and Qbt2, which are below the project excavation base and at higher *in-situ* pressures, the M-C failure envelope was replotted at 0-80 psi and 0-100 psi, respectively. The resulting modified M-C and H-B properties are shown graphically in Figures VI-22 to VI-25 and summarized in Table VI-5.

G. Compaction and Shear Strength Characteristics of Remolded Tuff Samples

It is expected that much of the *in-situ* material excavated for the proposed CAT I Facility, namely units Qbt4 and $Qbt3_U$, will be stockpiled and later used as compacted structural fill. However, we understand that much of Qbt4 material will be used prior to CMRR construction as embankment fill for the new TA-55 parking lot. Laboratory testing was performed to evaluate the characteristics of these materials when used as compacted structural fill. This included selecting samples that would likely be used for compacted structural fill, disaggregating these materials, remolding the disaggregated materials, and performing laboratory tests on these remolded samples.

The characteristics of the remolded materials at the CMRR site were determined from Modified Proctor tests (ASTM D-1557, Method A) and direct shear tests (ASTM D-3080). The tests were performed on representative samples from units Qbt4 and $Qbt3_U$ collected from borings throughout the site.

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Hammers and other hand tools, simulating the effects of heavy equipment, were used to mechanically disaggregate the samples used in testing. Materials from one to three borings were mixed for each of eight composite samples. Grain-size distribution tests were performed to evaluate the classification of the composited material. The composited materials were used in subsequent Proctor and direct shear testing. In addition to the eight composite specimens, five samples from borings SSC-1, SSC-2, SSC-3, and SSC-4 were composited in the same manner and used in Modified Proctor testing during the phase one portion of the program.

Based on the grain-size distribution tests, the eight composite specimens were categorized as a silty sand (SM) material, in accordance with the Unified Soil Classification System, with fines from 21 to 33 percent. The five composite samples were also broken up and recompacted, but appear not to have broken down to the same degree as the latter samples because they showed considerably less fines, ranging from 8 to 13 percent. These materials classified from a silty sand (SM) to a poorly graded silty-gravel (GP-GM).

Two of the eight composite specimens were composed of material from unit Qbt4. The other six composited specimens and the five phase one samples were composed of material from unit $Qbt3_U$. The estimated maximum dry density and optimum moisture content results for each specimen are presented in the GDR. The range of test results and average moisture density relationships are presented in Table VI-6.

Direct shear tests were performed on the eight composite specimens compacted to at least 95% maximum dry density per ASTM D-1557. These specimens were tested to failure at three different states of stress, allowing for the estimation of shear strength parameters. The average remolded material characteristics of each static profile unit are summarized in Table VI-6. Figures VI-26 and VI-27 graphically display the Mohr-Coulomb peak failure envelope for units Qbt4 and $Qbt3_U$.

H. Stress Path Controlled Triaxial Compression Tests

Load controlled stress-path triaxial testing was performed on samples of $Qbt3_U$ and $Qbt3_L$ material. The purpose of the testing was to simulate the effect of the excavation and construction of the CAT I Facility on the underlying materials. The stress-paths were obtained from an analysis of the excavation and construction using the explicit, finite-difference program FLAC (Itasca, 2006). The analysis simulated the excavation and construction (loading) sequence of the CAT I Facility. The history of displacement, volumetric and shear strain along

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with major and minor principal stresses were monitored. The values for major and minor principal stress developed in analysis time (step time, not physical time) were used to develop the loading sequence and timing of the change in loading. The effect of principal stress rotation is not possible to simulate in the triaxial test device, however principal stress rotation did occur in the analysis. Examples of the stress-paths in $Qbt3_U$ and $Qbt3_L$ are shown in Figures VI-28 and VI-29. The stress-paths are plotted in p - q space using the Cambridge definition (Atkinson, 1993) summarized below.

$$p = \frac{1}{3}\left(\sigma_1 + 2\sigma_3\right)$$

$$q = \left(\sigma_1 - \sigma_3\right)$$

where σ_1 = major principal stress and

σ_3 = minor principal stress

A loading rate of 2.0 lb/min was chosen for the stress-path tests. The major and minor principal stresses were reduced to loads using the dimensions of the sample, and the change in load between each analysis step was divided by the loading rate to create an artificial time step at which to conduct the analysis. The testing was conducted using samples at their natural water content, essentially unsaturated, as such, the loading is considered drained.

As the loading was developed from the analysis it is assumed that the loading follows the exact stress-path developed, exclusive of potential principal stress rotation effects. The remaining measures for quantifying the material behavior are the axial and volumetric strain. The discussion of the results of the stress-path testing is included in Section IX, Geotechnical Analysis and Discussion.

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VII. DYNAMIC ENGINEERING PROPERTIES

The proposed CAT I Facility has been classified as PC-3, which requires that site-specific seismic design criteria be developed and utilized for design. The MAIN will be a large, rigid below-grade facility. Therefore, soil structure interaction (SSI) analysis must be performed to ascertain the influence of the interaction of the structure and the surrounding soil on the response of the structure to the predicted site free-field ground motion. Others will perform the site free-field ground motion analysis as part of the PSHA. Input to the ground motion analysis includes baseline dynamic properties, which form the basis of the dynamic soil column beneath the site.

This section addresses the baseline (non-strain-compatible) dynamic engineering properties and profile for CMRR including:

- a site-specific unit profile for use in dynamic analyses,
- descriptions and comparisons of various field and laboratory dynamic test results performed for CMRR,
- mean values of dynamic engineering properties, and
- characterization of associated uncertainties of dynamic properties used for SSI in accordance with ASCE 4-98.

The results of this section were developed for input into an equivalent linear ground response analysis, which will generate strain-compatible soil properties (iteratively adjusted for the earthquake strain levels) and free-field (ground surface) three-component time histories through iterative procedures. The free-field three-component time histories and strain-compatible soil properties will then be used by others as input into an SSI analysis. The properties developed and presented herein include the following:

- Shear wave velocity,
- Compression wave velocity,
- Poisson's Ratio,
- In-Situ Unit Weight,

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- Low-strain shear modulus,
- Low-strain material damping ratio, and
- Nonlinear relationships indicating the variation of normalized shear modulus and material damping with shear strain.

These properties were developed for each unit of the dynamic profile from the ground surface to the basement rock level, which is the dacite layer encountered at approximately El 6597.5 for CMRR. The spatial variability of these properties across the footprint of the MAIN was evaluated to estimate parametric variability and uncertainty.

The dynamic profile is similar in most respects to the geologic/stratigraphic profile; however, its unit boundaries are determined based on dynamic properties. The primary methodology for selecting the units of the dynamic profile was based on interpretation of the field geophysical test results (discrete shear wave velocity measurements). The changes in geophysical properties with depth at a particular site do not necessarily correspond to the changes in stratigraphic layering. However, the *in-situ* geophysical tests information, as well as geophysical data developed from other portions of this project (e.g., natural gamma logs and SPT blow counts), correlated well with the stratigraphy and were used in the development of the dynamic profile.

Shear and compression wave velocities were measured through three seismic velocity field surveys consisting of downhole, crosshole, and suspension log techniques. Two laboratory test programs were also conducted; one at the University of Texas at Austin (UT), consisting of fixed-free Resonant Column/Torsional Shear (RC/TS) tests and unconfined, free-free resonant column (URC) tests; the other, at the University of California, Berkeley (UCB), consisting of cyclic simple shear (CSS) tests. Data from these field and laboratory programs are included in the GDR.

In the process of evaluating data from these various sources, it was apparent that there were inherent differences between data types. Typically, the data are weighted based on evaluation of quality and uncertainty to account for:

- Differences between methods used to measure the same property; e.g., seismic velocity in the field,

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- Sampling disturbance,
- Differences between mass properties and intact, small-scale sample properties, and
- Differences between stress and deformation boundary conditions in-situ versus those conditions in the laboratory.

Although a numerical weighting factor was not ultimately applied because of the difficulty of quantifying the many differences, the relative value or credibility of data types was evident.

A. Dynamic Site Profile

The geologic nomenclature of Broxton and Reneau (1995) for the description of the geologic profile was used as the preliminary basis for selecting the layers of the dynamic profile. As noted in Broxton and Reneau, additional subunits may be specified within the units to identify significant differences in lithology or rock properties. Due to the difference in properties within Qbt3, this unit was subdivided by KA for this project into the moderately hard, moderately welded $Qbt3_U$ and the soft, poorly welded (includes non-welded) $Qbt3_L$. The presence of this softer layer between two stiffer tuff units $Qbt3_U$ and Qbt2 is important with respect to the seismic ground response of the site because of the proximity of $Qbt3_L$ to the base of the MAIN and the relatively high impedance contrasts that exist between the units near the foundation of the MAIN.

1. Deep Profile Units

The lowest element of the dynamic profile is the basement rock. The basement rock underlies geologic layers with relatively low velocity, causing the basement rock to act like a seismic platform or basement below which seismic velocities of rock units have no substantial influence on SSI analysis. The basement rock of this site was encountered in boring DSC-1B at a depth of about 697.5 ft (El 6597.5) and consists of Tschicoma dacitic lava (dacite). At least three distinct flows were identified in the 43.5 ft of basement rock penetrated at the bottom of boring DSC-1B, but the total thickness is probably several hundred feet. The upper boundary is heavily fractured and vesicular, which reduces the overall rock mass stiffness.

The preliminary dynamic profile above the basement rock was based on the stratigraphy logged in the test borings. Five types of field measurements were used to refine the preliminary profile. The first two methods, suspension logging and gamma logging, were used to characterize and

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differentiate the deep layers below 150 feet. Crosshole velocity, SPT measurements, and downhole velocity logging were used to evaluate units within the upper 150 feet.

Suspension logging measures seismic wave velocities over discrete vertical intervals, making it especially useful in detecting boundaries between different units. Figure VII-1 presents the Receiver 1 to Receiver 2 (R1-R2) suspension log results from borings DSC-1B and DSC-2A superimposed with the preliminary stratigraphic data. There is reasonably good agreement between the geologic logging of deep strata and the geophysical results for both borings. As anticipated, there is more heterogeneity observed in the sedimentary Qct layer.

Natural gamma logging detects *in-situ* gamma emissions within the formation surrounding the borehole. Two adjacent but different geologic units will generally have different gamma emissions. This difference in gamma emission, termed signature, was useful in delineating different units. The natural gamma results exhibit notably distinct signatures between the various cooling units of tuff as illustrated in Figure VII-2. The gamma signature records show a good correlation between the two borings; only Units Qbt1g and Qct appear to have notably different gamma signatures between boreholes.

Based on the good correlation between these two geophysical methods and the stratigraphy, the logged strata breaks were used to define the deep profile units extending from Qbt2 to dacite. The only exception is the Guaje Pumice layer and adjacent alluvial sediments layer that were combined because of their relatively small thickness.

2. Shallow Profile Units

Crosshole seismic velocity testing was used to measure shear wave velocity at discrete elevations between adjacent boreholes. The results of the crosshole testing are illustrated in Figure VII-3. In this figure, the colorized background delineates the preliminary dynamic profile, which was refined from the stratigraphic profile using interpretation of the geophysical data and to a lesser degree the SPT results. The results of both the crosshole and SPT support the creation of transition layers above and below $Qbt3_L$. The four dashed lines from top to bottom, shown in Figure VII-3, illustrate the logged top of stratigraphic units Qbt4, $Qbt3_U$, $Qbt3_L$, and Qbt2, relative to the initial transition zones selected.

While the depth interval of downhole travel time measurements was not small enough to interpret the transition zones, the points of best-fit lines through adjacent layers could be used to connect the travel times through the transition zones. The downhole results were useful in

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defining the upper layer of the dynamic profile. First, the upper materials at the site (includes existing fill and residual soils, all of Qbt4, and approximately the upper ten feet of $Qbt3_U$) exhibit relatively similar seismic downhole and crosshole shear wave velocity. Consequently, these materials are classified as the Near Surface layer. It is uncertain why this upper ten feet of $Qbt3_U$ exhibits lower seismic velocity than the remaining $Qbt3_U$, but it is likely the result of greater weathering and/or higher fracture frequency. The inclusion of the $Qbt3_U$ in the Near Surface unit is primarily justified based on the seismic velocity data, but was also supported by static laboratory test data, including unit weight and compression testing.

RC/TS testing was performed on samples from eight separate stratigraphic units. While there is a difference in lab versus field values, as discussed later in this section, the shear wave velocity and material damping per layer support the dynamic profile model.

3. Summary of Dynamic Profile

Based on the analysis of geophysical field and laboratory testing, a dynamic profile was developed for the site from the ground surface to the basement rock and includes the following units:

- Near Surface
- Qbt3U
- Qbt3L-t
- Qbt3L
- Qbt2-t
- Qbt2
- Qbt1v
- Qbt1g
- Qct
- Qbo
- Pumice and Sediments
- Dacite

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B. Linear Dynamic Properties

Linear dynamic properties are those that characterize the dynamic response of the dynamic profile units at relatively low shearing strains, where material response is linear. The following linear properties were evaluated for the dynamic profile units at the CMRR site:

- Shear and compression wave velocities (VS, VP),
- Poisson's ratio (ν),
- Low-strain shear modulus (Gmax), and
- Low-strain damping ratio (Dmin).

The low-strain linear dynamic properties presented in the following section were developed from *in-situ* seismic velocity measurements and laboratory RC/TS testing.

1. Coefficient of Variation

Uncertainty in soil properties consists of the inherent randomness, as determined from statistical evaluation of soil data, plus increased uncertainty when only a limited number of data points (borings) are available (small sample population). The coefficient of variation (COV) includes the components related to inherent randomness of soil properties, COV_{IR}, and additional uncertainty due to the limited number of data points, COV_{RS}. COV is defined by the following equation:

$$\mathrm{COV} = \sqrt{(\mathrm{COV_{IR}})^2 + (\mathrm{COV_{RS}})^2}$$

Where:

$$COV_{IR} = \frac{\sigma_x}{\bar{x}} = \frac{\text{standard deviation}}{\text{mean}}$$

$$\mathrm{COV_{RS}} = \frac{\mathrm{COV_{IR}}}{\sqrt{\mathrm{n}}}$$

where n is number of samples.

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SSI analyses will consider the soil down to a lower soil boundary as specified by ASCE 4-98. This lower soil boundary is well below the 150-foot depth of the four SSC borings within MAIN. As a result, the two deeper borings, DCS-1 and DCS-2, were drilled to greater depths to provide soil data needed for SSI analyses. Fewer samples at greater depth are acceptable because the properties at these greater depths generally have a diminishing influence on the SSI analyses.

Per ASCE 4-98, uncertainties in defining the structural model and soil properties may be adequately accounted for by considering three soil cases based on varying the low-strain shear modulus, G_{max}, for all locations throughout the soil profile. The three cases are:

1. best estimate soil case based on G_{max}
2. lower bound soil case based on G_{max} divided by (1+COV)
3. upper bound soil case based on G_{max} multiplied by (1+COV)

COV values were calculated for low-strain velocity measurements, as discussed in the following sections, to characterize the variability of the material between test locations and between various geophysical methods. A summary of best estimate low-strain wave velocities and recommended COV (considering corroborating measurement techniques) is presented in Section VIII of this report. It is the responsibility of the SSI modeler to establish the upper and lower bound soil cases in accordance with ASCE-4-98.

2. Seismic Wave Velocities

The measured seismic shear and compression wave velocities (V_s and V_p) were evaluated by different comparative methods for shallow and deep profile units. The deep unit profiles (150 to 700 ft) illustrate V_s and V_p measured using the downhole and suspension log techniques. The upper unit profiles (0 to 150 ft) show the shear and compression wave velocities measured using the downhole and crosshole techniques.

Detailed shear wave velocity profiles were represented previously in Figure VII-1 for the suspension logging and in Figure VII-3 for the crosshole logging. Each of these *in-situ* measurement techniques was analyzed in the downhole or simulated downhole travel time format. In this simulated analysis technique, the wave travel time for successive measurements were plotted over a vertical distance converted from the discrete wave velocity. By this method, average velocities may be compared over dynamic profile units, whose boundaries were iteratively changed if necessary.

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The interpretation of the raw data included the following:

- Conversion of data to simulated downhole format,
- Least-squares analysis to determine the best fit line over the unit profile,
- Comparison of best-fit lines to dynamic profile units and iteratively changing the profile to match the linear data trends,
- Adjust the overall travel time best-fit relationships by engineering judgment so that the end points of adjacent slopes are connected,
- Calculation of average wave velocities per layer,
- Comparison of average wave velocities with measurements at other borings to develop mean and standard deviation values if sufficient number of tests are available, and
- Calculation of Coefficient of Variation (COV), (with engineering judgment used as needed due to level of uncertainty to increase this value).

