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ACRONYMS AND ABBREVIATIONS

ACC	Accession Number
ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
atm	atmosphere, atmospheres (unit of measurement)
AWWA	American Water Works Association
BGS	below ground surface
BSC	Bechtel SAIC Company
cm	centimeter, centimeters
CRWMS	Civilian Radioactive Waste Management System
DIRS	Document Input Reference System
DOE	U.S. Department of Energy
DON	Department of the Navy
DTN	Data Tracking Number
E	Young's modulus or secant Young's modulus
Eq.	equation
EFUW	equivalent fluid unit weight
ft	foot, feet (unit of measurement)
ft/s	feet per second
G	shear modulus
G_{max}	small-strain (maximum) shear modulus
in.	inch, inches
kips/ft^2	kips per square foot
kips/ft^3	kips per cubic foot
kN	kilonewton
kPa	kilopascal
lbf	pounds-force
lbf/ft^2	pounds-force per square foot
lbf/ft^3	pounds-force per cubic foot
lbm/ft^3	pounds-mass per cubic foot
m	meter
M&O	Management and Operating Contractor
pcf	pounds per cubic foot

psi	pounds per square inch
"Q"	"quality"
QA	quality assurance
Qal	Quaternary alluvium
Rev.	revision
REV.	revision
SASW	spectral analysis of surface waves
TIC	Technical Information Center
Tpc	Tiva Canyon Tuff
TDMS	Technical Data Management System
Tmbt1	pre-Rainier Mesa Tuff bedded tuff
Tmr	Rainer Mesa Tuff of the Timber Mountain Group
Tpbt4	pre-Tiva Canyon Tuff bedded tuff
Tpbt5	pre-Tuff unit "x" bedded tuffs (also known as post-Tiva Canyon Tuff bedded tuffs)
Tpcpll	Tiva Canyon Tuff: crystal-poor member, lower lithophysal zone
Tpcpln	Tiva Canyon Tuff: crystal-poor member, lower nonlithophysal zone
Tpcpmn	Tiva Canyon Tuff: crystal-poor member, middle nonlithophysal zone
Tpcpul	Tiva Canyon Tuff: crystal-poor member, upper lithophysal zone
Tpcpv	Tiva Canyon Tuff: crystal-poor member, vitric zone
Tpcr	Tiva Canyon Tuff: crystal-rich member, including the vitric zone (Tpcrv), the nonlithophysal zone (Tpcrn), and the lithophysal zone (Tpcrl)
Tpcrn	Tiva Canyon Tuff: crystal-rich member, nonlithophysal zone, but used in BSC (2002) to mean the Tpcr member
Tpcrv	Tiva Canyon Tuff: crystal-rich member, vitric zone
Tpki	Tuff unit "x"
USBR	U.S. Bureau of Reclamation
USN	U.S. Department of the Navy
ver.	version
v_p	compression-wave seismic velocity
v_s	shear-wave seismic velocity
WHB	Waste Handling Building
YMP	Yucca Mountain Site Characterization Project

GLOSSARY

This glossary presents definitions for geologic and geotechnical terms as used in this report. Other definitions may be used in other disciplines or in other contexts.

bedded tuff - a rock unit composed of volcanic ejecta that was deposited in layers and that exhibits distinct planes of weakness (bedding planes) parallel to layering; deposited either by water or by compositional sorting by air fall.

bulk density - synonym of density.

coefficient of uniformity - the ratio of D_{60} to D_{10}, where D_n is the sieve opening that would allow n percent of the soil particles (on a dry mass basis) to pass. In practice, D_n is determined by interpolation of the results of a particle-size distribution test.

coefficient of vertical subgrade reaction, k (mass per length squared per time squared, e.g., pound-force/ft^3 or kN/m^3) - the ratio of the vertical pressure acting at the foundation/subgrade interface at a point to the settlement at the same point.

compression-wave velocity - velocity of the compression (P) wave from a seismic energy source.

density, ρ (mass per length cubed, e.g., pound-mass/ft^3 or kg/m^3) - the total mass (solids plus liquid plus gas) per total volume. Synonyms: bulk density, total bulk density, moist density, total density, wet density.

density of solid particles, ρ_s (mass per length cubed, e.g., pound-mass/ft^3 or kg/m^3) - the mass of solid particles divided by the volume of solid particles.

dry density, ρ_d (mass per length cubed, e.g., pound-mass/ft^3 or kg/m^3) - the mass of solid particles per the total volume of soil or rock.

embed - to found a foundation at a certain distance below the ground surface (see embedment).

embedment - the depth at which the base of a foundation is situated below the ground surface.

engineered fill - an artificial fill (i.e., a fill constructed by man) that meets several criteria, typically including: (1) the fill is designed to meet established criteria (e.g., bearing capacity, settlement) for a particular purpose (building, embankment, etc.); (2) criteria are established on drawings and in a written specification for the material placed in the fill; (3) the fill is placed in accordance with drawings and written specifications; (4) the fill placement operations are observed by a geotechnical engineer (usually a geotechnical technician working under the geotechnical engineer's supervision); (5) the material being placed in the fill is sufficiently tested to establish its geotechnical characteristics; (6) the degree of compaction of the fill is verified by either (a) in situ density tests and compaction tests if relative compaction or relative density is specified, or (b) documenting adherence to a method specification, depending on which acceptance criteria is stipulated in the construction contract documents; (7) all fill material and all compacted fill that do not meet the contract requirements is either removed and replaced or reworked in an appropriate manner; (8) the geotechnical engineer prepares detailed written daily reports stating the geotechnical engineer's observations for the day, which are distributed on a daily basis; and (9) the geotechnical engineer writes and files a report at the conclusion of earthwork construction summarizing the geotechnical engineer's observations and testing made during construction and providing his opinion that the fill was or was not constructed in accordance with the specifications and is suited or not for its intended use.

fines content - the percent of a material's particles, on a dry weight basis, that pass through a U.S. Standard 75-micron sieve (U.S. Alternative No. 200 sieve).

kip - a unit of force (weight) equal to one thousand pounds-force (1000 lbf).

lithophysae - hollow, bubble-like structures composed of concentric shells formed by the concentration of gasses during cooling of portions of a volcanic flow deposit.

lithophysal - containing lithophysae.

low-amplitude shear modulus - see shear modulus, low-amplitude.

moist density - synonym of density.

non-engineered fill - an artificial (man-made) fill that does not meet the definition of engineered fill.

nonwelded tuff - a volcanic rock consisting of fragments that were deposited with insufficient heat to have become fused.

obliquity - the ratio of the major to minor principal effective stresses, σ_1'/σ_3'.

overburden pressure - at point A at depth, d, $\sigma_v = \int_0^d \gamma \, dz$ where γ is unit weight and z is depth below the point on the ground surface directly above Point A. Note: For this report, no groundwater needs to be considered, so effective overburden pressure is taken to be the same as total overburden pressure.

Poisson's ratio - in Hooke's Law for isotropic materials, for a material subjected to a stress in some direction, the ratio of the strain in the transverse direction to the strain in the direction of stress application.

relative compaction - the ratio, expressed as a percentage, of the dry unit weight of a soil mass to the reference maximum dry unit weight of the material as determined by a test, such as ASTM D 1557-00, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft³ (2,700 kN-m/m³))*.

relative density - the ratio of (1) the difference between the void ratio of a cohesionless soil in the loosest state and its actual void ratio, to (2) the difference between the void ratios in the loosest and in the densest states.

saturated density, ρ_{sat} (mass per length cubed, e.g., pound-mass/ft^3 or kg/m^3) - the total mass per total volume of completely saturated soil or rock.

separation – refers to the apparent relative displacement of a tabular body or surface across a fault. It is the distance between displaced parts measured in any specified direction. It is distinguished from slip, which refers to the actual relative displacement of the two walls of a fault. To classify a fault in terms of slip it is necessary to know the direction and sense of translation. If the direction and sense of displacement is not known, then a fault can be classified in terms of separation.

shear modulus - the stiffness factor for a material under shear stress, expressed by the relationship of the applied shear force to the change in position produced by this force, calculated as the product of the total mass density (total unit weight divided by gravity) and the square of the shear wave velocity. Symbol: G.

shear modulus, low-amplitude - shear modulus determined as the ratio of the shearing stress divided by the shearing strain at low strain values (< 0.001%). Symbol: G_{max}. Synonym: small-strain shear modulus.

shear-wave velocity - velocity of the shear (S) wave from a seismic energy source.

shear-wave velocity, low-amplitude -the velocity of a seismic body wave propagating with a shearing motion that oscillates particles at right angles to the direction of propagation measured at low strain values (< 0.001%). Synonym: small-strain shear-wave velocity.

small-strain shear modulus - synonym of low-amplitude shear modulus.

small-strain shear-wave velocity - synonym of low-amplitude shear-wave velocity.

total density - synonym of density.

total unit weight - synonym of unit weight.

unit weight, γ (mass per length squared per time squared, e.g., pound-force/ft^3 or kN/m^3) - the total weight (solids plus liquid plus gas) per total volume. This parameter is also referred to as "moist unit weight," "wet unit weight," or "total unit weight."

unit weight, dry, γ_d (mass per length squared per time squared, e.g., pound-force/ft^3 or kN/m^3) - the total weight of solid particles per total volume.

unit weight, total - synonym of unit weight.

vitric tuff - an indurated deposit of volcanic ash composed mainly glassy fragments blown out of a volcano during a volcanic eruption.

water content - the ratio of the mass of water contained in the pore spaces of soil or rock material, to the solid mass of particles in that material, expressed as a percentage. Also referred to as gravimetric water content. Note that adsorded water is not considered part of the water in the pore spaces but as water bound to the solid particles. Syn: moisture content.

welded tuff - a rock consisting of volcanic fragments that has been indurated by the heat retained by particles and the enveloping gases.

wet density - synonym of density.

