LA-UR-11-03899

*Approved for public release;
distribution is unlimited.*

Title: PHASE I GROUND MODIFICATION ALTERNATIVES FEASIBILITY STUDY
CHEMISTRY AND METALLURGY RESEARCH REPLACEMENT (CMRR)
NUCLEAR FACILITY
LOS ALAMOS NATIONAL LABORATORY
KLEINFELDER PROJECT NO. 101492 Kleinfelder DCN 101492.5.3-ALB10RP001
LANL No.: 100320-RPT-00029
Revision 0
February 22, 2010

Author(s): Catherine A. Flavin
Kleinfelder

Intended for: Reference for CMRR-NF Supplemental Environmental Impact Statement

PHASE I
GROUND MODIFICATION ALTERNATIVES
FEASIBILITY STUDY
CHEMISTRY AND METALLURGY RESEARCH
REPLACEMENT (CMRR) NUCLEAR FACILITY
LOS ALAMOS NATIONAL LABORATORY
KLEINFELDER PROJECT NO. 101492

Kleinfelder DCN 101492.5.3-ALB10RP001
LANL No.: 100320-RPT-00029

Revision 0

February 22, 2010

Prepared for:

Los Alamos National Security
Los Alamos, New Mexico

Submitted to:

ARES Corporation

Prepared by:

Kleinfelder, Inc.
8300 Jefferson NE, Suite B
Albuquerque, NM 87113

PHASE I
GROUND MODIFICATION ALTERNATIVES FEASIBILITY STUDY
CHEMISTRY AND METALLURGY RESEARCH REPLACEMENT (CMRR) NUCLEAR FACILITY
LOS ALAMOS NATIONAL LABORATORY
KLEINFELDER PROJECT NO. 101492

Kleinfelder DCN 101492.5.3-ALB10RP001
LANL No.: 100320-RPT-00029

Prepared for:

Los Alamos National Security
Los Alamos, New Mexico

Submitted to:
ARES Corporation

Prepared by:

KLEINFELDER
Bright People. Right Solutions.

8300 Jefferson NE, Suite B
Albuquerque, New Mexico 87113

Prepared by:	Alan Kuhn, Ph D, P.E. Senior Principal Consultant	02/22/10 Date
Reviewed by:	Roy McKinney, PMP, P.G. Senior Program Manager	2-23-10 Date
Approved by:	John North, P.E. Project Manager	2/23/2010 Date

TABLE OF CONTENTS

TABLES

APPENDICES

ACRONYMS

amsl	above mean sea level
ASR	alkali-silica reactivity
ASTM	American Society for Testing of Materials
CLSM	Controlled Low-Strength Material
CMRR	Chemistry and Metallurgy Research Replacement
cm/sec	centimeters per second
CSM	cutter soil mixing
cm	centimeter
cy	cubic yard
DMM	deep mixing method
DOE	Department of Energy
fps	feet per second
ft	feet
GM	ground modification
GMA	ground modification alternatives
hr	hour
ISRS	in-structure response spectra
GPa	gigapascals
ksi	1,000 pounds per square inch
LANL	Los Alamos National Laboratory
LANS	Los Alamos National Security
m	meter
M	million
MPa	megapascals
m/sec	meters per second
PC-3	Performance Category 3
PCA	Portland Cement Association
PG	permeation grouting
PIDADS	Perimeter Intrusion Detection, Assessment and Delay System
PRT	peer review team
PSA	peak spectral acceleration
PSHA	probabilistic seismic hazard analysis
psi	pounds per square inch
RCC	roller-compacted concrete
RCTS	resonant column torsional shear
RLUOB	Radiological Laboratory/Utility/Office Building
SASW	spectral analysis of surface waves
sf	square feet
SME	subject matter expert
TRD	Trench Re-Mixing and Cutting Deep Wall
UCS	unconfined compressive strength
USCS	Unified Soil Classification System
V_s	shear wave velocity

1 INTRODUCTION

Kleinfelder, West Inc. (Kleinfelder) presents this report for Phase I of the Ground Modification Alternatives (GMA) Feasibility Study as part of its scope of work in support of the Chemistry and Metallurgy Research Replacement (CMRR) project at Los Alamos National Laboratory (LANL) located in Los Alamos, New Mexico. The work was performed by Kleinfelder as a subcontractor to ARES Corporation (ARES) in accordance with subcontract agreement 0833300-009 and the Work Plan (Kleinfelder, 2009) for this task. ARES has a prime contract with Los Alamos National Security (LANS), which is the operator of LANL for the Department of Energy (DOE). The term LANL will be used to mean both the facility and the operating entity (LANS) from this point forward throughout this document.

This report addresses Phase I of four planned phases of work that will lead to selection and implementation of one or more ground modification (GM) methods to improve the characteristics of the lower portion of Unit 3, Bandelier Tuff ($Qbt3_L$) under the building footprint. Phase I involved conducting a feasibility study of candidate GM methods and the down-selection of up to three methods from an initial list of 10 methods. Phase II will include the development of conceptual design for the methods that are recommended based on the results of Phase I. Phase III will include a pilot field and laboratory program to 1) verify that the favored GM methods conceptually designed in Phase II produce the desired dynamic and geotechnical properties of $Qbt3_L$, and 2) select the final GM method. The Phase IV scope will include the development of design drawings and specifications for construction of the selected GM method.

The foundation of the proposed CMRR building will be located at a depth of about 75 feet (ft) below original site grade, which corresponds to an elevation of 7,226 above mean sea level (amsl). At this depth, the foundation will bear on or near the contact between the upper ($Qbt3_U$) and lower ($Qbt3_L$) Unit 3 Bandelier Tuff, thereby reducing the thickness of, or entirely eliminating the more rock-like $Qbt3_U$. $Qbt3_L$ has been well characterized over the course of the project and consists of a poorly-welded volcanic tuff.

Section 1.1 provides the background for the genesis of the Ground Modification Alternatives Study, including concerns raised by the LANL peer review team (PRT) that the $Qbt3_L$ should be treated or improved to increase its dynamic and geomechanical properties to the requisite values of the CMRR Performance Category 3 (PC-3) nuclear facility.

1.1 Background

Geologic and Geotechnical Conditions Requiring Modification

Based on previous work performed by Kleinfelder (Kleinfelder 2007a, 2007b), $Qbt3_L$ ranges from about 54 to 57 ft in thickness across the building plan and averages approximately 56 ft thick with upper and lower transition zones comprised of slightly stiffer and slightly more competent material. $Qbt3_L$ has lower bearing capacity and is more compressible than the $Qbt3_U$

layer above it or the Qbt2 layer below it. $Qbt3_L$ exhibited a best-estimate small-strain shear wave velocity (V_s) of about 1,050 feet per second (fps). According to LANL and based on the Probabilistic Seismic Hazard Analysis (PSHA) work done by others, the earthquake strain-compatible V_s for $Qbt3_L$ ranges from about 350 fps for the lower-bound soil properties case to 600 fps for the best-estimate soil properties case. $Qbt3_L$ has a relatively high porosity of approximately 48 % and exhibits little cohesion. $Qbt3_L$ exhibited an angle of internal friction of approximately 33 degrees. Laboratory hydro-collapse tests (consolidation test with inundation at foundation pressure) generated vertical strains ranging from 1 to 3 % with an average of about 2 % strain, suggesting that this layer has a slight to moderate potential for hydro-collapse due to wetting.

The properties of $Qbt3_L$ that are most problematic are those that affect the seismic response of the unit. Specifically, the V_s is substantially lower than the 4,000 fps value that is desired for the foundation subgrade. The density and shear modulus need to be modified (increased) to achieve a more favorable seismic response, as well as behavior in bearing and settlement.

Design Concerns Arising from Ground Conditions

The existing properties of $Qbt3_L$, coupled with its vertical proximity to the CMRR foundation grade and its lateral proximity to the slope of Two-Mile Canyon, have led to potentially significant issues for the design team and the PRT. The five design concerns are:

- potential for static deflection (compression),
- potential for hydro-collapse due to wetting,
- potential for excessive movement of buttress due to dynamic slope instability,
- inadequate resistance to dynamic sliding forces, and
- seismic shaking and building response.

LANL is concerned that an attempt to solve all five of these issues through more detailed analysis could have moderate to severe impacts to the project schedule and even the project's viability. Accordingly, LANL believes that there is less risk to the project by mitigating these issues by means of ground modification of the $Qbt3_L$ unit below the CMRR foundation.

Ground modification was initially discussed as a way to mitigate the first four issues identified above. Recently completed sensitivity analysis performed by LANL suggested that with ground modification, the in-structure response spectra (ISRS) could be reduced, providing even greater justification for ground modification. Based on the aforementioned sensitivity analysis performed by LANL (Mertz et. al., 2009), it appears that an improvement of the small-strain V_s of the $Qbt3_L$, from 1,050 (current in-situ best estimate value) to 2,000 fps would reduce the floor peak spectral acceleration (PSA). An improvement from 1,050 to 4,000 fps in the V_s of the $Qbt3_L$ would appear to reduce the PSA even further, but additional increases in V_s above 4,000 fps appear to have diminishing returns. Reduction of the motions by these levels could generate significant cost savings to equipment and components and could reduce the thickness of the mat-slab foundation.

Target Zone for Ground Modification

The target zone for ground modification is the volcanic tuff of $Qbt3_L$ between the foundation grade at elevation 7,226 ft amsl and the top of Qbt2 at approximately elevation 7,170 ft amsl. The 56-ft thick zone would include all of the $Qbt3_L$ vertically below the CMRR foundation. The horizontal limits of the target zone include the CMRR building footprint plus an additional zone specified by LANL that extends 20 ft beyond the maximum plan dimensions. The maximum plan dimensions of the CMRR building, as presented on the "Construction Access and Limits of Excavation Plan", Drawing No. C-54634 prepared by Sargent & Lundy, are 303 ft by 341 ft. Including the additional 20 foot zone, the total plan dimensions of the ground modification zone will be 343 ft by 381 ft. Based on these dimensions and a $Qbt3_L$ average thickness of 56 ft, the total volume of material to be modified will be approximately 271,000 cubic yards (cy) (Appendix G).

1.2 Purpose of the Study

Phase I of the GMA Feasibility Study was conducted to identify and assess the feasibility of one or more candidate GM methods that will be subsequently designed preliminarily in Phase II (conceptual design) and field tested in Phase III (field demonstration). The Phase I study was primarily a qualitative evaluation to support comparisons between candidate GM methods to select those best suited for further evaluation in subsequent phases. The qualitative evaluation was based on research of methods, professional experience with similar types of methods, relevant information from industry experts, and very simplified analyses. GM methods that are down-selected from Phase I will be designed conceptually in Phase II, which will include more sophisticated quantitative, analytical evaluation of each recommended GM method.

1.3 Scope of Phase 1

In the course of Phase 1, Kleinfelder, in close consultation with LANL, ARES, and the PRT, completed the following scope of work:

- identified performance objectives for the candidate GMAs (Table 1),
- developed evaluation criteria for the candidate GMAs based on the performance objectives and the evaluation methodology (Section 2),
- identified CMRR-specific factors that affect constructability and implementation of GMAs (Section 3)
- identified and characterized candidate GMAs (Section 4),
- evaluated the GMAs against the performance criteria and relative cost and schedule (Section 5),

- recommended up to three GMAs for conceptual design in Phase II (Section 6), and
- prepared this technical report of the Phase I GMA Feasibility Study.

2 PHASE I EVALUATION METHODOLOGY

2.1 Evaluation Criteria

In preparation for the GMA evaluation process, a set of evaluation criteria were developed for use in comparing between and selecting candidate GMs. The long-term GMA evaluation criteria, to be assessed in detail for the selected GMAs in Phases I through III, include:

- potential for meeting the GM target performance objectives (Table 1),
- constructability,
- uniformity and variability of modified ground,
- ability to verify the results,
- cost and schedule,
- risks and pitfalls (e.g., geochemical stability, effects on PF-4 facility),
- environmental impacts (e.g., noise, air quality, vibration, storm water pollution prevention), and
- availability of materials.

These criteria were further refined, and in some cases combined, for application in the GMA screening process, described below and on Tables 2, 3, and 4.

2.2 Evaluation Methodology

The GMA evaluation methodology consisted of three steps, as originally described in the Ground Modification Feasibility Study Work Plan (Kleinfelder, 2009). In the first step, an initial list of GM methods was compiled. In the second step (Level 1 screening), this initial list was screened against a set of pass/fail criteria, based on the criteria identified in Section 2.1 above, to eliminate GMAs that had fatal flaws. In the third step (Level 2 screening), the GMAs that had passed the Level 1 screening were evaluated against each other using a set of criteria consolidated from the Level 1 criteria and including constructability, cost and schedule.

2.2.1 Identification of Candidate GMAs

An initial list of candidate GMAs was compiled by Kleinfelder in consultation with LANL, ARES, and the PRT. The list included methods known to the participants through personal experience, through familiarity with the foundation construction industry, or through literature search. Methods were included that appeared worthy of consideration at least through an initial screening.

The initial list of candidate GMAs included:

<u>In-Situ Ground Treatments</u>

- Jet Grouting
- Conventional Deep Mixing Method
- Permeation Grouting with Micro-Fine Cement
- Trench Re-Mixing and Cutting Deep Wall Method
- Cutter Soil Mix Method

<u>$Qbt3_L$ Excavation and Replacement with</u>

- Roller-Compacted Concrete
- Controlled Low-Strength Material
- Concrete Fill
- Soil Cement

<u>Deep Foundation</u>

- Structural reinforcement with Reticulated Type II Micro-Pile Network

Hybrid systems (e.g., use of a combination of two or more GMAs) were not separately identified and evaluated because the merits or drawbacks of each would be related to its component GM methods. Hybrids will be considered in a later phase if one or more GMAs appear to be inadequate on their own.

Information on these 10 GMAs was collected through:

- Literature search
- Specialty contractor communications
- Subject Matter Expert (SME) consultation

Kleinfelder limited its information collection efforts for the Level 1 screening to those needed to support a fatal flaw assessment. This was done in order to screen the candidate GMAs from an initial list of 10 potential alternatives down to a short list that have the greatest likelihood of satisfying the performance objectives and comparing well on the evaluation criteria, Additional information was collected and evaluated for those GMAs that exhibited no fatal flaws and advanced to the Level 2 screening.

2.2.2 Level 1 – Pass/Fail Screening

The first level of GMA screening was for fatal flaws using pass/fail criteria for:

- Ability to protect PF-4 and other surrounding facilities. If the use of a GMA (solely or with the "hybrid" inclusion of other techniques) would cause an unmitigated risk to adjacent facilities of high importance during construction (i.e., PF-4), this GMA was eliminated from consideration (failed) prior to the full analysis.
    - Is excavation below foundation grade required? If yes, can the excavation below foundation grade be supported sufficiently to protect PF-4?
    - Is the GMA able to prevent or adequately limit the risk of settlement of PF-4?
    - Is the GMA able to keep ground vibrations at PF-4 to acceptable limits?
- Ability to meet the minimum requirement of material properties of the ground under CMRR, including:
    - Is a V_s of at least 4,000 fps in the treated ground reasonably achievable?
    - Can relatively uniform properties of the treated ground be delivered?
    - Can a methodology for verification of V_s of the treated ground be developed?

The Level 1 pass/fail screening is documented on Table 2. A pass/fail judgment for each of the two primary criteria was made, and comments are provided to support each pass/fail score.

2.2.3 Level 2 – Relative Evaluation

Candidate GMAs that passed the Level 1 pass/fail screening were evaluated using criteria proposed by Kleinfelder and finalized through close consultation with LANL, ARES, and the PRT. Because all candidate GMAs passed the Level 1 screening for impact and safety during construction with respect to protection of PF-4, this criterion was not factored into the Level 2 evaluation. The other criteria were retained and reformulated for relative evaluation of GMAs in Level 2 as follows:

- performance of the $Qbt3_L$ treated volume under CMRR,
    - materials properties and their uniformity throughout the treated area or volume
    - variability of the physical properties following ground modification treatment
    - ability to readily verify the ground modification treatment
- constructability,
- cost, and
- schedule.

This second level of screening evaluated each GMA against each of these criteria using relative, qualitative scales for comparison. Categories or “bins” of suitability (e.g., high, medium, low) were established to classify the GMAs into approximately equal groupings for each criterion. The definitions for high, medium, and low are criterion-specific and are provided in Table 3.

Information used in the Level 2 evaluation of the GMAs included:

- description of the GMA, including required equipment;
- history of application;
- methods for verification;
- logistical considerations – available working space, laydown area, utilities, traffic, and transportation impacts;
- operational limits of equipment;
- costs – mobilization/demobilization, unit prices, and contingency; and
- time to complete.

3 CONDITIONS AFFECTING CANDIDATE GMAs

3.1 Geological and Geotechnical Conditions

The CMRR site is located on Pajarito Mesa in TA-55, and the facility will be constructed within Units 3 (Qbt3) and 4 (Qbt4) of the Tshirege Member of the Bandelier Tuff. The site geology and geotechnical conditions are described in detail in the Geotechnical Engineering Report (Kleinfelder, 2007b). The overlying unit, Qbt4, is soft volcanic tuff with slight to moderate welding and substantial random fracturing. Some fractures are deeply weathered and clay-filled. The upper part of Unit 3 ($Qbt3_U$) is similar to Qbt4 but less fractured and weathered. The lower part of Unit 3 is non-welded to slightly welded, is weak and friable, does not sustain fractures, and exhibits more soil-like properties. The ground modification target is the lower portion of Unit 3 ($Qbt3_L$)

Some excavation of the CMRR site has been performed previously, removing the uppermost unit (Qbt4) and some of the topmost part of the upper Unit 3 ($Qbt3_U$) within the CMRR footprint The original ground surface of the CMRR site sloped toward the east and south from about elevation 7,293 ft amsl at the northwest corner. The previous excavation has lowered the ground surface at the site to an elevation of 7,270 to 7,276 ft amsl. Excavation required for the proposed CMRR building will extend to at least an elevation of 7,226 amsl, or approximately 45 to 50 ft below existing grade. Excavation to this elevation will require excavation support in Qbt4 and $Qbt3_U$ to resist loosening and displacement of the tuff along fractures and to strengthen the rock mass to support loads imposed by construction activities along the excavation perimeter.

Additional excavation support will be required if an excavate-and-replace GMA (excavation to the base of $Qbt3_L$) is selected. This support would be applied in the $Qbt3_L$ to resist degradation of the exposed weak volcanic ash and stiffen the ground to support loads from the overlying $Qbt3_U$/ Qbt4 and construction loads.