The dynamic profile of the various stratigraphic units has been subdivided into three zones including 1) the basement rock, 2) the deep dynamic profile units, and 3) the shallow dynamic profile units. The following subsections present the seismic wave velocities developed for each of the various units as grouped by the three zones listed.

a) Basement Rock

Because of the limited penetration of downhole and suspension logging surveys into the dacite unit and the fractured nature of this material, there is increased uncertainty in the velocities measured in the basement rock zone than the layers above it. The reinterpreted V_S and V_P values of downhole travel times measured between a depth of 700 and 720 feet were 5,280 ft/sec and 7,920 ft/sec, respectively (Figure VII-4a). Suspension log data converted to a simulated downhole format indicates average V_S and V_P values of 3,540 ft/sec and 6,990 ft/sec, respectively between roughly this same interval (Figure VII-5a). As illustrated in Figure VII-2, the discrete R1-R2 measurements are highly variable in the upper 10-ft of the dacite layer, likely the result of variable fracturing in this depth interval. However, a constraint on the range of possible velocities is provided by the laboratory measurements of intact dacite core. Unconfined free-free resonant column (URC) tests resulted in average V_S and V_P values of

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about 9,200 ft/sec and 14,800 ft/sec, which provide an upper bound on the velocities of the rock mass because the intact core does not contain the macro-scale discontinuities present in the field. KA estimated rock mass modulus reduction values based on RMR and GSI values to be applied to URC results of intact dacite core. Boring geomechanical data that was recorded during the core runs of this layer were used to develop a dacite layer average RMR and GSI in accordance with Hoek and Brown (1997). Hoek and Diederich's method (2006) of reducing elastic modulus was used and subsequently converted to equivalently reduced wave velocity values. From this comparative analysis, V_S and V_P values of 5,800 fps and 9,300 fps, respectively, were estimated for the rock mass dynamic property of the dacite layer. V_S and V_P values of 5,500 ft/sec and 8,800 ft/sec, respectively, and a COV of 0.41 was selected as the best estimate wave velocity values for dacite based on engineering judgment.

b) Deep Dynamic Profile Units

Deep dynamic profile units consist of the layers above the dacite basement rock through Qbt2. Travel-time plots and the interpreted average velocities for the two deep borings are shown in Figures VII-4a and VII-4b for downhole measurements and in Figures VII-5a and VII-5b for suspension log measurements. The linear interpretation of wave velocities across the deep profile units was possible without iteratively changing the layer boundaries. While there was often only a modest increase in wave velocities between successive layers of Qbt1g through Qbo, similar trends and averages were developed throughout the deep profile for the two measurement locations and between the two techniques.

The good corroboration between the measurements is evident in the average V_S and V_P values within each unit (Figure VII-6, Tables VII-1a and VII-1b). V_S and V_P measured in the laboratory at the estimated mean effective stress are also shown in Figure VII-6. In the more rock-like materials (e.g. Qbt2 and the dacite) the field measurements are lower than those on small diameter laboratory specimens due to the effects of discontinuities in the former that are not present in the latter. In the more soil-like units, which do not readily support fractures, the velocities measured in the field are greater than those measured on small diameter laboratory specimens, and are likely attributed to sample disturbance in the laboratory specimens.

The agreement between the downhole and suspension log measurements is good, and the small number of measurements limited the opportunity to perform rigorous statistical analyses. Therefore, the actual measurements, which range in number from two to four at any given depth, have simply been averaged to obtain mean vertical wave velocities (Figure VII-7).

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Three or four measurements may be considered insufficient for rigorous statistical analyses; nevertheless, the standard deviation and coefficient of variation (COV) values presented in Tables VII-1a and VII-1b support discussions about the good data agreement in the preceding paragraphs.

The COVs listed in Tables VII-2a and VII-2b include both the inherent variability of the material and random errors in the measurements. They do not include systematic errors that might be contained in any one set of measurements or errors due to the boring locations being unrepresentative of the footprint as a whole (additional forms of epistemic uncertainty), although these errors are believed to be small in this case.

While aleatory uncertainty is important in some kinds of predictive computations, such as the assessment of seismic hazard, the aleatory component of uncertainty in wave velocities, and in modulus reduction and damping curves that will be discussed subsequently, is less significant in site response calculations because, within a limited area such as the footprint of the CMRR building, compatibility of displacements requires that adjacent columns or elements of soil vibrate together so that the material properties must be effectively averaged. Only when a broader area, such as the entire LANL complex, is considered should the possible variations in the profiles and properties be accounted for. Thus, the COV values that are shown in Tables VII-1a and VII-1b may be conservative for application to site response analyses conducted only for the CMRR project.

c) Shallow Dynamic Profile Units

Travel time plots and the interpreted average velocities of the shallow profile units are shown in Figures VII-8a to VII-8f for downhole measurements and in Figures VII-9a to VII-9d for crosshole measurements. Because there are insufficient data points within the transition zones themselves, these slopes were evaluated only after the estimated slopes above and below the transitions were completed by connecting the travel time lines through the top and bottom of a particular transition zone. With the delineation of the near surface layer and the transition layers, acceptable slopes of the travel times were consistently attained for both measurement methods and each upper unit.

The interpreted average downhole V_S and V_P values within each upper unit are consistent when compared between borings (Figure VII-10 and Tables VII-2a and VII-2b), particularly for Unit

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$Qbt3_L$. Similar relations are evident for the crosshole velocity measurements in Figure VII-11 and Tables VII-3a and VII-3b.

V_s and V_p values measured by the laboratory RC/TS and URC tests compare well to crosshole test results (Figure VII-11). In particular, there is exceptionally good agreement between lab and field values for Unit $Qbt3_L$.

The arithmetic averages of the shear wave velocities measured using the downhole and the crosshole techniques are concordant, as illustrated in Figures VII-12. The vertical and horizontal compression wave velocities, however, are significantly different presumably due to the anisotropic nature of the tuffs at the CMRR site. Confirmation of anisotropic properties in $Qbt3_L$ is detailed in the report by the University of Texas at Austin (Stokoe et. al., 2006a) on the crosshole testing, which describes how measured V_P increased as the direction of the compression waves was rotated downward from horizontal to vertical. As explained by Dr. Stokoe, there is no difference for shear wave velocity (and shear modulus) for anisotropic material because the plane of shear wave propagation is equivalent for both vertically and horizontally propagating shear waves.

The uncertainty in the field measurements of downhole and crosshole seismic velocities is indicated by the COVs values, computed assuming a normal distribution, that are shown in Tables VII-2a, VII-2b, VII-3a, and VII-3b. Although these values have been computed using the average velocities for each unit rather than the point velocities at a given depth, the uncertainties are generally rather small, indicating both uniform materials and high quality measurements. For the critical $Qbt3_L$, the average V_s values measured using the downhole and crosshole techniques are 1040 ft per second (ft/sec) and 1065 ft/sec, respectively, and the coefficient of variation in each case is estimated to be 0.04. Formal error propagation analyses have not been conducted for any of the field velocity measurement techniques. However, the good agreement between the results for the downhole and suspension log techniques in the deep borings and the downhole and crosshole techniques at shallower depths suggests that any systematic errors are small and negligible.

Estimates of uncertainty for use in site response analyses should also take into account the possibility of areal variations in the materials away from the sampled and tested boreholes. However, these appear to be small, and there is no geophysical reason to expect significant or systematic variations of properties within the building footprint.

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3. Poisson's Ratio

The inferred values of Poisson's Ratio, ν, for the dynamic profile units at the CMRR site were obtained using the theory of wave propagation through a homogeneous, isotropic, elastic media. The ratio of compression to shear wave velocity is related to ν by the following equation:

$$\frac{\mathrm{Vp}}{\mathrm{Vs}} = \sqrt{\frac{2-2\nu}{1-2\nu}}$$

Since there is apparently some structural or stress-induced anisotropy at the site, all assumptions of the equation are not correct. However, it is proposed that the data from dynamic measurements in the vertical direction (i.e. downhole and suspension logging) be used for SSI calculations since the analysis is based on the upward propagation of seismic waves. These values will be designated as vertical Poisson's Ratio, ν_v. The horizontal Poisson's Ratio presented for comparison is designated as ν_h.

Figure VII-13a illustrates the inferred Poisson's Ratio calculated from the downhole and suspension log velocities in the deep profile. Both the downhole and suspension log surveys measure wave velocities in the vertical direction. Therefore, it is appropriate to use both the downhole and suspension log wave velocities to obtain an averaged deep-profile ν_v, presented in Figure VII-13b and Table VII-4.

Figure VII-14a shows the inferred ν_v values evaluated from downhole wave velocities for the upper units. Likewise, Figure VII-14b shows the inferred ν_h determined from crosshole measurements. Because crosshole surveys measure horizontal compression wave velocities, the resulting Poisson's Ratio is also only relevant to the horizontal direction. Figure VII-14c, Tables VII-5a, and VII-5b present the inferred ν_h and ν_v values based on averaged crosshole and downhole wave velocities. As expected, considerable difference between the vertical (downhole) Poisson's Ratio and the horizontal (crosshole) Poisson's Ratio can be seen in the figure with ν_h at 0.12 or less and ν_v ranging from 0.24 to 0.39.

4. Low-Strain Shear Modulus and Material Damping Ratio

Low-strain shear modulus was determined using the shear wave velocity for each unit of the dynamic profile. To obtain the low-strain shear modulus, G_{max}, the following relationship was used:

$$G_{max} = \gamma_t / g * V_s^2$$

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where G_{max} is shear modulus at low shear strain (typically lower than 10^{-3} percent) and γ_t is moist unit weight and g is acceleration due to gravity. The moist unit weight used in the calculation was obtained from the average laboratory-measured intact unit weight of each layer.

As illustrated in Figure VII-15, G_{max} is often affected by the state of confinement, particularly for the poorly-welded tuff and alluvial soil layers. The linear stiffness relationships of the eight RC/TS test layers fall into five groups, as detailed in Stokoe, et al, 2006a. The log G_{max} versus log σ_o relationship for Unit $Qbt3_L$ is similar to the trendline for loose sand (Menq, 2003), which is parallel but slightly lower in magnitude. Units Qbt1v, Qbt1g, Qct, and Qbo form an aggregate trendline that is also parallel to, but slightly higher than, the loose sand trendline. Conversely, the more rock-like tuff units are less affected by confinement. This is shown by the relatively flat trendlines in which the overall magnitude of stiffness increases for $Qbt3_U$, Qbt2, and dacite.

Low-shear strain material damping ratio, D_{min}, was also measured from RC/TS tests. D_{min} for all materials was between 0.3 and 2.0 percent. There was typically a trend of decreasing D_{min} with a corresponding increase in confining pressure, but the difference between units was not as clear as in the G_{max} data (Stokoe et al., 2006a).

C. Nonlinear Dynamic Properties

Nonlinear dynamic properties are those that characterize the dynamic response of the stratigraphic units at higher shearing strains (typically greater than 10^{-3} percent to more than 10^{-1} percent). The nonlinear properties determined for the geologic units at the CMRR site were presented as normalized shear modulus (G/G_{max}) and material damping ratio (D) relationship with log-shearing strain based on RC/TS testing by Stokoe et al. (2006a). Families of normalized shear modulus and material damping ratio versus log-shearing strain relationships have been presented in Stokoe et al (2006a), but the data are re-presented herein so that adjustments to the laboratory results can be made for individual dynamic profile units.

1. Normalized Nonlinear Shear Modulus and Material Damping Ratio Relationships

Relationships of the laboratory data for G/G_{max} and D versus shearing strain are shown by dynamic profile unit in Figures VII-16a to VII-16h. Best-fit lines through the data are also shown in these figures using the same modified hyperbolic relationship of Darendeli (2001) used by Stokoe et al (2006a). In the context of this relationship, the reference strain, γ_r, is the shearing strain at which G/G_{max} equals 0.5. The parameter “a” is a curvature coefficient that alters the shape of the hyperbola and thus the nonlinear curves.

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Stokoe et al. (2006a) used the same values for γ_r and parameter "a" that were established for fitting the G/Gmax data to show a relationship between damping ratio and shear strain; however, for equivalent linear analyses, the shear modulus reduction and damping curves can be independent. Thus, γ_r and parameter "a" were evaluated independently to obtain the best-fit lines through the material damping ratio results.

There was a good correlation between the hyperbolic curve fits and the actual test data. The hyperbolic curve fits were extended beyond the range of actual test strain, as indicated by the dashed lines, to 1 percent. The rock-like samples could possibly fail in shear prior to reaching this strain level during an earthquake. Based on axial failure strain from triaxial tests that were subsequently converted to shearing strain, a failure strain of about 0.3 to 0.7 percent is estimated for units Qbt2 and $Qbt3_U$.

For the $Qbt3_L$ samples, nonlinear relationships for each of eight samples were individually evaluated using the Darendeli approach for G/G_{max} (Figure VII-17a) and D (Figure VII-18a). Distributions of the values at $3x10^{-2}$ percent shearing strain, a common reference point used by ground response modelers, are plotted on Figures VII-17b and VII-18b. While these distributions are commonly assumed to be log-normal, in this case there does not appear to be strong support for that assumption. The COV for G/G_{max} at a shearing strain of $3x10^{-2}$ percent is less than 0.03 assuming a normal distribution; this is consistent with other measurements, as well as visual inspection of samples suggesting that $Qbt3_L$ is a rather uniform material.

The scatter in the measured damping ratios is greater than shear modulus but still yields a COV on the order of only 0.1 assuming a normal distribution. This scatter results from two causes; one is that there may be variations in the actual material properties from point to point (again, aleatory uncertainty, which can be averaged for site response purposes), and the other is that test specimens may have inhomogeneities, such as the lithics or pumice fragments found in the $Qbt3_{L,}$ or micro fractures that may be found in the more rock-like specimens, which adversely affect a particular test. These inhomogeneities might generally be expected to make the behavior more nonlinear so that there will be a systematic error in that direction, but any such errors appear to be relatively small.

The best-fit or mean normalized shear modulus and material damping ratio curves for each layer are shown in Figure VII-19. For reference, the families of curves for generic low plasticity index (PI) soils as a function of depth included in EPRI (1993) are shown as background in the

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figure. The nonlinear G/G_{max} relationship for the $Qbt3_L$ is generally similar to the EPRI curve for 120 to 250 ft depth, suggesting that this material behaves like dense sand in the nonlinear shearing strain range. However, the tuffs generally appear to show lower damping values than standard low PI soils. These curves are simply a best fit through the laboratory data and take no account of possible differences between field and laboratory conditions as discussed later in this section.

Sampling and testing of soil and rock for the purpose of measuring G/G_{max} and D can be affected by two or more factors that work in opposite directions, including:

- Confining Pressure
- Number of Loading Cycles
- Plasticity/Overconsolidation Ratio
- Natural Cementation/Welding
- Sample Size
- Sample Disturbance

The uncertainty associated with these factors may be significantly greater than the uncertainty that results from the scatter in measured values from multiple test specimens. Well-indurated but fractured or fissured rocks and materials such as stiff, fissured clays may be expected to show higher velocities and more elastic behavior in relatively small samples tested in the laboratory.

Other soils and soft rocks may be affected more by the stress relief and disturbance that are inevitable, even with relatively high-quality sampling. These effects reduce the measured velocities and, presumably, yield more nonlinear modulus reduction and material damping ratio values. These effects were minimized by taking larger diameter samples and carefully trimming test specimens.

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2. Adjustment of Normalized Nonlinear Shear Modulus and Material Damping Ratio Relationships

The difference in field- and laboratory-measured shear wave velocities provides some indication of whether the sampling factors and rock mass properties presented in the previous section are significant. However, because of the difficulty of measuring nonlinear shear modulus and material damping ratio relationships *in-situ*, it has historically been difficult to adjust laboratory values to field values. One approach, which has been selected for use herein, is based on the observed relationship between low-strain modulus (or shear wave velocity) and γ_r, as defined by Darendeli (2001). The relationship, as discussed in Roblee and Chiou (2004), indicates that the normalized reference strain ratio is proportional to the laboratory-to-field velocity ratio. This premise is based on suggestions by Pyke (1998) as part of the ROSRINE project, which involved extensive field and laboratory measurements at accelerometer sites in Southern California.