1. PURPOSE

1.1 PURPOSE

The purpose of this report is to summarize calculations made for foundations for a potential Waste Handling Building (WHB) in the protected area of the North Portal Operations Area. This report has been prepared under *Technical Work Plan for Testing and Monitoring*, TWP-MGR-MD-000018 REV 00 (BSC 2001). This report documents geotechnical evaluations of, and geotechnical recommendations for, foundations for potential waste handling facilities near the North Portal of the Exploratory Studies Facility of the potential monitored geologic repository. These recommendations have been developed for use in design of the potential waste handling facilities to a level suitable to support License Application. The potential waste handling facilities near the North Portal will be referred to in this report as simply the "waste handling facilities" or "waste handling buildings."

1.2 SCOPE

The scope of work documented in this report supports work package P4D1226TH2 "Surface Facility Characterization," the planning for which is documented in Technical Work Plan TWP-MGR-MD-000018 REV 00 (BSC 2001). Work package P4D1226TH2 supports the completion of field and laboratory studies for the waste handling facilities and the preparation of two reports, one of which is this report. The following specifics of the work scope described in the TWP are provided in *Update to Preliminary Geotechnical Investigation of the Waste Handling Building* (Misiak 2001):

> "WASTE HANDLING BUILDING SOILS REPORT
>
> A site-specific geotechnical/soils engineering report is needed to accurately complete the License Application design of the foundation for the Waste Handling Building (WHB).
>
> Current preliminary foundation concepts for the WHB are described in the *PRELIMINARY Geotechnical Investigation for Waste Handling Building, Yucca Mountain Site Characterization Project* (DI: BCB000000-01717-5705-00016 REV 00)[1]. This report is based upon one boring located within the footprint of the WHB and on information from several earlier borings and test pit programs at the North Portal pad.
>
> The update to the geotechnical report is needed to remove the uncertainty of the soil data below the WHB as a result of using data from only one boring.
>
> Specific issues involving the design and construction of the WHB foundation include:

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[1] The report referred to by Misiak (2001) is CRWMS M&O (1999a).

1. The existing North Portal pad was developed using tunnel muck of a varying thickness (5 to 25 ft thick). The muck was placed using both controlled and uncontrolled methods (i.e., layer thickness and mechanical compaction). There is no documentation of inspection of the muck placement, nor is there qualified foundation design information for allowable bearing pressures, soil density, lateral (active and passive) earth pressures, the potential for settlement under building loading, etc. As recommended by the *PRELIMINARY Geotechnical Investigation for Waste Handling Building, Yucca Mountain Site Characterization Project*, the tunnel muck will be removed and replaced with engineered fill.

2. The foundation for the WHB will be a mat foundation as recommended in the *PRELIMINARY Geotechnical Investigation for Waste Handling Building, Yucca Mountain Site Characterization Project.*

A site specific geotechnical report should be prepared, and include as a minimum:

1. A chart illustrating the soil classification criteria and the terminology and symbols used on the boring logs;

2. The identification of the ASTM standards and test methods used;

3. A plot plan giving dimensioned locations of test borings;

4. Vertical sections for each boring plotted and graphically presented showing number of borings, date of start and finish, surface elevations, description of soil and rock and thickness of each layer, depth to any significant loss or gain of drilling fluid, hydraulic pressure required or number of blows per foot (N value);

5. Locations of strata containing organic materials, weak materials or other inconsistencies that might affect engineering conclusions;

6. A description of the existing surface conditions;

7. A summary of the subsurface conditions;

8. A profile and/or topographic map of rock and other bearing stratum;

9. A report on laboratory determinations of soil properties including shrinkage and expansion properties;

10. Water table depth.

The report shall include the responsible geotechnical engineer's recommendations for foundation design and construction including the following:

1. Foundation support of the WHB including allowable soil bearing pressures, bearing elevations, modulus of subgrade reactions, foundation design recommendations and anticipated static and dynamic settlements;

2. Lateral earth pressures, pressure coefficients (active, passive, and at rest) and internal friction angles for design of walls below grade, soil density, and requirements for backfill, compaction and subdrainage;

3. Soil material and compaction requirements for site fill, construction backfill, and for the support of structures;

4. Design criteria for temporary excavation and temporary protection of existing structures such as sheet piling and underpinning. It is anticipated that there will be a construction uniform surcharge of 300 psf starting at the edge of excavation and a line load of 2000 pounds per linear foot 4 feet from the edge of excavation.

5. Stability of slopes;

6. Frost penetration depth and effect;

7. Analysis of the effect of weather and construction equipment on soil during construction;

8. Investigation of soils to evaluate the presence of dispersive potentially expansive, deleterious, chemically active or corrosive material or conditions including soil conductivity/resistivity;

9. Recommendation of a foundation system and with alternative workable systems.

DESCRIPTION OF THE WASTE HANDLING BUILDING

The following is a brief description of the WHB structure and the approximate columns and loads. This information will be used in developing the site-specific geotechnical report needed for the structural foundation design of the WHB.

The WHB is the primary nuclear waste handling surface facility. The WHB footprint plan dimensions at grade are 590 ft by 700 ft. The building's roof structure has several levels at 64 ft, 71 ft, and 91 ft above the ground-operating floor. There are also two cross-transfer corridors at 117 ft above ground. Approximately one-half of the WHB will be constructed of heavy, reinforced concrete walls and roof slabs for radiation shielding purposes. Walls will be up to 5 ft thick for a height of up to 30 ft above the ground-operating floor and will reduce to 1.5 ft thick for the remaining heights up to the roof levels. The remainder of the WHB will be heavy steel framing with metal clad walls and concrete slabs for equipment rooms and roof slabs. Overhead cranes ranging in capacities of 15 tons to 140 tons will operate over the entire building. Water pools 50 ft deep below grade will be located in the northwest quadrant of the WHB.

See attached WHB floor loading plan."

As defined in BSC (2001), this input was to be provided in two reports, the first of which is Scientific Analysis Report ANL-MGR-GE-000003, *Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project* (BSC 2002). In the scope above, the ten items that the site-specific geotechnical report should include are covered in BSC (2002). This report covers the first eight of the nine items for foundation design and construction.

As a result of a meeting on September 26, 2001 and later telephone conversations concerning the scope of the report and the nature of the WHB, several modifications were made to the above scope (Wong 2001). Specifically:

1. The description of the WHB in Misiak (2001) was deleted due to changes in the layout, as will be discussed in Section 5 of this report.

2. For Recommendation Item 1 in Misiak (2001), settlement analysis was removed from the scope, and ultimate bearing capacities were to be presented based on shear strength alone, without consideration of the effects of settlement on allowable bearing pressures. Settlement will be computed as part of the engineering analyses for the waste handling facilities.

3. Recommendation Item 9, "Recommendation of a foundation system and with alternative workable systems," in Misiak (2001) was removed from the scope. Initially, preliminary soil-structure engineering calculations indicated that performance of the WHB (as described in Misiak 2001) under seismic conditions was unsatisfactory. Consideration was given to performing analyses for foundation types other than a mat foundation. However, to develop a recommendation as to the preferred foundation system and to identify alternative workable systems would require many more analyses than was intended when the request was made for a soils report. Consequently, Item 9 was deleted from the scope to expedite the report, and an assumption (see Section 5, Assumption 8) was made that a mat foundation would be satisfactory for the WHB (Wong 2001).