3.2 Dimensional and Spatial Constraints

The top of $Qbt3_L$ is within a few feet below foundation grade, elevation 7,226 ft amsl, and the bottom of $Qbt3_L$ (top of Qbt2) is at average elevation of 7,170 ft amsl, giving a thickness of $Qbt3_L$ of approximately 56 ft. The plan dimensions of the CMRR building will be 303 ft by 341 ft, the volume of the prism to be treated by any GMA includes $Qbt3_L$ material under the building footprint plus 20 ft beyond each side, or approximately 271,000 cy (Appendix G). For in-situ GMAs, this prism will be left in place, but for excavate-and-replace GMAs this prism will be completely excavated to the top of Qbt2 before replacement with the selected material. All GMAs will require excavation to foundation grade and support of the excavation to that level (elevation 7,226 ft), but excavate-and-replace methods will require support of excavation walls from foundation grade to the top of Qbt2 (elevation 7,170 ft), as well.

For purposes of this evaluation, Kleinfelder assumes that all candidate GMAs will require the same logistical support, access, and working space including lay-down area and batch plant, spoils disposal, and utility services. The space within and adjacent to the CMRR site is limited by the Perimeter Intrusion Detection, Assessment and Delay System (PIDADS) along the west and north sides and the Radiological Laboratory/Utility/Office Building (RLUOB) building adjacent to the east side of CMRR. Pajarito Road along the south side of CMRR will be relocated further to the south but will still limit available space to a narrow strip between the south edge of CMRR and the road. Consequently, activities like concrete batching, equipment lay-down and maintenance, and stockpiling will be limited to off-site locations identified by LANL. For purposes of cost evaluation, the concrete batch plant(s) are assumed to be located approximately one mile east of CMRR, and the excavation spoils are assumed to the trucked to a site five miles east of CMRR for disposal.

Ingress and egress at the CMRR excavation will also be constrained by the limited space available. Primary access to foundation grade (elevation 7,226 ft amsl) will be by relatively steep (9 to 12% grade) ramps along the east an1d south sides of the excavation (LANL, 2009). Tower cranes are planned to be situated outside the northeast and southwest corners of the building near foundation grade. Due to the weak nature of the $Qbt3_L$ ash, access ramps below foundation grade through $Qbt3_L$ within the excavation might not be feasible if an excavate-and-replace GMA is used. In this situation, it is likely that the GMA will incorporate a provision for hoisting or conveying, a method for ingress/egress through $Qbt3_L$ that is incorporated into the GMA, or both.

3.3 Constructability

In the context of CMRR ground modification, constructability means the ability to complete a GMA to meet the material properties criterion (V_S at least 4,000 fps) in the treated ground, within the constraints of space and depth imposed by the site, using available equipment and established practices, and without extraordinary measures that have large cost or schedule penalties. Both the geotechnical conditions and the dimensional/spatial constraints described above impact the constructability of the candidate GMAs. Constructability is impacted by equipment availability and performance limits, as well. Cost and schedule are treated separately in this GMA evaluation but clearly are impacted by the foregoing constructability factors. Uncertainty is also a factor in constructability evaluation; the more precedent for a GMA in comparable applications, the better. Finally, constructability at CMRR must recognize potential negative impacts to security and operations issues at adjacent facilities. Table 3, which provides an evaluation of the GMAs, includes relative evaluation of constructability.

4 DESCRIPTION OF CANDIDATE GMAs

4.1 Candidate GMAs Eliminated in Level I

All 10 GMAs passed the criteria for protection of PF-4. However, based on the screening for the material-properties criteria, only six of the 10 candidate GMAs passed. The Level I screening results are presented in Table 2. To be retained for Level 2 screening, “pass” scores on both PF-4 protection and material properties criteria were necessary. The GMAs that failed the material-properties criterion screening are:

Trench Re-mixing and Cutting Deep Wall Method

Trench Re-mixing and Cutting Deep Wall (TRD) method involves a continuous vertical cut in soil, up to 34 inches wide and 170 ft deep, made by a chain saw-like apparatus. The cuttings are simultaneously mixed with a cement-based grout, creating a soil-cement wall. TRD has been used over the past two decades on a large number of projects, primarily for installation of cut-off walls. For application at CMRR, a pattern of intersecting orthogonal walls (waffle pattern) would have to be installed, but the method is not well-suited for the short runs in a waffle pattern and does not provide a means for assuring that gaps of soil are filled between the adjacent walls. Therefore, this method was failed on the uniformity criterion. The large amount of space required for the TRD equipment was also a consideration.

Cutter Soil Mixing

The Cutter Soil Mixing (CSM) method mixes in-situ soil with cement slurry using cutting wheels that rotate about a vertical axis. CSM was developed for construction of cut-off and retaining walls and, therefore, would produce a waffle-like pattern in which gaps of untreated soil may be left, requiring additional passes or a secondary method of treatment. Therefore, this method was failed on the uniformity criterion.

Controlled Low-Strength Material Concrete

Controlled low-strength material (CLSM) is a flowable cementitious material used primarily as backfill in place of compacted fill and consisting of fine to coarse sand as an aggregate, ordinary Portland cement Type I, fly ash, and water. Its primary application is as backfill in utility excavations and other confined spaces. By definition, its strength is low enough that the cured material can be excavated by hand using shovels. It differs from lean concrete by the amount of cement and water in the mix, and from soil cement by the aggregate used. Despite the advantages of using a flowable fill to backfill the space left from excavating $Qbt3_L$, the reviewed literature indicated that cured CLSM could not meet the material property (V_S) criterion and was dropped from further evaluation.

Reticulated Type II Micro-pile Network

Although not a ground modification method per se, micro-piles were included in the original list of candidate methods because they offered the possibility of stiffening $Qbt3_L$ and providing the necessary horizontal shear resistance for the foundation. Micropiles are small diameter piles that can be installed in locations with restricted headroom or working space. However, uniformity and verification are both uncertain enough for this method that it failed on both criteria.

4.2 Candidate GMAs Evaluated in Level 2

The six candidate GMAs that passed Level 1 screening included three in-situ and three excavate-and-replace methods, described in detail in the appendices A through F of this report and summarized in the following sections. The volumes that apply to the candidate GMAs are described above in Section 3.2 and in Appendix G.

All GMAs evaluated in Level 2 are considered to be constructible using the criteria in Section 3.3. All Level 2 GMAs can be implemented within the space available and to the required depths. Equipment and practices exist in the construction industry for implementation of each Level 2 GMA at CMRR. The CMRR application should not require measures or costs beyond those typically associated with each method. Each GMA has some precedent of successful application that is relevant to CMRR. Potential negative impacts to security and operations issues at adjacent facilities do not differ significantly between the Level 2 methods and are manageable.

The cost data presented in Table 4 are based on a variety of sources including published documents, conversations with specialty contractors, industry publications, local practice and pricing, and data forwarded by Kleinfelder's SME. There are many parameters that define the unit cost. Some of these parameters are well known and understood while others are not. Therefore, Kleinfelder has included variable contingencies for the six GMAs evaluated in Level 2. Low contingency values represent a method that is relatively well understood and exhibited few uncertainties, whereas a higher contingency value represents a method that is not as well understood and has more uncertainty.

4.2.1 In-Situ Methods

Jet Grouting

Jet grouting (Appendix A) involves high pressure/velocity injection of grout slurry to disturb and replace the soil with grout, creating grouted columns. The method is used in a variety of soil types, including silty sand similar to the volcanic tuff that constitutes $Qbt3_L$. A borehole is drilled to the depth of the target treatment zone (bottom of $Qbt3_L$), and then grout slurry is introduced through horizontal radial nozzles at the bottom of the drill string at high pressure/velocity to disturb the subsurface soil and to mix the material with grout. High pressure air or water may also be used in addition to the grout slurry to aid in cutting the subsurface material. The jet grouting system is slowly rotated and raised from the bottom to the top of target zone to achieve a cylindrical geometry (column). Grouted columns are formed at a regular gridded interval, and a second set of columns can be formed to fill gaps between columns in the first pattern.

Several configurations of nozzles and other apparatus can be used, depending on the size of column desired and the erodibility of the soil. To establish contiguous or overlapping columns in

$Qbt3_L$, an arrangement of nozzles specially designed to deliver a highly focused stream of grout and air is likely to be most effective in creating large-diameter grouted columns.

Unconfined compressive strengths of up to 3,000 pounds per square inch (psi) and P-wave velocities of up to 8,000 fps in jet grouted silty sand have been reported in the literature. Both laboratory and field testing methods are available to verify these properties, which may be both non-uniform and variable through the treated column and between columns.

Logistical requirements (access and space for equipment operations) can be accommodated within the CMRR excavation and adjacent laydown area, and space for at least two rigs is available.

Jet grouting can be performed from the initial (existing in 2010) ground surface or from foundation grade, although the latter would require movement of the rigs down one of the steep construction ramps.

Approximately 3,800 grout columns with an average diameter (horizontal spacing) of 6 ft and depth of 56 ft would be required to treat the $Qbt3_L$ prism, approximately 271,000 cy of ground, through 213,000 lineal ft of grout column. At an average advance rate of 300 ft per rig-day, 710 rig-days would be required. Using two rigs, the work should be performed in 355 days. The estimated cost is $300 per cy and the contractor cost is estimated to be $82M, including mobilization. Using a 20% contingency, the total estimated cost is $98M.

<u>Deep Mixing</u>

The Deep Mixing Method (DMM), described in more detail in Appendix B, is a soil treatment technology that combines cementitious materials in either wet or dry form with native soils using rotating shafts that are tipped with a cutting tool. The method is applied to a variety of soil types including silty sand. A number of different proprietary DMMs have been developed; these differ by the specific equipment, type of binder, energy of injection, and mixing principle (along the shaft or at the end of the shaft). The shafts are either equipped with discontinuous auger flights or mixing blades or paddles. In some methods, the mixing action is enhanced by simultaneously injecting fluid grout at high pressure through nozzles in the mixing or cutting tools. One to eight shafts are mounted on each crawler-mounted carrier. Column diameters typically range from 2 ft to 5 ft and may extend to 132 ft in depth. Soils treated with DMM typically show unconfined compressive strengths of up to 1,450 psi in granular soils and shear wave velocities for sands treated with DMM of 4,100 fps. Both laboratory and in-situ methods are available to verify properties of treated soil, which may be both non-uniform and variable through the treated column and between columns.

Logistical requirements (access and space for equipment operations) can be accommodated within the CMRR excavation and adjacent lay-down area. Space might be available for two rigs. DMM would be performed from foundation grade, requiring movement of the rigs into and out of the excavation using one of the steep construction ramps.

Assuming a treated column diameter of 3.3 ft and a column depth of 56 ft, the expected advance rate for a wet mix emplaced at the end of the cutter tool is 665 cy per rig day, requiring 408 rig-days for the $Qbt3_L$ block treatment. Two rigs should be able to work together, reducing the schedule to 204 days. The estimated cost is $180 per cy and with mobilization the total cost is estimated to be $50M. Including a contingency of 25%, the total estimated cost is estimated to be $63M.

Permeation Grouting

The permeation grouting method (Appendix C) involves filling soil matrix pore space with grout without causing large displacement or disturbance of the soil. The grout is low viscosity cement or chemical grout and is delivered at relatively low pressures in order to minimize disturbance to the in-situ ground. The grout components are mixed at ground surface and injected at the desired subgrade depth through a pipe, drill rod, or hollow stem auger. The $Qbt3_L$ consists of volcanic ash with grain size distribution equivalent to a silty sand with Unified Soil Classification System (USCS) classification of SP to SM. According to Baker (1982), soil with the grain size distribution of $Qbt3_L$, with 2 to 20% fines, should be moderately groutable.

Three types of grout can be used in the permeation grouting method: particulate grout (such as aqueous suspensions of cement, fly ash, bentonite, microfine cement, or a combination of materials); colloidal grouts (such as sodium silicate gel or phenolic or acrylic resins); and true solutions. Recent laboratory tests on intact samples of $Qbt3_L$ produced hydraulic conductivity values of 10^{-2} centimeters per second (cm/sec) to 10^{-4} cm/sec, suggesting that a cement grout, as opposed to a chemical grout, may be more appropriate for the treatment of this stratum.

According to Hayward Baker (2004), the unconfined compressive strength of silicate permeation grouted soil can range from 50 to 300 psi. Another study of cement grout showed unconfined compressive strength of up to 2,700 psi for a mix of 1:1 water: cement. Both laboratory and field test methods can be conducted to verify properties of the treated ground which may be both non-uniform and variable through the treated column and between columns.

Permeation grouting can be performed with small drill rigs and an on-site mixing/batching plant. Three or more permeation grouting operations can be accommodated within the CMRR excavation. Permeation grouting can be performed from the initial (existing in 2010) ground surface or from foundation grade, although the latter would require movement of the rigs into and out of the excavation using one of the steep construction ramps.

The volume of ground grouted from any grout hole is highly dependent on the apparent viscosity of the mix and the injection pressure. Assuming a 6-ft radius of treatment from each grout hole, 3,800 grout holes would be needed. At a cost of $240 per cy of grouted ground, the estimated cost would be $66M, including mobilization. Including a contingency of 30%, the total estimated cost would be $86M. It is assumed that each rig could complete 225 ft or four holes per day, so the entire treatment would require 950 rig-days. Actual schedule can be reduced by using multiple rigs; four rigs and 200 days are assumed for schedule estimating.

4.2.2 Excavate-and-Replace Methods

RCC, concrete fill, and soil cement ground modification methods will require complete excavation and disposal of the $Qbt3_L$ layer within the CMRR footprint. Because excavation from existing grade to foundation grade will be required for all in-situ and excavate-and-replace GMAs, excavation above foundation grade is not considered in this evaluation; only the excavation below foundation grade to the top of Qbt2 is considered here. As described in Section 3.2, this below-grade excavation will remove $Qbt3_L$ tuff from an area 381 ft by 343 ft and 56 ft deep, a volume of approximately 271,000 cy of $Qbt3_L$.

Below-grade excavation can be accomplished with dozers, loaders, and excavators that would feed the spoil onto a belt conveyor for removal from the excavation area. After excavation is completed, fixed or mobile conveying or hoisting systems would feed the filling material (concrete or cement) from the batch plant to the placement site. Typical conveying systems include rotating retractable conveyors, hoppers and drop chutes, and cranes and buckets to move the mixed material to the point of placement.

Excavation rates are assumed to be a function of the production of a D6 dozer, with a short haul distance averaging 150 ft, for 350 cy per hr. All other equipment is expected to be selected to work within this production rate. Therefore, a total of 775 operating hours will be required for excavation, and at 80 % availability and assuming 8-hr days, the excavation will require 121 days. Mobilization/ demobilization is estimated to be $1M. The excavation unit cost is estimated to be $8.10 per cy for a total cost of $2.3M. If the excavated tuff is not used in the fill material and must be disposed of off-site at a location assumed to be approximately five miles from CMRR, an additional cost $650,000 to $1M will be incurred based on assumed hauling costs of $4 to $6.50 per cy. The total estimated cost of mobilization, excavation and disposal of the $Qbt3_L$ tuff is $4.2M.

Methods and costs of excavation support between foundation grade and top of Qbt2 have not been evaluated for Phase I, but for purposes of comparing all cost and schedule factors, Kleinfelder has assumed a cost of $150 per sf and 81,200 sf (1,450 ft perimeter distance x 56 ft depth) of supported surface for a cost of $12.5M including mobilization. Although ground support and excavation are likely to proceed concurrently, an additional 50% or 56 days of schedule, is assumed to account for delays and time lost to interference between excavation and support activities, bringing the schedule for excavation and excavation support to approximately 180 days.

Roller-Compacted Concrete

Roller-Compacted Concrete (RCC), described in more detail in Appendix D, is concrete placed at no-slump consistency as a moist (not saturated) mix that is able to support a roller during compaction. Properties of hardened RCC can be similar to those of conventionally placed concrete. RCC typically contains aggregate, Type II Portland cement, and pozzolan (Class F or C fly ash) or ground slag. Mixes are adjusted to site-specific conditions and design requirements. Class F fly ash contributes to a lower heat generation at early ages, a potential advantage for CMRR application, and may be used to replace cement (generally up to approximately 50% by volume). RCC can be placed under closely controlled conditions with standard earthmoving and compaction methods, providing confidence in uniformity of the results.

RCC will be performed either concurrently with the excavation phase or after completion excavation of $Qbt3_L$ and placement of ground support in the walls of the excavation. RCC will be batched in a near-by batch plant and transported to the excavation, where it will be deposited in the excavation and spread in controlled lifts (typically 12-inches thick) and compacted with smooth drum rollers.

Typical unconfined compressive strength of RCC ranges from 1,000 psi to as high as 4,000 psi and is primarily a function of cement content. The properties can be verified by standard laboratory and field test methods at whatever frequencies are desired. In-situ testing of field density using nuclear gauges can be performed. During construction, compressive strength tests on recovered core are performed to verify strength development with time (7, 28, 90, 180, and 365 days). Because cement hydration has the potential to result in thermal cracking of mass concrete, thermal properties can be tested. Resonant column torsional shear (RCTS) testing of cured cylindrical samples can be used to confirm intact properties including shear wave velocity. Seismic cross-hole and down-hole tests, as well as surface wave methods such as Spectral Analysis of Surface Waves (SASW), can be performed for verification of shear wave velocities of the in-place cured material after construction.

Typical RCC production rates may range from 50 to 1000 cy per hr. For estimating schedule, 200 cy per hr and 8-hr days have been assumed for RCC batching and placement, giving a total of 170 days for RCC placement and, with excavation, a total schedule is approximately 350 days. RCC costs are based on Portland Cement Association data and escalated to 2010 at $95 per cy, giving an RCC placement estimated cost of $26M. Accommodations for the cost of managing the thermal effects of cement hydration have not been considered. With excavation and ground support costs included, the estimated cost of the RCC method is $43M, and with a 10%contingency, the total estimated cost is $47M.

Concrete Fill

Concrete fill (Appendix E) is a flowable mixture of Portland cement, coarse/fine/aggregate, and water placed by conventional methods. Due to potential issues of alkali-silica reactivity (ASR), the coarse/fine aggregate used for this alternative consists of an imported coarse/fine aggregate not subject to ASR. For conventional concrete fill, the coarse/fine/aggregate particles are bonded by the cement paste, and completely coated through mixing of the cement, the coarse and fine aggregate, and other additives. The concrete mix can be designed for specified properties of aggregate, cement type and content, and water-cement ratio to achieve the required elastic and strength properties with proper quality control methods.