The laboratory-to-field shear wave velocity ratios for the CMRR samples are shown in Figure VII-20a for the $Qbt3_L$ and in Figure VII-20b for all other units. Figure VII-20a indicates that the data plot near the line of equity between the laboratory- and field- V_s values, suggesting that there was minimal disturbance to these samples. The difference in the field and laboratory measured values might, in fact, be due to the laboratory samples being reconsolidated to a state of stress that is different from *in-situ*. Figure VII-20b indicates V_s ratios (Vs_{lab}/ Vs_{field}) plot above the line of equity for $Qbt3_U$, Qbt2 and dacite materials and below the line of equity for the other materials. This is expected because the former materials are more rock-like; V_s of the rock mass is expected to be lower than that of intact specimens because of fracturing, while the other materials are more soil-like and thus are more subject to sample disturbance. The most anomalous behavior is shown by the $Qbt3_U$ unit, which, although generally rock-like in appearance and behavior, shows the most pronounced nonlinearity (Figure VII-19).

The average laboratory-to-field shear wave velocity ratios are listed in Tables VII-6 and VII-7 along with the reference strains for the modulus reduction and damping curves, respectively. Neglecting the possible effects of the differences in laboratory and field states of stress and applying the average V_s ratio for each unit as a correction to the reference strain, adjusted reference strains were computed (Tables VII-6 and -7). In turn, adjusted nonlinear G/G_{max} and D relationships are shown in Figure VII-21. EPRI curves are superimposed upon Figure VII-21 to show the relationships between these data.

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Tables VII-6 and VII-7 list the values of G/G_{max} and D at a shear strain of $3x10^{-2}$ percent before and after the adjustment of the reference strains. The changes in the values of G/G_{max} are generally less than 0.05 while the changes in the damping values are more typically on the order of 0.2.

D. Dynamic Compression Properties

Seismically induced compaction (SIC) is a geologic hazard associated with earthquake ground shaking. During the phase one field investigation, concern was raised regarding the potential for SIC to occur within the $Qbt3_L$ unit. A testing program was initiated with phase SBT and carried through to the phase two field investigation. The laboratory-testing program consisted of performing cyclic simple shear (CSS) tests on undisturbed samples of $Qbt3_L$. A total of fourteen production tests were performed on these materials to evaluate their dynamic compression properties.

Production CSS testing was performed using the estimated parameters of cyclic shear stress ratio (CSR) and number of significant stress cycles. Cyclic Shear Stress Ratio is depth-dependent and is a measure of the average shear stress developed during an earthquake divided by the vertical effective stress. One simplified expression for the CSR developed by an earthquake is given by the equation:

$$CSR \approx 0.65 \times (a_{max}/g) \times (\sigma_o / \sigma_{o'}) \times r_d \quad \text{(Seed and Idriss, 1982)}$$

where

a_{max} = Peak ground surface acceleration in units of gravity

g = acceleration of gravity

σ_o = Total vertical stress

$\sigma_{o'}$ = Effective vertical stress

r_d = Stress reduction coefficient which is a function of depth below the ground surface.

Cyclic stress ratios or shearing strains relative to a design basis earthquake can also be estimated by means of a site-specific ground response analysis. For the work described in this report, design CSR values were estimated using empirical equations including the equation provided above.

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The CSR value at any given depth is influenced by the magnitude of an earthquake, the total and effective confining stress, and the depth below the ground surface. In general, a stronger earthquake corresponds to a higher CSR value.

An approximate value of CSR was first estimated using the simplified method given above (Seed and Idriss, 1982). A subsequent CSR value was estimate using a more recent method proposed by Seed et al (2003). The values given by these two equations were averaged and used as the baseline CSR value (CSR_B) for laboratory cyclic simple shear (CSS) testing discussed in Chapter IV of this report.

The design earthquake magnitude and peak ground acceleration used to determine CSR_B were derived from the 1995 Woodward Clyde Federal Services seismic investigation of Los Alamos (WCFS, 1995). Two analyses were performed in that investigation, a Probabilistic Seismic Hazard Analysis (PSHA) and a Deterministic Seismic Hazard Analysis (DSHA). The PSHA analyses estimated a peak ground acceleration of 0.33 g and an earthquake magnitude of 6.0 for a 2,000-year return period event. The DSHA analyses estimated a peak ground acceleration of 0.5 g and an earthquake magnitude of 7.0. The PSHA analysis parameters were chosen to calculate the CSR_B values because the 6.0 magnitude event is the Design Basis Earthquake (DBE) for the CMRR facility.

Using the two empirical estimates, the CSR ranged from 0.12 to 0.19, and the results were averaged to obtain an estimated CSR_B of 0.155. This value was used as the baseline value of loading for the CSS test program.

For CSS testing, a series of three tests were run on each production sample at three different load cases including:

- Lower Bound – CSRL equal to 50 percent of CSRB (approx. 0.078),
- Baseline – CSRB (approximately 0.155), and
- Upper Bound – CSRU equal to 200 percent of CSRB (approx. 0.31).

The tests were run for a number of cycles estimated based on the earthquake magnitude. For the baseline load case an estimate of 10 stress cycles was utilized. This was also used for the upper bound load case of CSR_U. For the lower bound load case, CSR_B, 5 stress cycles were used.

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The results of the production CSS testing support the hypothesis that vertical strain increases for both increasing CSR and number of stress cycles. The results are summarized below:

Result	Vertical Strain Measured, Percent		
	CSR_L	CSR_B	CSR_U
Maximum	0.07	0.45	1.45
Minimum	0.03	0.12	0.25
Best Fit Curve	0.06	0.29	0.91

The best-fit curve values are based on Figure VII-22a through VII–22c, which is an interpolation of all of the production test results and yields a family of curves for varying CSR values and number of stress cycles.

E. Estimated Dynamic Properties of Imported Backfill

KA performed an evaluation of potential backfill materials, at the request of LANL, with regard to their estimated dynamic properties. A New Mexico Department of Transportation (NMDOT) Type II base course material was evaluated, as well as several other locally available pit-run materials. Although no specific published dynamic properties for these materials were available, an estimate of their linear and nonlinear properties was developed based on published empirical correlations (Menq, 2003 and Darendeli, 2001) related to sample index properties. The average gradational properties for the NMDOT Type II base were utilized along with typical moisture-density relationships from two local aggregate suppliers. The results of the nonlinear shear modulus and material damping relationships compared favorably to other published curves (e.g.: EPRI, Gilroy #2 Site). The preliminary estimates developed from this evaluation were presented in memorandum submitted to DMJM H+N on January 18, 2007 and were subsequently used by ARES in their SSI analysis for the CAT I Facility.

Due to the important relationship between the shear wave velocity (and shear modulus) with confining pressure, additional analyses were performed. The backfill thickness was divided into six, eight-foot-thick sublayers to refine the dynamic properties of the backfill. The results of this evaluation were presented in an supplemental memorandum, dated January 29, 2007. Because both of these analyses were completed on an expedited basis, NQA-1 level calculations and design verifications have not yet been performed. We recommend that for future phases of design, that resonant column / torsional shear testing be performed to confirm

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the actual dynamic properties of the backfill material and reduce the uncertainty of these parameters.

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VIII. GEOMECHANICAL MODEL

The static and dynamic engineering properties, discussed in Sections VI and VII respectively, display similar trends and contrasts between geologic units. As a result, the model of the subsurface based on static properties is very similar to the subsurface model based on dynamic properties, and both are reasonably well predicted by the geologic section. Therefore, a single geomechanical model was developed that depicts spatial distribution of both statically and dynamically equivalent materials as layers. The geomechanical profiles and corresponding static and dynamic properties presented herein may be used by the project's designers / builders for their respective calculations and construction considerations.

A. Development of Geomechanical Model

Geomechanical characterization for each boring location was based on boring log data, static properties, and dynamic engineering properties. For the development of the geomechanical model, layers were identified as those materials in a portion of the subsurface section that exhibit generally similar geologic characteristics, but more importantly exhibit static and dynamic engineering properties that are comparable. Transition layers were identified where changes in properties were gradual. Geomechanical units and transition layers were delineated by changes in engineering properties with respect to elevation.

Field and lab test data in the *gINT* database were plotted on two-dimensional subsurface profiles (See Figures V-6 and V-7, for example). In addition to the geomechanical property trends within each boring, data patterns in the lateral directions were identified by comparison between the two-dimensional profiles.

Boring log data used for geomechanical characterization included degree of welding, hardness, and RQD. For this analysis, units and transition layers were distinguished by logged changes in these geomechanical properties rather than the mineralogical and stratigraphic criteria of Broxton and Reneau (1995). For example, the contact between transition zone $Qbt3_L$-t and $Qbt3_L$ was typically delineated as the point at which RQD measurements were no longer feasible since the stratum became more soil-like. Also, there was often a change in logged hardness from "soft to medium hard" or "medium hard" to "soft". Likewise, there was often a change to "poorly welded" at this contact, although the changes in these latter two properties were sometimes logged at slightly higher elevations. The interpretation of this data set was influenced by the length of the core runs, which was typically 5 feet. Where changes in these

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logged properties were not noted within the five-foot core run, layer boundaries were often interpreted to occur within the core run.

Geomechanical boundaries based on laboratory static properties were evaluated from unconfined compressive strength, triaxial compressive strength and elastic modulus. As discussed in Section VI and illustrated in Figures VI-4, VI-8, and VI-15, there are order-of-magnitude differences of these properties in $Qbt3_L$ relative to Units $Qbt3_U$ and Qbt2. For example, unconfined compressive strength, σ_{ci} for $Qbt3_L$ ranged from about 5 to 40 psi, compared to $Qb3_U$ where the results cluster from about 200 to 1,200 psi, and Qbt2 with results between 1,500 to 3,000 psi. However, intermediate lab test values were measured between these observed data clusters, revealing the existence of transition layers above and below $Qbt3_L$. Consequently, transitional unit $Qbt3_L$-t included σ_{ci} in the range of 40 to 200 psi and Qbt2-t included σ_{ci} in the range of 40 to 1,500 psi.

Dynamic properties utilized in this analysis included field geophysical test results such as discrete shear wave and compression velocity measurements, as well as gamma log measurements and SPT blow counts. As discussed in Section VII-A, there was good correlation between the velocity measurements and the geologic units from Qbt2 through dacite. Thus, these boundaries were used for the geomechanical model below Qbt2.

The transitional zones above and below $Qbt3_L$ were readily observable by the crosshole test results plotted versus elevation (Figure VII-3). The clear, polarized waveforms around this zone suggest direct travel waves as opposed to head waves from refracted boundaries. However, it is possible that the waves could take a slightly bowed travel path from source to receiver through transitionally stiffer material. While such a travel path could potentially create an apparent, but false transition zone, the presence of transitional layers was confirmed by secondary means such as SPT tests in which N values ranged from about 30 to 40 blow per foot within $Qbt3_L$. The more welded tuff layers ($Qbt3_U$ and Qbt2) generally encountered SPT refusal (N = 50 blows/6 inches or less) where the rock mass is not weakened by increased weathering or fracture frequency. As shown in Figure VIII-3, there is a strong correlation in the transition of the SPT results and shear wave velocity from crosshole test.

B. Geomechanical Model and Profiles

Static and dynamic models were compared and found to correlate well based on geological, geophysical, and laboratory data. Therefore, one geomechanical model was developed for the

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CAT 1 Facility area for use in project design and construction. This model is biased by data from measurements of *in-situ* properties, which account best for the rock mass properties of the geomaterials. The CMRR geomechanical model is shown in Figure VIII-1, where it is juxtaposed with the static and dynamic profiles. For ease of use and to avoid confusion with other terms of reference, the geomechanical model is divided into Layers A-M. Layers are identified and differentiated based on geomechanical properties, rather than geologic nomenclature, because the former determines the engineering behavior of the ground. The geomechanical model also includes a designation for recompacted tuff structural fill (Layer A), assumed to be excavated/recompacted $Qbt3_U$ geomaterial. These layers and their corresponding static and dynamic strata are listed below.

Static Profile	Dynamic Profile	Geomechanical Model
		Layer A Structural Fill
Fill	Near Surface	
Qbt4		Layer B
$Qbt3_U$	$Qbt3_U$	Layer C
$Qbt3_L$-t	$Qbt3_L$-t	Layer D
$Qbt3_L$	$Qbt3_L$	Layer E
Qbt2-t	Qbt2-t	Layer F
Qbt2	Qbt2	Layer G
	$Qbt1_V$	Layer H
	$Qbt1_G$	Layer I
	Qct	Layer J
	Qbo	Layer K
	Guaje Pumice Sediments	Layer L
	Dacite	Layer M

Where there were sufficient data to do so, contour plots of the geomechanical layers were generated to characterize spatial variability within the limits of the CAT I Facility. Contour plots of the tops of geomechanical layers were developed from data for each boring (Table VIII-1) using a Delaunay triangulation routine in the computer program, *IGOR Pro* (Wave Metrics, Inc.,

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2004). Two types of contour plots were created including: a) elevation contours that define tops of geomechanical layers (Figures VIII-2 through VIII-6) and b) isopach contours of layer thickness (Figures VIII-7 through VIII-10). The contour intervals were selected for interpolation at one- or two-foot intervals.

The contour plots suggest that within the CAT 1 Facility area and from the El 7300 report datum, there is a general dip of the contact elevation of the shallow units toward the south. The plots also indicate other localized trends (slight indulations or mounds) that are often observed in successive layers. Undulations in pyroclastic flow are common and can be caused by former levees, channels, flow lobes or erosional features after initial tuff deposition (Rowely, et al (1981) and Wilson and Head (1981)).

The isopach maps were reviewed for trends and anomalies. For example, the remaining thickness of Layer C is thinner toward the south, presumably due to historical surface erosion. Layer E apparently increases in thickness from about 42.5 feet in the south to about 50 feet in the northern portion of the MAIN footprint. Conversely, Layer D exhibited a slightly greater thickness in the southern portion of the excavation area (about 10 feet, compared to 6 feet in the north). This suggests that where the transitional Layer D is locally thicker, Layer E may be correspondingly thinner. Although there was a smaller sample population in Layer F, a large variation of transitional thickness was not observed in this lower transition because the estimated thickness varied only from 7 to 7.5 feet. There was insufficient data to continue the contouring below the surface of Layer G.

The subsurface geologic profiles displayed in Section V were modified to include the geomechanical layers (Figures VIII-11 through VIII-16). These 2-D profiles were subsequently used for the analyses discussed in Section IX. Layers C, D, E and F are the layers within the depth of influence of the foundation for MAIN. Layer G is the lowest layer in which geomechanical properties were used to define layer boundaries; deeper layers generally conform to site stratigraphy as shown by the boring logs and the results of *in-situ* geophysical testings.

C. Static Properties for Geomechanical Model

Static geotechnical engineering properties and design parameters for the more soil-like units including Layer A (recompacted tuff structural fill) and Layer E of the geomechanical model are presented in Table VIII-2a; static properties and design parameters for rock-like layers are

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presented in Table VIII-2b. These values are based on the static test results discussed in Section VI. Many of the properties, such as unit weight, moisture-density relationships, specific gravity, void ratio and porosity, are based on the mean average of test results. In the case of the shear strength properties of structural fill (Layer A), that were previously evaluated by best-fit lines through composite test data, the apparent cohesion was neglected because cohesion is not realistic for *in-situ* conditions, particularly under states of low confinement or increased moisture content. Likewise, cohesion was neglected for the residual shear tests along fractures in rock-like tuff. As discussed in Section VI the peak shear strength and modulus values of the intact tuff (excluding Layer E) have been reduced to account for rock mass properties. The rock mass values presented in Tables VIII-2a and VIII-2b were utilized for static deformation, bearing capacity, and slope stability analyses, as discussed in Section IX, and may be used by the CMRR design team.

Some of the design values requested by S & L, including bearing capacity and modulus of subgrade reaction, are dependent on factors such as foundation dimensions, depth of embedment, and the elevation of the foundation with respect to underlying geomechanical layers. These design values will be presented in Section X.

D. Dynamic Properties and Uncertainty for Geomechanical Model

Dynamic properties have been developed for subsequent use in the ground response and SSI analyses from the results of *in-situ* geophysical tests and laboratory dynamic tests of high-quality, intact samples from the CAT1 Facility borings. The resulting linear dynamic properties were assigned to the appropriate layers of the geomechanical model (Table VIII-3). Based on the comparison of *in-situ* geophysical data in the downhole or simulated downhole format, KA calculated standard deviations and corresponding COV values (Tables VII-1a through VII-3b). The concordance of the wave velocities within the SSC/DSC borings are reflected in the relatively low COV values, particularly for Layer E, which was the main focus of the crosshole testing. Although there was some difference between the dynamic versus geomechanical profile, it is relatively small and does not significantly affect the interpreted wave velocities.