1.3 LIMITATIONS

This report is intended to provide geotechnical input for foundations for the waste handling facilities to support License Application. The locations of individual structures and the site grading plan were not defined at the time the work described in this report was performed. The input was developed in this report to cover a variety of potential layouts. When the borrow area is identified and the locations of individual structures and the site grading plan become known, the data and interpretations in this report should be reviewed to evaluate whether any changes are required and some confirmatory boreholes and velocity measurements may be required. This report may not contain sufficient information for purposes other than those for which it has been prepared.

Only a very small part of the subsurface conditions at the project site has been observed. In view of the general geology of the project area and the presence of non-engineered fill, the possibility of different subsurface conditions cannot be discounted. Conclusions and recommendations presented in this report are based upon the current understanding of the project and the assumption (Section 5, Assumption 5) that the subsurface conditions do not deviate appreciably from those disclosed by the site subsurface exploration and the assumption (Section 5, Assumption 9) that alluvium logged in borehole UE-25 RF#21 between about 70 and 115 feet is in fact bedrock.

The bearing capacity calculation in this report is based on level ground conditions. According to Section 4.2, the final grade over the pad area shall have a nominal slope between 2 and 3 percent (CRWMS M&O 1999b, Section 1.2.1.7). Slopes of 2 to 3 percent are considered sufficiently horizontal for the values of ultimate bearing capacity in this report to be used (subject to consideration of settlement). However, if a foundation is located near a slope, the allowable bearing capacity should be reviewed. For the purpose of triggering a review, "near" may be taken to mean within four times the footing width.

For the lateral earth pressures (including passive, active, at-rest, and compactor-induced) developed in this report to be valid, the ground surface in the zone behind the wall must be horizontal or slope downhill away from the wall for the active, at-rest, and compactor-induced conditions or slope uphill away from the wall for the passive condition. According to Section 4.2, the final grade over the pad area shall have a nominal slope between 2 and 3 percent (CRWMS M&O 1999b, Section 1.2.1.7). Slopes of 2 to 3 percent are considered sufficiently horizontal for the values in this report to be used. In addition, if a slope or grade change (retaining wall) occurs within or at the edges of the engineered fill pad, there must be sufficient distance between the wall/foundation where the passive resistance develops and the slope or grade change (retaining wall) (see Table 5).

Any persons using this report for bidding or construction purposes should perform such independent investigations as they deem necessary to satisfy themselves as to the surface and subsurface conditions to be encountered and the construction procedures and methods to be used in the performance of work on this project.

2. QUALITY ASSURANCE

The activities documented in this calculation were evaluated in accordance with procedure AP-2.21Q, *Quality Determinations and Planning for Scientific, Engineering, and Regulatory Compliance Activities*, and they were determined to be subject to the Yucca Mountain quality assurance program. This evaluation is documented in BSC (2001, Attachment I).

The control of the electronic management of data for this document included:

- Backing-up on tape of all project electronic files twice daily, with tapes being sent weekly to an offsite storage location.

- All electronic data transfers were checked for alteration using a file-size comparison, using a zip file format, using a Project-approved file comparison software (signature generation and compare routine), or visually comparing the electronic file with a printed copy from the Technical Data Management System (TDMS).

- Write-protecting files before including them in a scientific notebook or other permanent record.
- Backing-up of all unique physical records and storing the backup in a dual location.
- Saving any intermediate analysis records required to understand how acquired data were processed/analyzed.

3. USE OF SOFTWARE

Table 1 lists the computer programs used in developing the parameter values in this Calculation:

Table 1. Computer Software Used for This Calculation

Software Name	Software Tracking No.	Computer Type
Grapher ver. 3.02	Exempted by Section 2.1.2 of AP-SI.1Q*	IBM PC-compatible
Microsoft Excel, ver. 97 SR-2	Exempted by Section 2.1.1 of AP-SI.1Q	IBM PC-compatible
Microsoft Word, ver. 97 SR-2	Exempted by Section 2.1.1 of AP-SI.1Q	IBM PC-compatible
AutoCAD 2000	Exempted by Section 2.1.2 of AP-SI.1Q	IBM PC-compatible
Adobe Acrobat, ver. 4.0 and 5.0	Exempted by Section 2.1.2 of AP-SI.1Q	IBM PC-compatible

* the title of AP-SI.1Q is *Software Management.*

In accordance with Section 2.1.1 of AP-SI.1Q, Microsoft Word version 97 SR-2 and Microsoft Excel version 97 SR-2 are exempted software products. In accordance with Section 2.1.2 of AP-SI.1Q, AutoCAD 2000 and Adobe Acrobat versions 4.0 and 5.0 are exempted software products. The use of Grapher for visual display of data is exempted by Section 2.1.2 of AP-SI.1Q.

4. INPUTS

4.1 DATA AND PARAMETERS

The input data used or considered in this Calculation are summarized in Table 2. The Q-status of each of these inputs is provided in the electronic Document Input Reference System (DIRS).

Table 2. Summary of Input Data Used in This Calculation

Description	DTN or Reference
logs of boreholes UE-25 RF#13 to #26 & #28, #29	GS020383114233.003
logs and photographs of test pits TP-WHB-1 to -4	GS020383114233.001
borehole geophysical data (gamma-gamma density) in boreholes UE-25 RF#16, #18, #20, #21, #22, #24, #28	MO0112GPLOGWHB.001
suspension seismic data, UE-25 RF#14 to #26, #28, #29	MO0204SEPBSWHB.001
resonant column and torsional shear data	MO0203DHRSSWHB.001*

Table 2. Summary of Input Data Used in This Calculation (continued)

Description	DTN or Reference
downhole seismic velocity profiles, boreholes UE-25 RF#13 to #16, #18 to #26, #28, #29	MO0111DVDWHBSC.001
downhole seismic velocity profiles, UE-25 RF#13, #17	MO0110DVDBOREH.000
velocity profiles for SASW lines SASW-1 to SASW-37	MO0110SASWWHBS.000
geotechnical laboratory test results – TP-WHB-1 to -4	GS020483114233.004
geotechnical laboratory test results – Fran Ridge borrow	MO0203EBSCTCTS.016
geotechnical laboratory test results for core from NRG and SD boreholes	SNL02030193001.001 to SNL02030193001.027, except SNL02030193001.025, SNL01A05059301.005
contact depths for core from boreholes NRG#2, 2a, 2b, 3, 6	GS940308314211.009
contact depths for core from borehole NRG#7/7A	GS940708314211.032
contact depths for core from boreholes USW SD-9, SD-12	GS941108314211.052, GS940908314211.045
field resistivity data from North Portal area	GS930283114233.001
logs and laboratory test results for NRSF-series test pits	GS920983114220.001
in situ density tests (sand cone and nuclear gage) in pit SFS-3	Ho et al. (1986, p. 14)*
particle-size distributions for samples from borehole RF#3b and test pit SFS-3	CRWMS M&O (1999a, pp.C-2 and H-1)*
particle-size distributions for samples from borehole RF#13	CRWMS M&O (1999a, Appendix L)*
locations of Test Pits TP-WHB-1 to -4	MO0012GSC00405.000
elevation contours for ground surface before existing fill pad was constructed near the North Portal	MO9906COV98462.000

* indicates the reference was used for corroboration or reference purposes only.

4.2 CRITERIA

Criteria used or referred to in this report are:

1. The layout shall locate the surface waste handling facilities above the probable maximum flood (CRWMS M&O 1999b, Section 1.2.1.6).

2. The final grade over the pad area shall have a nominal slope between 2 and 3 percent (CRWMS M&O 1999b, Section 1.2.1.7).

3. The configuration of pads shall prevent pooling of water (CRWMS M&O 1999b, Section 1.2.1.8).

4. Site drainage shall contain and route stormwater from natural surface water drainage ways around surface facilities and provide water drainage for systems located on pads (CRWMS M&O 1999b, Section 1.2.1.10).

5. Site drainage shall be designed for the runoff from the probable maximum precipitation event (CRWMS M&O 1999b, Section 1.2.1.11).

6. Fill slopes shall be designed with a slope no steeper than 2:1 (horizontal to vertical) (CRWMS M&O 1999b, Section 1.2.1.12).

7. The layout shall locate all surface waste handling facilities away from faults which have 2 in. (5 cm) or more displacement over the past 100,000 years (CRWMS M&O 1999b, Section 1.2.1.15).

8. The maximum grade of the surface pathways used by the Waste Emplacement System shall be ±2.5 percent (CRWMS M&O 1999b, Section 1.2.4.9). The Waste Emplacement System is the system that will transport the loaded and sealed waste packages, each placed on its dedicated emplacement pallet, from the Waste Handling Building to the emplacement area, which contains the emplacement drift.

9. The layout shall locate the Waste Handling Building System near the North Portal of the repository (CRWMS M&O 1999b, Section 1.2.4.12).