Concrete fill will be performed after excavation of $Qbt3_L$ and placement of ground support in the walls of the excavation, as for RCC. Concrete fill will be batched in a near-by batch plant and pumped to the excavation, where it will be placed using standard concrete placement methods. The excavated walls will serve as the forms for the concrete fill, which will be placed in lifts.

Heat of hydration is a concern for mass concrete placement and it will affect the advance rate. Past experience, such as the Building 9720-82 mass concrete fill at the Y-12 National Security Complex, suggests this issue can be controlled. Lifts would probably be limited to a maximum of 3.0 ft thickness, with successive lift placement separated by at least 72 hours (Y-12 National Security Complex, 2005). Chilled aggregate or cooling pipes can be used as needed to control the heat of hydration.

Although concretes are available with 28-day unconfined compressive strengths from 2,500 psi to as high as 10,000 psi, a typical range of unconfined compressive strengths is from 3,000 to 7,000 psi. During construction, compressive strength tests are performed at prescribed times (7, 28, 90, 180, and 365 days). Other standard ASTM tests are available for characterizing the strength and deformability of concrete fill in addition to RCTS and SASW testing mentioned previously.

Typical production rates for concrete fill are from 110 cy per hr to 800 cy per hr. Assuming a production rate of 200 cy per hr and 8-hr days, the required time for emplacement is approximately 170 days. Heat–of-hydration dissipation will also limit concrete placement rates to approximately 200 cy per hr. The estimated placement costs would be $115 per cy, giving a concrete fill placement cost of $31M. Accommodations for the cost of managing the thermal effects of cement hydration have not been considered in the placement of the concrete fill. With excavation and ground support costs included, the estimated cost of the concrete fill method is $48M and total schedule is approximately 350 days. Using a relatively low contingency of 10%, the total estimated cost is $53M.

Soil Cement

Soil cement (Appendix F) is a densely compacted mixture of Portland cement, soil aggregate, and water (ACI, 1997). Soil cement differs from RCC in that, while RCC uses aggregate selected to include sand and gravel, the soil cement uses the native soil as partial to full replacement for aggregate. For CMRR, the aggregate replacement for this alternative would consist of the volcanic ash of the $Qbt3_L$ unit that has the consistency of silty sand to sand (SM to SP soil). For soil cement, the soil aggregate particles are bonded by the cement paste, but unlike concrete, the individual particles are not completely coated with cement paste. Soil cement is used for base materials under pavements, slope protection for dams and embankments, liners for channels, and mass soil-cement placements for dikes and foundation stabilization.

In addition to the soil (crushed volcanic ash) aggregate as a partial to full replacement of sand and gravel, the mix would contain Type II Portland cement and pozzolan (Class F or C fly ash) or ground slag. It may be possible to adjust mixes to account for potential ASR by adding fly ash; however, this option cannot be considered until initial ASR testing is completed. Class F fly ash also contributes to a lower heat generation at early ages, a potential advantage for CMRR application, and may be used to replace cement (generally up to approximately 50% by volume). Soil cements are prepared at or slightly wet of optimum water contents to achieve this target density.

The properties of soil cement are influenced by several factors including type and proportion of soil, cementitious materials, water content, compaction, uniformity of mixing, curing conditions, and age of the compacted mixture. Because of these factors, a wide range of values for specific properties exists. However, soil cement using light-weight volcanic tuff aggregate has been tested with unconfined compressive strengths as high as 8,600 psi.

Soil cement properties can be verified by standard laboratory and field test methods at whatever frequencies are desired. In-situ testing of field density using nuclear gauges can be performed. ACI 230 contains guidance for quality control during construction. During construction, compressive strength tests on recovered core can be performed to verify strength development with time (7, 28, 90, 180, and 365 days). Because cement hydration has the potential to result in thermal cracking of mass concrete, thermal properties can be tested.

Verification testing can be divided into three categories. These include quality control tests during placement, standard engineering tests on recovered core or fabricated cylinders, and in-situ test methods. The standard engineering tests on recovered core can be performed using standard ASTM methods. Moisture density testing controls compaction for emplaced soil cements. In general, a density requirement ranges from 95 to 100% of the maximum density of the cement-treated soil. The most common standard test methods for determining in-place density before curing include the nuclear method (ASTM D2922 and D3017); the sand-cone method (ASTM D1556) and the balloon method (ASTM D2167). RCTS testing can be used to confirm intact properties including shear wave velocity. Seismic cross-hole or down-hole methods and SASW are available to assess shear wave velocities of the in-place cured material.

As described above for RCC, soil cement for the CMRR ground modification will require complete excavation and disposal of the $Qbt3_L$ layer within the CMRR footprint. After excavation is completed, fixed or mobile conveyors would feed soil cement from the batch plant (a continuous flow pugmill) plant to the placement site. Typical installations include a rotating, retractable conveyor that deposits the soil cement on the lift surface via a drop chute. Because it is assumed that nearly all the excavated $Qbt3_L$ will be used in the soil cement mix, no cost would be incurred for off-site disposal, producing a cost saving of $1M compared to the other excavate-and-replace methods

Typical soil cement production rates are expected to be the same as those for RCC, 50 to 1,000 cy per hr. For estimating the production schedule, 200 cy per hr and 8-hr days have been assumed, giving a total of 170 days for soil cement batching and placement and, with excavation, a total of approximately 350 days. Soil cement costs should be similar to those for RCC, less the cost of the aggregate. Based on PCA data escalated to 2010 and subtracting cost of aggregate at $30 per cy, the estimated cost of soil cement is $65 per cy or $18M total. Accommodations for the cost of managing the thermal effects of cement hydration have not been considered. With excavation cost included, the estimated cost of the soil cement method is $33M. With contingency of 15%, the estimated total cost is $38M.

5 EVALUATION OF GMAs

5.1 Level 1 Evaluation

The rationale and process for Level 1 screening for fatal flaws was described in Section 2.2.2 and the results shown in Table 2. The candidate GMAs that failed the material-properties criteria screening and were eliminated from further evaluation in Level 2 are:

- Trench Re-mixing and Cutting Deep Wall Method,
- Cutter Soil Mixing,
- Controlled Low-Strength Material Concrete, and
- Reticulated Type II Micro-pile Network.

5.2 Level 2 Evaluation

GMAs that passed the Level 1 screening, receiving a "pass" score of the criteria described in Section 2.2, were then screened against the Level 2 criteria. The evaluations are documented in Table 3, and cost estimates are presented in Table 4. Relative ratings of high, medium, and low were used to compare the GMAs. A key to these ratings is provided in Table 3.

5.2.1 Evaluation of In-Situ Methods

The three in-situ GMAs share several features – all would be applied from approximately the same depth above the final foundation grade, would avoid excavation below foundation grade, and involve using or leaving in place the $Qbt3_L$ volcanic ash as a constituent of the improved ground. The estimated time for completion is longest for jet grouting (assuming two rigs) at 355 days, giving it a low schedule rating, and shortest for permeation grouting (assuming four rigs) at 200 days for a high rating for schedule.

ASR is a potential issue in all three in-situ GMAs. ASR evaluation is beyond the scope of Phase I, but is identified here for attention in the near term. ASR degrades the physical properties of cementitious mixtures in which the aggregate (in this case, $Qbt3_L$) reacts with Portland cement in the presence of moisture to form a gel. The gel can cause expansion and cracking of the cured mix, reducing its strength. Because the ambient moisture of $Qbt3_L$ is very low, water used in the mix will be essentially consumed during hydration, and the CMRR design will minimize the risk of deep infiltration from ground surface, the ASR potential for any of the in-situ methods is probably low. Nevertheless, the ASR potential needs further evaluation.

Jet Grouting is rated as high in material properties and variability. Case histories indicate that a V_s of 4,000 fps is achievable in a relatively uniform sand medium like $Qbt3_L$ Constructability is rated as medium because of the need to maneuver rigs down steep ramps if jet grouting is performed from foundation grade, and multiple or overlapping passes might be needed to achieve adequately uniform treatment. Although the results of jet grouting cannot be directly observed, verification can be made using several field and laboratory methods to give verification a medium rating. The cost rating is low because of the high unit cost and the 20% contingency added to account for potential variability.

Jet grouting, applied to form a wall around the CMRR excavation in $Qbt3_L$, is also being considered as an excavation support method.

DMM rates as medium in all three performance criteria. Continuity between adjacent panels is the primary uncertainty in materials properties and uniformity, requiring either overlap of panels or a hybrid approach using grouting as the secondary treatment. Constructability is rated as medium because of the need to maneuver rigs down steep ramps if DMM is performed from foundation grade. Although the results of DMM cannot be directly observed, verification can be made using several field and laboratory methods to give verification a medium rating. Deep mixing can be completed in approximately 204 working days, which gives this method a high schedule rating. The cost rating is medium because of the intermediate cost relative to the other methods and the 25% contingency added to account for variability concerns.

Permeation grouting rates low on properties and variability due to uncertainties, arising from both case histories and assessment of $Qbt3_L$ properties, about distribution of grout through the intact tuff pore spaces. Although laboratory permeability tests indicate favorable conditions for pushing grout through the pore space, the fragile structure of the tuff creates uncertainty about the potential for hydro-fracturing under grouting pressures. Verification can be achieved through testing of laboratory samples and field tests, giving permeation the same medium rating as the other in-situ methods. Constructability is rated as high because of the ability to grout with small rigs from any excavation level. The cost rating is low because of the high estimated total cost and the 30% contingency added to account for material property and variability concerns. Assuming that four grouting rigs can work simultaneously in the CMRR space, the schedule of 200 days is rated high compared to all other methods except DMM.

5.2.2 Evaluation of Excavate-and-Replace Methods

The three excavate-and-replace GMAs are similar, and all would require excavation to the top of Qbt2 and support of excavation below foundation grade. All three involve batching and placing cementitious fill. The distinction between the RCC and soil cement is the aggregate, with a percentage of $Qbt3_L$ volcanic ash being used in the latter. Constructability is rated as high for RCC and concrete fill because well-established practices and standard equipment can be used for excavation, excavation support, and material placement. Constructability for soil cement is rated medium only because of the uncertainty regarding ASR of the volcanic ash. The cost and schedule of excavation would be the same, about $16-17M and 180 days, for all three methods: RCC, soil cement, and concrete fill. All three methods are rated low on schedule, requiring approximately 350 days (including excavation and excavation support) to complete.

The ASR potential described in Section 4.2.1 applies to the soil cement method, but not to RCC or concrete fill. RCC and concrete fill use aggregate selected from an off-site source for its good properties, including low ASR potential. Soil cement is subject to whatever ASR potential is inherent in the $Qbt3_L$ volcanic ash; however, the lower water content of soil cement versus the in-situ methods would mean lower risk of ASR in soil cement. Increasing percentages of fly ash are likely to ameliorate the ASR problem and its affect can be evaluated by future laboratory testing.

RCC is rated as high in material properties, control of variability, and verification. The properties of RCC can be selected and closely controlled through selection of aggregate, cement types and additives as well as mix design to produce properties on par with structural concrete. Placement can be visually observed, and the RCC can be tested in both the laboratory and field using standard methods to verify properties and variability. RCC costs are relatively low (high rating) and are predictable at about $95 per cy with a relatively small contingency of 10%. Even with the cost of excavation and support included, the cost of RCC rates high (relatively low cost) compared to the in-situ methods.

Concrete fill, like RCC, rates high in the majority of categories and the only real difference is related to the higher unit cost for the material, which makes concrete fill more expensive than RCC. Concrete fill also has the shortest schedule for placement of the three excavate-and-replace GMAs. However, concrete fill has a significant issue related to control of the heat of hydration and prevention of thermal cracking.

Soil cement is rated as medium in material properties and high for variability and verification. Although the properties of soil cement can be controlled through selection of cement types, additives, mix design and compactive effort, the volcanic ash is not an ideal aggregate and its ASR potential remains uncertain, requiring a lower ranking compared to RCC and concrete fill. Placement can be visually observed, and the properties can be tested in both the laboratory and field using standard methods to verify properties and variability. Because of the cost savings of using the excavated $Qbt3_L$ ash as aggregate, the cost of soil cement is lower than RCC, making it the lowest cost GMA at approximately $65 per cy with the highest cost rating and a low contingency of 10%.

6 RECOMMENDATIONS

6.1 GMAs for Phases II and III Evaluation

The Level 2 evaluation demonstrates that jet grouting is the best of the three in-situ methods and that RCC, concrete fill and soil cement are superior to the in-situ methods in terms of performance criteria and cost. If ASR is not a concern for the long-term material properties of the treated ground, then soil cement should remain in consideration. If ASR is a concern that cannot be managed by selection of cement types and additives, then only RCC and concrete fill remain as viable GMAs. Therefore, Kleinfelder makes the following recommends:

1. Deep mixing and permeation grouting should be eliminated from further consideration and not addressed in Phase II.
2. Laboratory testing for ASR of the $Qbt3_L$ should begin as soon as possible using both short-term testing (ASTM C1260 or ASTM C1567) and long-term testing (ASTM C1293). Results from ASTM C1260 or ASTM C1567 should be evaluated to support a decision about whether to retain jet grouting and soil cement in Phase II, and to make a decision to continue with the long-term testing.
3. Until results from #2 above are available, assume that ASR will not limit selection of the treatment method.
4. In Phase II, RCC, concrete fill and soil cement should be further evaluated as GMAs and jet grouting should be evaluated for excavation support.
5. Include both RCC and soil cement as variations of a single excavate-and-replace method that involves placement and compaction of a no-slump cementitious mix. Concrete fill using imported aggregate, locally excavated $Qbt3_L$volcanic ash, or a combination of these materials placed as a flowable mix constitutes another variant. Unless there is a technical or programmatic reason to eliminate one of these replacement materials, trial mixes of all three variants should be planned for batching and testing as part of the Phase III laboratory pilot testing.
6. RCC and soil cement should be tested during the initial construction lifts to verify mixes and placement practices. Concrete fill would not require field testing.
7. The Phase III field test should be limited to jet grouting, and only to evaluate its use for excavation support in $Qbt3_L$.
8. Begin evaluation of potential ground support systems to be used with excavate-and-replace methods.

7 REFERENCES

American Concrete Institute (ACI), 1997, State-of-the-Art Report on Soil Cement, ACI 230.1R-90 (Reapproved 1997) ACI Committee 230 Report, American Concrete Institute, Detroit, Michigan.

Baker, W. (Ed.), 1982, “Planning and Performing Structural Chemical Grouting.” Proceedings of the Conference on Grouting and Geotechnical Engineering, ASCE, pp. 515-539.

Hayward Baker, 2004, "Ground Improvement Solution Chart." http://www.haywardbaker.com/docs/HB-Chart_Final3.pdf (accessed December 14, 2009).

Kleinfelder, 2007a, “Geotechnical Data Report Chemistry and Metallurgy Research Facility Replacement (CMRR) Project Los Alamos National Laboratory,” Rev. 0, May 2007.

Kleinfelder, 2007b, “Geotechnical Engineering Report Chemistry and Metallurgy Research Facility Replacement (CMRR) Project Los Alamos National Laboratory,” Rev. 0, May 2007.

Kleinfelder, 2009, “Work Plan Ground Modification Feasibility Study Chemistry and Metallurgy Research Replacement Nuclear Facility Los Alamos National Laboratory Los Alamos, NM, Work Plan,” CMRR Document Number: 100320-DSD-00020, DCN Number: 101492.5.1-ALB09WP001, Revision: 0, November 2009

Los Alamos National Laboratory, 2009, CMR Replacement (CMRR) Nuclear Facility, CONSTRUCTION ACCESS AND LIMITS OF EXCAVATION PLAN. Drawing No. C-54634, Rev. F, Sheet CS-50, sheet number 2 of 8. Drawing prepared by Sargent & Lundy for Los Alamos National Laboratory. Los Alamos, NM.

Mertz, Greg, et. al., 2009, “Sensitivity of CMRR of Seismic Response to Soil Improvement,” LANL D5:09-053 Rev. 1, Sept. 2, 2009.

Y-12 National Security Complex, 2005, Seven Day Requirement for the Mass Concrete Fill for the Building 9720-82 Project, Report RP-ST-972082-A005, Prepared by the Building 9720-82 Project Team, Y-12 National Security Complex, Oak Ridge, Tennessee.

TABLES

Table 1

Ground Modification Target Performance Objectives

Target performance objectives of ground modification are the physical property values and the static and dynamic behaviors that the modified $Qbt3_L$ should display to be able to perform as required under the design loads and seismic ground motions. Performance objectives also include the effects on adjacent structures and facilities that must be considered. Examples of performance objectives, to be further developed and refined in later phases of the ground modification program or other elements of work, include:

- Minimum, strain-compatible (earthquake-level strain) shear wave velocity of 4,000 fps (average minus one standard deviation of tested values).
- Minimum allowable static bearing capacity of 10 kips per square foot. This assumes acceptable factor of safety relative to the ultimate bearing capacity and the associated deflection of the foundation element in question does not exceed criteria established by others.
- Minimum allowable static plus dynamic (e.g., seismic or wind) overturning bearing capacity of 20 kips per square foot. This assumes acceptable factor of safety relative to the ultimate bearing capacity and the associated deflection of the foundation element in question does not exceed criteria established by others.
- Static and dynamic total settlement of the base mat (compression of the surface of the foundation subgrade) does not exceed criteria established by others for any location anywhere over the footprint area of the building. Static and dynamic differential settlement (angular distortion) of the base mat (relative vertical displacement between two points divided by the horizontal distance between those same two points) does not exceed criteria established by others for any location over the footprint area of the building.
- Elimination of the potential for hydro-collapse over the footprint area of the CMRR building.
- Shear resistance capacity of at least 200,000 kips at the base mat/foundation subgrade interface.
- Absolute static deflection (combined horizontal and vertical) of ground equal to or less than 0.10 inches within 20 horizontal ft of the southern building face of the PF-4 facility and the western building face of Radiological Laboratory Utility Office Building (RLUOB). This will be considered during selection and design of excavation support for the overall CMRR foundation excavation.
- Absolute dynamic deflection (combined horizontal and vertical) of ground equal to or less than 0.50 inches within 20 horizontal ft of the southern building face of PF-4 and the western building face of RLUOB. This will be considered during selection and design of excavation support for the overall CMRR foundation excavation.
- Absolute static deflection (combined horizontal and vertical) equal to or less than 1.00 inches at and north of the southernmost Perimeter Intrusion Detection, Assessment, and Delay System (PIDADS) fence. This will be considered during selection and design of excavation support for the overall CMRR foundation excavation.
- Absolute dynamic deflection (combined horizontal and vertical) equal to or less then 2.00 inches at and north of the southernmost PIDADS fence. This will be considered during selection and design of excavation support for the overall CMRR foundation excavation.