Discrete measurements from suspension logging and crosshole tests were also reviewed to characterize the variation of wave velocities within corresponding geomechanical layers. For example, all of the crosshole measurements within each geomechanical layer were statistically compared (Table VII-3a). For Layer E, a COV of 0.07 was calculated for Vs, which provided an alternate confirmation of the tight fit of data within this unit. Conversely, greater variation of

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discrete velocity data (relative to the average simulated downhole data) was encountered. For Layer C, the calculated COV from crosshole tests was 0.21 and the best estimate upper and lower bounds were increased by adjusting the mathematical COV. Also, some of the deeper units exhibited greater discrete wave velocity variation. As shown in Figure VI-1, the discrete suspension logging V_S measurements for Layer H generally varied between about 1550 and 2200 ft/sec. The COV of V_S was accordingly increased to 0.20 for this layer so that the resulting lower and upper bound could envelope this variation (Table VIII-3b). The corresponding COV value for G_{max} is 0.44.

E. Nonlinear Dynamic Properties and Uncertainty for Geomechanical Model

Because dynamic properties of soil and rock are strain-dependent, it is necessary to characterize these properties over a range of shearing strains that could occur during an earthquake. Nonlinear, normalized shear modulus and material damping relationships for subsequent use in ground response / SSI analyses were developed using laboratory RC/TS tests in which shearing strains ranging from less than 0.001 up to 0.3 percent were applied. Adjustment factors were applied to the reference strain to account for differences in lab versus field measurements of shear wave velocity (Tables VII-6 and VII-7). The tables also include benchmark G/G_{max} and D_{min} values (unadjusted and adjusted) at a shearing strain of 0.03 percent. Samples from $Qbt3_L$ (Layer E) exhibited consistent nonlinear properties as indicated by COV values of 0.03 and 0.10, respectively (Figures VII-17a, VII-17b, VII-18a and VI-18b). The COV for G/G_{max} and D_{min} at 0.03 percent shearing strain was conservatively increased to 0.05 and 0.20, respectively, for Layer E; for other layers where only two RC/TS tests were completed, COV values for this benchmark were selected at 0.10 and 0.20 for G/G_{max} and D_{min}.

The adjusted, nonlinear shear modulus and material damping relationships for Layers B through M are presented in tabular form for shearing strains ranging from $1x10^{-5}$ to 0.9 percent (Table VIII-4a). For Layer B, in which no RC/TS tests were performed, the nonlinear relationship for Layer C may be used. For Layers D and F, the Layer E relationship may be used. Finally, for Layer L, the relationship for Layer J is appropriate.

As indicated on the geomechanical profiles, there will be a substantial amount of excavation for building construction followed by placement of the structural fill (presumably recompacted $Qbt3_U$) adjacent to the MAIN. To our knowledge, there are no dynamic nonlinear test data available for recompacted Bandelier Tuff material. In lieu of site-specific data, the EPRI (1993)

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curves for sand in the 20- to 50-foot range may be used for preliminary considerations (Table VIII-4b).

F. Spatial Variability for Geomechanical Model

In addition to the variability of the dynamic properties, it is important to characterize the uncertainty of geomechanical unit thickness (or depth to each layer) across the footprint of PC-3 structure like the MAIN. As illustrated in Figures VIII-2 through VIII-10, the layer contact contours and isopachs vary across the footprint of MAIN. Deep seismic characterization boring DSC-1B, located at the approximate center of the MAIN, was used to develop a composite column for the site. The layer contacts at this location are similar to the average contact elevations of the geomechanical model (Table VIII-1). For borings within the CAT 1 Facility area, the standard deviation for layer contact elevations was used to characterize spatial variability about the DSC-1 column.

As illustrated in Figure VIII-17, the depths to DSC-1B contacts of the shallow units are based on El 7300 ft The standard deviations of the depths to contacts are shown relative to the DSC-1B contacts. The SSI analyst may evaluate the variability of the DSC-1B column by varying the depth to each layer within each respective uncertainty range, as well as varying the dynamic properties within the COV range (Table VIII-3) for sensitivity analyses. For the purpose of maintaining layer thickness within one standard deviation of its average thickness, the depth contacts should be varied no more than ½ of their respective standard deviation. This constraint is illustrated in Figure VIII-17 and summarized in Table VIII-5 where the resulting range of minimum and maximum depths (and layer thickness) are shown.

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IX. GEOTECHNICAL ANALYSIS

Using the geomechanical model, geotechnical analyses were performed to evaluate the geotechnical characteristics of the site and to determine their influence on the design and construction of the proposed CAT I Facility. The geotechnical analyses performed for the CAT I Facility focused on the following foundation design and construction elements:

- Foundation design and performance,
- Lateral earth pressure on below grade walls and tunnels, and
- Excavation and cut slopes.

The analyses and discussions presented in this section are based in part on design drawings provided by Sargent & Lundy (S&L). These include the following:

- General Arrangement drawings GA-1001, -1002, -1003, -1004, -1005, -3001, -3002, and –3003, all last revised March 3, 2006,
- Site and Building Location Plan drawing G-0002, last revised March 3, 2006,
- Construction Access and Limits of Excavation drawing CS-001, last revised March 17, 2006, and
- Soil bearing contours, last revised April 28, 2006 and building loads, last revised May 15, 2006.

Preliminary seismic sensitivity evaluations were also performed during phase one of the G/SIP. These evaluations considered 1) the effects of new ground motion attenuation relationships on seismic hazards, and 2) seismic amplification effects associated with topographic high points such as the TA-55 mesa (Pajarito Mesa). These evaluations were preliminary in nature and additional study may be warranted; however, the follow-up site response analyses for CMRR was deleted from KA's scope of work and is being performed by others. Discussion of the preliminary sensitivity evaluations is provided at the end of this section.

Detailed discussions of these analyses are provided within this section of the report. Calculations performed during these analyses have been submitted separately and are not included in this GER. The analyses discussed in this section are used in the development of geotechnical design recommendations presented in Section X of this report.

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A. Foundation Design and Performance

The CAT I Facility will include two buildings, two pedestrian tunnels, site retaining walls, and several ancillary elements including a receiving / handling bay, utility pads and utility ducts. The buildings, tunnels, and ducts will be supported on mat foundations. Conventional footing foundations will be used to support the remaining structural elements.

1. General Configuration of CAT I Facility Elements

The MAIN will be supported by a heavily reinforced mat foundation as shown on GA-3001, -3002, and –3003. Based on this information, the mat will be founded at a depth of about 60 ft below existing site grade near the central part of the MAIN footprint. The excavation area for the CAT I Facility will cover an area of approximately 150,000 sq ft or about 3.5 acres, while the mat foundation for the MAIN will cover about 95,000 sq ft or about 2.25 acres.

Based on available information, the mat foundation of the MAIN will have finish floor at El 7241. With the nominal mat thickness of 3 ft shown on the referenced drawings, and an assumption of 1 foot of overexcavation to facilitate placement of a lean concrete (mud) mat, the finish grade of the excavation will be at approximately El 7237. Elevator pits will be approximately 5 ft deeper than the nominal depth of the MAIN mat foundation. Using the same assumptions about slab thickness and overexcavation, the finish excavation in these areas will be at El 7232. A liquid waste sump is also planned near the center of the west side of the laboratory portion of the MAIN; this sump will have an invert depth of up to 10 ft below the finish grade of the MAIN mat foundation. Using the same assumptions about mat thickness and overexcavation, the finish excavation of the waste sump will be at El 7227. A summary of these excavation elevations is presented below:

Facility Element	Elevation of Excavation Grade (feet amsl)
Overall MAIN Excavation	7237
Elevator Pits	7232
Liquid Waste Sump	7227

Figure I-3 (based on Drawing CS-001) shows proposed excavation limits and excavation subgrade at El 7238 that are consistent with values given above when the 1-foot overexcavation is considered. This same plan shows construction access at excavation grade around the entire

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perimeter of the MAIN. Along the east, south, and west sides of the MAIN, the construction access varies from 30 to 50 ft wide. In these areas of the excavation, a 70-degree construction slope is shown on Figure I-3. On the north side of the MAIN, the construction access is a maximum of 10 ft wide and becomes as narrow as 5 ft wide. The majority of the north side also uses a similarly inclined construction slope; however, vertical excavation support is shown around the northeast corner. Excavation depths will range from as much as 63 ft in the north to as little as 32 ft in the south along Pajarito Road.

The PUMP will be located to the west of the MAIN and will bear at El 7260 as indicated on the GA drawings referenced. According to the referenced drawings, the foundation for the PUMP will bear on a variable thickness of structural fill overlying competent rock; about 22 ft of fill on the north end versus about 5 feet of fill on the south end.

2. MAIN Mat Foundations

Several sets of analyses were performed for the MAIN mat foundation. The static performance of the MAIN foundation was primarily evaluated using a series of numerical modeling analyses. A conventional settlement calculation based on Westergaard and Boussinesq pressure distributions was also performed by hand to compare to the results of the numerical modeling. Analyses to evaluate the potential for seismically induced compaction (SIC) and settlement due to hydro-collapse were performed using calculations and parameters determined during laboratory testing. Analysis of the PUMP foundation was performed using conventional elastic settlement methods and based on elastic properties of the foundation materials. Details of these analyses are presented in the following sections of this report.

a) Analysis of Static Deformation

Construction of the MAIN will require a deep excavation over a large area followed by construction of a relatively heavy building (MAIN). As a result, the state of stress at foundation grade will change significantly over a short period of time, first dropping to zero as excavation progresses, then increasing to levels near the initial state and higher (for heavily loaded areas) as structural loads of the MAIN are imposed. This analysis was performed to evaluate the influence of the excavation (unloading) step and that of the subsequent structural loading on mat performance.

The MAIN will be supported on a reinforced concrete mat foundation bearing at approximately El 7238 and within Layer C. At this bearing elevation, the required excavation will remove a

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significant quantity of existing material, thus relieving vertical stress. Based on the laboratory-measured unit weight values for the materials being excavated, the loss of overburden pressure at El 7238 will be approximately 3000 to 5500 psf, depending on location within the excavation.

Loss of overburden pressure during excavation will be largely offset by subsequent loads applied by the MAIN, but loads will not be uniformly applied across the mat. This variation of pressure compensation will affect the performance of the mat foundation. The MAIN is expected to be approximately 320 ft in the longest plan dimension; consequently, the zone of influence could extend to a significant depth below foundation grade if the underlying geomaterials were all soil. However, within relatively close proximity of the MAIN foundation mat, Layers C and G are rock materials with moduli approximately two orders of magnitude greater than Layers D, E, and F. As a result of this higher stiffness, the MAIN mat contact pressures (Figure IX-1) will be distributed more widely in Layer C and, therefore, the layers of greatest concern are Layers D, E, and F. Layer G is the stiffest geomaterial at the site; therefore, any increase in vertical stress at the base of Layer G is considered negligible.

1. Numerical Modeling

Numerical modeling was conducted to estimate the response of Layers C through G geomaterials and the mat foundation to peak static loading. This analysis is detailed in Calculation No. 13568-109-XCAL-012, Static Deformation Due to Building Excavation and Building Construction, and is summarized in this section. The numerical model input included stress-strain parameters of the Layers C through G geomaterials, particularly those developed for Layer E by stress-path triaxial testing. The foundation conditions and performance of the MAIN mat slab were evaluated in a sequence of analyses that included the following:

- Preliminary numerical modeling to determine applicable initial geomechanical laboratory testing parameters,
- Evaluation of the predicted versus actual laboratory test response,
- Recalibration of the numerical model to generate final geomechanical laboratory testing parameters, and
- Analyses using the final laboratory test results in the refined numerical model to develop design recommendations for the MAIN mat slab.

Analysis of the static displacements due to the excavation and construction of the MAIN were modeled using the explicit, finite difference modeling program Fast Lagrangian Analysis of

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Continua (FLAC) (Itasca, 2006). Two geomechanical model cross sections were analyzed, E-E' and I-I', as shown in Figures VIII-14 and VIII–16.

An initial series of numerical modeling analyses were performed on the MAIN to evaluate the expected stress distribution in Layers C, E, and G. The excavation and building geometry as well as structural loads used for this modeling were approximate and based on simplified information provided by LANL from the Conceptual Design Report (CDR).

An initial calibration procedure was performed to assist in modeling the properties of key strata, Layers C ($Qbt3_U$) and E ($Qbt3_L$). The calibration procedure consisted of developing the initial material parameters using laboratory test data that included triaxial compression, direct shear and unconfined compression tests. Three constitutive models, Duncan-Chang Hyperbolic (DCH), modified Cam Clay (MCC) and elastic-perfectly plastic Mohr-Coulomb (EPPMC), were used to calibrate the Layer C and E materials. Calibration was performed by matching the materials response in triaxial compression, as obtained from the laboratory testing, to a series of triaxial compression tests modeled in the FLAC software. Layer C was not amenable to calibration by the DCH and MCC models because the strength of the material made an accurate calibration impossible. As a result, the EPPMC model was used for the Layer C material. The Layer E material could be calibrated to all three of the previously referenced models; however, the DCH model provided the most reasonable calibration of the three. Layers A, B, F and G were modeled as linear elastic materials, with properties based on testing as described in Section VI. Transition Layer D was modeled using EPPMC properties also described in Section VI.

The calibration process proceeded and a series of points within the Layer C and Layer E materials were chosen as points to obtain the histories of the major and minor principal stress, y-direction displacement, shear and volumetric strain as the excavation, construction and backfill were modeled. After review and acceptance by the analysis team, the FLAC-generated stress paths were used to guide the stress path triaxial testing as discussed in Section VI-H.

The volumetric and axial strain data from the laboratory stress path triaxial testing were compared with the volumetric and axial strain histories that were obtained in the FLAC model. The results of the initial round of testing indicated that there was reasonable agreement with the volumetric strain output from FLAC for Layer E. However, Layer C materials showed greater volumetric strain in the laboratory testing than in the FLAC results. The model was not

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recalibrated at that time; one laboratory test was not sufficient justification to make changes in the material parameters used in the FLAC model. The first phase of FLAC modeling was then completed using the results of the first phase of TRXSP testing.

A refined model was then prepared for the second phase TRXSP testing and FLAC modeling. The refined model used updated excavation and building geometry based on the March 3, 2006 GA Drawings and more detailed definition of the predicted bearing stresses (May 1 and May 15, 2006 data provided by S&L). Based on available structural loading information provided to KA (Dead Loads plus ½ Live Loads), the contact pressure at the base of the MAIN foundation will range from 2 to 12 ksf (Figure IX-1). In general, the SNM area of the MAIN will impart the highest contact pressures, which will range from 5 to 8 ksf. Perimeter loads in portions of the SNM area are projected to be as high as 8 to 12 ksf. Within the LAB and AUX areas of the MAIN, the contact pressures will be lower and will generally range from 2.0 to 4.0 ksf. For a majority of the MAIN foundation, the expected mat contact pressure will be offset, or compensated for, by vertical stress relief from excavation of material. In these areas of the MAIN, it is expected that some minor amounts of heave followed by minor amounts of settlement will occur during both construction and service life of the building. However, for some areas of the MAIN, particularly the SNM area, the compensating effect of excavation is less than the expected mat foundation contact pressure. In these areas of the MAIN, it is expected that additional settlement will occur.

The finite difference grids for Cross Sections E-E' and I-I' (Figure IX-2 and IX-3) were changed to take into account the changes in the geometry of the excavation and the layout of the facility and the changes in loading. Once changes to the grid were complete, the full model of the excavation, construction and backfill of the planned construction was completed, producing new stress paths. Due to changes in the loading pattern, the areas of highest loading required the use of several analyses to assist in capturing the information needed for the revised stress paths, moving the points at which histories were taken to reflect the changes in loading concentration. The revised stress paths were provided to ATT, and once a second round of testing was complete, data were again compared between the laboratory and FLAC results. The same general agreement was noted in the Layer E material, and the general incompatibility between predicted and measured strain was also noted in the Layer C material. If additional numerical modeling is performed for a later project phase, it may be beneficial to perform isotropic compression tests to better define the bulk modulus of Layer C material. Table IX-1

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presents the parameters for the DCH model that were used to represent the Layer E geomaterial.