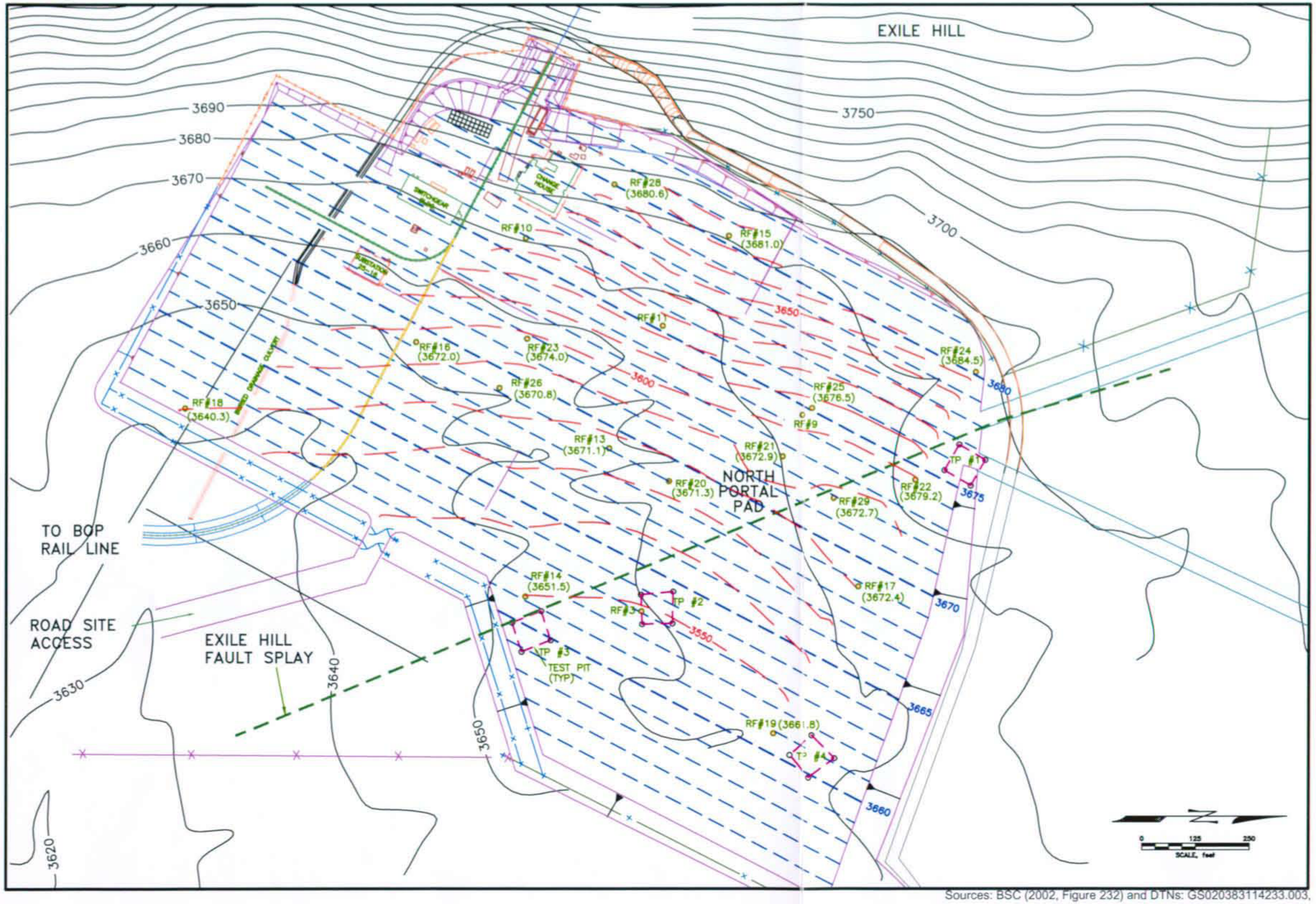
5. ASSUMPTIONS

The following assumptions have been used in this Calculation. The assumptions are used in Sections 6 through 16. None of these assumptions requires confirmation for this level of preliminary design supporting License Application.

1. The waste handling facilities are to be constructed immediately east of Exile Hill, near the North Portal (DOE 2001, pages 14-15, CRWMS M&O 1999b, Section 1.2.4.12). In the past, a fill pad was constructed in this area to support construction of the Exploratory Studies Facility. In addition, muck from tunneling operations was discharged around the perimeter of the fill pad. Based on Misiak (2001) and Wong (2001) and as recommended in CRWMS M&O (1999a, Section 7.3), it is assumed in this report that the existing fill pad and the tunnel muck previously placed near the North Portal will be removed prior to constructing the waste handling facilities.

2. After removal of the existing fill, it is assumed that engineered fill will be required to achieve the final grades at the building sites. A grading concept was assumed based on the criteria listed in Section 4.2, particularly the first and second criteria. The finished grade at the North Portal was maintained near its present elevation. The finished grade was assumed to slope downward at 2.0 percent to the southeast from the North Portal to provide for surface drainage. The assumed slope, 2.0 percent, is the flatter limit on the allowable range of slopes (CRWMS M&O 1999b, Section 1.2.1.7). The flatter limit was assumed because the pad should be kept above the probable maximum flood (CRWMS M&O 1999b, Section 1.2.1.6). The finished grade contours based on these principles and assumed in this report are shown on Figure 1.

3. It is assumed that the engineered fill will be constructed entirely of material from the Fran Ridge Borrow Area, and that the single composite sample of material from that borrow pit is representative of the material that will be used for construction. The Fran Ridge Borrow



Legend

- RF#22 (3679.2) Borehole number (ground elevation in feet in parentheses)
- TP #2 Test pit number
- Conceptual finish grade contours based on 2% grade (1-ft intervals)
- Ground surface elevation prior to construction of North Portal construction support pad (10-ft intervals)
- Top of bedrock contours (10-ft intervals)

Sources: BSC (2002, Figure 232) and DTNs: GS020383114233.003, MO0012GSC00405.000, MO9906COV98462.000

Figure 1. Interpreted Top-of-Bedrock Contours and Estimated Engineered Fill Contours.

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Area is an active borrow site, and thus has undergone required environmental and permitting reviews for its current use. It provides sand and gravel material, which is a commonly available type of material in the site vicinity and one that typically can be used to construct engineered fills with excellent engineering properties. Recommendations for using the Fran Ridge material for the engineered fill are provided in Section 7 and foundation recommendations incorporating use of the Fran Ridge material for the engineered fill are provided in Sections 9 through 15.

4. The WHB referred to in Misiak (2001) was a single, large building intended to house multiple operations (DOE 2001, pages 28-29). During development of this calculation, the concept of a single large WHB was replaced by several smaller buildings, each intended for a different function or process (Wong 2001). The number of buildings may change as the project is further developed; however, it is assumed in this report that the final layout will include several specialized buildings. The types of buildings that are likely are dry- and wet-process buildings and buildings for receiving transporters and preparing disposal containers. This assumption does not require confirmation because the analyses reported herein were performed to accommodate an unknown number of buildings and different sizes of buildings.

5. It is assumed that the subsurface conditions do not deviate appreciably from those disclosed by the site subsurface exploration. This is a normal, if not universal, assumption to make in geotechnical practice.

6. It is assumed that the wet-process building will include a pool (or group of pools) that extends approximately 50 feet below the main floor level, and the excavation for the pool mat will extend about 55 feet below the main floor level, similar to the description of the pools in Misiak (2001). The remainder of the wet-process building and all the other buildings were assumed to have a main floor level close to the elevation of the surrounding fill surface, and to have no subsurface rooms or tunnels. The rational for the main floor slab being at the approximate level of the surrounding engineered fill pad surface is that the waste transporters must travel along a relatively flat gradient from building to building and then into the repository via the North Portal (CRWMS M&O 1999b, Section 1.2.4.9), and it would be difficult to design and costly to construct a system for raising and lowering the waste containers inside the buildings. Thus, the only potential significant excavation is assumed to be for the wet-process building pool.

7. It is assumed that the excavation for the pool will occur after the existing fill is removed, and either before or after any engineered fill is placed at this location. Thus, the geotechnical properties of the existing fill do not need to be considered with regard to maintaining ground stability during excavation. This construction sequence is logical if the existing fill is to be removed prior to construction of the surface waste handling facilities. If the excavation is made before the engineered fill is constructed, it is assumed that the excavation will involve only alluvium, whereas if the engineered fill is constructed before the excavation is made, it is assumed that the excavation will involve engineered fill and alluvium. If the pool were located much nearer Exile Hill than has been shown in the past, part of the excavation might involve bedrock; however, the pressures on the temporary shoring should be more favorable (i.e., lower) than those associated with excavation in alluvium.