LEVEL 1 PASS/FAIL SCREENING FOR GROUND MODIFICATION ALTERNATIVES

ABILITY TO PROTECT PF-4 AND SURROUNDING FACILITIES						ABILITY TO MEET MINIMUM MATERIAL PROPERTIES				
REQUIRES XCAVATION BELOW OUNDATION ADE? (IF YES, NSWER 3)	1) ABLE TO PREVENT/LIMIT SETTLEMENTS OF PF-4?	2) ABLE TO KEEP VIBRATIONS AT PF-4 WITHIN ACCEPTABLE LIMITS?	3) CAN EXCAVATION BELOW FOUNDATION GRADE BE SUPPORTED SUFFICIENTLY TO PROTECT PF-4?	ADEQUATE PROTECTION	COMMENTS	4) IS SHEAR WAVE VELOCITY (V_s) OF 4,000 FT/SEC REASONABLY ACHIEVABLE?	5) CAN RELATIVELY UNIFORM PROPERTIES BE CONSTRUCTED?	6) CAN A METHODOLOGY BE DEVELOPED FOR VERIFICATION OF Vs > 4,000 FT/SEC?	PERFORMANCE OBJECTIVES	
				PASS/FAIL					PASS/FAIL	
NO	YES	YES	N/A	PASS	Can apply from foundation grade. Ground vibrations not larger than those from RLUOB construction or CMRR lay-down area excavation.	YES	YES	YES	PASS	Mul may met ASF
NO	YES	YES	N/A	PASS	Can apply from foundation grade. Ground vibrations not larger than those from RLUOB construction or CMRR lay-down area excavation.	YES	YES	YES	PASS	Mul may met ASF
NO	YES	YES	N/A	PASS	Can apply from foundation grade. Ground vibrations not larger than those from RLUOB construction or CMRR lay-down area excavation.	YES	YES	YES	PASS	Mul may met ASF
NO	YES	YES	N/A	PASS	Can apply from foundation grade. Ground vibrations not larger than those from RLUOB construction or CMRR lay-down area excavation.	YES	NO	YES	FAIL	Only (ie < wou of a
NO	YES	YES	N/A	PASS	Can apply from foundation grade. Ground vibrations not larger than those from RLUOB construction or CMRR lay-down area excavation.	YES	NO	YES	FAIL	Only (ie < wou of a
YES	YES	YES	YES	PASS	Requires excavation to Qbt2. Ground vibrations not larger than those from RLUOB construction or CMRR lay-down area excavation.	YES	YES	YES	PASS	Prov all t
YES	YES	YES	YES	PASS	Requires excavation to Qbt2. Ground vibrations not larger than those from RLUOB construction or CMRR lay-down area excavation.	NO	YES	YES	FAIL	Will stre
YES	YES	YES	YES	PASS	Requires excavation to Qbt2. Ground vibrations not larger than those from RLUOB construction or CMRR lay-down area excavation.	YES	YES	YES	PASS	Prov all t
YES	YES	YES	YES	PASS	Requires excavation to Qbt2. Ground vibrations not larger than those from RLUOB construction or CMRR lay-down area excavation.	YES	YES	YES	PASS	Prov all t
NO	YES	YES	N/A	PASS	Can apply from foundation grade. Ground vibrations not larger than those from RLUOB construction or CMRR lay-down area excavation.	YES	NO	NO	FAIL	Unif que

g facilities, the questions for preventing/limiting settlements and keeping vibrations to within acceptable limits must be answered in the affirmative (yes).
aterial properties, all questions relating minimum shear wave velocity, uniformity, and verification methodology must be answered in the affirmative (yes).

Table 3
LEVEL 2 EVALUATION FOR GROUND MODIFICATION ALTERNATIVES

		RELATIVE EVALUATION					
GROUND MODIFICATION CATEGORY	GROUND MODIFICATION ALTERNATIVE (GMA)	Performance of the $Qbt3_L$ Block under CMRR			Constructability	Cost ***	Schedule
		Materials Properties** (including shear wave velocity)	**Variability** of the Result of the Ground Modification Treatment	**Verification** of the Ground Modification Treatment			
IN-SITU METHODS	JET GROUTING	High	High	Medium	Medium	Low	Low
	DEEP MIXING METHOD	Medium	Medium	Medium	Medium	Medium	High
	PERMEATION GROUTING	Low	Low	Medium	High	Low	High
EXCAVATE-AND-REPLACE METHODS	ROLLER-COMPACTED CONCRETE	High	High	High	High	High	Low
	LEAN CONCRETE FILL	High	High	High	High	High	Low
	SOIL CEMENT	Medium	High	High	Medium	High	Low

Suitability Evaluation Key		Description					
High		Highly likely to achieve desired properties with this method.	Very likely to have uniform properties.	Verification can be performed during construction by direct testing methods.	Can be constructed with equipment and resources also needed for other CMRR construction. GMA-specific equipment is readily available. Effective to required depths and within available space.	Best combination of volume of excavation, volume of spoiled material, mobilization cost and unit price per treated volume.	Able to complete in one construction season. Minimal impacts from inclement weather. Minimal excavation support required. Continuous, multi-rig operations.
Medium		Likely to achieve desired properties with this method but might have to be followed by a secondary method.	Likely to have treated masses with uniform properties separated by interfaces.	Verification can be performed after construction by indirect methods and testing on extracted samples.	GMA-specific equipment from remote locations needed. Lower efficiency at required depths and within available space. Shares some equipment and resources needed for other CMRR construction.	Intermediate combination of volume of excavation, volume of spoiled material, mobilization cost and unit price per treated volume.	Able to complete in one construction season. Minimal impacts from inclement weather. Substantial excavation support required. Continuous, multi-rig operations.
Low		Likely to achieve desired properties only with multiple passes or if combined with another method.	Likely to have treated masses with uniform properties separated by untreated zones.	Verification can be performed after construction by indirect methods only.	Little or no precedent to follow. Requires specialty equipment not shared by other CMRR construction. Limits of space and depth limits might be exceeded. Ground conditions may impede progress.	Poorest combination of volume of excavation, volume of spoiled material, mobilization cost and unit price per treated volume.	Completion in one construction season is uncertain. Potential down-time from inclement weather. Substantial excavation support required.

High, medium, and low are used for comparison between methods and do not imply absolute suitability

** ASR not considered

*** The estimated cost of excavation support below foundation grade is included.

LEVEL 2 CONSTRUCTION COST ESTIMATES FOR GROUND MODIFICATION ALTERNATIVES

	Excavation (1, 2, 3)				Replacement (3)				In-Situ Treatment				TOTAL ESTIMATED COST $ (6, 8)	CONTINGENCY % (4)	C
me of vation cy	Mob/ Demob $ (5)	Unit Price $/cy	Excavation Cost $	Excavation Support $ (7)	Volume of Replacement cy	Mob/ Demob $ (5)	Unit Price $/cy	Replacement Cost $	Volume of Treated Ground cy	Mob/Demob $ (5)	Unit Price $/cy	Treatment Cost $			
									$271,000	$100,000	$300	$81,400,000	$82,000,000	20	
									$271,000	$400,000	$180	$49,180,000	$50,000,000	25	
									$271,000	$60,000	$240	$65,100,000	$66,000,000	30	
1,000	$100,000	$15	$4,165,000	$12,213,200	$271,000	$50,000	$95	$25,795,000					$43,000,000	10	
1,000	$100,000	$15	$4,165,000	$12,213,200	$271,000	$50,000	$115	$31,215,000					$48,000,000	10	
1,000	$100,000	$8	$2,268,000	$12,213,200	$271,000	$50,000	$65	$17,665,000					$33,000,000	15	

are not included in this estimate. Excavation over area 343 ft x 381 ft and from base map elevation to top of Qbt2 (56 ft) is included where applicable.
qual for all excavation scenarios.

osts for Phase I evaluation.

nveyor (500 miles).

jacent to CMRR.

iles).

ot3$_L$ exposed surface (1450 ft x 56 ft) at $150/sf plus $50,000 mobilization.

APPENDIX A

Jet Grouting

APPENDIX A
JET GROUTING METHOD

A.1 Description of Method

Jet grouting is an in-situ ground modification method that involves the creation of grouted columns through high pressure injection of grout slurry to erode and mix the soil with grout [Rowe (2001) and Hayward Baker (2004a)]. Jet grouting applies to nearly all soil types with multiple placement geometries in limited workspace. Specific subsurface soil layers can be targeted and treated. Jet grouting does not induce ground vibrations.

In the jet grouting method, a borehole (typically 6 inches in diameter) is drilled to the depth of the target treatment zone. High pressure grout slurry is then applied at the bottom of the borehole through horizontal radial nozzles to erode the subsurface soil and to mix the material with grout. High pressure air or water may also be used in addition to the grout slurry to aid in cutting the subsurface material. The jet grouting system is rotated and lifted to achieve a cylindrical geometry.

The jet grouting technique can be used to create columns of strengthened material at a regular gridded interval. Continuous panels can also be created from the columns, or the columns may be placed adjacent to each other so that a large cell pattern is formed. The generalized jet grouting methodology is shown in Figure A-1.

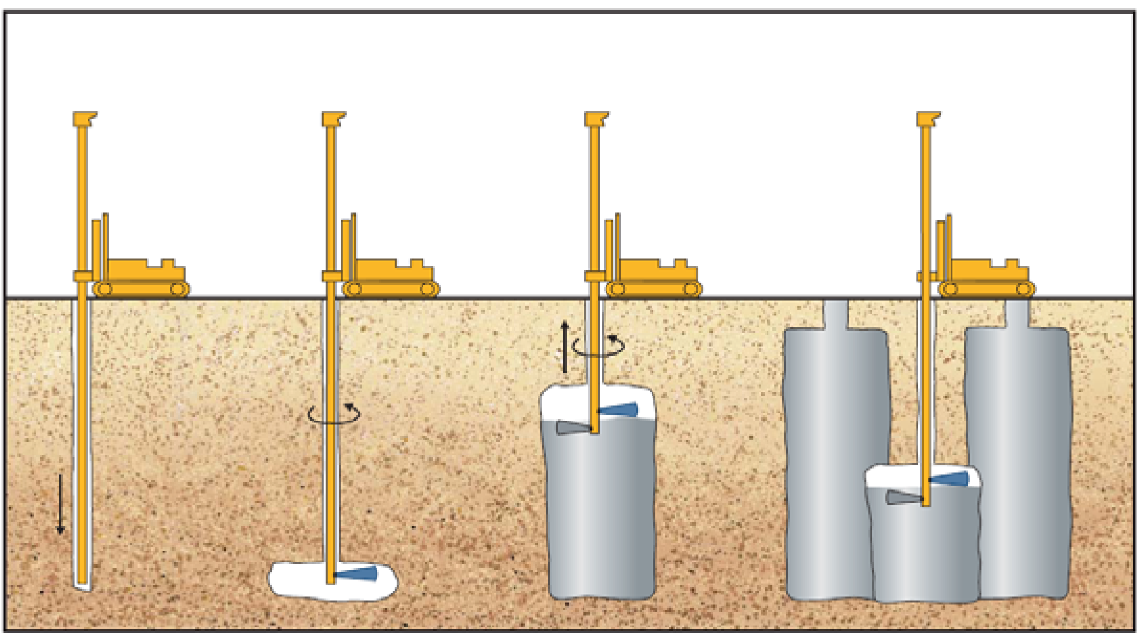

Figure A-1. General Jet Grouting Methodology (Hayward Baker, 2004a)

Several jet grouting systems (Figure A-2) are available for the jet grouting method:

- Single Fluid Jet Grouting – In this jet grouting technique, a single fluid consisting of cement grout is injected at high pressures through horizontal radial nozzles to cut and mix with the in-situ soil. A typical column diameter of 2 to 4 ft (0.6 to 1.2 m) can be achieved with this method.

- Double Fluid Jet Grouting – Cement grout is injected at high pressures through horizontal radial nozzles to cut and mix with the in-situ soil. The cutting process is increased by using a second fluid (water or air) from a secondary horizontal radial nozzle to erode the in-situ soil. A typical column diameter of 3 to 6 ft (0.9 to 1.8 m) can be achieved with this method.

- Triple Fluid Jet Grouting – The cutting process is achieved by injecting water at high pressures through horizontal radial nozzles to cut the in-situ soil. The cutting process is increased by injecting air from a secondary horizontal radial nozzle to erode the in-situ soil. A third horizontal radial nozzle placed lower on the jet grouting system injects cement grout at a lower pressure which achieves mixing of the cut soil with the cement grout. A typical column diameter of 3 to 8 ft (0.9 to 1.8 m) can be achieved with this method.

- Super Jet Grouting – Cement grout and air are injected at high pressures through horizontal radial nozzles to cut and mix with the in-situ soil. The nozzles are specially designed to deliver a highly focused stream of grout and air. Very slow rotation and lift is used to create the column. A typical column diameter of 10 ft (3.3 m) to 14 ft (4.3 m) can be achieved with this method.

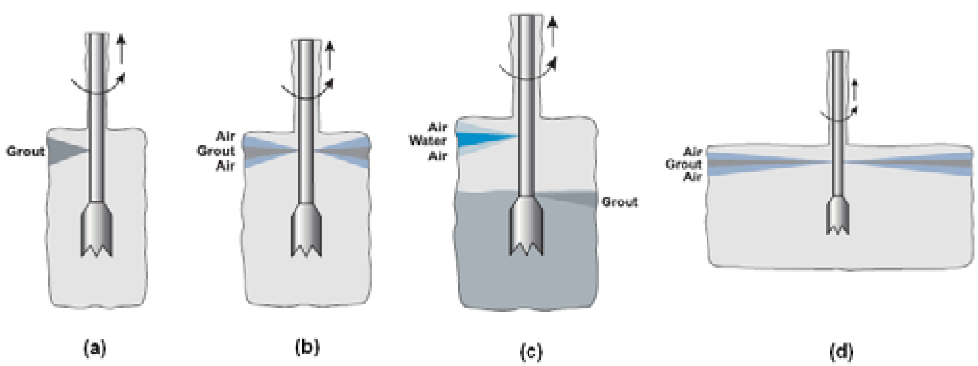

Figure A-2. Jet Grouting Systems: (a) Single Fluid, (b) Double Fluid, (c) Triple Fluid, and (d) Super Jet (Hayward Baker, 2004a)

A.2 Typical Results

According to Rowe (2001), the jet grouting soil improvement method is suitable for most soil types from fine-grained soils to gravels; however, highly plastic clays are generally difficult to jet grout, as are coarse gravels.

A general estimate of strength of jet-grouted soil according to Hayward Baker (2004a) is shown in the following Figure A-3.

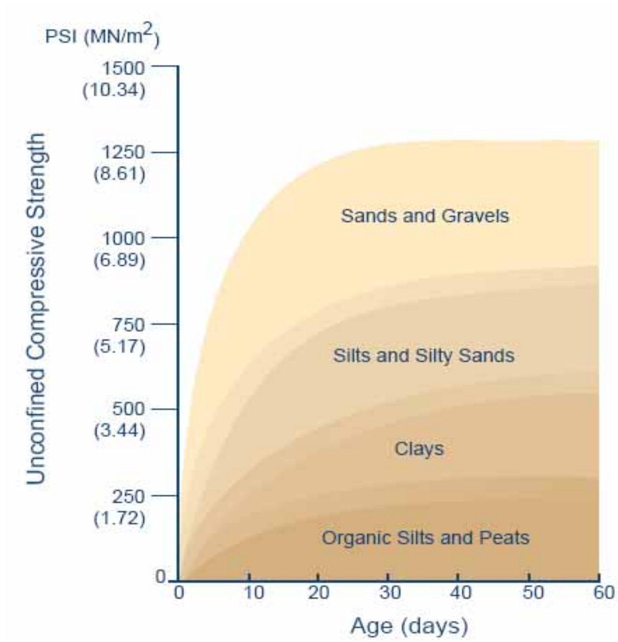

Figure A-3. Unconfined Compressive Strength for Jet Grouted Soils (Hayward Baker 2004a)

According to Hayward Baker (2004b), a general unconfined compressive strength of 100 to 1,000 psi (0.7 MPa to 6.9 MPa) can be achieved for jet-grouted soils. Unconfined compressive strength depends on soil type. A maximum unconfined compressive strength of 1,250 psi (8.6 MPa) is achievable for jet-grouted sands and gravels (Hayward Baker (2004a). Borden and Byle (1995) report compressive strengths ranging from 300 to 1,500 psi (2.1 to 10.3 MPa). However, other sources have reported higher compressive strengths. Bell and Burke (1994) state that the compressive strength for triple-jet grouted soils in both the UK and the US ranges from 145 to 4,350 psi with average unit weights of jet-grouted soil ranging from 123 to 130 pcf. Remedial Construction Services (2009) provide a general unconfined compressive strength estimate of up to 4,000 psi (27.6 MPa) for jet-grouted columns. They also point out that the compressive strength as determined through laboratory testing on core samples is less than the compressive strength determined through *in situ* testing.

Mechanical properties of jet grouted soil (soilcrete) were determined for samples obtained from the Nankang Line tunnel of the Taipei rapid transit project in Taipei, Taiwan (Fang et. al., 1994). The unconfined compressive strength of the grout-treated silty sand ranged from 730 to 3,000 psi (5 MPa to 20.7 MPa); the maximum unconfined compressive strength value represents a 50% increase from the unconfined compressive strength of the untreated silty sand. The large range in unconfined compressive strengths may be due to non-uniform mixing of the soil with grout during the jet grouting process. The axial failure strain of the grout-treated silty sand ranged from 0.3 to 0.62%. The modulus of elasticity at 50% of the unconfined compressive strength ($E_{t,50}$) ranged from 320 to 1,030 ksi (2.2 GPa to 7.1 GPa). The Poisson's ratio (v_{50}) ranged from 0.14 to 0.28. The P-wave velocity of the silty sand ranged from 6,100 to 8,000 fps (1,900 m/s to 2,400 m/s). The mechanical properties of the silty sand determined during this study increased with increasing dry density of the subsurface soils.

A.3 Verification Procedures

Verification methods include testing wet samples of grout injected into the ground (Bell and Burke, 1994), testing core samples, and in situ methods. Wet samples are obtained from a piston sampler. Test cylinders or cubes are then cast from the wet sample and can be tested after they are cured. This method does not work well for gravelly soils.
Core samples or samples obtained from test pits are tested for compressive strength and to determine the amount of voids or non-homogeneous material in the treated zone. In situ methods include load tests to evaluate the capacity of the grouted columns and seismic down-hole or cross-hole methods. In addition to assessing strength and deformability, laboratory permeability tests or borehole permeability tests determine the hydraulic conductivity of the grouted soil.