The initial FLAC analyses were performed in two-dimensional space using plane stiffness characteristics of the mat slab provided by S&L including slab thickness, modulus of elasticity, and moment of inertia. These initial analyses did not account for overall three-dimensional building system stiffness (heavily reinforced shear and moment resistant perimeter and internal walls and floor slabs) that would have significantly increased the plane stiffness of the mat slab. Recognizing that the three-dimensional building system stiffness was significant, KA was directed to perform a supplemental analysis using a stiffened mat slab. Based on evaluation of the building system, the structural design team and LANL directed KA to reanalyze the model using a plane mat slab stiffness approximately 1000 times that of the original analyses.

Mat slab settlement profiles were again developed from this supplemental analysis and are presented in Figures IX-4 and IX-5 for geomechanical cross sections E-E' and I-I', respectively. Both settlement profiles represent the mat displacement following building and backfill loads; the net displacement estimated by incorporating stress relaxation due to excavation is not included in these profiles. As illustrated in Figures IX-4 and IX-5, there is a reduction of the maximum displacement and differential settlement between the mat slab only (original) and the stiffened mat slab (supplemental) analyses.

2. Conventional Methods

A separate evaluation of settlement of the MAIN was performed (Calculation No. 13568-109-XCAL-017) to provide a check of the settlement estimates developed from the FLAC modeling described in the previous section. The comparison was performed for geomechanical cross sections E-E' and I-I'. The estimate was based on load distributions averaged between those proposed by Boussinesq and Westergaard (Duncan and Buchignani 1976). The subsurface model was simplified by considering Layers D, E, and F as one thick layer with properties of Layer E. Layer C was considered incompressible, as was Layer G. The effect of having a much stiffer Layer C overlying the composite Layer D-E-F was ignored during the estimation of stress distributions. Thus, no inherent "bridging" over the more compressible layer was assumed by this comparative method. The layer thickness values used were also based on the geomechanical model described in Section VIII of this report, except for the combined D-E-F layer.

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As detailed in Calculation No. 13568-109-XCAL-017, average loads applied to the foundation subgrade (Figure IX-6) were selected based on those provided by S&L (Dead Load plus ½ Live Load), which simplified as a uniform mat load of 3.5 ksf for the LAB and 6.5 ksf for the SNM. As shown one Figure IX-1 (Dead Load plus Full Live Load), the actual load contours exceed these averages in select locations such as building perimeter walls. A backfill load of 5.5 ksf was also added to include the effect of backfill within the excavation area. The principle of superposition was used to estimate the average stress distribution along the cross sections.

Stress-strain properties for Layer E were developed based on the final stress path controlled triaxial compression (TRXSP) tests discussed in Section VI of this report. A bi-linear curve for recompression and compression, respectively, was based on six separate TRXSP tests that were plotted to generate the best-fit lines shown on Figure IX-7. This analysis estimates settlement that will occur following the applied building loads and backfill and assumes that all stress relaxation (heave) due to excavation unloading was complete.

As shown in Figures IX-8 and IX-9, slightly larger settlements are estimated using the Boussinesq pressure distribution versus the Westergaard pressure distribution. As expected, relatively small settlement, about 0.4 to 0.6 inches, was estimated in the LAB where the maximum past pressure is not exceeded. Estimates of settlement within the SNM, where a portion of the influence zone is subject to virgin loading, ranges from 0.6 to 1.2 inches using the Westergaard pressure distribution and 0.8 to 2.7 inches by the Boussinesq pressure distribution. Settlement estimates by the average of the two pressure distributions is also illustrated in Figures IX-8 and IX-9.

3. Discussion of Static Deformation Results

Two different methods were used to estimate settlement of the MAIN. Table IX-2 presents a summary of the results from both the FLAC analysis and the conventional approach. The table presents the maximum and minimum values estimated by both approaches along both cross section lines. The table also presents estimates of the maximum angular distortion (δ/l) which is equal to the settlement difference between two points divided by the horizontal distance between these two points) that was estimated from the settlement profiles. The settlement along Cross Sections E-E' and I-I' for the mat slabs only and stiffened mat slab cases estimated using the FLAC analyses, as well as the settlement estimates from the conventional analysis using the averaged pressure distribution (Boussinesq and Westergaard) are compared as shown on Figures IX-10 and IX-11.

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The shapes of the curves are generally similar between the two different methods. For Cross Section E-E' (Figure IX-10), the curves for the stiffened mat slab (FLAC) and conventional case are nearly parallel but offset by approximately 0.5 inches of settlement. The curves for the stiffened mat slab only (FLAC) case is also similar and parallel over the middle 80 percent of the cross section length, but settlement is greater at the ends of the slab with a two-fold increase over the conventional case. This is likely the result of edge effects that are inherently modeled in the FLAC analysis that were not considered in the conventional case.

For Cross-Section I-I' (Figure IX-11), the curves , while generally showing similar trends, show moderate to significant variation between the two FLAC cases and the one conventional analysis. Again, as with Section E-E', the conventional analysis shows less total settlement than that estimated using FLAC. There also does not appear to be any edge effects indicated for I-I' as were shown in E-E', but this may be the result of the generally higher loads along I-I' that may be masking this effect. Significant variation between the three curves is noted in the middle to south end of the cross section. This is likely the result of underestimating the average bearing stress transmitted into the underlying geomaterial in the conventional analysis. Because the stresses used in the FLAC analysis were also actual predicted values, it is concluded that the settlement curves for the FLAC case are more representative.

The FLAC models considered the stress-strain characteristics and thickness of all of the layers based on the geomechanical model properties. The FLAC models also considered all of the various steps associated with excavation, building construction, and the added load of the backfill. However, several of the properties used in the FLAC analysis were based on limited data. One of those properties, the unload-reload curve in the DCH model, was likely represented softer in the model than would have been expected based on correlations with properties such as triaxial compression and SPT N values. The resulting effect is that the FLAC model is likely slightly over-predicting the settlement that might be expected.

The influence of the backfill steps on the overall settlement profile is clearly shown in Figure IX-12, where it can be seen that after building construction but before backfill, the settlement profile along E-E' is a relatively straight line with more settlement toward the west (SNM area) and less toward the east (LAB/AUX). As the backfill is added in steps, the deflection curve takes on a concave-down shape and the total maximum settlement nearly doubles along the sides of the building, whereas in the middle of the building the deflected shape remains relatively constant.

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b) Modulus of Subgrade Reaction

The modulus of vertical subgrade reaction, K_v, is estimated to assist the designer with structural modeling of a beam on elastic foundation. The beam, in this case, the mat foundation, is modeled as a structural element with spring support provided at discrete locations. For the in-situ geomaterials within the zone of influence below the mat, there is no difference between the short-term and long-term K_v.

1. Numerical Modeling

Based on the FLAC-determined deflected shape of the subgrade below the mat, K_v was estimated by dividing the design mat contact pressure by the node-averaged deflected amount. The resulting values for flexible and rigid cases are presented in Figures IX-13 and IX-14 for Sections E-E' and I-I', respectively. As anticipated, the rigid cases exhibit a more uniform K_v across the 2-D section, although the average magnitude is about 20 pounds per cubic inch (pci) for both cases. Additionally, K_v is relatively consistent through both the LAB and SNM areas of MAIN.

2. Conventional Methods

Based on the deflected subgrade in the Westergaard and Boussinesq methods, relative to the average loads, alternately-estimated K^v values were obtained, as shown in Figures IX-15 and IX-16. Using the average between the Westergaard and Boussinesq methods, K_v ranges from about 25 to 70 pci, averaging about 45 pci.

As detailed in Calculation No. 13568-109-XCAL-009, K_v was also estimated by simplified methodology referenced by Terzaghi (1955). In this method, K_v was estimated based on elastic theory using weighted stiffness values of the underlying geomaterials. The size of the influenced area is approximately one half the column spacing or the span between parallel walls as suggested by Terzaghi. Width B is limited to column to column spacing or wall spacing (25 ft maximum). Using this methodology a K_v value of 150 pci was estimated for the LAB, while a K_v of 70 was estimated for the SNM. The actual response of the beam on elastic foundation is influenced by the stiffness of the beam and underlying geomaterials as well as the location relative to the entire foundation.

The following equation was then used to calculate the 1 ft x 1 ft value for comparison to published textbook values only. $Kv_1 = K_v\ (2B/B+1)^2$. Table IX-3 presents the values of K_v and

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K_{v1}, for the various elements of the site based on this simplified method. This table includes K_v and K_{v1}, values for the MAIN as well as for the PUMP and tunnels which are discussed later in section IX.

3. Recommended Subgrade Modulus and Recommendations for Future Work

The subgrade modulus values presented previously are based on analysis of preliminary structural configurations. Based on the analysis results, we recommend that a K_v value of 20 pci be used for preliminary analysis, however, based on the current project data and analyses, we estimate that site- and building-specific K_v may range from 10 pci to 200 pci. For detailed final analysis, we recommend that an iterative solution be employed to refine the modulus values. This iterative approach would consider the final structural configurations and loading and would be performed in a series of cycles until convergence of both the structural and geomaterial deflections and modulus values is achieved. Based on discussions with S&L, we recommend the following steps be performed during final structural design:

1. Using the final GA drawing configurations, structural designer (S&L) should perform an analysis to estimate the subgrade contact pressures and deflections of the mat slab foundation. As appropriate, this analysis should model the sequential steps associated with complete construction of the facility including excavation backfill and expected static service loads. This analysis should be performed using simulated linear elastic springs proportioned based on an initial modulus of subgrade reaction of 20 pci. Output from the analysis should include a plan showing final bearing pressure intensity and final slab deflections over the entire building footprint.

2. Using the final bearing pressure intensity plan, the geotechnical engineer should perform a subsequent analysis that models the characteristics of the subgrade geomaterial to appropriate three-dimensional limits and estimates the slab deflections over the entire building footprint. In this analysis, the stiffness characteristics of the final structural configuration of the building should be considered. This includes increasing the plane stiffness of the mat slab foundation to appropriately account for the three-dimensional building system stiffness that will be realized as the internal elements (shear walls and support floors) are completed. Output from Step 2 analysis will be a final deflected shape of the mat slab foundation as well as revised modulus of subgrade reaction values.

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3. Compare the deflected shapes and modulus values from Steps 1 and 2. If the agreement is good between these steps than the two different models have converged and the analysis is complete. If the agreement is poor, re-perform Step 1 with revised modulus of subgrade reaction values (from Step 2) and generate new output for Step 1 and re-perform Step 2. Re-perform Step 3, evaluate, and proceed as appropriate.

c) Seismically Induced Compaction

At the completion of the phase-one field investigation, seismically induced compaction (SIC) was considered as a possible geologic hazard to the CMRR project after review of data. The phase SBT and phase two field investigations were subsequently planned to permit the collection and laboratory testing of samples from Layer E to evaluate the potential for SIC.

SIC is a process in which low-density, cohesionless materials compact or densify during a seismic event. Materials most susceptible to SIC include loose, unsaturated sands. Because of sample collection difficulties during the phase-one field investigation, concern was raised that Layer E could be equivalent to a loose or meta stable sand-like material.

To address this concern, initial classification and cyclic simple shear (CSS) testing were performed on block samples collected during phase-SBT. After initial testing, the phase-SBT CSS testing protocol was modified, additional experimental CSS tests were performed, and a final laboratory testing procedure was established. Production CSS testing was completed on a total of fourteen samples as discussed in Section IV of this report. The parameters used for the CSS testing and the properties developed are presented in Section VII.

Based on the CSS results, the vertical strain for the baseline CSR value (0.155) ranges from about 0.1 to 0.5 percent with an average of 0.29 percent. Based on results of the geomechanical model analysis, Layer E varies in thickness from 42.5 to 50 ft excluding transition zones. Applying the baseline case average strain over the full height of Layer E results in a vertical settlement of between 1.5 to 1.7 inches for the range of thicknesses of this layer.

A portion of the relatively small vertical strain measured in CSS testing may be attributable to sample disturbance during the physical steps required to extract, transport, prepare, and test these materials. Considering these possible contributions to sample disturbance, a relatively small part of the vertical strains measured by these tests is attributable to intact matrix compressibility. As summarized in Riemer (2006), “the degree of vertical strain, when

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interpreted as volumetric strain, is relatively low compared to similar testing performed in simple shear on clean, moderately dense sands, and compacted fills (Whangs et al, 2000), including the Ottawa sand specimens tested in this study." Considering the relative uniformity of $Qbt3_{L,}$ an earthquake excitation (if sufficient to generate shearing strains in the range of CSS testing) would likely result in general lowering of the entire site, with insignificant differential displacement of the structure.

d) Hydro-Collapse

Analyses were performed to evaluate the magnitude of hydro-collapse that could occur due to wetting of Layer E. Results of laboratory testing, simulating the effect of wetting of Layer E, indicate that vertical strains range from 1.2 to 3.1 percent with an average of 2.2 percent. Although inundation of this layer due to rising groundwater is not likely (present depth to groundwater is greater than 700 ft bgs), there is a potential that a portion of this layer could become saturated due to unmitigated infiltration of surface water or as a result of a ruptured pipe carrying water or some other fluid.

An analysis was performed to estimate the magnitude of total and differential settlement that could occur as a result of wetting of this layer. The analysis assumed that 5 feet of Layer E was subject to a wetting front. By applying the laboratory-measured vertical strains due to saturation of the upper 5 feet of Layer E, the analysis indicates that a range of 0.7 to 1.9 inches of vertical strain could occur with an average of 1.3 inches. While this analysis gives a general magnitude of settlement due to hydro-collapse, the actual amount could be very different depending on actual thickness and aerial extent of the wetted zone.

Slight to moderate collapse (vertical strain) was measured during the inundation step of the continued compression tests. These strains are attributable, at least in part, to sample disturbance during the physical steps required to extract, transport, prepare, and test these materials. Considering the likelihood of some sample disturbance, it is reasonable to assume that only a portion of the measured strain is due to collapse. Judgment should be used in evaluating the potential for collapse and any mitigations employed. Recommendations to reduce or mitigate the potential for inundation and thus hydro-collapse are discussed in Section X.B.7 of this report.

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3. Other Mat Foundations

The referenced drawings provided by S&L indicate that the foundation for the PUMP as well as the pedestrian tunnels and utility duct will bear on a variable thickness of compacted tuff structural fill overlying competent rock. This foundation condition is shown in the various geomechanical model cross sections presented on Figures VIII-12, VIII-13, VIII-14, and VIII–16.

Hand calculations were performed to evaluate the variable foundation conditions that are currently indicated for the PUMP (Figures VIII-13, VIII-14 and VIII-16). The calculations considered the range of thicknesses of structural fill shown and the average compressibility properties based on published data for compacted sand (Winterkorn & Fang, 1975).

Section D-D' shows that about half of the south end of the PUMP foundation bears directly upon a variable thickness of fill, and calculations indicate this end of the PUMP could settle approximately ¼ to ½ inches. For the north end of the PUMP, shown in Figure VIII-16, which bears on approximately 22 ft of structural fill, the calculation indicates that approximately 1 to 1-1/2 inches of settlement could occur; differential settlement of the PUMP could result, as well. Without specific compressibility data of compacted tuff, it is difficult to accurately predict the short and long-term settlement of structural fills constructed with tuffaceous material. If an imported fill materials such as NMDOT Type II base course is used, static properties can be estimated from standard industry references.

The calculations discussed previously evaluated the settlement of the PUMP in its long-axis, north-south direction. A separate calculation was not performed for the short-axis, east-west direction of the PUMP; by inspection, this foundation condition at the south end of the PUMP is also undesirable.

Values of K_v were also developed for the PUMP mat. The values were developed using the method described previously for the MAIN. Based on this analysis, the K_v values range from 60 for a section underlain by 22 ft of fill (Layer A) to 90 pci for sections with 7 ft of fill over Layer C.

Similar foundation conditions are indicated for the two pedestrian tunnels and the utility tunnel that extend from the MAIN. Uniform support of foundations is essential for satisfactory performance. The variable foundation support conditions identified for these elements of the CAT I Facility could lead to differential settlements where tunnels/ducts are rigidly connected to a relatively unyielding structure like the MAIN. These expected differential settlements could

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lead to stress concentrations and cracking. Recommendations to reduce or mitigate the potential for damage to these elements are provided in Section X of this report.

An analysis was performed to estimate the K_v values for the tunnels and duct. Using the conventional method for an assumed 10-ft wide tunnel slab, the K_v value for these elements underlain by Layer A geomaterial is about 40 pci and where bearing on Layer B/C geomaterial, the K_v value is about 240 pci.