8. It is assumed that the buildings can be founded on shallow spread footings and mat foundations. These types of foundations are commonly used where site conditions similar to those at the site (dense granular soils and a deep water table) are encountered. It is assumed that some of the buildings with greater loads (for example, reinforced concrete structures) may be founded on mat foundations, while some of the buildings (for example, steel-frame structures) with lighter loads may be founded on spread footings. The choice between spread footings and a mat foundation may depend on several factors, such as the percent of the building footprint that spread footings would occupy. This report considers the validity of this assumption from the standpoint of static conditions, but additional calculations will be required to consider the validity of this assumption from the standpoint of seismic conditions. This assumption does not require confirmation because if seismic calculations indicate that shallow foundations are not appropriate, then the recommendations of this report will simply be inapplicable.

9. It is assumed that the alluvium logged in borehole UE-25 RF#21 between about 70 and 115 feet (DTN: GS020383114233.003) is in fact bedrock (refer to Sections 8.2.1 and 16.2.1). The unit weight of the alluvium measured by the gamma-gamma survey between depths of about 70 and 115 feet BGS in borehole UE-25 RF#21 is anomalously low (see Section I.2.1 of Attachment I for details). This could indicate that the engineering properties of the deep alluvium are different and less favorable than those of the shallower alluvium. Alternatively, this could be due to misidentification of the drill cuttings from borehole UE-25 RF#21, which was not otherwise sampled. For this calculation it is assumed that the drill cuttings in borehole UE-25 RF#21 were misidentified and that the material is actually bedrock. This assumption is based on the lack of evidence of similar low-density alluvium near the base of the alluvial deposit in other boreholes and by absence of a low or decreased shear-wave velocity in the depth interval from 70 to 115 feet BGS in borehole RF#21 (BSC 2002, Figure 12). (Unfortunately, no suspension seismic measurements were made in this depth interval.) This assumption applies to the material properties developed in Section 8 and Attachment I, which are used in Sections 9 through 15.

10. It is assumed that the fill logged in borehole UE-25 RF#20 between about 9 and 28 feet (DTN: GS020383114233.003) is in fact alluvium (refer to Sections 6.2 and 16.2.1). Based on the pre-fill ground surface contours (Figure 1), there should be only about 9 feet of fill at that location. Unless there are utilities or structures that were buried at this location, this 19-foot discrepancy suggests that the alluvium in this vicinity was identified during fill construction as being unsuitable to support the fill and was removed. However, this could also be explained by misidentification of the drill cuttings in borehole UE-25 RF#20, which was not otherwise sampled. For this calculation, it is assumed that the drill cuttings in borehole UE-25 RF#20 were misidentified and that the material is actually alluvium. During removal of the existing fill, this material can be examined and its nature determined. The recommendations in Attachment II provide for examination of the subgrade after removal of the existing fill so that any loose material can be removed. This assumption applies to the material properties developed in Section 8 and Attachment I, which are used in Sections 9 through 15.

6. SITE SUBSURFACE CONDITIONS

The types of subsurface materials in the North Portal area and their engineering properties were investigated by cored borings, rotary wash borings, borehole geophysical measurements (downhole seismic, suspension seismic, gamma-gamma), SASW (spectral analysis of surface waves) surface-based geophysical measurements, test pits, in-place density tests, and static and dynamic laboratory tests. Material from a potential borrow source, known as the Fran Ridge Borrow Area, was sampled at four locations, and static and dynamic laboratory tests were performed on a composite of these samples.

The data acquired by these activities are summarized in BSC (2002), which should be considered to be a companion volume to this report. For brevity, only a few of the principal findings of BSC (2002) will be repeated in this Calculation. It is expected that the users of this report are conversant with BSC (2002).

6.1 EXISTING FILL

Non-engineered fill was encountered in 10 of the 16 boreholes that have been advanced in the North Portal area since the construction of the existing fill pad began in 1992 (BSC 2002, Tables 4 and 5; DTN: GS020383114233.003). This fill was placed in part to create a working platform for the construction operations that supported scientific experiments conducted in the Exploratory Studies Facility. Tunnel muck was also discharged around the edge of the construction pad. Section 3.9 of CRWMS M&O (1999a) provides more information about the fill. As noted in Section 5 (Assumption 1), it is assumed that this fill and tunnel muck will be removed. Hence, the existing fill is not discussed further in this report. Test pits TP-WHB-1 through -4 did not encounter fill because they were deliberately located off of the fill and on the alluvial surface (BSC 2002, Section 6.2.4, Attachments III and IV; DTN: GS020383114233.001).

6.2 ALLUVIUM

The geologic ages of surficial deposits in the North Portal Area, which include alluvium, colluvium and minor windblown deposits, range from early to middle Pleistocene to present (Whitney 1996, p. 4.3-4). The oldest deposits are found near the bottom of the alluvium unit; the youngest occur in presently active channels of small, ephemeral stream channels. In this report and in the companion report by BSC (2002), these deposits are referred to simply as "alluvium (Qal)."

Of boreholes UE-25 RF#13 through #29, all but RF#15 encountered alluvium (Qal) at the ground surface or beneath 5 to 28 feet of fill (BSC 2002, Tables 4 and 5) (however, see Assumption 10, Section 5). All four of test pits TP-WHB-1 through -4 encountered alluvium over their full depth of approximately 19 to 20 feet below ground surface (BGS) (BSC 2002, Section 6.2.4).

The most representative exposures of the alluvium were in the sideslopes of the test pits and trenches excavated in the North Portal area, including TP-WHB-1 through -4 (DTN:

GS020383114233.001) and older test pits and trenches (Swan et al. 2001, pages 8-21 and Plates 4, 5, 6, and 9). Based on these exposures and on particle-size distribution test results for samples from the test pits (BSC 2002, Table 13; DTNs: GS020383114233.001, GS020483114233.004), the alluvium consists primarily of interbedded caliche-cemented and non-cemented, poorly sorted, coarse-grained gravel with sand and some fines, cobbles, and boulders. The fines content of samples of alluvium from the test pits and boreholes are generally low, generally between 3 and 20 percent, but can be as high as 40 percent in the near-surface colluvium near Exile Hill (CRWMS M&O 1999a, pages C-2 and H-1, Appendix L; BSC 2002, Table 13; DTNs: GS020483114233.004, GS920983114220.001). Based on visual logging (BSC 2002, Attachments III and IV), the alluvium classifies in the Unified Soil Classification System (USCS) (ASTM D 2487-00, *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)* and ASTM D 2488-00, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*) primarily as poorly graded gravel (GP), poorly graded gravel with silt (GP-GM), and well-graded gravel with silt (GW-GM). Lesser amounts of silty sand (SM) and silty gravel (GM) were also logged. Most of the mapped soil units also have some cobble and boulder content. Based on laboratory tests (BSC 2002, Attachment XI), some of the alluvium also classifies as well-graded gravel (GW), well-graded gravel with silt (GW-GM), poorly graded sand with silt (SP-SM), and well-graded sand with silt (SW-SM).

It is worth noting that the USCS soil groups identified based on particle-size distribution tests on samples from the test pits tends to be different than those identified based on visual-manual methods. Using visual-manual procedures, geologists mapped some GM and SM soil groups in the test pits, but the laboratory test results did not indicate the presence of such soils. On the other hand, GW, GW-GM, SP-SM, and SW-SM soils were identified by the particle-size distribution tests, but none were mapped using visual-manual procedures. These results suggest that the mappers had a tendency to view the material as having more fines (silt- and clay-size particles) than it actually has. The mappers did not identify any of the material as being well-graded, and so appear to have viewed the material as less well-graded than it is.

The surface on which the alluvium was deposited was probably irregularly eroded, and the thickness of the alluvium likely varies considerably at some locations from the thickness implied by the relatively smooth bedrock contours shown on Figure 1.

Based on exposures in Fortymile Wash, and data from test pits and trenches discussed above, it appears that the alluvium in this area is highly variable in both the vertical and lateral directions. Several layers of calcite-cemented material (caliche) are present in the soil column, indicating episodes of low sediment accumulation and relatively advanced pedogenic soil development. Uncemented layers are also common, which would cause rapid changes in soil strength both with depth and laterally.

High blow counts were recorded for most of the alluvial material encountered in borehole UE-25 RF#13 during driving of a Modified California sampler. In general, the equivalent SPT N_1 values (standard penetration test result corrected to overburden pressure of 2,000 lbf/ft^2) are greater than 50 (CRWMS M&O 1999a, Section 6.2). However, the alluvial material from about 35 to 42 feet below ground surface appeared to be significantly less dense than the overlying and underlying alluvial material. The one Modified California sample driven in this interval yielded an equivalent SPT N_1 value of 12 (CRWMS M&O 1999a, Section 6.2). However, this isolated low blow count could also be caused by driving the sampler through slough and disturbed material at the bottom of the borehole. The shear-wave velocities measured in the sampled interval (36.9 to 38.5 feet BGS) are not particularly low (BSC 2002, Figure VII-1) and support the possibility that this blow count is not reliable.