A.4 Equipment and Logistical Requirements

Jet grouting equipment depends on the system of jet grouting to be used and the geometry for the ground modification. Drill rig support depends on the jet grouting method employed and the depth of application. A minimum of 600 square ft for batching and pump set-up is needed (Hayward Baker, 2004b).

A.5 Costs

Costs depend on equipment mobilization costs for the site location and the nature of the subsurface emplacement. Hayward Baker (2004b) estimates unit costs from \$100 to \$300 per yd^3. Mobilization, site preparation and demobilization costs range from \$30,000 to \$50,000. Remedial Construction Services (2009) estimate unit costs from \$200 to \$400 per yd^3.

A.6 Rate of Advance

Advance rates depend upon the size of the grouted columns. Remedial Construction Services (2009) report an advance rate from 100 to 500 ft (30.5 to 152 m) of column per shift for columns 2 to 4 ft (0.6 to 1.2 m) in diameter over a range of depths of 10 to 150 ft (3.0 to 45.7 m).

A.7 Constraints

Constraints for jet grouting include the potential for soil inclusions with incomplete mixing among the grouted columns. In the case of the Tuttle Creek Dam (Stark et al., 2009), jet grouted columns contained 40 to 50% unmixed soil clods as shown in Figure A-4. These soil inclusions may be caused by material sloughing from the top of the jet grouted column and sinking down to the base of the column through the uncured grout. The presence of soil inclusions within a jet grouted column can decrease the compressive strength and increase the permeability of the grout treated soil.

Figure A-4. Soil Inclusions in Jet Grouted Column (Stark et al., 2009)

The jet grouting process may produce a volume of spoil equal to the injected soil volume (Remedial Construction Services, 2009). This necessitates reclamation for the spoil pile and increases costs depending on the location of the spoil pile.

A.8 Relevant Case Histories

Based on his experience with many jet grouting projects, Burke (2004) states that jet grouting is generally not effective in most stiff clays. Gravels, cobbles, and boulders are also difficult to erode. Burke describes cases when the erosive action of the jet grouting process is repeated (double-cutting) in soils that are difficult to erode. The double-cutting process removes additional soils that have not eroded during the initial soil-cutting process. Burke also describes cases where variable soil conditions have led to variable soilcrete quality, and states the importance of ensuring a uniform jet grouting column geometry by accounting for variable soil conditions.

Jet grouting was used on silty sand soils in the Nankang Line tunnel of the Taipei rapid transit project in Taipei, Taiwan (Fang et al., 1994). A 50% increase in unconfined compressive strength was observed after jet grouting was completed.

In a project at Tuttle Creek Dam near Manhattan, Kansas, jet grouting was performed to create an upstream cutoff wall and to improve the ground under the embankment slopes for seismic stability (Stark et al., 2009). Subsurface conditions consisted of fine-grained alluvial soils from 8 to 25 ft (2.4 to 7.6 m) and fine to coarse sands with some gravel from 25 to 60 ft (7.6 m to 18.3 m). The study found that the jet grouting technique often leads to soil inclusions within the grouted columns. In some instances, soil inclusions were 50 to 60% of the total column volume. Soil inclusions occurred in both clayey and sandy soils.

A.9 References

Bell, A. L. and Burke, G.K., 1994, The Compressive Strength of Ground Treated using Triple System Jet Grouting. Proceedings of the conference organized by the Institution of Civil Engineers and held in London on 25-26 November 1992, Thomas Telford, London.

Borden, R.H. and Byle, M.J. (Eds.), 1995, Verification of Geotechnical Grouting. a report from the ASCE Committee on Grouting of the Geotechnical Engineering Division and papers presented at the ASCE Convention in San Diego, California, October 23-27, 1995. ASCE Publications, New York.

Burke, G.K., 2004. Jet Grouting Systems: Advantages and Disadvantages. Geosupport, 2004: Drilled Shafts, Micropiling, Deep Mixing, Remedial Methods, and Specialty Foundation Systems. Geotechnical Special Publication No. 124, Proceedings of the Conference of American Society of Civil Engineers, held in Orlando, Florida from January 29-31, 2004.

Fang, Y.S., Liao, J.J. and Lin, T.K., 1994, Mechanical Properties of Jet Grouted Soilcrete. Quarterly Journal of Engineering Geology, Vol. 27, Issue 3, pp. 257-2085.

Hayward Baker, 2004a. Jet Grouting.
<http://www.haywardbaker.com/services/jet_grouting.htm> (accessed December 9, 2009).
.
Hayward Baker, 2004b, Ground Improvement Solution Chart."
http://www.haywardbaker.com/docs/HB-Chart_Final3.pdf (accessed December 14, 2009).

Remedial Construction Services (RECON), 2009, E-mail sent on 12/9/2009. Subject: Cost of Jet Grouting and Soil Mixing.

Rowe, R. (Ed.), 2001, Geotechnical and Geoenvironmental Engineering Handbook. Kluwer Academic Publishers, Norwell, Massachusetts, 429-461.

Stark, T., P. Axtell, J. Lewis, J. Dillon, W. Empson, J. Topi, and Walberg, 2009, Soil Inclusions in Jet Grout Columns. *Deep Foundations Institute Journal*, Vol. 3, No. 1, pp. 33-44.

APPENDIX B

Deep Mixing

APPENDIX B
DEEP MIXING METHOD

B.1 Description of Method

The Deep Mixing Method (DMM) is a soil treatment technology that combines cementitious materials in either wet or dry form with native soils. During the 1980s, a number of different deep mixing methods were developed in Japan with unique names. Due to the large number of techniques, a classification system of DMM was developed as shown in Figure B-1 when the use of deep mixing became world-wide. The type of binder (Wet or Dry), the energy of the grout injection (low pressure Rotary, or high pressure Jet) and the mixing principle (all along the Shaft or only at the End), characterize the current methods in use.

Although many variations of these technologies evolved in several countries, the basic method uses rotating shafts that are tipped with a cutting tool (Figure B-2). The shafts are either equipped with discontinuous auger flights or mixing blades or paddles. One to eight shafts can be mounted on each crawler mounted carrier. Column diameters typically range from 2.0 to 4.0 ft (0.6 to 1.5 m) and may extend to 130 ft (40 m) in depth. In some methods, the mixing action is enhanced by simultaneously injecting fluid grout at high pressure through nozzles in the mixing or cutting tools.

DMM can be applied to a broad range of soil conditions including those similar to the subsurface conditions at the CMRR. The Federal Highway Administration (FHA) (2000, Table 3) describes DMM applications in native soils including those described as dense soils or sands.

B.2 Typical Results

Information exists on engineering properties using standard laboratory tests on recovered core and in situ methods. The soil type, amount and type of binder, water cement ratio, degree of mixing and curing conditions affect the engineering properties obtained by DMM treatment. The FHA (2000, Table 3) presents information on the range of cement contents and water cement ratios and the attendant range of unconfined compressive strengths and Young's Modulus. Soils treated with DMM show unconfined compressive strengths of up to 1,450 psi (10 MPa) in granular soils and up to 2,900 psi (20 MPa) in very hard soils. Young's Modulus typically ranges from 150 to 300 times the unconfined compressive strength. Larsson (2005, Figure 4.7) reports shear wave velocities for sands treated with DMM of 4,100 fps (1,250 m/sec).

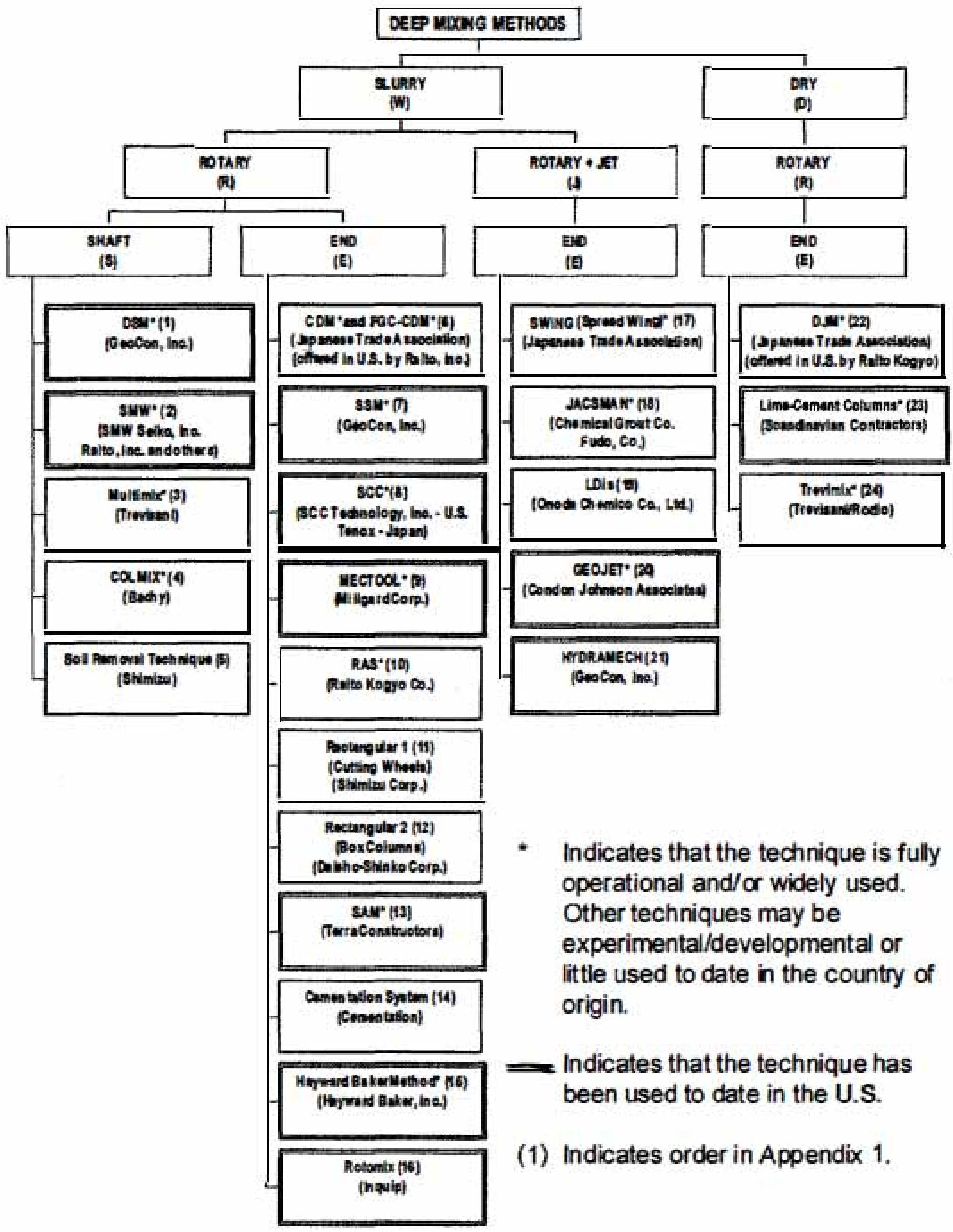

Figure B-1. Classification of DMM (FHA, 2000)

Figure B-2. DMM in Japan (Larsson, 2005)

Figure B-3. DMM in the United States (Larsson, 2005)

B.3 Verification

Many interactive factors influence the engineering properties of interest. The properties of treated ground can be predicted and/or verified by the following broad groups of tests: (1) laboratory testing of laboratory samples (before construction); (2) wet grab sampling of fluid in situ material (during construction); (4) coring of hardened in situ material (after construction); (5) exposure and cutting of block samples (after construction): and standardized tests for the engineering properties of interest are available for tests on recovered core. Excellent recent overviews of Scandinavian verification practice have been provided by Larsson (2009).

B.4 Equipment and Logistical Requirements

There are many variations in the use and combinations of equipment for DMM. The basic equipment layout consists of a large drill rig with rotating shafts with a cutter mounted at the ends of the shaft. Also, the equipment includes a mixing plant and pumps for slurry preparation and slurry injection. For large projects, cement may be stored in cement silos or hoppers. The excavator tractor vehicle supports the rotating shafts. The maximum placement depth is 130 ft (40 m) with larger tractors required for support of longer shaft columns as illustrated in Figure B-4.

Figure B-4. DMM Triple Axis Machine (Bruce and Sills, 2009)

European countries (Larsson, 2009) also developed DMM equipment. The Colmix method involves mixing the soil with a water-based or dry binder by means of a helical tool. The binder is injected as the tool penetrates the soil. Mixing and compaction take place as the tool is withdrawn. The Trevimix method was developed in Italy in the early 1980s and uses both dry and wet binders. Keller and May Gurney have used wet deep mixing since the end of the 1990s.

In Japan, it has also been a common practice to install rectangular columns (Mizutani et al., 1996). The main objective of the use of rectangular columns is to avoid overlap between circular columns.

B.5 Advance Rate

Advance rate depends upon the soil being treated, the equipment employed and depths to which the soil treatment is applied. FHA (2000, Table 3) quotes ranges for production rates for a wet mix using rotary drilling emplaced at the end of the cutter tool from 520 yd^3 per shift to 2000 yd^3 per shift (400 to 1500 m^3 per shift). Advance rates are higher for emplacement methods where mixing occurs along all, or a significant portion, of the drilled shafts. FHA (2000) quotes ranges from 1080 to 1600 ft^2 (100 to 150 m^2 per shift).

B.6 Cost

Costs depend on the equipment being used; the mobilization/demobilization of such equipment; and the cement content used to achieve properties. Costs range from $180 per cy to $300 per yd^3 ($235 to $390 per m^3). Mobilization costs are high due to the use of specialized equipment. Mobilization/demobilization costs for larger DMM systems are typically $80,000 to $200,000 per deep mix unit.

B.7 Constraints

Large power equipment is used to support the rotating shafts and cutting tool that pose logistical problems in small areas. The heavy equipment requires a large space with no overhead restrictions (Figure B-4).

B.8 Relevant Case Histories

A number of domestic and foreign companies use DMM in soils that are comparable to the properties of the $Qbt3_L$ unit for the CMRR. Geo-Con, Inc and SMW Seiko, Inc. commonly use a wet slurry-rotary drilling-shaft method in sands (FHA, 2000, Table 3). SCC Technology, Inc. uses a wet slurry-rotary drilling-end method in sands. Taisei Corporation uses a wet slurry-jet-end method in sandy soils (FHA, 2000).

B.9 References

Bruce, D. and G. Sills, 2009, Seepage Cut-offs for Levees: A Technology Review, Managing Our Water Retention Systems. Proceedings of the 29th Annual Meeting and Conference of the U. S. Society on Dams, Nashville, Tennessee. U.S. Society of Dams, Denver, Colorado.

Federal Highway Administration, 2009, An Introduction to the Deep Mixing Method as Used in Geotechnical Applications, Geosystems, FHWA-RD-99-138, Washington D.C.

Larsson, S, 2005, State of Practice Report Execution, monitoring and quality control, Proceedings of the Deep Mixing Conference.

Mizutani, T., S. Kanai, and M. Fujii, 1996, Assessment of the Quality of Soil-Cement Columns of Square and Rectangular Shaped Formed by a Deep Mixing Method. Proc. of the IS-Tokyo´96, Tokyo, 637-642.

APPENDIX C

Permeation Grouting

APPENDIX C
PERMEATION GROUTING METHOD

C.1 Description of Method

The permeation grouting method involves filling pore space in soil or fissures in rock with grout without causing large displacement of the soil or fracturing of the rock (Borden and Byle, 1995). Permeation grouting can strengthen the soil mass without causing displacement caused by damage to the soil matrix (Warner, 2004). The grout used in permeation grouting is low apparent viscosity and is delivered at relatively low pressures in order to minimize disturbance to the in-situ ground. The permeation grouting method requires mixing of the grout components in a batch or in a continuously metered process and pumping the grout to an injection pipe. The grout is then injected at the desired subgrade depth through an opened ended pipe, such as a drill rod or hollow stem auger, or through sleeve port pipe, which has ports at various intervals through which the grout is delivered.

According to Baker (1982), the potential for permeation grouting correlates to the percent fines in the soil:

- 1 to 2% fines are easily groutable,
- 2 to 20% fines are moderately groutable, and
- 20 to 25% fines are marginally groutable.

Brachman et al. (2004) provides a chart (Figure C-1) that provides guidelines for grouting (Baker, 1982). The chart shows that the grain size distribution for the Qbt3L (Kleinfelder, 2007, Figure V-1) would fall within the groutable range.

Three types of grout can be used in the permeation grouting method: particulate grout (such as aqueous suspensions of cement, fly ash, bentonite, microfine cement, or a combination of materials), colloidal grouts (such as silica gel), or solution grouts (phenolic or acrylic resins) (Kramer, 1996). Colloidal and solution grouts typically exhibit a lower viscosity than particulate grouts and may be used in finer grained material. Byle and Borden (1995) provide a grout type selection process based on grain size and permeability, as shown in Table C-1.

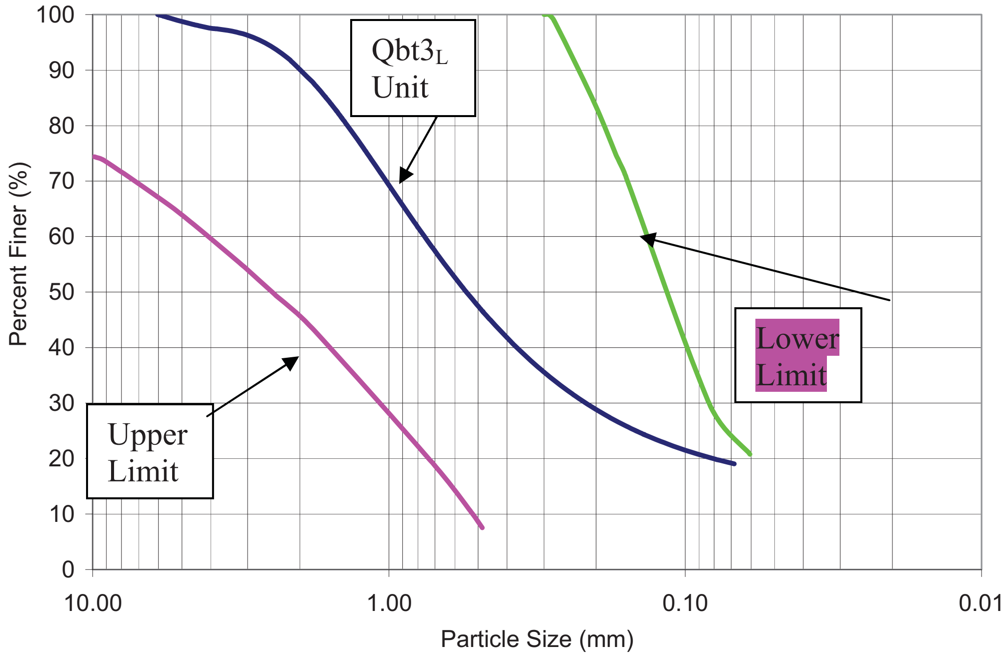

Figure C-1. Qbt3$_L$ Grain Size Distribution Curve to Guidelines for Permeation Grouting (Baker, 1982 and Kleinfelder, 2007).