4. Conventional Foundations

Various ancillary structures and retaining walls for the CAT I Facility will be supported using conventional foundations. The majority of these structures will bear on structural fill; however, at least one proposed foundation will bear on rock-like geomaterial. The parameters for the compacted structural fill are based on the assumption that the material consists of excavated, disaggregated on-site tuff and tuffaceous soil derived from excavation of the CAT I Facility. This fill material will be predominantly $Qbt3_U$ material but may also include Qbt4 and some previously placed granular fill materials. Geotechnical engineering properties for the compacted structural fill were developed as part of the laboratory testing program, and design properties were presented previously in Section VIII. Because the majority of the material used for the compacted structural fill will be from Layer B and C tuff, the analyses were based on cohesionless soil parameters.

Bearing capacity analyses for spread footing foundations were performed using the general bearing capacity equation with applicable bearing capacity factors. For footings on Layer A geomaterial (compacted tuff structural fill) the equations by Bowles (1988) and Das (1994) were used. Footings bearing on rock-like geomaterials were based on Wyllie (1992). In the analysis for footings on Layer A, the apparent cohesion value has been neglected and the bearing capacity values based solely on the cohesionless properties of the compacted structural fill. For footings on rock-like geomaterials, rock mass properties were used based on Hoek and Brown (1997) and Wyllie and Mph (2004). To account for the foundation geometry, the analyses considered discrete minimum footing widths for strip footings. The analyses were based on unfactored ultimate bearing capacity without consideration of settlement. Factors of safety were then applied to these ultimate bearing capacities to generate maximum allowable bearing pressures.

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Ranges of immediate settlement, resulting from compression of the Layer A geomaterial (structural fill) in response to application of the allowable bearing pressures, were also estimated; long-term settlement is not a major consideration because the bearing geomaterials are relatively coarse-grained and unsaturated. These predicted settlements were evaluated relative to tolerable settlements for the proposed elements of the structure. For this analysis, the assumed tolerable total settlement was set at 1.0 inch. Where the predicted total settlement for a given footing shape exceeded this settlement limit, the bearing capacity was reduced by an amount sufficient to meet this settlement criterion.

B. Lateral Earth Pressures

Lateral earth pressures for retaining walls and below-grade walls have been analyzed based on the referenced drawings provided by S&L. The analyses included evaluation of static loading conditions. Lateral earth pressures for dynamic conditions will be developed as part of the SSI analysis performed by others.

Lateral earth pressures can be estimated using earth pressure coefficients defined by the Rankine State of Stress, which neglects wall friction between the support element and the surrounding soil (Das, 1994). Although this method is conservative, it provides a reasonable upper bound for estimating static lateral earth pressures. The analyses considered 1) active pressures for freestanding retaining walls not restrained against rotation but otherwise statically stable, and 2) at rest pressures for rigid below-grade walls of the MAIN, PUMP, and facility tunnels. The analyses also considered additional lateral stress induced by backfill compaction and applicable surcharge pressures. Recommendations for lateral earth pressures, including backfill-induced pressures, and discussed in Section X.C.6.

C. Excavation and Support

To construct the CAT I Facility, an open excavation about 40 to over 60 ft deep will be made through Layer B and into Layer C to El 7237 ft over most of the CAT I Facility footprint. To limit the amount of excavation and to prevent intrusion of the PIDAS perimeter, cut slopes will be steep. Although these cut slopes will be backfilled after CAT I Facility construction is completed, the stability of these slopes must be maintained for the construction period needed to complete building structural work. This section describes the rock structures that will influence slope stability, the kinematic (Markland) analysis of plane and wedge failure, and global slope stability analyses.

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1. Rock Fractures

Natural rock fractures, or joints, were investigated as described in the GDR and in previous sections of this report. Figure IX-17 displays the results of the test trench mapping and optical televiewer surveying on stereonet plots of the poles of each fracture plane. The test trench data, based on a total of 67 fractures mapped, indicate that most of the fractures are steeply dipping, with angles of greater than 60°. By contrast, the optical televiewer recorded only five fractures out of 18 observed dipping greater than 60°, and most fractures (11 of 18) had moderate (31-60°) dips. However, there is bias inherent in both sampling methods that affect the results. Each test trench, being an essentially horizontal one-dimensional sampling window, will intersect more steeply dipping fractures, especially those steep fractures that strike at large angles (close to perpendicular) to the direction of the trench. Low-angle fractures will be missed in trench mapping unless they are very shallow. The test trenches were arranged at angles of 90° to 120° apart to sample as widely as possible in the x-y plane, but provided limited sampling in the vertical direction. The optical televiewer surveys had the opposite bias, sampling along the vertical line of the borehole, intercepting those fractures that are closer to horizontal and tending to miss fractures that are very steep. Nevertheless, the dominance of steep fractures (67% of fractures dip greater than 70°) indicated by the total of all 85 recorded fractures is consistent with the pattern of cooling fractures common in pyroclastic volcanic rocks and documented elsewhere (Broxton and Reneau, 1995; Gardner, 1998; Woehlitz, 2004).

The horizontal direction, or strike, and dip azimuth of each mapped fracture are illustrated in Figure IX-17 as well as Figure IX-18, which illustrates the number and relative angle of fractures per quadrant. These figures show distinct clustering of very steep (81-90°) fractures dipping toward the northeast (Set A) and some lesser clustering of 71-80° fractures toward the northwest (Set B). Otherwise, fracture directions appear to be randomly distributed.

Figure IX-19 shows the orientations of fractures according to the sampling locations – TP-1/SSC-1, TP-2, TP-3/SSC-3, and TP-4/SSC-4. Each of these locations corresponds to one of the quadrants of the CAT I Facility. The NE steep cluster described above is evident in each data set, indicating that Set A is persistent across the site. Set B is evident in the TP-1/SSC-1 and TP-4/SSC-4 data but not in the other two sets, implying that Set B is limited largely to the south half of the site and is possibly related to greater stress relief and weathering along the south side of the site.

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2. Kinematic Analysis

The rock fractures described above were plotted on stereonets and analyzed for their kinematic potential for failure when exposed in an excavation cut slope. This type of analysis examines the position and orientation of cut slopes and fractures to identify the geometries that would be favorable for failures in the form of sliding of rock along a fracture plane or by toppling of a fracture-bounded rock block. This analysis does not predict failures; it simply identifies conditions needed for failure to occur.

The ROCKPACK III© software created by C.F. Watts and Associates (Watts, 2003) was used to plot the stereonets and perform the Markland kinematic analysis. The Markland analysis tests two conditions:

- The fracture's dip is steeper than the friction angle, and
- The fracture will daylight in the cut slope in the down-dip direction.

Both conditions must be met for the rock to fail by sliding along the fracture plane. For failure to actually occur, additional conditions must be met that are not analyzed by the Markland analysis; i.e., a set of conditions that result in the driving force for rock displacement being greater than the forces resisting movement. The Markland analysis does not take into consideration any cohesion along the fracture or the effects of asperities on sliding resistance; therefore, this form of analysis is inherently conservative.

a) Fractures with Infilling

Many of the fractures mapped in test trenches had infilling of clay, roots, or some combination of clayey soil and roots. No testable sample of fracture infilling was obtained for testing of strength parameters, but tests on remolded samples of Layers B and C indicate an average friction angle of about 32°. This value was used for the Markland analysis of fractures with infilling, as illustrated in Figures IX-20a through IX-20d.

Infilling is most common in fractures that daylight to ground surface or the surficial soil. However, fractures infilled with clay were observed in some rock core to depths below 30 feet. Fracture infilling was apparently present in some other fractures but was washed out during rotary drilling with mud. Because the maximum depth of fracture infilling is not known, this analysis assumes that fracture infilling can exist along all fractures within Layers B and C.

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b) Fractures without Infilling

With depth, the observed fractures contained less infilling. Lost circulation while drilling test borings DSC-1, -1A and –1B indicated that filling may be incomplete or absent along fractures below about 30 feet and that some of these fractures have apertures wide enough to reduce rock contact along the fractures and lower the effective friction that could act along the fracture surface during initial movement. In this case, the friction angle of 45.5° could be unconservative. Nevertheless, the residual friction angle of 45.5° is based on laboratory direct shear tests of Layer C samples and represents the best available estimate of the friction angle of unfilled fractures. This value was used for the Markland analysis of fractures without infilling, as illustrated in Figures IX-21a through IX-21d.

c) Markland Analysis

On Figures IX-20a-20d and IX-21a-21d, which are dip-vector plots of all fractures mapped in test trenches and in boreholes by optical televiewer, the crescent-shaped shaded areas delineate the critical zones in which both Markland criteria for potential movement are satisfied. The triangular shaded areas indicate the zones of potential toppling. Dip vector points within the shaded areas identify fractures that are geometrically capable of movement in response to excavation of the cut slope. For these analyses, four uniformly sloping cuts at 60° (0.57H: 1V), 70° (0.36H:1V), 80° (0.18H:1V), and 90° (vertical) were considered.

The Markland method can also be used for kinematic analysis of potential rock wedges that are defined by two or more fracture sets. Because the spatial distribution of fractures did not reveal distinct sets, a wedge failure analysis was not performed. Additional fracture data may be obtained from mapping of the excavation of Layer B, completed Fall-Winter 2006; these data may support additional plane and wedge kinematic analyses.

1. Southwest wall

The south or southwest wall of the excavation will face in the azimuth direction 37.6°±. Approximately half of all mapped fractures (42 of 85) have dip vectors that could emerge from a vertical southwest wall, and 31 of these dip 70-90°. This means that, if the 85 measured fractures are representative of all fractures, nearly ¾ of all fractures would not daylight from a 70° or flatter cut slope.

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2. Northwest wall

The west or northwest wall will face in the azimuth direction 127.6° ±. The distribution of fractures that could affect the northwest wall is virtually the same as for the southwest wall: approximately half of all mapped fractures (42 of 85) have dip vectors that could emerge from a vertical northwest wall, and 31 of these dip 70-90° and would not daylight from a 70° or flatter cut slope.

3. Northeast wall

The northeast or north wall will face in the azimuth direction 217.6° ±. Approximately half of all mapped fractures (43 of 85) have dip vectors that could emerge from a vertical northeast wall, and approximately 60% of fractures that could affect the northeast wall dip 70-90° and would not daylight from a 70° or flatter cut slope.

4. Southeast wall

The southeast or south wall will face in the azimuth direction 307.6° ±. Approximately half of all mapped fractures (43 of 85) have dip vectors that could emerge from a vertical southeast wall, and approximately 60% of fractures that could affect the northeast wall dip 70-90° and would not daylight from a 70° or flatter cut slope.

5. Summary of Markland Analyses:

- Only a few fractures (7%) are flatter than the friction angle of 32° and could not support movement regardless of cut slope angle or direction,
- About 15% of fractures are inclined between 32° and 60°. Most of these dip northwesterly and would be potentially problematic for the southeast excavation wall,
- The distribution and orientation of fractures steeper than 60° will have similar effects on all four excavation cut slopes, or walls. Differences will develop primarily from individual fractures positions within each slope and from the height of each slope,
- Because of the predominance of fractures steeper than 70°, limiting cut slopes to 70° or less will minimize the number of fractures encountered and the need for ground support. Cut slopes steeper than 70° are expected to encounter more fractures and require ground support, and
- Because of the high percentage of steep fractures, toppling is likely to be a concern for all cut slopes, becoming more acute as the cut slope angle increases.

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The value of the friction angle ϕ may range from the residual value (estimated at 45.5°) to the remolded value, for which 32° was used based on laboratory direct shear test results. The actual ϕ will make a difference in kinematic analysis of fractures that dip at angles close to ϕ, but because the majority of fractures have higher angles, the ϕ value is relatively unimportant in this analysis.

Markland analysis was not performed for rock wedges. Wedges, defined by two or more intersecting fractures have the potential for displacement into the excavation. Because rock fractures appear to be spatially oriented in all directions rather than grouped into well-defined sets, the analysis of fractures described above takes into account the combinations of fractures that could form wedges.

3. Slope Stability Analyses

Slope stability analyses were performed to evaluate the general stability of temporary excavation slopes for two different conditions that include 1) global mass stability and 2) local block stability. The global mass stability analysis assumed homogenous, albeit reduced, rock mass properties for the geomaterials and uniform layer thicknesses based on the geomechanical model for cross-section geometries that were considered to be critical. The local block stability analysis assumed properties for the geomaterials and evaluated stability along a set of assumed fracture surfaces that were developed based on the geologic characterization of discontinuities found at the site. Both types of analyses were performed using the computer program *SLIDE*, version 5.0 developed by Rocscience, Inc. Slope stability calculations (13568-109-XCAL-016) have been submitted separately from this report.

a) Excavation Slope Models

Two separate cross section models were analyzed including 1) with the maximum excavation slope height, and 2) with a slightly lower excavation slope height but with the maximum thickness of Layer B. The first model, for maximum slope height, is based on a cross section near the northern corner of the CAT I Facility excavation. At this location the excavation slope height used was 60 feet, which is approximately the maximum for the entire site. Two separate analyses were performed at this location; a) a 70 degree (up from horizontal) slope inclination, and b) a vertical excavation cut.

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The second model considers a layer geometry having a maximum thickness of Layer B, which is found near the eastern corner of the CAT I Facility. At this location, the excavation slope height is approximately 55 feet high and Layer C is 24 feet thick.

The modified Bishop and adjusted Janbu methods of slope stability were used in the SLIDE program to evaluate the two-dimensional limit equilibrium of the slopes modeled. The computer program performs analyses along a series of either computer-generated or pre-established failure surfaces and provides the critical failure surfaces and their minimum factors of safety. For the circular failure surfaces, the limits of computer-generated failure traces were extended to 200 feet either side of the toe or top of slope, thus providing a broad band of evaluation.

b) Global Mass Stability

The global mass stability of excavation cut slopes was analyzed using a generalized analysis of stability through layers with homogenous material properties. Soil-like layers, such as Layer E, were modeled using Mohr-Coulomb strength parameters as presented in section VIII. For more rock-like layers, such as Layer C, the Hoek-Brown rock-mass properties were used. For this analysis, the failure surfaces were assumed to be circular and passed through various homogeneously modeled layers including the rock-like layers. This failure mode is an idealized condition and serves as only a representation of one potential mode of failure.

The analyses for the global mass stability indicate that the 70-degree slope has a minimum factor of safety against failure of 1.8. For the vertical, externally unsupported slope, the analysis indicates that the minimum factor of safety is 1.0 suggesting that unsupported vertical excavation cut in the homogenously-modeled layers is at or very near incipient failure. This analysis indicates that some form of support should be utilized for vertical excavation cuts.

c) Local Block Stability

The analysis for local block stability assumes fractures that daylight, or are exposed, somewhere along the excavation slope surface. In this scenario, the key assumption is the overall system friction angle of the assumed fractures. The overall system friction angle considers not only the frictional resistance along a fresh planar fracture surface but also considers non-planar asperities that would otherwise increase the overall resistance to sliding. In this analysis, the assumption was made that the overall system friction angle was equal to the fresh planar fracture surface friction angle measured in laboratory testing, thereby not considering the increase in friction due to non-planar asperities.

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The analysis for the two cross section models indicates that the minimum factor of safety against slope instability substantially is less than 1.0. Following this analysis, a check was performed to estimate the maximum slope inclination that could be used to generate a factor of safety of at least 1.0 (incipient failure). The result was a slope inclination of 32 degrees that, as expected, approximates the residual friction angle of Layer B.

This analysis confirms the findings of the kinematic modeling discussed previously in that blocks or wedges formed by intersecting joints could be unstable where cut slope inclinations exceed the friction angle along plane fracture surfaces. However, as also discussed previously in the kinematics section of this report, the predominant discontinuities are relatively high angle.

D. Seismic Sensitivity Evaluations

In accordance with the G/SIP, design ground motions that had been developed during the initial probabilistic seismic hazard study (PSHA) were to be used for CMRR seismic design analyses. The initial plan was to deconvolve the ground surface motion presented in the WCFS (1995) report and generate a basement rock motion at the CMRR site. This basement rock motion would then be utilized in a ground response analysis that considered the site-specific baseline dynamic properties and profiles described in Section VII of this report. As part of this work, the ground response analysis was going to consider advances made in the state of practice on the seismic modeling. These advances include the influence of updated seismic attenuation relationships and the effects of topographic irregularities that exist at the CMRR site.

Two separate sensitivity evaluations were performed at the early stages of the geotechnical investigations and include the following:

- Evaluation of the effects of new ground motion attenuation relationships on seismic hazards in terms of spectral accelerations for the proposed new CMRR facility, and
- Evaluation of topographic effects due to a focusing of energy near ridge crests and the interaction of the primary (incident) wavefield with outgoing scattered surface waves.