No drive tube samples were taken in the alluvium encountered in boreholes RF#14 to RF#29. However, the dense nature of the alluvial material is also indicated by the relatively high values of shear-wave velocity measured in the downhole and suspension surveys at boreholes UE-25 RF#13 through #29 and by SASW surveys on and adjacent to the existing fill pad (DTNs: MO0111DVDWHBSC.001, MO0110DVDBOREH.000, MO0204SEPBSWHB.001, MO0110SASWWHBS.000). The shear-wave velocity results are discussed in detail in BSC (2002, Sections 6.2.5 to 6.2.7, Section 6.7, and Attachments V through IX).

It is noted that DTN: GS020383114233.003 indicates that the existing fill extends from the ground surface to a depth of 28 feet BGS in borehole UE-25 RF#20. Based on the pre-fill ground surface contours (Figure 1), there should be only about 9 feet of fill at that location. Unless there are utilities or structures that were buried at this location, this 19-foot discrepancy suggests that the alluvium in this vicinity was identified during fill construction as being unsuitable to support the fill and was removed. However, this could also be explained by misidentification of the drill cuttings in borehole UE-25 RF#20, which was not otherwise sampled. For this calculation, it is assumed that the drill cuttings in borehole UE-25 RF#20 were misidentified – it seems likely that the material is actually alluvium (Section 5, Assumption 10).

It is assumed (Section 5, Assumption 9) that the alluvium logged in borehole UE-25 RF#21 between about 70 and 115 feet (DTN: GS020383114233.003) is in fact bedrock (refer to Sections 8.2.1 and 16). The unit weight of the alluvium measured by the gamma-gamma survey between depths of about 70 and 115 feet BGS in borehole UE-25 RF#21 is anomalously low (see Section I.2.1 of Attachment I for details). This could indicate that the engineering properties of the deep alluvium are different and less favorable than those of the shallower alluvium and would require further investigations in the field and in the geotechnical laboratory. This could also be explained by misidentification of the drill cuttings from borehole UE-25 RF#21, which was not otherwise sampled. For this calculation it is assumed that the drill cuttings in borehole UE-25 RF#21 were misidentified and that the material is actually bedrock.

Based on the granular nature of the alluvium and paucity of clay particles (as inferred from results of liquid and plastic limit tests (BSC 2002, Table 13)) that could swell or shrink with changes in water content, the alluvium's expansion potential is insignificant. Based on the relative density values measured by in situ density tests in test pits TP-WHB-1 to -4 (BSC 2002, Table 6; DTN: GS020483114233.004), the alluvium has a relative density generally greater than 50 percent and is generally medium dense to dense. Only four (of the 22) relative density results were less than 50 and they ranged from 28 to 48 percent. Based on this, the alluvium does not appear to have a high potential for collapse (hydroconsolidation) if it should become saturated, which itself seems unlikely. Nonetheless, it is recommended that some in situ collapse tests be performed during future site exploration (Section 16.2.1).

6.3 BEDROCK

Sections 6.2.2 through 6.2.4 of BSC (2002) provide information about the stratigraphy and structure of rock units in the North Portal Area subsurface. Figure 233 (BSC 2002) identifies the upper bedrock units that underlie the North Portal Area. Section 6.6 of BSC (2002) provides interpretations of the subsurface structure in the North Portal Area subsurface. In particular, Figure 224 (BSC 2002) shows the interpreted locations of high-angle faults that crisscross the area; Figures 225 through 231 (BSC 2002) show vertical geologic cross sections. Figure 232 (BSC 2002) shows top-of-bedrock contours (these contours are reproduced on Figure 1 in this report). Note that Assumption 9 (Section 5) was used to construct the contours in the vicinity of borehole RF#21, as shown on Figure 1.

A few of the main conclusions and interpretations of BSC (2002) regarding stratigraphy and structure are:

- Beneath the surface deposits of fill and alluvium are welded and nonwelded units of the Timber Mountain and Paintbrush groups. Nonwelded units beneath the site include the pre-Rainier Mesa Tuff bedded tuffs (Tmbt1)[2] of the Timber Mountain Group, and the Tuff unit "x" (Tpki) and pre-Tuff unit "x" bedded tuffs (Tpbt5) of the Paintbrush Group. Beneath these nonwelded units is the Tiva Canyon Tuff, which is generally densely welded. The Tiva Canyon Tuff has been divided into two members; the younger crystal-rich member (Tpcr)[3] and the older crystal-poor member (Tpcp). These members are further divided into zones, for example, the Tiva Canyon Tuff crystal-rich nonlithophysal zone (Tpcrn) (BSC 2002, Section 6.6.2). For detailed geologic descriptions of the various zones of Tiva Canyon Tuff encountered in the boreholes, refer to BSC (2002, Section 6.6.2 and Attachments I and II).

- The bedrock generally dip at about 13 to 25 degrees towards the east-southeast, more or less parallel to the axis of the North Ramp. The interpreted dip angle is generally flattest in the western part of the area and steepest in the eastern part, and is relatively constant within fault-bounded blocks (BSC 2002, Figures 225 to 231).

- High-angle normal and reverse faults crisscross the area. The trend of the faults is generally north-northeast to north-northwest (BSC 2002, Figure 224).

- One interpreted fault, referred to informally as the "Exile Hill fault splay" (Figure 1), exhibits up to 300 feet of down-to-the-northeast separation. This has resulted in a thick downdropped sequence of pre-Rainier Mesa Tuff bedded tuff (Tmbt1) being present immediately below the alluvium in the northeast part of the area and being juxtaposed

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2 See Acronyms and Abbreviations for meaning of abbreviations of geologic unit and subunit names.

3 Note that Figure 233 and the report it is in (BSC 2002), as well as certain data (DTNs: GS020383114233.001 and GS020383114233.003), adopt the symbol Tpcrn for the Tpcr member. In fact, the Tpcr member consists of the Tpcrv, Tpcrl, and Tpcrn zones.

laterally with various zones of the Tiva Canyon Tuff (Tpcrn[4], Tpcpul, Tpcpmn, Tpcpll, and Tpcpln) (BSC 2002, Figures 226, 228, and 231).

6.4 GROUNDWATER

Section 6.6.3 of BSC (2002) reviews groundwater data relevant to the North Portal area. The groundwater table is located at a typical depth of 1270 feet below the present ground surface, and is over 1,000 feet below the top of bedrock in the North Portal area. Thus, groundwater is not a factor in the geotechnical calculations reported in this report.

7. RECOMMENDATIONS FOR ENGINEERED FILL AND EARTHWORK

After removal of the existing fill that was placed at the North Portal during 1992-94 to support construction of the Exploratory Studies Facility (Section 5, Assumption 1), engineered fill will be required to achieve final grade (Section 5, Assumption 2). Based on a comparison of estimated final grade contours (Figure 1) and the elevations of the existing construction-support pad (which is mostly a fill surface, but is partly in cut) at points of known elevation (that is, at boreholes RF#13, #15, #16, #20 to #26, #28, and #29[5]) (DTN: GS020383114233.003), the estimated final grade is generally lower than the existing grade by up to about 3 feet, and exceptionally as much as 5 at the location of UE-25 RF#22 near the north margin of the estimated fill pad. Relative to the grades existing before the existing fill was constructed (original grade), engineered fill up to 25 feet thick will be required in the southwest part of the pad and excavation up to 11 feet deep will be required at the north end of the pad. The excavation at the north end of the pad that is implied by Figure 1 seems, however, unlikely to occur because of the drainage problem this would entail, and it is likely that some other grading scheme will be developed to eliminate this situation. As mentioned in Section 5 (Assumption 3), it has been assumed that the engineered fill will be constructed of alluvial sand and gravel from the Fran Ridge Borrow Area.

Attachment II provides recommendations for the engineered fill in the form of guideline specifications. It addresses only the general fill required to bring the site to rough grade. It does not cover earthwork required for subgrade for foundations and slabs-on-grade, structure backfill, utility trench excavations and backfill, roadway subgrade and base, or sidewalk subgrade. The following paragraphs provide commentary on particular issues.

The composite sample of the alluvial deposits in the Fran Ridge Borrow Area consisted mainly of sand and fine gravel (DTN: GS020483114233.004). It was estimated that less than 5 percent of the material was greater than 3 inches in size (Lung 2002). However, it is evident from a visual survey of the borrow area that some cobbles and boulders are present. There are two approaches to dealing with a coarse material like the Fran Ridge Borrow Area sand and gravel: (1) specify a relatively small maximum allowable particle size and pay a premium to screen/crush the material, or (2) specify a relatively large maximum allowable particle size and

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[4] Recall that Tpcrn as used in BSC (2002) means Tpcr (Tiva Canyon Tuff crystal-rich member).