Table C-1. Field Applications of Grouts for Granular Soils (AFTES, 1991)

GROUT				Strengthening (C) or Watertightening (W)	Normal Field of Application; Limited by Cost						
PARTICULATE	CEMENT			C							
	CLAY-CEMENT			WC							
	GROUT with filler Cellular Grout			WC							
	CLAY GEL BENTONITE (deflocculated, strengthened)			W							
	GROUTS with improved penetration			WC							
COLLOIDAL	SILICATE GEL	Strengthening	concentrated	C							
			low viscosity	C							
		Watertightening	concentrated	W							
			very diluted	W							
SOLUTIONS	RESINS	ACRYLIC		W							
		PHENOLIC		C							
GROUND PROPERTIES	Initial Permeability (k), cm/s				10^{-7}	10^{-6}	10^{-5}	10^{-4}	10^{-3}	10^{-2}	10^{-1}
					Coarse pre-treated alluvial. Fine alluvial (gravels and sand, sands, silty sands)				Coarse grounds scree. Coarse alluvial.		

The hydraulic conductivity of the Qbt3L ranges from 10-2 to10-4 cm/sec. Table C-1 suggests that a particulate grout would be applicable, as opposed to a chemical grout for ground modification. However, the choice of grout also depends on the physical size of the pore spaces within the Qbt3L unit. Chemical permeation grouting may pose hazardous handling issues (such as flammability) or may cause environmental issues based on the toxicity of the chemical used in grouting (Borden and Byle, 1995).

C.2 Typical Results

Permeation grouting reduces the permeability and strengthens the soil. The grout injection rate and pressure must be carefully controlled to prevent hydraulic fracturing of the soil matrix. If fracturing occurs, the grout will fill the fracture and will not permeate the mass of the soil, which will decrease the strength of the overall soil mass (Warner, 2004).

According to Hayward Baker (2004), the Unconfined Compressive Strength (UCS) of silicate permeation grouted soil can range from 50 to 300 psi (0.3 to 2.1 MPa) with a reduction of permeability of 1 x 10-6 cm/s.

Kramer (1996) states that soils improved by permeation grouting can exhibit shear strength of 50 to 300 psi (0.34 to 2.1 MPa) with an estimated UCS from 170 to 1040 psi (1.2 to 7.2 MPa).

Brachman et al. (2004) showed an increase in shear wave velocity for permeation grouted sands from 820 ft/sec to 1,640 ft/sec (250 m/sec to 500 m/sec). Crouthamel and Daemen (1990) showed that permeation grouting reduced the permeability of the fractured tuff at Yucca Mountain from 10-5 to 10-7 cm/s.

The mechanical properties of sodium silicate grouted sand were determined in a laboratory setting by Gonzales and Vipulanandan (2007). The grout used in the study was composed of sodium silicate with 5 to 9% dimethyl ester. The sand used was classified as a poorly graded sand (SP). The test results showed that the compressive strength of the treated sand ranged from 41 to 270 psi (0.3 to 1.9 MPa); the Young's modulus ranged from 29 to 73 ksi (0.2 to 0.50 GPa). The strain at failure ranged from 0.4 to 2%.

A study of UCS on microfine cement grouted sand by Schwarz et al. (2007) provided UCS values for various water/microfine cement ratios and analyzed the effect of pumping rates on UCS. The following UCS values were determined in this study:

- For a 1:1 (by weight) Water Cement Ratio: UCS ranging from 2,500 psi to 2,700 psi.
- For a 2:1 (by weight) Water Cement Ratio: UCS ranging from 600 psi to 1,400 psi.
- For a 4:1 (by weight) Water Cement Ratio: UCS ranging from 250 psi to 720 psi.

C.3 Verification Procedures

According to Borden and Byle (2005), verification methods for permeation grouted materials include quality control testing on grouts during permeation grouting; laboratory testing on recovered core and in situ testing using seismic cross hole or down hole methods or plate loading tests. Direct testing on of the grouted material obtained through coring or test pits includes standards tests for compressive strength and elastic modulus of the material. Hydraulic methods, such as laboratory permeability tests or borehole permeability tests, are used to determine the permeability of the grouted material.

C.4 Equipment and Logistical Requirements

The permeation grouting method requires a small drill rig and approximately 300 ft2 for a batching and pumping station (Hayward Baker, 2004); a larger area would be required to accommodate the equipment required to transport materials to the batching and pumping station, and to transport the grout to the work site. The grout may be injected in different ways. In stage grouting, a boring is advanced a short distance before grout is injected through the end of the drill rod. After the grout sets up, the boring is advanced another short distance and grouted again. This process continues until grout has been placed to the desired depth. In the tube-a-manchette approach, a grout tube with injection ports every 12 to 24 in. along its length is installed in a borehole. Rubber sleeves (manchettes) that serve as one-way valves cover the injection ports on the outer surface the grout tube and internal packer systems are used to control the depths at which grout is injected.

The MaxPerm Grouting System was developed as a reliable ground modification method for soft ground reinforcement. The grouting method applies principles using the Dual-tube Double Packer Grouting (Figure C-2). MaxPerm injection is performed at a very low pressure. It allows smooth improvement directly under existing structures (such as oil tank and bridge pier), as well as improvement of soft ground in general.

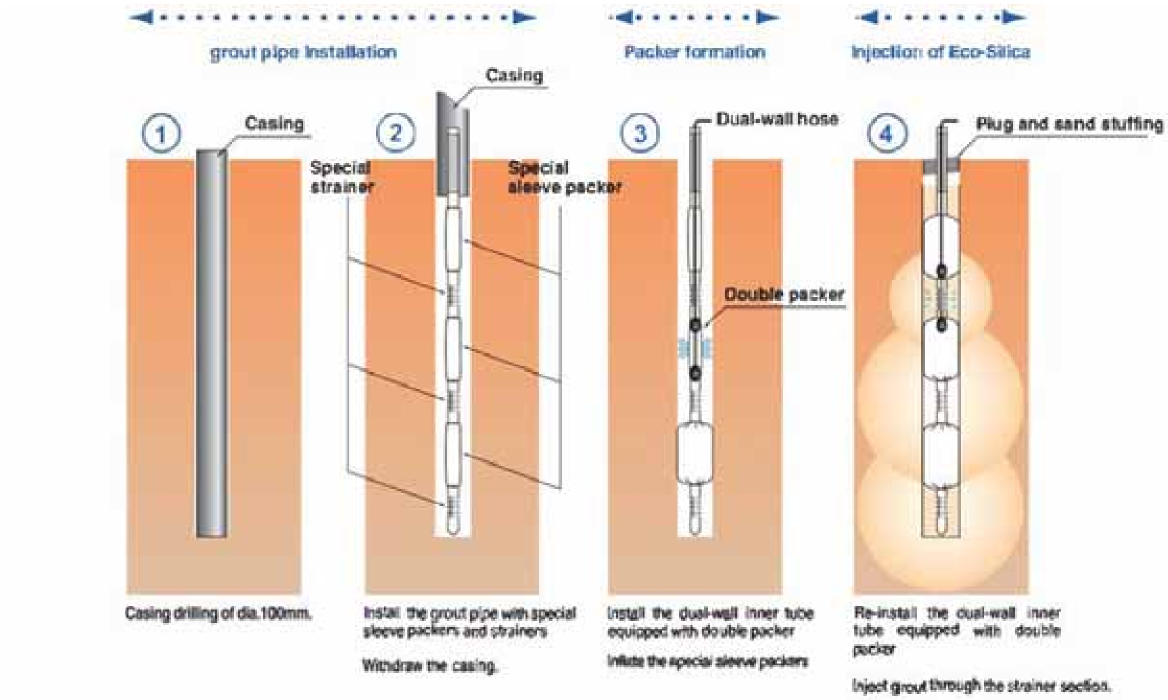

Figure C- 2 Sequence of Permeation Grouting Operations (Raito Kogyo Co, 2010).

C.5 Costs

Estimated material costs for permeation grouting range from $110 to $330 per yd3. Estimated costs for sleeveport pipe installation range from $15 to $50 per linear foot. Estimated costs for mobilization, site preparation and demobilization range from $10,000 to $15,000 (Hayward Baker, 2004).

C.6 Rate of Advance

Grout injection advance rates depend on a number of factors including the capacity, grout viscosity and grouting patterns. Injection rates for the Max Perm (Raito, 2009) permeation grouting system range from 3.9 to 5.2 gpm (15 to 20 liters/min) which is equivalent to 1.2 to 1.6 yd3/hr (0.9 to 1.2 m3/hr) assuming the soil porosity is 50%. Assuming grout holes are spaced 3 ft apart, the linear advance rate equals 220 to 290 ft per day.

C.7 Constraints

Permeation grouting may not increase the UCS of soils as much as other ground modification methods; Hayward Baker (2004) provides an average UCS of 50 to 300 psi for permeation grouted soils, which was confirmed in other case studies researched in this report. However, this is highly dependent on the type of grout used to treat the ground and the water/cement ratio within the grout (Schwarz, 2007).

Chemical permeation grouting may pose hazardous handling issues (such as flammability) or may cause environmental issues based on the toxicity of the chemical used in grouting (Borden and Byle, 1995).

C.8 Relevant Case Histories

Field trials were conducted to evaluate three different permeation grouts in a medium-dense, silty sand outwash deposit in Edmonton, Alberta, Canada (Brachman et al, 2004). Sodium silicate, microfine powder, and microfine cement based grouts were used. Observations from boreholes, a large-diameter vertical shaft, and two drifts indicated (a) the microfine cement based grout did not produce a uniformly grouted mass; (b) the microfine powder grout appeared to permeate the outwash sands, but did not harden one month following injection into the ground. Low temperatures, lack of oxygen in the ground and (or) poor mixing of the two components in the ground likely limited the curing of the grout. The conventional sodium silicate grout successfully permeated the outwash sand deposit, producing a hard material with massive structure. Shear wave velocity from crosshole testing showed improvement from 250 m/sec to 500 m/sec (820 to 1,640 fps).

Permeation grouting in tuff at Yucca Mountain were described by Crouthamel and Daemen (1990). The effectiveness of permeation grouting to reduce the permeability of the tuff was determined by testing two grout mixtures: grout consisting of Portland cement and granular bentonite and another mixture consisting of Microfine cement. Overall, the permeation grouting applications reduced the permeability of the fractured tuff from 10-5 to 10-7 cm/s.

Permeation grouting was used to treat the settlement of the Unit 8 Main Output Transformer at the Ontario Power Generation Pickering B Nuclear Generating Station (Fuller et al., 2007). Treatment was focused on very loose to compact granular fill material consisting of silty sand with gravel to gravelly sand with silt. Permeation grouting successfully treated the settlement issues experienced by the transformer.

C.9 References

Association Francaise des Tunnels et de l'Espace Souterrain (AFTES, French Tunnelling and Underground Space Association), 1991, “Recommendations on Grouting for Underground Works.” Tunneling and Underground Space Technology, Vol. 6, No. 4, pp. 383-461.

Baker, W.H. (Ed.), 1982, Planning and Performing Structural Chemical Grouting. Proceedings of the Conference on Grouting and Geotechnical Engineering, ASCE, pp. 515-539.

Borden, R.H. and M.J. Byle (Eds.), 1995, Verification of Geotechnical Grouting. Report from the ASCE Committee on Grouting of the Geotechnical Engineering Division and papers presented at the ASCE Convention in San Diego, California, October 23-27, 1995. ASCE Publications, New York.

Brachman, R.W., C.D. Martin, and S.A. Gilliss, 2004, Grout Field Trials in Outwash Sands. Canadian Geotechnical Journal, Vol. 41, NRC Canada, pp. 1-11.

Crouthamel, D. and J. Daemen, 1990, Pressurized Grout Applications in Fractured Tuff for Containment of Radioactive Wastes. Geotechnical and Geological Engineering, Vol. 9, Iss. 1, pp. 53-62.

Fuller, A., J. Westland, and P. Blakita, 2007, Grouting Beneath a Main Output Transformer at the Pickering Nuclear Generation Station. Proceedings from the GeoDenver 2007 Conference, Grouting for Ground Improvement Session.

Gonzales, H.A. and C. Vipulanandan, 2007, Behavior of Sodium Silicate Grouted Sand. Proceedings from the GeoDenver 2007 Conference, Grouting for Ground Improvement Session.

Hayward Baker, 2004, Ground Improvement Solution Chart. http://www.haywardbaker.com/docs/HB-Chart_Final3.pdf (accessed December 14, 2009).

Kleinfelder, 2007, Geotechnical Engineering Report, Chemistry and Metallurgy Research Facility Replacement (CMRR) Project, Los Alamos National Laboratory. Kleinfelder Project No. 19435, Rev. 0, Kleinfelder, Albuquerque, New Mexico.

Kramer, S., 1996, Geotechnical Earthquake Engineering. Prentice Hall, Upper Saddle River, New Jersey, pp. 518-519.

Raito Kogyo Co., 2010, Max Perm Grouting System, Brochure Raito Kogyo Co., Tokyo Japan http://www.raito.co.jp/english/construction/pdf/maxperm.pdf

Schwarz, L.G. and M. Chirumalla, 2007, Effect of Injection Pressure on Permeability and Strength of Microfine Cement Grouted Sand. Proceedings from the GeoDenver 2007 Conference, Grouting for Ground Improvement Session.

Warner, J. 2004, Practical Handbook of Grouting: Soil, Rock and Structures. John Wiley and Sons, Inc., Hoboken, New Jersey, pp. 16-19.

APPENDIX D

Roller-Compacted Concrete

APPENDIX D
ROLLER-COMPACTED CONCRETE

D.1 Description of Method

The American Concrete Institute (ACI) 116R-00 defines Roller-Compacted Concrete (RCC) as "concrete compacted by roller compaction; concrete that, in its unhardened state, will support a roller while being compacted." Properties of hardened RCC can be similar to those of conventionally placed concrete. However, RCC can also be made with hardened properties that are outside the range of typical properties of conventionally placed concrete. The ACI also defines "roller compaction" as "a process for compacting concrete using a roller, often a vibrating roller."

Roller compacted concrete typically has the following constituents:

- Cement. Type II Portland cement is more commonly used with RCC because of its low heat of hydration generation characteristics at early ages and its longer set times;
- Pozzolan or ground slag. This constituent may be especially beneficial in RCC as a mineral filler and for its cementitious properties, as well as providing a degree of lubrication during compaction; and
- Coarse and fine aggregate.

Pozzolan typically occupies some of the paste volume otherwise occupied by cement and water. Class F fly ash is most commonly used as a pozzolan or mineral filler for RCC, but Class C fly ash has also been used. Class F fly ash contributes to a lower heat of hydration generation at early ages and may be used to replace cement (generally up to approximately 50% by volume). Class F fly ash reduces cost and acts as a mineral filler to improve workability and delay in the final set. Laboratory testing should be conducted to verify and evaluate the benefits of using pozzolans.

D.2 Typical Results

RCC strength depends upon the quality and grading of the aggregate, mixture proportions, and degree of compaction. Mixture proportioning controls strength serviceability for RCC. Methods for mixture proportioning differ significantly due to the location and design requirements of individual structures.

RCC structures are generally unreinforced and must rely on the concrete strength in compression, shear and tension to resist applied loads, as well as internal stresses caused by nonuniform temperatures (gradients). Typical compressive strength and elastic properties range from 1000 psi (6.9 MPa) to as high as 4,000 psi (27.6 MPa) (ACI, 1999, Tables 3.1, 3.2 and 3.5).

Also, the Corps of Engineers (COE) (COE, 2006) provide information on the unconfined compressive strength (UCS) as a function of cement content at various ages for RCC with and without pozzolan. Figure D-1 illustrates the relationship of UCS as a function of cement content. Since RCC is placed in lifts, the COE presents information on tensile strength and the design lift tensile strength (COE, 2006, Tables 4.2 and 4.3).

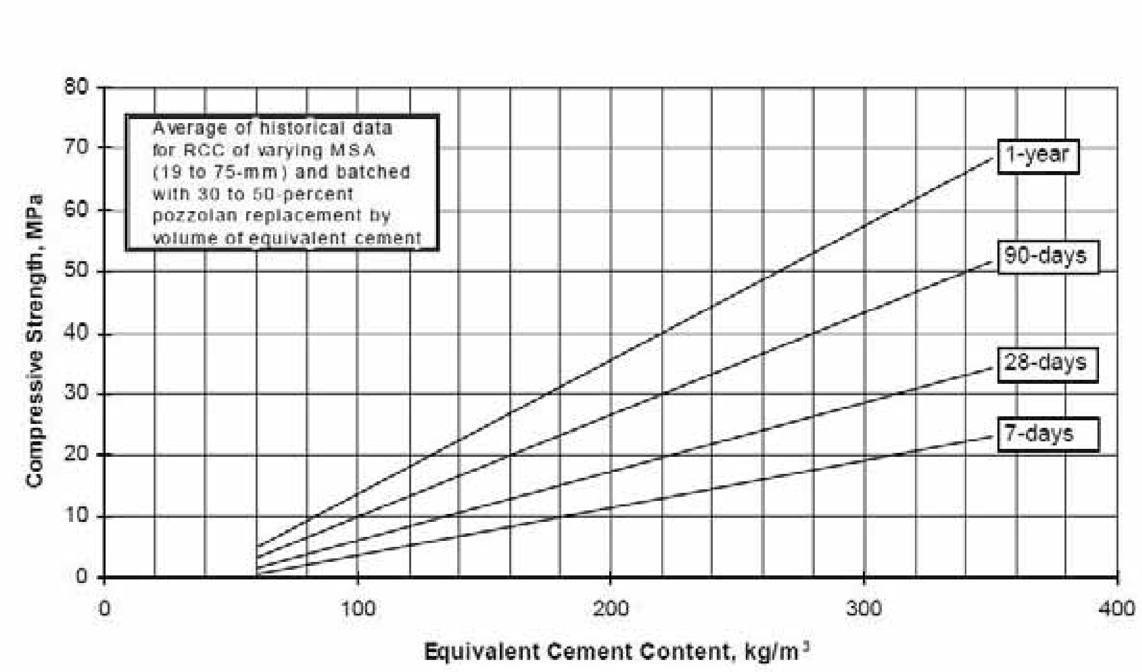

Figure D-1. Equivalent Cement Content Versus Compressive Strength; Average Historical Data for RCC Batched With Pozzolan (COE, 2000)

Basic elastic relationships can be used to estimate the expected shear wave modulus for RCC. Note that EM1110-2-2006 (COE, 2006, Equation 4.2) uses the normal weight concrete relationship between UCS and Young's Modulus for RCC. The elastic methods suggest that higher shear wave velocities can be achieved with a RCC with a higher cement content.