The results of these evaluations were completed and reports were prepared under separate cover. These reports are titled:

- Effects of New Attenuation Relationships on Seismic Hazards, Chemistry and Metallurgical Research Replacement Project, Los Alamos National Laboratory, Los

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Alamos, New Mexico, Kleinfelder Project No. 19435, Revision 0, DCN No. 19435.10725.B-ALB05RP001, August 26, 2005, and

- Evaluation of the Potential for Topographic Effects at the Proposed Location of the CMRR Facility (TA-55), Pacific Engineering and Analysis, Rev. 1, DCN No. PE&A002, August 26, 2005.

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X. GEOTECHNICAL DESIGN RECOMMENDATIONS

The geotechnical engineering opinions, conclusions, and design recommendations presented in this report are based on KA's field and office studies, the properties of the geomaterials encountered in the subsurface investigations, the results of the laboratory testing program, the interpretation of the data collected, and KA's understanding of the proposed development of the site. It is the responsibility of the final designer / contractor to determine the suitability of the information contained in this report and, at its discretion, to seek supplemental geotechnical investigation and assistance during design and construction. If the design and construction plans used as the basis for this report either change or are deemed inaccurate, or if the subsurface conditions encountered during construction differ from what was encountered during site characterization, KA should be given the opportunity to evaluate the newer information and to provide alternative recommendations.

The primary geotechnical considerations for design and construction of the proposed CAT I Facility include:

- **Layer E, which has a significant thickness and is relatively close to the mat foundation of the MAIN.** This material exhibits geomechanical properties similar to those of a medium dense to dense sand, has a high porosity, and exhibits relatively low strength and stiffness compared to the other more rock-like layers at the site such as Layer B, C, and G. Analyses indicates that this layer is slightly compressible under high bearing stresses from the MAIN foundation and that this layer could soften and compress slightly if it becomes wet. This material also has a relatively low seismic shear wave velocity (lowest of the underlying site geomaterials) and thus possesses shear modulus values that are significantly lower than those of the more rock-like layers both above and below it.

- **The proposed construction of facility elements over dissimilar and variably stiff foundation bearing materials.** Review of the current construction plan indicates that the PUMP building will bear on an uneven thickness of structural fill (Layer A); about 5 ft at its south end and about 22 feet at its north end. The structural fill will tend to densify and strain vertically over time as a result of its own weight and the imposed structural loads. This proposed foundation condition could result in uneven elastic compression of underlying structural fill, which could lead to slight structural distortion of the PUMP building. Non-uniform bearing conditions also exist for the proposed pedestrian tunnels and utility ducts. These facility elements exit the MAIN and extend to either the RLUOB

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or PF-41. In the area where these elements connect to the MAIN, they bear upon structural fill. As these linear elements extend beyond the excavation limits of the MAIN, they cross over into Layer B and C, which consist of relatively stiff rock-like geomaterials. While these tunnel and duct elements are essentially zero net-load elements, the thick section of structural fill supporting them will tend to densify and strain vertically over time as a result of its own weight. This vertical strain will impose stress concentrations at both the structural connection with the MAIN and near the bearing transition from fill to rock-like geomaterials.

- **The variably fractured nature of Layer B and Layer C as it relates to excavation slopes during construction.** These geomaterials exhibit natural fractures that are predominantly high angle. Where excavation cut slopes expose these fractures, potential exists for block and wedge failures. Excavation slopes cut at inclinations of 70 degrees or shallower will serve to reduce, but not eliminate, the potential for rock block failure. The final designer/contractor should be prepared to flatten slopes or selectively scale to remove potentially unstable rock blocks or to install ground support.

Detailed discussion of these primary considerations as well as geotechnical engineering recommendations for design and construction of the CAT I Facility are presented in the following sections of this report.

We recommend that on-site geotechnical observation and materials testing services be employed during site earthwork, excavations, and foundation construction. This will allow LANL the opportunity to compare actual conditions with those encountered in the site investigation and, if necessary, to make design changes if warranted by the exposed subsurface conditions. We also recommend geotechnical review of the final foundation and earthwork plans and specifications. It has been our experience that this review provides an opportunity to detect misinterpretation or misunderstanding of these recommendations prior to the start of construction.

A. Geomechanical Design Parameters

Geotechnical engineering and design parameters have been requested by S&L for use in the structural design of the CAT 1 facility. A summary of recommended design values for general use is presented in Table X-1. Additional static engineering properties were previously summarized in Tables VIII-2a and VIII-2b. Some of the design parameters, including bearing

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capacity and modulus of subgrade reaction, are dependent on factors such as foundation dimensions, depth of embedment, and the elevation of the foundation with respect to underlying geomechanical layers. Therefore, the tabulated values are limited to the conditions stated in the table notes. Detailed recommendations, are presented in the following subsections.

B. Earthwork and Excavations

1. Site Clearing

The CAT I Facility site should be cleared of all obstructions, including but not limited to base course and deleterious fill; concrete parking bumpers; light standards and any other existing structures and their associated foundations; designated chain-linked fencing and foundation posts; buried utility lines; trees, shrubs, and associated root systems; designated equipment; and any other below-grade obstacles encountered during this operation. Base course removed as part of site clearing may be used as fill provided it meets the size requirements for fill material.

2. Excavations and Cut Slopes

After site clearing is complete, excavations for various below-grade structures can be performed. All excavations should be performed in accordance with the requirements of the Occupational Safety and Health Administration (OSHA).

Construction of the CAT I Facility will require excavation of Layers B and C to approximately elevation 7235 to 7238 ft Both layers may be excavated by mechanical means, although more resistant zones within these layers may be encountered which can be excavated using specialized equipment such as a hoe-ram or ripper. Although the contractor may choose to rip Layer B or C geomaterial to expedite the work, most excavation in Layer B or C should not require ripping. Both layers should be excavated by mechanical methods that will limit overbreak and disturbance of the ground beyond excavation neat lines. Provided that the unexcavated ground beyond the neat line is not disturbed, the response of both layers to the excavation will be dominated by natural fractures (joints).

Because of the expected predominance of high-angle fractures, the cut slopes of the excavation walls should be not steeper than 70 degrees (2.75V:1H) to reduce the number of fractures that are daylighted by the cut slope. Although flatter cut slopes will reduce the number of fractures intercepted by excavation, the potential exists for sliding or toppling of fracture-bounded rock

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blocks from any slope. Fractures with large apertures or clay infilling may be especially susceptible to initiation of rock movement.

Although the global stability of rock mass was estimated to have a Factor of Safety (FS) of at least 1.3, during excavation/construction localized rock block failures are possible for slope angles of 45 degrees or steeper; for rock blocks where clay or weathered infilling of fractures exists, block failures are possible for slope angles of 32 degrees or steeper. Due to the limited depth of trench logs, there are insufficient data to determine where such adverse fractures may daylight at the site.

The contractor is responsible for achieving stable cut slopes. During excavation the cut slopes should be observed by a qualified engineering geologist, who should map all fractures as they are exposed and advise the contractor about potential instability related to sliding or toppling of rock blocks in the cut slope. Fracture-bounded rock blocks that are evaluated as potentially unstable may require corrective action from the contractor to provide protection of personnel, equipment during construction, and the work in progress. Such measures may include scaling to remove loose rock and/or providing ground support (e.g.; rock dowels, steel straps or channels, wire mesh, shotcrete) to stabilize rock. Design of ground support and final excavation slopes is the responsibility of the contractor.

3. Subgrade Preparation

Subgrade preparation for Layer B and C rock-like geomaterials should be performed in a manner that limits disturbance. To limit disturbance of these geomaterials during excavation and construction, KA recommends that the last one to two feet of excavation be performed using an excavator equipped with a smooth-bladed bucket. Subgrade for the MAIN mat should be overexcavated approximately 6 to 12 inches to eliminate high spots. Once the subgrade is properly excavated and cleared of loose material, a lean fill concrete should be placed as a working mat. The lean fill concrete should have a minimum 28-day compressive strength of 1000 psi but should be designed to achieve 50 percent of its 28-day strength within 24 hours of placement to facilitate continued construction operations. The lean fill concrete working mat will protect the integrity of the rock-like subgrade materials during foundation mat construction.

Within the pit areas of the MAIN, final subgrade excavation should be performed using lightweight equipment and hand labor to minimize disturbance of Layers D and E where encountered. Subgrade geomaterials that are disturbed during excavation should be entirely removed and those areas also filled with lean fill concrete.

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The bearing surfaces for spread and strip footings supporting ancillary structures and retaining walls should be neatly excavated. Where rock-like geomaterials provide foundation support, the excavation methods should limit disturbing and fracturing the bearing geomaterial. A smooth-toothed bucket or blade should be used for the final one to two feet of excavation to minimize surface disturbance of the geomaterial. Where disturbance/fracturing of footing subgrade is unavoidable, the disturbed geomaterial should be over-excavated and replaced with a lean fill concrete as discussed previously.

All exposed subgrade should be protected during construction to prevent disturbance. This includes minimizing saturation of the subgrade during rain or snowfall events and redirecting surface water away from open excavations and towards appropriate discharge or collection systems. As necessary, the contractor should provide the means and methods to collect and discharge surface water runoff that accumulates in the bottom of the excavations. The contractor is responsible for the means and methods to control surface water.

4. Material For Fill

In general, the existing site geomaterials with an organic content of less than three percent and free of any deleterious materials may be used as structural fill provided they are disaggregated sufficiently to meet the structural fill gradation criteria. Based on the site investigation data and discussions with LANL, KA understands that the majority of excavated material to be used as structural fill will consist of Layer C geomaterial. This geomaterial is expected to disaggregate by the process of excavation, handling, and track rolling; however, more vigorous disaggregation methods may be necessary for more resistant zones. The Layer C geomaterials generally disaggregate to a silty sand-sized material. All structural fill should meet the gradation criteria presented in Table X-2.

Granular base material used as a leveling course beneath mats and slabs should consist of durable aggregate meeting the gradation criteria given in Table X-3. This material generally meets the specification for Type II base course detailed in Section 304 of the New Mexico Department of Transportations Specifications for Highway and Bridge Construction. The notable exception is that the granular base should have a plasticity index of no greater than 3. The granular base should be compacted in accordance with the requirements for structural fill.

Material used for the permeable drainage layer behind retaining walls should consist of durable aggregate meeting the gradation criteria given in Table X-4. This material corresponds to the

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specification for open graded base course (OGBC) detailed in Section 304 of the New Mexico Department of Transportations Specifications for Highway and Bridge Construction.

5. Fill Placement and Compaction

Fill material consisting of Layer A (compacted tuff structural fill) geomaterial meeting the gradation requirements presented previously in Table X-2 should be used for the following applications:

- Support of structures not bearing on Layer B or C geomaterial,
- Backfill against below-grade walls of the MAIN, PUMP, tunnels, and ducts,
- All other locations where fill is required within the entire excavation for the CAT I Facility, and
- Backfill behind site retaining walls.

Layer A structural fill should be spread in layers not greater than 8 inches in uncompacted thickness, moisture-conditioned as necessary, and compacted. Layer A fill should be moisture conditioned to within -4 percent and +2 percent of optimum and compacted to not less than 95 percent of the maximum dry density as determined by ASTM Test Method D1557, Modified Proctor. At a minimum, a new moisture-density relationship test (Modified Proctor) should be performed on materials that have a gradation difference (change in the percent passing by weight) of more than ±5 percent of the specified percent passing any of the sieve sizes indicated. Section 02310, Grading, Excavating, and Trenching, of the LANL Master Specifications should be used as a guideline specification for fill placement and compaction. Where differences exist between the recommendations contained herein and in Section 02310, the more stringent requirement applies. Common fill material and reworked native soils used outside of areas indicated above should be compacted to at least 90 percent of maximum dry density.

Anticipated moisture-density values at 95% Modified Proctor density are presented in Table VI-6 for Layer B and C (Unit Qbt4 and $Qbt3_U$) geomaterial. However, the contractor should perform additional sampling and testing to determine moisture-density relationships and total unit weights of excavated material selected for use as Layer A geomaterial.

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All Layer A geomaterial that has been previously placed should be proof-rolled to evaluate for post-filling disturbance. Proof-rolling should be performed using a heavy wheeled vehicle having a weight of not less than 20 tons. Where disturbance of Layer A is identified, the disturbed material should be entirely excavated down to a level, undisturbed surface and the excavation filled with structural fill placed and compacted in accordance with recommendations provided in this Section. During fill placement and compaction, moisture conditioning of the exposed material and the excavated, recompacted material may be required and may include drying where subgrade has become saturated.

6. Trench Excavations and Backfill

KA expects that excavation for utility trenches in Layer A can be made with either a backhoe or trencher. Trenches in Layer B or C geomaterial can be made with either an excavator or trencher equipped with teeth designed to cut soft rock. All excavation should be performed in accordance with either OSHA or LANL requirements, whichever are more stringent. Excavations should be located so that no structures are located above a plane projected 45 degrees upward from any point in an excavation, regardless whether it is shored or unshored.

For underground utilities to be placed below-grade, installation should be performed according to LANL Engineering Standard Drawing No. ST-G30GEN-4, Rev. 2, Single Trench Detail. As stated in Section G10, Item 3.0 of the LANL Engineering Standards Manual, the sand bedding for underground piping should be placed in layers not to exceed 6 in. and compacted to at least 85% of the maximum dry density (ASTM D 1557), except under roadways or structures where a minimum density of 95% is required. Trench backfill to be placed above the bedding zone should meet the requirements of Section X.B.4 of this report.

Backfill for trenches or other excavations within pavement areas should be compacted by mechanical means in thin lifts not exceeding 5 inches loose thickness to not less than 95 percent relative compaction based on ASTM D1557, Modified Proctor. Jetting and flooding of backfill should not be permitted.

7. Long-Term Surface Drainage

For satisfactory long-term performance of the CAT I Facility, the site geomaterials should be protected against long-term water infiltration and/or saturation. Positive drainage away from structures and below-grade walls should be provided and may include hardscaping (concrete or asphalt paving) to prevent infiltration of surface water into the site geomaterials. During

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construction, the floor of the excavation should be shaped to provide drainage of water to a collection point, and ground surface adjacent to the excavation should be sloped away from the excavation. In addition, a secondary subsurface barrier and collection system could be used. This secondary system could include a barrier made of either a compacted clay or a synthetic liner. The barrier should slope away from the building at a minimum 2% grade, toward an appropriate drain outlet.

Special care should be taken in the design and construction of sub-floor water and sewer lines to reduce the possibility of leaks. Hydraulic infiltration into soils due to leaks could cause settlement of the structures. Landscaped areas should not be allowed adjacent to structures. Hardscaped surfaces should be graded to drain surface water away from the structures and off-site. Water should not be allowed to pond adjacent to structures or pavement areas.

8. Weather Limitations

Structural fill should not be placed when the atmospheric temperature is below 35 degrees Fahrenheit. When the temperature falls below 35 degrees, areas of completed work should be protected against detrimental effects of ground freezing, and any areas affected by freezing should be reconditioned in conformance with the above requirements. Subgrade that is allowed to freeze should be excavated and replaced with properly compacted structural fill. Any soils disturbed due to wetting, drying, excavation, or other causes should also be reconditioned before placing additional fill or constructing foundations or other structural elements. Reconditioning should include scarification, moisture conditioning, and recompaction.

9. Construction Observation

Continuous on-site observation and testing should be provided during over-excavation, subgrade preparation, and placement of structural fill to determine compliance with the specifications. Following excavation to the desired elevation, bearing surfaces should be inspected by a geotechnical engineer to confirm that they are suitable for placement of structural concrete or backfill. Compliance testing should be performed according to the following minimum intervals:

- One moisture-density relationship test (Proctor per ASTM D1557) for every 2000 cubic yards of each type of material, as determined by sieve analysis and plasticity index, to be used as structural fill,

- One field density test per 2000 sq ft per lift of structural fill placed,

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- One field density test at the location of each foundation structural element where structural fill is required, at least one per 1000 square feet per lift immediately prior to concrete placement or placement of reinforcing steel, and

- One field density test per 5000 square ft of paving on both finished subgrade and base course (if included in the pavement structural section).

C. Foundation Recommendations

Structures for the CAT I Facility will utilize both mat foundations and conventional spread footing foundations for support of structural loads. As presently planned, the MAIN, PUMP, pedestrian tunnels, and utility ducts will be supported on mat slabs. For the MAIN, the mat slab will be supported by Layer C geomaterials, and the PUMP will be supported by Layer A geomaterials. The pedestrian tunnels and utility ducts will have segments supported on both Layer A and B geomaterials. Shallow spread and strip footing foundations will be used to support site retaining walls and various structural elements supported at or near finish grade such as awnings and truck bay structures. The following sections of the report provide recommendations for design and construction of foundations.