[5] Boreholes RF#14, #17, #18, and #19 are located off the existing fill pad. Boreholes RF#22 and #29 may or may not be on the fill pad – no fill was logged in those boreholes (BSC 2002, Table 4).

address all the added complexities presented by a coarser material and pay a premium for material testing during design and construction as well as to screen material to remove the cobbles and boulders. Either alternative would yield a fill with excellent engineering properties if the finer fraction of each is compacted to the same degree. However, it is more likely that the first alternative will be result in a material that is compacted more uniformly to at least the minimum required and hence is preferable from the standpoint of engineering properties. It is recommended that the first alternative be selected and that the engineered fill consist of particles that pass a 37.5 mm U.S. Standard Sieve (1½ inch U.S. Alternative Sieve), with no more than 20 percent of particles retained on the 19.0 mm U.S. Standard Sieve (3/4-inch U.S. Alternative Sieve).

These material gradation limits will permit the use of the 6-inch Proctor mold (ASTM D 1557-00, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3)))* with rock corrections for compaction characteristics and the sand cone test for in-place density verification. As an alternative to compaction control by a relative compaction criterion linked to ASTM D 1557, the use of relative density using ASTM D 4253-00, *Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table*, and ASTM D 4254-00, *Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density*, was considered. However, based on the limited laboratory testing performed to date and experience with similar materials, the relative compaction method using ASTM D 1557 is simpler and less costly to implement and is less sensitive to measurement errors. Consequently, it is recommended that a relative compaction criterion linked to ASTM D 1557 be adopted unless counterindicated by future laboratory or field testing of the borrow material.

The lateral earth pressures acting on structures and utilities that are not free to displace will be augmented by compaction and other equipment operating nearby or materials positioned nearby. These pressures are estimated in Section 10 for certain ranges of equipment and operational parameters. Attachment II includes definitions of classes of equipment and defines where the equipment can operate so as to be consistent with Section 10. Note that the guideline specifications in Attachment II do not allow heavy equipment, including compactors, close to structures or utilities in a lateral sense in order to limit stresses that could cause damage to the structures or utilities. In addition, the guideline specifications in Attachment II do not allow heavy equipment, including compactors, to operate or park above structures or utilities without the authorization of the Engineer, in order to limit stresses that could cause damage to the structures or utilities. This approach requires strict procedures and supervision to ensure that the construction crews do not operate a heavy compactor or heavy equipment closer to walls, structures, or utilities than is permitted.

Because the alluvium and engineered fill material are primarily sand and gravel, typical weather conditions at the site are expected to have minimal effect on these materials.

As noted in BSC (2002, Section 6.5.2) the engineered fill material from the Fran Ridge Borrow Area exhibited significant particle-size reduction as a result of the compaction characteristics test. Consequently, heavy construction equipment is expected to cause some breakdown of the alluvial material, though the degree of breakdown is often less in the field than is observed in the ASTM D 1557 compaction characteristics test.

8. MATERIAL PARAMETERS

To develop the geotechnical recommendations in this report, values for specific parameters are required for the three major subsurface materials: engineered fill, alluvium and bedrock. The values selected for these parameters are discussed in Attachment I and in this section. Table 3 summarizes the recommended parameter values as a convenient reference. However, users of Table 3 must have a thorough understanding of the limitations of these values, which can only be obtained by a thorough knowledge of the rest of this report and referenced data.

8.1 ENGINEERED FILL

As mentioned in Section 7, an engineered fill will be constructed to achieve the required grades. Based on a review of test results, the sand and gravel material available in the Fran Ridge Borrow Area appears to be acceptable as engineered fill material and is assumed to be the material source (Section 5, Assumption 3).

For the calculations to be performed in this report, the moist density, shear strength, compressibility characteristics, and interface friction coefficient of the engineered fill are required. In addition, a value of the coefficient of subgrade reaction is desired for subsequent use in structural calculations; as this parameter is not considered to be a basic soil property, it will be discussed in Section 11.

8.1.1 Moist Unit Weight

The moist unit weight of the engineered fill is needed for analyses of ultimate bearing capacity, passive resistance to lateral loads, lateral earth pressures acting on subterranean walls, and stability of permanent and temporary slopes.

The compaction characteristics of a composite sample of Fran Ridge Borrow Area material, which consisted of poorly graded sand with gravel, were measured in accordance with ASTM D 1557. The test was performed on the material after it had been scalped on the one-half inch sieve.[6] The compaction test results (BSC 2002, Figure 215; DTN: MO0203EBSCTCTS.016) indicate a maximum dry unit weight of 114.5 pounds-force per cubic foot (pcf or lbf/ft^3) and an optimum water content of 11 percent. These values for the one-half inch maximum material were adjusted to reflect the maximum dry unit weight that would have been measured had the complete minus 3-inch material been tested (the adjusted values would also be applicable if the complete material is crushed so that all the material passes the 1.5-inch sieve). Based on an estimate that the average relative compaction of the minus ¾-inch fraction of the as-constructed fill will be approximately 96 percent and the average water content will be one percentage point above the optimum water content, the average moist unit weight of the engineered fill would be

[6] The one-half inch size corresponds to the maximum particle size that was included in the static triaxial and the resonant column/torsion shear test specimens that were fabricated using a portion of the composite sample.

128 lbf/ft^3. See Section I.1.1 of Attachment I for details.

Table 3. Parameters for Subsurface Materials

Material/Parameter/Type of Analysis	Parameter Value(1, 2)	Section
ENGINEERED FILL		8.1, I.1
moist unit weight	128 lbf/ft^3	8.1.1, I.1.1
shear strength		8.1.2, I.1.2
general case	$\tau_{ff} = \sigma'_{ff} \tan(54° - 16° \log(\sigma'_{ff}/p_a))$	8.1.2, I.1.2.3
for passive pressure	$\tau_{ff} = \sigma'_{ff} \tan 55°$	8.1.2, I.1.2.4
for ultimate bearing capacity	$\tau_{ff} = \sigma'_{ff} \cdot 1.9655(\sigma_a/\sigma'_m)^{0.3331}$	8.1.2, I.1.2.5
for slope stability	$\tau_{ff} = 1.6636572(\sigma'_{ff})^{0.7543251}$ τ_{ff} and σ'_{ff} in kips/ft^2	8.1.2, I.1.2.6
compressibility	$E = 911.19(\sigma')^{0.4541}$ E and σ' in kips/ft^2	8.1.3, I.1.3
interface friction coefficient (3)	$\tan \delta = 0.55$	8.1.4, I.1.4
ALLUVIUM		8.2, I.2
moist unit weight	114 lbf/ft^3 in upper 8 ft; 117 lbf/ft^3 at deeper depths	8.2.1, I.2.1
shear strength		8.2.2, I.2.2
general case	$\tau_{ff} = \sigma'_{ff} \tan(39° - 3° \log(\sigma'_{ff}/p_a))$	8.2.2, I.2.2.1
for passive pressure for 55-foot deep wall	$\tau_{ff} = 169$ lbf/ft^2 + $\sigma'_{ff} \tan 36.5°$	8.2.2, I.2.2.2
for passive pressure for soldier piles	$\tau_{ff} = 78$ lbf/ft^2 + $\sigma'_{ff} \tan 42.4°$	8.2.2, I.2.2.3
for ultimate bearing capacity	$\tau_{ff} = \sigma'_{ff} \cdot 0.8299\ (\sigma_a/\sigma'_m)^{0.0486}$	8.2.2, I.2.2.4
for slope stability	$\tau_{ff} = 0.98808555(\sigma'_{ff})^{0.95450603}$ τ_{ff} and σ'_{ff} in kips/ft^2	8.2.2, I.2.2.5
compressibility	$E = 777.37(\varepsilon)^{-0.6505}\sigma^{0.5}$ E & σ' and kips/ft^2	8.2.3, I.2.3
interface friction coefficient (3)	$\tan \delta = 0.55$	8.2.4, I.2.4
BEDROCK		8.3, I.3
moist unit weight	100 lbf/ft^3	8.3.1, I.3.1
shear strength	greater than the overlying alluvium	8.3.2, I.3.2
compressibility	E = 55,000 kips/ft^2	8.3.3, I.3.3

Notes: (1) Some of the equations in Table 3 are the result of a regression analysis and may be shown with more digits than are justified by the precision of the data used. When the equations are used in a calculation, appropriate rounding of the final result should be performed.
(2) Symbols are defined in the remainder of Section 8 and in Attachment I.
(3) Ultimate values of interface friction coefficient between concrete cast-in-place on the subgrade.