D.3 Verification

Construction quality management policy and guidance are provided in ER 1180-1-6, "Construction Quality Management," and identify the requirements and procedures for Contractor Quality Control and Government Quality Assurance. The manual provides a table for frequency of testing materials.

Verification testing can be divided into three categories. These include quality control tests during RCC placement, standard engineering tests on recovered core or fabricated cylinders and in situ test methods.

The COE (2000, Table 7-1) and ACI (1999, Table 6.1) provide a series of tests, certifications, and frequency of testing for quality control. Manufacturers provide certification on cement, pozzolan, and admixtures. Other testing is performed for the grading of fine and course aggregate. Also, they include tests on RCC for moisture content, wet density, and UCS.

During construction, compressive strength tests on recovered core are performed to verify strength development with time (7, 28, 90, 180, and 365 days). Other ASTM and COE methods are available to test for other properties such as creep tensile strain capacity and volume change. Since cement hydration has the potential to result in thermal cracking of mass concrete, thermal properties are tested.

In situ testing of field density using nuclear gauges is performed and quality control charts for consecutive testing are used (ACI, 1999, Figure 6.1). Cross-hole and down-hole tests can be performed for verification of shear wave velocities.

D.4 Equipment and Logistical Requirements

Based on telephone conversations with local northern New Mexico based contractors (Lafarge and Gears Inc.), a concrete mobile batch plant requires an operating area of 3 to 5 acres. This includes room for a mobile batch plant, material stockpiles, and equipment needed to support the batching process. According to these contractors, the capacity of a single mobile concrete batch plant ranges from 225 to 300 yd^3 per hour.

Fixed conveyors feed RCC from the batch plant to the placement site (COE, 2000). Typical installations include a rotating, retractable conveyor that deposits the RCC on the lift surface via a drop chute. These systems require the addition of more rotating/retracting units to cover large placement areas.

More recent implementations have replaced the rotating/retracting unit with a mobile conveyor. One method is for the RCC supply belt to be installed over the full length of the emplacement area. At desired locations, the RCC is diverted from the belt to a secondary belt feeding a track-mounted rotating/retracting conveyor. This mobile unit is capable of positioning a drop chute at any location on the lift surface as illustrated in Figures D-2 and D-3. This system practically eliminates the need for vehicles to transport RCC on the foundation surface.

Figure D-2. Conveyor System with Self-Propelled Crawler-Placer (COE, 2000)

Figure D-3. Conveyor System with Mobile Side Discharge Belt (COE, 2000)

A sloping layer method (COE, 2000) has been used recently to construct lifts of multiple layers. RCC is placed in layers approximately 8 to 12 inches thick for a total thickness of 10 to 13 ft. Each lift is compacted with a vibrating steel-wheel roller. A variety of vibratory rollers provide adequate compaction of RCC. These compactors range from relatively small and light asphalt rollers used extensively for compaction of RCC in Japan, to heavy single-drum units designed to compact rock fills.

D.5 Advance Rate

Typical production rates may range from 50 to 230 yd^3/hr (35 to 150 m^3/hr) for a small RCC project, 230 to 460 yd^3/hr (150 to 350 m^3/hr) for a moderate-size RCC project and 460 to 1,000 yd^3/hr (350 to 750 m^3/hr) for a large RCC structure (COE, 2000, Section 6.1). At the Elk Creek Dam in southwest Oregon, a maximum rate of 1,000 yd^3/hr (765 m^3/hr) was achieved with an average placement rate of 600 yd^3/hr (450 m^3/hr).

D.6 Cost

RCC costs depend on the cost of aggregate and cementing materials, the complexity of placement and the total quantities of concrete placed. Based on telephone conversations with Los Alamos Transit Mix, a local cement provider, the local cost of cement is approximately $142 per ton. The local price of Class F fly ash ranges from $97 to $100 per ton. Local aggregate ranges from $26 to $32 per ton, with the price dependent on the source of the aggregate. The Portland Cement Association provides information on the cost per cubic yard of RCC (PCA, 2009 and Choi and Hansen, 2005). The information suggests economies of scale for RCC and that RCC costs are from 25 to 50% less than the cost of conventional concrete. Kleinfelder has estimated a placement unit cost of 95/yd^3 for RCC using cost data provided by contractors and in the RSMeans Heavy Construction Cost Data 2010 Manual.

D.7 Constraints

Since cement hydration has the potential to result in thermal cracking of mass concrete, thermal properties are necessary to assess cement hydration effects. ACI 207.1R-96 (ACI, 1996) provides a specification for materials and mix proportioning, the properties of mass concrete and construction methods. Methods available for controlling cement hydration temperatures include the use of a low heat of hydration cement; chilling of aggregate and the circulation of water through cooling pipes. Lift thicknesses are controlled.

D.8 Relevant Case Histories

RCC also has a wide range of applications and throughout its development, somewhat different names have been used for it, such as rollcrete. Historically, RCC has been used as a construction material for concrete dams, embankment protection, pavements and slope protection (Choi and Hansen, 2005). COE (2006) mention use of RCC for a massive open foundation and base slabs.

D.9 References

American Concrete Institute (ACI), 1999, Cement and Concrete Technology, Report by ACI Committee 207, ACI 116R-00, Detroit Michigan.

American Concrete Institute (ACI), 1999, Roller-Compacted Mass Concrete, Report by ACI Committee 207, ACI 207.5R-99, Detroit Michigan.

Choi, Y. and Hansen, K., 2005, RCC/Soil-Cement: What's the Difference?, Journal of Materials in Civil Engineering, ASCE, New York.

Corps of Engineers (COE), 2000, Construction Quality Management, Department of the Army, ER 1180-1-6, U.S. Army Corps of Engineers, Washington, DC.

Corps of Engineers (COE), 2006, Engineering Design Roller-Compacted Concrete Department of the Army EM 1110-2-2006 U.S. Army Corps of Engineers, Washington, DC.

Gears Inc., 2010, Telephone conversation, January 29, 2010.

Lafarge, 2010, Telephone conversation, February 5, 2010.

Los Alamos Transit Mix, 2010, Telephone conversation, January 29, 2010.

Portland Cement Association (PCA), 2009, http://www.cement.org/water/dams_rcc.asp.

RSMeans Heavy Construction Cost Data, 2010, Construction Publishers and Consultants, Kingston, MA, 24th Annual Edition.

APPENDIX E

Concrete Fill

APPENDIX E
CONCRETE FILL

E.1 Description of Method

Concrete fill is a mixture of Portland cement, coarse/fine/ aggregate, and water (ACI, 1997) placed by conventional methods. Due to potential issues of alkali-silica reactivity (ASR), the coarse/fine aggregate used for this alternative consists of an imported coarse/fine aggregate not subject to ASR. For conventional concrete fill, the coarse/fine/aggregate particles are bonded by the cement paste, and completely coated through mixing of the cement, the coarse and fine aggregate, and other additives.

E.2 Typical Results

The ACI publishes a series of specifications covering a number of topics for plain concrete. ACI 207.1 provides a specification for mass concrete that is defined as "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking." Generic information is known (ACI 1997, Table 3.1.3), and the information demonstrates that concrete mix designs can be developed to achieve the minimum shear wave velocity requirement for concrete fill. High confidence exists that the concrete mix design for specified properties of aggregate can be selected with a cement content and water cement ratio to achieve the required elastic and strength properties with proper quality control methods.

The use of concrete fill for the CMRR foundation would require that heat of hydration effects be controlled through a combination of mix design and placement methods. The COE publishes ETL 1110-2-542 that presents three levels of analyses for addressing heat of hydration effects. The ETL discusses using finite element analysis for critical structures where cracking poses a significant risk. To address the potential thermal cracking issue, Level III calculations using finite element or finite difference analysis methods should be used to demonstrate that for cement mix and placement methods selected that heat of hydration effects are controlled for the concrete fill placement.

E.3 Verification

Verification testing can be divided into three categories. These include quality control tests during concrete fill placement, standard engineering tests on recovered core or fabricated cylinders and in situ test methods. The standard engineering tests on recovered core can be performed using standard ASTM methods. In situ cross-hole or down hole methods are available to assess shear-wave velocities.

The COE Manual of Concrete Practice (COE, 1994, Chapter 9) provides quality control for aggregate grading, slump, air content, and concrete temperature which are important for controlling cement hydration, and compressive strength for concrete fills.

As discussed subsequently, the high volume required for the CMRR treatment might require a central mixing plant. Cement control is achieved by weighing the coarse and fine aggregates and cement, and then adjusting weights until the correct amount of cement is being discharged. ACI 221R covers the processing, handling, and quality control of aggregate.

For central-plant-mixed concrete fill, the uniformity is usually checked visually at the mixing plant. It can also be checked at the placement area in a manner similar to the method used for mixed-in-place construction. The mixing time necessary to achieve a uniform mixture will depend on the aggregate gradation and mixing plant used.

E.4 Equipment and Logistical Requirements

The ACI 304R series of specifications discuss the measuring, mixing, transporting, and placing of concrete fill. Mixers can be stationary parts of central mixture plants or of portable plants. Mixers can also be truck mounted. Satisfactorily designed mixers have a blade or fin arrangement and drum shape that ensure an end-to-end exchange of materials parallel to the axis of rotation or a rolling, folding, and spreading movement of the batch over itself as it is being mixed.

Central-mixed concrete is mixed completely in a stationary mixer and then transferred to another piece of equipment for delivery. This transporting equipment can be a ready-mixed truck operating as an agitator, or an open-top truck body with or without an agitator. The tendency of concrete to segregate limits the distance it can be hauled in transporters not equipped with an agitator.

Sometimes the central mixer will partially mix the concrete with the final mixing and transporting being done in a revolving-drum truck mixer. This process is often called “shrink mixing” as it reduces the volume of the as-charged mixture. When using shrink mixing, ASTM C94 limits the volume of concrete charged into the truck to 63% of the drum volume.

Based on telephone conversations with local contractors (Lafarge and Gears Inc.), a concrete mobile batch plant requires an operating area of 3 to 5 acres. This includes room for a mobile batch plant, material stockpiles, and equipment needed to support the batching process.

E.5 Advance Rate

Advance rates depend principally on the number of plants used in batching. Newman and Choo (2003) stated that production rates for batch plants range from 110 yd^3/hr (80 m^3/hr) to 800 yd^3/hr (610 m^3/hr). These rates assume the production feed of cement supports the production rate of the batching plant. Local contractors (Lafarge and Gears Inc.) have provided Kleinfelder with an estimated mobile concrete batch plant capacity of 225 to 300 yd^3 per hour.

E.6 Cost

Based on telephone conversations with Los Alamos Transit Mix, a local cement provider, the local cost of cement is approximately $142 per ton. The local price of Class F fly ash ranges from $97 to $100 per ton. Local aggregate ranges from $26 to $32 per ton, with the price dependent on the source of the aggregate. Kleinfelder has also used cost data provided in the RSMeans Heavy Construction Cost Data 2010 Manual to aid in determining a unit placement cost for concrete fill. Based on this information, Kleinfelder has estimated a placement unit cost of $115/yd^3$.

E.7 Constraints

It is important to achieve uniform vibration, curing, and to control hydration for concrete fill. Cement hydration temperatures are controlled through several methods by controlling lift thicknesses, chilling aggregate prior to placement, and circulating water through pipes after placement. Aggregates can be cooled by evaporation through vacuum, by inundation in cold water, by cold air circulation, or by liquid nitrogen.

E.8 Relevant Case Histories

Numerous case histories exist for the placement of concrete in foundations and major dams in the United States. These include such noteworthy dams as the Grand Coulee Dam, the Hoover Dam, and the more recent Glen Canyon Dam.

E.9 References

American Concrete Institute (ACI), 1997, Mass Concrete ACI 207.1R-96. (Reapproved 1997) ACI Committee 207 Report, American Concrete Institute, Detroit Michigan.

American Concrete Institute (ACI), 1996, Guide for Use of Normal Weight and Heavyweight Aggregates in Concrete, ACI 221R-96, ACI Committee 221 Report, American Concrete Institute, Detroit Michigan.

American Concrete Institute (ACI), 2000, Transporting, and Placing Concrete, ACI 304R-00, ACI Committee 304 Report, American Concrete Institute, Detroit Michigan.

Gears Inc., 2010, Telephone conversation, January 29, 2010.

Lafarge, 2010, Telephone conversation, February 5, 2010.

Los Alamos Transit Mix, 2010, Telephone conversation, January 29, 2010.

Newman, J. and B. Choo, 2003, Advanced Concrete Technology 3, Butterworth-Heinemann, Woburn, MA.

RSMeans Heavy Construction Cost Data, 2010, Construction Publishers and Consultants, Kingston, MA, 24th Annual Edition.

US Army Corps of Engineers (COE), 1994, Standard Practice for Concrete for Civil Works Structures, EM 1110-2-20003, U.S. Army Corps of Engineers, Washington, D.C..

APPENDIX F

Soil Cement

APPENDIX F
SOIL CEMENT

F.1 Description of Method

Soil cement is a densely compacted mixture of Portland cement, soil/aggregate, and water (ACI, 1997). The aggregate used for this alternative consists of the poorly welded tuff of the $Qbt3_L$ unit that has the consistency of medium dense sand. Granular soils are preferred fine aggregate since they more easily pulverize and mix than fine grained soils. For soil cement, the soil/aggregate particles are bonded by the cement paste, but unlike concrete, the individual particles are not completely coated with cement paste. Although the primary use of soil cement is for base materials under pavements, other uses include slope protection for dams and embankments; liners for channels, and mass soil-cement placements for dikes and foundation stabilization.

Soil-Cement mixture proportioning treats the mix as an earthfill, and uses test methods such as the Standard Proctor compactive effort (ASTM D698) (Choi and Hanson, 2005). Soil cements are prepared at or slightly wet of optimum water contents to achieve this target density. Proportioning procedures for soil cements involve mainly changing the cementitious content to satisfy a set of design criteria that include mainly compressive strength, but could include tensile strength, rate of heat generation, and durability.

Pozzolans such as fly ash have been used where the advantages outweigh the disadvantages of storing and handling an extra material. The quantity of cement and pozzolan required should be determined through a laboratory testing program using the specific cement type, pozzolan, and soil to be used in the application.

F.2 Typical Results

The properties of soil cement are influenced by several factors, including (a) type and proportion of soil, cementitious materials, and water content; (b) compaction; (c) uniformity of mixing; (d) curing conditions; and (e) age of the compacted mixture. Because of these factors, a wide range of values for specific properties exists.

ACI (1997, Table 4.1) reports unconfined compressive strengths at 7 and 28 days for soil cement used with granular soils. The strengths ranged from 300 to 600 psi (2.1 to 4.1 MPa) for strength at 7 days. The ACI (1997, Table 4.2) presents unconfined compressive strengths from 400 to 1000 psi (2.8 to 6.9 MPa) at 28 days. Figure F-1 illustrates relationships between the unconfined compressive strength, and cement content by weight (%) for coarse grained soils.

Smadi and Migdady (1991) report on unconfined compressive strength tests as high at 8,700 psi (60 MPa) at 90 days for a concrete consisting of a lightweight tuff aggregate (Figure F-2). The cement content was 500 kg/m^3. Mixture proportioning followed ACI 211 for structural lightweight concrete. Also, the laboratory studies developed correlations of the Young's Modulus and other material properties with unconfined compressive strength similar to those for normal weight concrete.