1. MAIN Foundation

Analysis has been performed that indicates that the mat foundation will provide satisfactory support to the proposed MAIN. However, the planned elevator pits and liquid waste sump could nearly or completely penetrate through Layer C.

For design of the mat foundation for the MAIN and SNM, KA recommends using an initial modulus of vertical subgrade reaction (K_v) value of 20 pci, as presented on Figure X-1. As discussed in Section IX.A.2.b.3, an iterative approach is recommended for a later phase of design to converge the mat slab deflected shape and subgrade modulus values. For design of elevator and waste sump pits, KA recommends using the same modulus value as for the mat slab foundation assuming these two elements will be structurally connected and will deflect as a monolithic element.

The mat slab contact pressures expected by S&L were used in both the FLAC and conventional analyses discussed previously in Section IX of this report. Specifically, as indicated on Figure IX-1, the expected contact pressures ranged from less than 2.4 ksf to as much as 12.0 ksf. These expected contact pressures are magnitudes lower than the ultimate or factored bearing capacity of the foundation geomaterials based on theoretical limit equilibrium analysis.

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However, with foundation systems such as these, the controlling criteria are the tolerable settlements. The analyses indicate that slight to moderate total settlements could occur as a result of the expected contact pressures; it is understood by Kleinfelder that this magnitude of total settlement is within tolerable limits for the proposed facility. As such, the expected contact pressures are suitable for the foundation geomaterials and the proposed facility. As necessary, the mat slab contact pressure could be increased to a level of 20,000 ksf or locally higher; however, if the overall contact pressure is increased, it follows that additional mat slab total deflections will occur above and beyond those discussed earlier in Section IX. Where an increase in mat slab total deflection is intolerable, it will be necessary for the structural design team to consider alternative measures including stiffer mat slab foundation system, redistribution of structural inputs to the mat slab foundation, or some form of ground improvement to reduce total settlements to tolerable amounts. In any case, if contact pressures higher than those used in the analysis presented in Section IX are expected, additional similar analysis (FLAC) should be performed to determine the impact to the overall mat slab total deflection profile. The expected contact pressures used in the referenced analysis are for dead plus realistic live loads and may be increased by ⅓, without further analysis, to account for transient loads such as wind or seismic.

Data from the mat slab total deflection curves presented in Section IX was used to develop angular distortion values. Angular distortion (δ/L) is the measure of relative differential settlement (δ) between two known points along the total deflection curve, divided by the horizontal distance (L) between the two known points. Table IX-2 presents the range of δ/L values measured from the mat slab deflection curves for both the mat slab only and stiffened mat slab FLAC analysis cases. As indicated in Table IX-2, the δ/L values range from 0.08 to 0.25 percent. The structural designer should evaluate the δ/L values presented in Table IX-2 and determine if these values are tolerable for the building structural system and materials, and for the functional and operational parameters required for the facility. If these δ/L values are excessive, it will be necessary for the structural design team to consider alternative measures (similar to those previously recommended) to reduce these expected δ/L values to within tolerable ranges.

The vertical modulus of subgrade reaction values for the MAIN are based on rock mass properties of the supporting geomaterials and the geometry of the loaded area. These values assume that the supporting geomaterials are relatively undisturbed. Subgrade for the MAIN

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mat foundation should be prepared in accordance with the recommendations provided previously in this Section.

2. PUMP Foundation

The mat slab foundation planned for the PUMP will also be heavily reinforced and 3 ft thick. Contact bearing pressures for the PUMP are not known at this time but are assumed to range from 1 to 3 ksf on the mat bearing at El 7260. At this elevation, the south end of the PUMP will partially bear on a variable thickness of Layer A structural fill material. At its north end, the PUMP will be underlain by about 22 feet of structural fill (Layer A). Based on standard geotechnical engineering practice, this variably-supported foundation is undesirable. However, an evaluation was performed to estimate the potential settlement that could occur under this condition. The evaluation indicates that about ½ to 1 ½ inches of total and differential settlement could occur between the north and south ends of the PUMP, which could result in some minor distortion of this structure. Although our preliminary evaluation estimates indicate that the differential settlement will not be significant, the structural designer should determine if these movements are within the performance tolerances for the structure.

Several alternatives exist for providing satisfactory foundation support for the PUMP. These include 1) adjusting the planned excavation geometry such that the PUMP bears on uniformly thick foundation materials, 2) excavating all or a portion of the temporary construction access ramp and replacement with structural fill, or 3) supporting the entire PUMP mat on deep foundations. In consideration of cost and performance, KA recommends either alternative 1) or 2). Recommendations for the PUMP mat foundation are based on the current grading plans as discussed in the preceding paragraph.

KA developed recommended vertical modulus of subgrade reaction values K_v for mat foundation design, which are presented on Figure X-1. As with the MAIN foundation, two different values are presented based on position within the mat foundation. Allowable bearing pressures for the PUMP mat should not exceed 5,500 psf. Foundations should be placed on properly prepared Layer A geomaterial. This allowable contact pressure is for dead plus realistic live load and may be increased by 1/3 to account for transient loads such as wind or seismic. The PUMP foundation should be designed to accommodate a maximum total settlement of 1½ inches and a maximum angular distortion (δ/L) of 0.001 or 0.10 percent. These total settlement and angular distortion values are based on the contact assumed; if contact pressures are higher than assumed herein, these settlement and angular distortion values may require modification.

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Foundation geomaterials for the PUMP should consist of compacted structural fill, and preparation of the foundation subgrade should be performed in accordance with earthwork recommendations provided earlier in this report. The PUMP mat foundation can be constructed on the properly compacted and prepared subgrade. At the discretion of the structural engineer, a 6- to 12-inch thick leveling course of crushed aggregate could be utilized beneath the mat foundation to provide a working mat and to reduce disturbance to the prepared subgrade.

3. Tunnels and Utility Ducts

As discussed in section IX, the tunnels/duct will span over dissimilar subgrade support conditions. To reduce the potential for deleterious cracking, KA recommends that the tunnels/duct structures be designed to accommodate differential movement within the approximate zone indicated on Figure X-2. To accommodate this type of differential movement, the structural designer may need to consider utilizing a series of short tunnel segments capable of tolerating the total settlements indicated.

The structural fill supporting the tunnels/duct mats will tend to densify and strain vertically over time as a result of its own weight, hydraulic densification due to surface water infiltration, and construction irregularities. These settlement estimates indicate that at its thickest, the structural fill could settle a total of approximately 1 to 2 inches. These settlement amounts should be taken as the magnitude of vertical displacement that will occur at the tunnels/duct connections with the MAIN. To minimize the deleterious effects of this expected settlement, KA recommends that the joint between the MAIN and the tunnels be a non-rigid, structurally separate connection capable of accommodating this estimated settlement.

Connections between adjacent tunnel segments should utilize a flexible waterstop to prevent surface water from leaking into these buried structures. A similar waterstop should be utilized at connections with the MAIN, RLUOB, and PF-41. The waterstops should be appropriately sized and designed by the project structural engineer to accommodate the estimated settlement.

Mat foundations for the tunnels and duct should be designed using vertical modulus of subgrade reaction K_v values listed in Table IX-3. The mat foundations should be designed to accommodate the structural weight of the elements as well as any surcharge loads exerted on the top of these structures including Layer A backfill, hardscape pavement structure, vehicles,

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and loads from stored or stockpiled materials. A summary of the design loads for the tunnel and duct structures is presented on Figure X-3.

4. Shallow Continuous Footing Foundations

Conventional shallow footing foundations are appropriate for site retaining walls and various structural elements supported at or near finish site grade. The recommended bearing pressures and anticipated settlements provided herein are based on a footing length to width ratio of 50, a minimum footing width of 5.0 feet, and a minimum footing depth below finish grade of 36 inches. Alternative allowable bearing capacities can be provided if footing geometries different than those assumed are utilized.

Continuous footings bearing entirely on Layer A may be used to support retaining walls and perimeter wall loads and can be designed with an allowable bearing capacity of 5,500 psf. Continuous footings bearing entirely on Layer C may be used to support retaining walls and perimeter wall loads and can be designed with an allowable bearing capacity of 14,500 psf. KA recommends that all footings bear a minimum of 36 inches below lowest adjacent finish grade and have a minimum width of 36 inches. As specified in Section III 1.7.3 of the structural section of the LANL engineering manual, the minimum embedment depth is 36 inches (unless the foundation bears directly on tuff) to account for frost heave.

The recommended bearing values have been selected to include a safety factor of 3 with regard to bearing capacity failure. The bearing pressures recommended herein apply to full-dead plus realistic live loads and may be increased by one-third for total loads, including wind and seismic forces. Foundations sized in accordance with these values should experience total and differential settlement of less than ¾ and ½ inch, respectively, after construction, provided that the site and footing subgrade are prepared in accordance with the recommendations contained in this report. Settlement response of the foundation system will be influenced more by the quality of construction than by geomaterial-structure interaction.

The allowable design bearing pressures presented in this section are based on the footings bearing on properly prepared subgrade. Recommendations for subgrade preparation are provided previously within this Section of the report.

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5. Slabs On Grade

Slabs-on-grade can be supported directly on neatly excavated, undisturbed, and properly prepared geomaterial. However, if desired as a leveling course, a 6- to 12-inch thick layer of granular base can be placed beneath the floor slab. Concrete slabs-on-grade loaded in excess of 500 psf should be considered as heavily loaded slabs. Heavily loaded slabs bearing directly upon properly prepared subgrade (at least 3 feet of structural fill) should be designed using a modulus of vertical subgrade reaction for a 1 ft x 1 ft loaded area (Kv_1) of 200 pci. The Kv_1 value can be increased to 300 pci if a 6-inch-thick layer of granular base is placed beneath the heavily loaded slabs.

A properly graded granular base will tend to act as a capillary barrier to the upward rise of moisture to slabs-on-grade. However, for floors with moisture-sensitive coverings such as epoxy, we recommend a suitable moisture barrier be provided beneath mats to prevent capillary rise and damp floors. An effective moisture barrier may consist of a visqueen membrane covered with a cushion of sand at least 2 inches thick. The sand cushion should be lightly moistened, but not saturated, immediately before placing concrete to resist shifting of the sand and to minimize differential curing of the slab. The final designer/builder should determine the need for a moisture barrier as part of the design process.

Concrete slabs should be designed and reinforced to minimize cracking as a result of shrinkage. Nominal reinforcement should be installed at mid-height in the slab or as required by the structural design.

Special precautions must be taken during the placement and curing of concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during hot, cold, or excessively windy weather conditions could lead to excessive shrinkage, cracking, and/or curling in the slabs. We recommend that concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) Manual.

6. Below-Grade Walls

Below-grade walls for the MAIN, PUMP, and tunnels/duct elements of the project should be designed to resist lateral pressures exerted by geomaterial backfill and adjacent surcharge loads. Below-grade walls for these structures will be heavily reinforced and supported against lateral movement by interior horizontal elements (mat, supported floors, and roof) as well as

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shear walls (MAIN and PUMP). Walls of this type are relatively rigid and considered restrained against lateral deflection and thus will be subjected to “at-rest” lateral earth pressures acting over the full height of the wall.

KA recommends that rigid below grade walls be designed to support static lateral earth pressures equivalent to those exerted by a fluid with a unit weight determined by multiplying the at-rest earth pressure coefficient (k_o) by the unit weight of the Layer A geomaterial. Additional lateral earth pressures may be exerted on below-grade walls as a result of nearby surcharge loads, backfill that slopes up away from walls, and induced stress from compaction of wall backfill.

Additional pressure due to surcharge loads should be added as a uniform lateral pressure over the full height of the wall and determined by multiplying the average surcharge pressure at its depth of contact along the wall by $k_{o.}$ Where finished grade behind the walls slopes upward, KA recommends that the equivalent fluid pressures be increased by one pound per cubic foot for every two degrees of slope inclination. Excess lateral earth pressures resulting from induced stresses during backfill compaction should be determined as detailed for tunnels and presented on Figure X-3.

The lateral earth pressures assume that there is no buildup of hydrostatic pressure due to groundwater or infiltration of surface water. Although it appears unlikely that groundwater will affect the project, special care should be given to prevent surface water infiltration into and behind the below grade walls both during construction and facility operations.

Where migration of moisture would be detrimental or undesirable, KA recommends that below-grade walls be waterproofed. The waterproofing should consist of an impermeable material, such as a bituminous compound, that can be uniformly applied over the full height of the wall.

Backfill behind below-grade walls should consist of Layer A geomaterial and should be placed and compacted in accordance with the recommendations given for structural fill presented earlier in this report.

7. Site Retaining Walls

Freestanding site retaining walls are anticipated for the CAT I Facility. It is our understanding that several retaining walls will be utilized to accommodate grade differential around the MAIN

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and between the MAIN and the PUMP buildings. Other retaining walls are anticipated to support grade differential between the CAT I Facility and Pajarito Road.

It is anticipated that site retaining walls will be freestanding and will not be structurally integral with elements of the CAT I Facility. For this condition, these retaining walls should be designed for yielding conditions, which assumes they are free to rotate at least 0.1 percent of the wall height at the top of the backfill. For this condition, retaining walls will be subjected to active lateral earth pressures equivalent to those exerted by a fluid with a unit weight determined by multiplying the active earth pressure coefficient (k_a) by the unit weight of the Layer A geomaterial. Additional lateral earth pressures may be exerted on freestanding retaining walls as a result of nearby surcharge loads, backfill that slopes up away from walls, and induced stresses from compaction of wall backfill.

Additional lateral earth pressures due to dynamic earthquake loading are not provided in this report. Design recommendations for earth pressures due to dynamic earthquake loading should be performed using the peak ground surface acceleration (PGA) applied using an accepted design method such as presented by Mononobe Okabe.

Freestanding retaining walls should be supported on conventional spread or continuous footings bearing on undisturbed Layer A or C geomaterials. Footings for these walls should be designed in accordance with the recommendations given previously for shallow spread footing foundations. Footing subgrade should be properly prepared as discussed previously in this Section of the report.

Freestanding site retaining walls should be fully backdrained to prevent buildup of hydrostatic pressures due to infiltration of surface water. Drainage may be provided by a 12-inch wide zone of permeable material consisting of NMDOT (Section 3 and 4, OGBC material), or by an open-graded gravel wrapped in a layer of non-woven geotextile filter fabric. The permeable material or open-graded gravel drain should extend from the base of the wall to the finished grade or to just below the all-weather surface (asphaltic or concrete pavement) if used. Weepholes with a nominal diameter of 1 to ½ inches should be provided at five feet on center. To prevent loss of granular materials from behind the wall, corrosion resistant screens should be placed across the weepholes at the back side of the wall.

Where migration of moisture would be detrimental or undesirable, we recommend that freestanding retaining walls be waterproofed. The waterproofing should consist of an

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impermeable material, such as a bituminous compound, that can be uniformly applied over the full height of the wall.

Backfill behind freestanding retaining walls should consist of Layer A geomaterial and should be placed and compacted in accordance with the recommendations given for structural fill presented earlier in this report.

8. Lateral Load Resistance

Lateral loads may be resisted by a combination of friction between the foundations and the supporting subgrade and passive pressure acting against the vertical face of foundations. KA recommends that the friction coefficients presented previously in this section of the report be used for design. The friction coefficients for geomaterials A, B, and C are applicable to the various structural elements identified in this Section (X) of the report and should be applied in accordance with the expected bearing material. These values are based on residual friction angles from direct shear tests of remolded tuff. Additional adhesion should not be included.

Additional lateral load resistance may be provided by passive pressures developed by backfill weight plus mechanical compaction against mats, footings, and rigid below grade walls. For static conditions, KA recommends that the passive pressure be equivalent to those pressures exerted by a fluid with a unit weight determined by multiplying the passive earth pressure coefficient (k_p) by the unit weight of the geomaterial in contact with the structural element. Additional lateral load resistance values used for rock-like geomaterials assume that the subject foundation structural elements are cast neat against undisturbed, intact faces of these geomaterials. If the rock-like geomaterials are disturbed during foundation excavation or preparation, reduced lateral load resistance values equal to Layer A (structural fill) geomaterials should be used.

Additional lateral load resistance due to dynamic earthquake loading is not provided in this report. Design recommendations for dynamic earthquake loading will be provided by others based on SSI analyses.

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