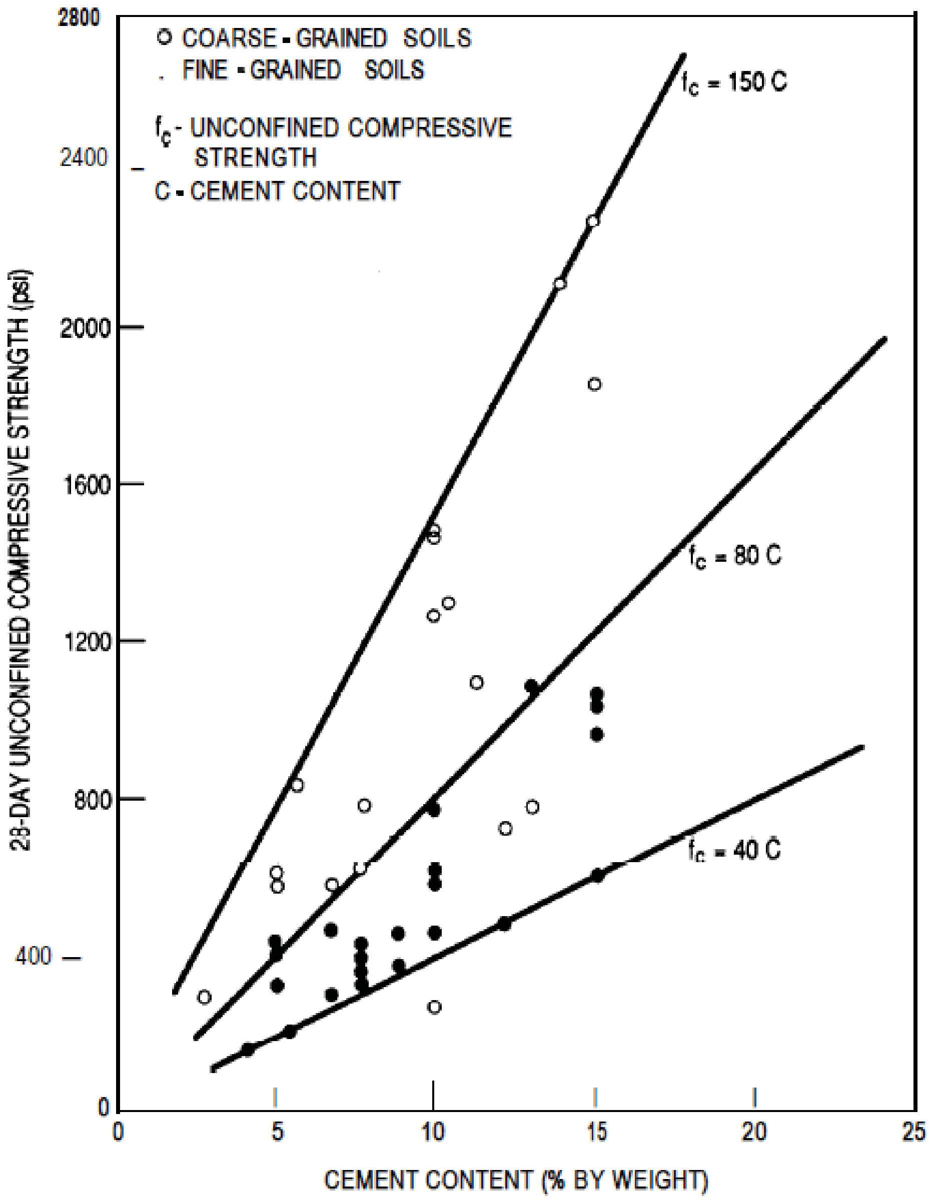

Figure F-1. Relationship Between Cement Content and Unconfined Compressive Strength for Soil-Cement Mixtures (ACI, 1997)

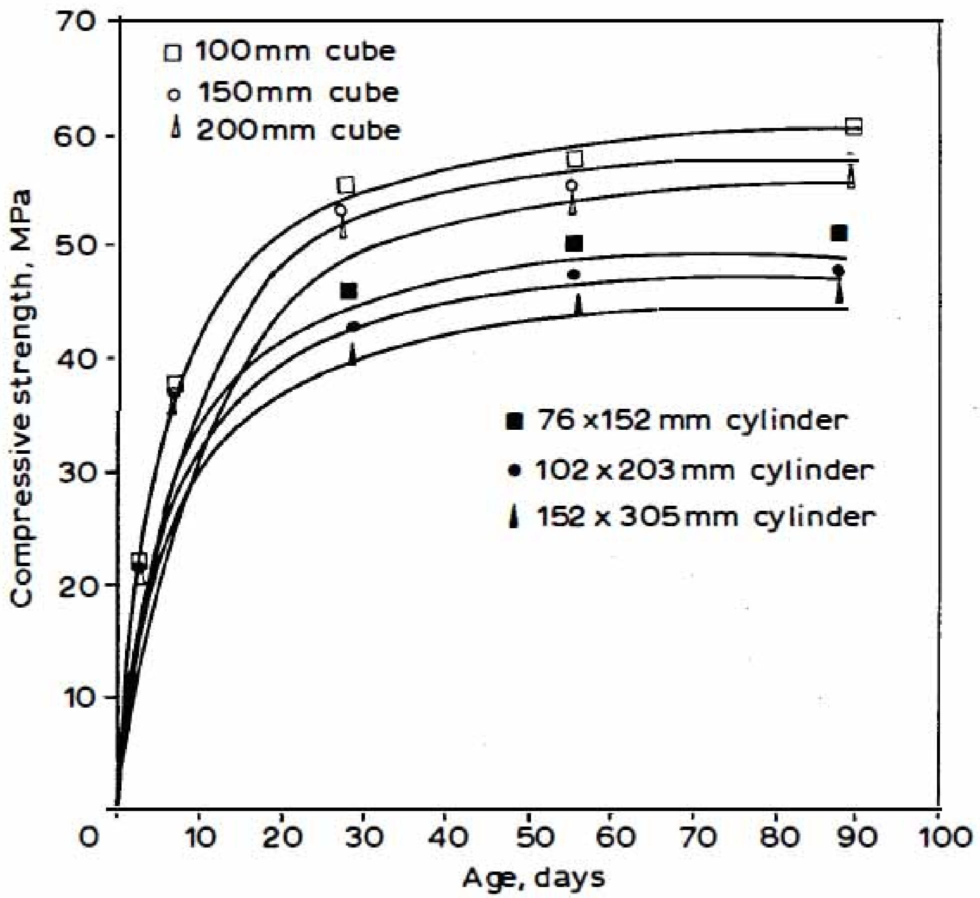

Figure F-2. Size Effect on Strength Gain of High Strength Lightweight Aggregate Concrete Cubes and Cylinders under Moist Curing (Smadi and Migdady, 1991)

F.3 Verification

Verification testing can be divided into three categories. These include quality control tests during RCC placement, standard engineering tests on recovered core or fabricated cylinders and in situ test methods. The standard engineering tests on recovered core can be performed using standard ASTM methods. In situ cross-hole or down-the-hole methods are available to assess shear wave velocities.

ACI 230 (ACI, 1997, Section 7) provides quality control for pulverization and gradation of the fine aggregate, cement content, moisture content, mixing uniformity, compaction, lift thickness , and curing for soil cements. The pulverization tests consist of screening a representative sample of the aggregate with a No. 4 sieve and expressing the dry weight of the retained material to the total dry weight of the material.

As discussed subsequently, the high volume required for the CMRR treatment would require a central mixing plant. Cement control is achieved by weighing the soil and cement, and then adjusting weights until the correct amount of cement is being discharged.

Proper moisture content is necessary for adequate compaction and for hydration of the cement. The proper moisture content of the cement-treated soil is determined by the moisture-density test (ASTM D 558 or D 1557). The optimum moisture content is used as a guide for field control during construction.

For central-plant-mixed soil cement, the uniformity is usually checked visually at the mixing plant. It can also be checked at the placement area in a manner similar to the method used for mixed-in-place construction. The mixing time necessary to achieve an intimate uniform mixture will depend on the soil gradation and mixing plant used. Usually 20 to 30 sec of mixing is required.

Moisture density testing controls compaction for emplaced soil cements. In general, a density requirement ranges from 95 to 100% of the maximum density of the cement-treated soil. The most common standard tests methods for determining in-place density include the nuclear method (ASTM D 2922 and D 3017); the Sand-cone method (ASTM D 1556) and the balloon method (ASTM D 2167).

F.4 Equipment and Logistical Requirements

ACI 230.1R (1997, Section 6) provides a detailed description of equipment and logistical requirements for soil cement. A central mixing plant might consist of a continuous flow pugmill plant. The plant consists of a soil bin or stockpile, a cement silo with a surge hopper, a conveyor belt to deliver the soil and cement to the mixing chambers, and a water storage tank for adding water. The pug mill mixing chamber consists of two parallel shafts equipped with paddles along each shaft. The twin shafts rotate in opposite directions, and the soil cement is moved through the mixer by the pitch of the paddles.

Based on telephone conversations with local contractors (Lafarge and Gears Inc.), a mobile batch plant requires an operating area of 3 to 5 acres. This includes room for a mobile batch plant, material stockpiles, and equipment needed to support the batching process. According to these contractors, the capacity of a single mobile concrete batch plant ranges from 225 to 300 yd^3 per hour.

For large projects, a conveyor system can be used to deliver the soil cement to the spreader eliminating the necessity for ramp construction, and earthmoving equipment.

Motor grader or spreader box attached to a dozer are the most commonly used means for soil cement emplacement. Spreading may also be done with asphalt-type pavers. Some pavers are equipped with one or more tamping bars, which provide initial compaction.

Soil cement is usually placed in a layer 25 to 50% thicker than the final compacted thickness. For example, an 8 to 9 inches loosely placed layer will produce a compacted thickness of about 6 inches. This relationship varies slightly with the type of soil, method of placement and degree of compaction. The actual thickness of the loosely spread layer is determined from contractor experience or trial-and error methods. Compacting, finishing, and curing follow the same procedures as for mixed-in-place construction.

F.5 Advance Rate

Advance rate depends principally on the number of plants used in batching. Reid (2009) stated that production rates for batch plants range from 190 yd^3/hr (140 m^3/hr) to 250 yd^3/hr (190 m^3/hr). These rates assume the production feed of cement supports the production rate of the batching plant. Under more typical situations where the advance would be constrained by the production feed of cement of high content, advance rates might range from 140 yd^3/hr (100 m^3/hr).

F.6 Cost

Costs for placement depend on a number of factors including water availability for soil cement mixing. Mobilization costs for two plants might range from $125,000 to $150,000. Replacement costs range from $10 to $30 per yd^3 under the assumption of Type I-II cement. Based on telephone conversations with Los Alamos Transit Mix, a local cement provider, the local cost of cement is approximately $142 per ton. The local price of class F fly ash ranges from $97 to $100 per ton. Kleinfelder has also used cost data provided in the RSMeans Heavy Construction Cost Data 2010 Manual to aid in determining a unit placement cost for soil cement. Based on this information, Kleinfelder has estimated a placement unit cost of $65/$yd^3$ for soil cement.

F.7 Constraints

Cement-treated soils undergo shrinkage during drying. Soil cement made with granular soils produces less shrinkage than fine grained soils , but larger cracks spaced at greater intervals (usually 10 to 20 ft or more apart) (ACI, 1997, Section 4.6). Methods suggested for reducing or minimizing shrinkage cracks include keeping the soil-cement surface moist beyond the normal curing periods.

F.8 Relevant Case Histories

Soil cement has been used as a massive fill to provide foundation strength and uniform support under large structures (ACI, 1997, Section 2.5). In Koeberg, South Africa, for example, soil cement was used to replace an approximately 18 ft thick layer of medium-dense, liquefiable saturated sand under two 900-MW nuclear power plants. An extensive laboratory testing program was conducted to determine static and dynamic design characteristics, liquefaction potential, and durability of the soil cement. Results showed that with only 5 percent cement content by dry weight, cohesion increased significantly, and it was possible to obtain a material with enough strength to prevent liquefaction.

The largest soil-cement project worldwide involved 1.2 million yd^3 of soil-cement slope protection for a 7000-acre cooling-water reservoir at the South Texas Nuclear Power Plant near Houston. Completed in 1979, the 39 to 52 ft (12 m to 16 m) high embankment was designed to contain a 15 ft high wave action that would be created by hurricane winds. In addition to the 13 miles (21 km) of exterior embankment, nearly 7 miles (11 km) of interior dikes, averaging 27 ft (8.2 m) in height, guide the recirculating cooling water in the reservoir.

F.9 References

American Concrete Institute (ACI), 1997, State-of-the-Art Report on Soil Cement, ACI 230.1R-90 (Reapproved 1997) ACI Committee 230 Report, ACI, Detroit, MI.

American Concrete Institute (ACI), 1981, Recommended Practice for Selecting Proportions for Structural Lightweight Concrete (ACI 211 - 2.81). ACI Committee 211, ACI Manual of Concrete Practice, Part I ACI, Detriot, MI.

Choi, Y. and K. Hansen, 2005, RCC/Soil-Cement: What's the Difference?, Journal of Materials in Civil Engineering, ASCE, New York.

Gears Inc., 2010, Telephone conversation, January 29, 2010.

Lafarge, 2010, Telephone conversation, February 5, 2010.

Los Alamos Transit Mix, 2010, Telephone conversation, January 29, 2010.

Reid, B., 2009, Estimated Costs for Soil Cement, Personal Communication, Las Vegas Nevada.

RSMeans Heavy Construction Cost Data, 2010, Construction Publishers and Consultants, Kingston, MA, 24th Annual Edition.

Smadi, M, and E. Migdady, 1991, Properties of High Strength Tuff Lightweight Aggregate Concrete, Cement and Concrete Composites, Vol. 13, pp. 129-135.
.

APPENDIX G

Volume of $Qbt3_L$ Ground Modification

APPENDIX G
Volume of $Qbt3_L$ Ground Modification

G.1 Introduction

The purpose of this appendix is to determine an approximate volume of ground requiring modification for use in Kleinfelder's Phase I Ground Modification Alternatives Feasibility Study for the CMRR project.

The scope of this appendix is to document the configuration and volume of $Qbt3_L$ that requires ground modification from foundation elevation to the bottom of Unit 3, Bandelier Tuff ($Qbt3_L$). The volume of $Qbt3_L$ that requires ground modification was estimated based on simple geometric relationships and dimensions provided in the design documents by Sargent and Lundy.

G.2 Basis

As determined from drawing number C-54634, Rev. F titled "Construction Access and Limits of Excavation Plan" provided by Sargent & Lundy, the foundation of the proposed CMRR building will be located at a depth of approximately 75 ft below original site grade, which corresponds to an elevation of 7,226 amsl. At this depth, the foundation will bear on or near the contact between the upper ($Qbt3_U$) and lower ($Qbt3_L$) Unit 3 Bandelier Tuff. The "Construction Access and Limits of Excavation Plan" (Drawing Number C-54634, Rev. F) also shows the CMRR foundation footprint of 341 ft long by 303 ft wide. The limits of the excavation are shown on the drawing as 20 ft in each direction beyond the foundation perimeter.

The target zone for ground modification is the volcanic tuff of $Qbt3_L$ between the foundation grade at elevation 7,226 ft amsl and the top of Unit 2 Bandelier Tuff (Qbt2). As shown in a contour plot in Figure VIII-5 of Kleinfelder's 2007 CMRR Geotechnical Engineering Report, the elevation of $Qbt3_L$ generally ranges from 7,172 to 7,169 ft amsl. Thus, the thickness of $Qbt3_L$ generally ranges from 54 to 57 feet.

- It is assumed that the foundation dimensions of the CMRR facility are 341 ft long by 303 ft wide as shown on the "Construction Access and Limits of Excavation Plan" drawing.
- It is assumed that the limits of the excavation are 20 ft in each direction beyond the foundation perimeter.
- It is assumed that the thickness of $Qbt3_L$ is approximately 56 feet with a corresponding $Qbt3_L$ bottom elevation of 7,170 ft amsl.

G.3 Methods and Calculations

The foundation footprint and excavation limits of the CMRR facility used in the volume of $Qbt3_L$ ground modification calculation are shown in Figure G-1.

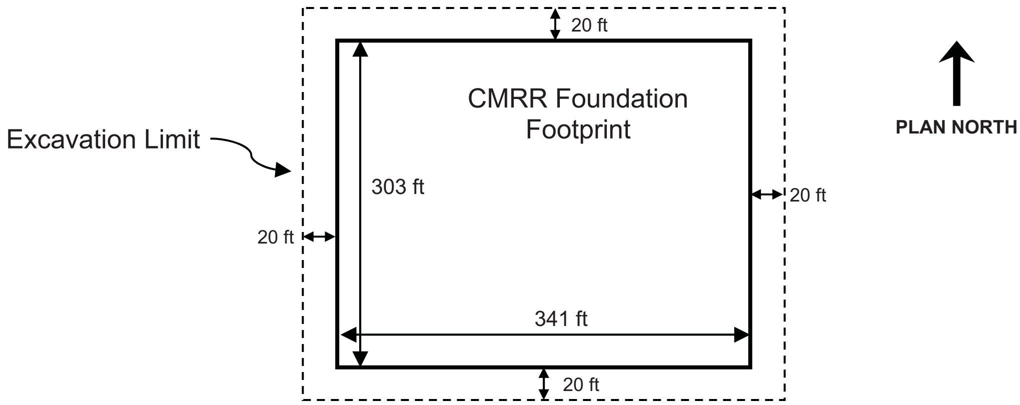

Figure G-1. Foundation Footprint and Excavation Limit Dimensions of the CMRR Facility

The ground modification lateral dimensions were calculated using the CMRR footprint foundation dimensions and the lateral excavation limits, which are 20 ft in each direction beyond the foundation perimeter:

<u>Eqn. G-1:</u>

Ground Modification Width Dimension = (CMRR Foundation Width) + (Excavation Width Dimension East of CMRR Foundation Footprint) + (Excavation Width Dimension West of CMRR Foundation Footprint) = (341 ft) + (20 ft) + (20 ft) = 381 ft

<u>Eqn. G-2:</u>

Ground Modification Length Dimension = (CMRR Foundation Length) + (Excavation Width Dimension North of CMRR Foundation Footprint) + (Excavation Width Dimension South of CMRR Foundation Footprint) = (303 ft) + (20 ft) + (20 ft) = 343 ft

<u>Eqn. G-3:</u>

The depth of the ground modification, which corresponds to the thickness of $Qbt3_L$, was calculated using the foundation elevation of the CMRR building and the elevation at the bottom of the $Qbt3_L$ Bandelier Tuff unit.

Ground Modification Depth Dimension = (CMRR Foundation Elevation) – (Elevation at bottom of unit $Qbt3_L$) = (7,226 ft amsl) - (7,170 ft amsl) = 56 ft

Eqn. G-4:

The total volume of the ground modification of the $Qbt3_L$ Bandelier Tuff was calculated using a simple geometric relationship.

Volume of Ground Modification = (Ground Modification Width Dimension) x (Ground Modification Length Dimension) x (Ground Modification Depth Dimension)

$$= (381\ \text{ft}) \times (343\ \text{ft}) \times (56\ \text{ft}) = 7{,}318{,}248\ \text{ft}^3$$

$$= 271{,}046\ \text{yd}^3$$

$$\approx 271{,}000\ \text{yd}^3$$

G.4 Results and Conclusions

The volume of the ground modification of the $Qbt3_L$ Bandelier Tuff is 271,046 yd^3. The rounded value of the volume of the ground modification is 271,000 yd^3.

The average thickness of the $Qbt3_L$ Bandelier Tuff was obtained from a contour plot in Figure VIII-5 of Kleinfelder's 2007 CMRR Geotechnical Engineering Report, the elevation of $Qbt3_L$. According to this reference, the thickness of $Qbt3_L$ generally ranges from 54 to 57 feet. The thickness was assumed to be 56 feet to determine the volume of the ground modification of the $Qbt3_L$.

The contour plot in Kleinfelder's 2007 report was created from limited field data collected by Kleinfelder at the time of the CMRR geotechnical field exploration; the actual thickness of $Qbt3_L$ may differ from the thickness indicated in this reference. This may result in a change in the actual volume of the ground modification at the time of construction. The actual thickness of the $Qbt3_L$ layer should be verified prior to construction to ensure that the volume of ground modification as determined in this appendix is still applicable.

G.5 References

Kleinfelder, 2007, Geotechnical Engineering Report, Chemistry and Metallurgy Research Facility Replacement (CMRR) Project, Los Alamos National Laboratory. Kleinfelder Project No. 19435, Rev. 0, Kleinfelder, Albuquerque, New Mexico.

Los Alamos National Laboratory. April 28, 2009, CMR Replacement (CMRR) Nuclear Facility, CONSTRUCTION ACCESS AND LIMITS OF EXCAVATION PLAN. Drawing No. C-54634, Rev. F, Sheet CS-50, Sheet Number 2 of 8. Drawing prepared by Sargent & Lundy for Los Alamos National Laboratory.