8.1.2 Shear Strength

One set of four triaxial tests were performed on the minus one-half inch fraction of a composite sample from the Fran Ridge Borrow Area. The triaxial test specimens were isotropically consolidated to confining stresses ranging from 1.18 to 8.70 kips per square foot (kips/ft^2) and sheared under drained conditions. The initial conditions and results are summarized in BSC (2002, Table 28). Plots of deviator stress, change in volume, and obliquity (i.e., ratio of major to minor principal effective stresses, σ'_1/σ'_3) versus axial strain are presented on Figure 216 in BSC (2002). Mohr circles based on the peak deviator stress for each of the confining stresses are shown on Figure 217 in BSC (2002). DTN: MO0203EBSCTCTS.016 tabulates the test data.

The strength parameters from the drained triaxial tests can be represented in different ways. For this set of test results on the engineered fill, a significant difference is apparent between the linear (Mohr-Coulomb) envelope and various reasonable nonlinear strength envelopes. BSC (2002, Figure 217) reported that the Mohr-Coulomb strength envelope derived from a linear fit to the four test results is:

$$\tau_{ff} = c' + \sigma'_{ff} \tan\phi' = 1790\,\text{psf} + 0.7588\,\sigma'_{ff} \qquad \text{(Eq. 1)}$$

where: τ_{ff} = shear stress acting on the failure plane at failure, i.e., the effective shear strength
c' = effective cohesion intercept
ϕ' = effective friction angle
σ'_{ff} = normal effective stress acting on the failure plane at failure.

Note that this linear regression equation (Eq. 1) has a very large value of apparent cohesion for a material that would generally be considered "cohesionless." The apparent cohesion is the result of projecting the straight-line fit to the test data back to zero normal stress. In reality, coarse-grained granular soils normally have a curved failure envelope (Maeda and Miura 1999, page 53). There are several nonlinear strength functions in use to describe the shear strength envelope, and this report uses three, two of which are required for particular calculation methods. Although some nonlinear strength envelopes are capable of incorporating tensile strength, which would yield a cohesion intercept, tensile strength was taken to be zero for the engineered fill and alluvium. The nonlinear representations of strength are more suitable in general, but are difficult to apply in some calculation methods that were developed for use with the linear Mohr-Coulomb strength envelope. For use in some calculations, a Mohr-Coulomb strength envelope is developed for the range of stress that is associated with the calculation. This is generally done by fitting a straight line to one of the nonlinear envelopes over a selected range of normal stress. Supporting details for the shear strength envelopes are provided in Section I.1.2 of Attachment I.

The nonlinear failure envelope described by equations 2A and 2B is recommended for general use (see Section I.1.2.3 of Attachment I for supporting details):

$$\tau_{ff} = \sigma'_{ff} \tan\phi' \qquad \text{(Eq. 2A)}$$

where:

$$\phi' = \phi'_1 - \Delta\phi' \log\left(\frac{\sigma'_{ff}}{p_a}\right) = 54° - 16° \log\left(\frac{\sigma'_{ff}}{p_a}\right) \qquad \text{(Eq. 2B)}$$

where: τ_{ff} = shear strength (shear stress acting on the failure plane at failure)

σ'_{ff} = normal effective stress acting on the failure plane at failure

ϕ' = the effective friction angle at a particular value of σ'_{ff}

ϕ'_1 = the effective friction angle at σ'_{ff} = 1 atmosphere

$\Delta\phi'$ = the decrease in ϕ' per log cycle change in σ'_{ff}

p_a = 1 atmosphere (approximately 2.11622 kips/ft^2).

Note that equations 2A and 2B are based on triaxial compression tests. Experimental studies have shown that triaxial compression tests yield lower strength than such other shear strength tests as triaxial extension, plane strain compression and plane strain extension (Kulhawy and Mayne 1990, page 4-14).

For use in calculating the passive pressure that mat foundations and spread footings with an embedment in engineered fill of less than 8 feet could potentially develop, the overall range of stresses in the engineered fill was estimated and a linear failure envelope was fit to equations 2A/2B, yielding (see Section I.1.2.4 of Attachment I for supporting details):

$$\tau_{ff} = \sigma'_{ff} \tan 55° \qquad \text{(Eq. 3)}$$

For use in calculating bearing capacity by the method developed by Ueno et al. (1998), the following equation should be used (see Section I.1.2.5 of Attachment I for supporting details):

$$\tau_{ff} = \sigma'_{ff} \tan\phi' = 1.9655\, \sigma'_{ff} \left(\frac{\sigma_a}{\sigma'_m}\right)^{0.3331} \qquad \text{(Eq. 4)}$$

where: τ_{ff} = shear strength (shear stress acting on the failure plane at failure)

σ'_{ff} = normal effective stress acting on the failure plane at failure

ϕ' = the effective friction angle as a function of σ'_m

σ_a = 1 atmosphere pressure expressed in the same units as σ'_m

σ'_m = ½ $(\sigma'_1+\sigma'_3)$.

For use in calculating slope stability by the method developed by Charles and Soares (1984), the following should be used (see Section I.1.2.6 of Attachment I for supporting details):

$$\tau_{ff} = 1.6636572 (\sigma'_{ff})^{0.7543251} \qquad \text{(Eq. 5)}$$

where: τ_{ff} = shear stress acting on the failure plane at failure in kips/ft^2

σ'_{ff} = normal effective stress acting on the failure plane at failure in kips/ft^2.

Note that the value of the constant multiplier in equation 5 depends on the system of units being used, while the exponent is independent of the system of units.

8.1.3 Young's Modulus

Secant Young's modulus is needed for analyses of coefficient of subgrade reaction. The following equation is recommended:

$$E = 911.19(\sigma')^{0.4541} \qquad \text{(Eq. 6)}$$

where: E = secant Young's modulus in kips/ft^2
σ' = the initial isotropic consolidation stress prior to loading in kips/ft^2.

Note that units of kips and feet should be used with equation 6. Supporting details for equation 6 are provided in Section I.1.3 of Attachment I.

8.1.4 Interface Friction

Interface friction is needed for calculation of sliding resistance to horizontal loading to be performed at a later time. Based on values recommended in the literature for this type of soil, an ultimate interface friction coefficient of 0.55 is recommended for concrete cast-in-place on undisturbed engineered fill. Refer to Section 10.2 for a discussion about the use of this parameter. Supporting details are provided in Section I.1.4 of Attachment I.

8.2 ALLUVIUM

For the calculations to be performed in this report, the moist unit weight, shear strength, compressibility characteristics, and interface friction coefficient of the alluvium are required.

8.2.1 Moist Unit Weight

The moist unit weight of the alluvium is needed for analyses of ultimate bearing capacity, passive resistance to lateral loads, lateral earth pressures acting on subterranean walls, and stability of temporary slopes.

The data (except from drive tube samples) shown on Figure 236 of the BSC (2002) (DTNs: MO0112GPLOGWHB.001, GS020483114233.004, GS920983114220.001) were replotted as moist unit weight versus depth below top of alluvium (see Section I.2.1 of Attachment I for details). Based on the trends in the data, it is recommended that moist unit weights of 114 and 117 lbf/ft^3 be used above and below a depth of 8 feet below the original ground surface (before fill was placed), respectively, measured relative to the original ground surface before the existing fill was constructed. For some calculations requiring moist unit weight of alluvium, one value or the other may be selected for simplicity.

During development of the engineering properties it was noted that the unit weight (measured by the gamma-gamma survey) of the material logged as alluvium between depths of about 70 and 115 feet BGS in borehole UE-25 RF#21 is anomalously low for alluvium (see Section I.2.1 of

Attachment I for details). This could indicate that the engineering properties of the deep alluvium are different and less favorable than those of the shallower alluvium. This could also be explained by misidentification of the drill cuttings from borehole UE-25 RF#21, which was not otherwise sampled. For this calculation it is assumed (Section 5, Assumption 9) that the drill cuttings in borehole UE-25 RF#21 were misidentified and that the material is actually bedrock. This assumption is based on the lack of evidence of similar low-density alluvium near the base of the alluvial deposit in other boreholes and by absence of a low or decreased shear-wave velocity in the depth interval from 70 to 115 feet BGS in borehole RF#21 (BSC 2002, Figure 12). (Unfortunately, no suspension seismic measurements were made in this depth interval.) This assumption applies to the material properties developed in Section 8 and Attachment I.

8.2.2 Shear Strength

Shear strength is needed for analyses of bearing capacity, resistance to lateral loads, and lateral earth pressures acting on subterranean walls. Because relatively undisturbed samples of the alluvial material were not obtained, shear strength was evaluated on the basis of correlations with parameters measured by in situ tests.

Due to the dense, granular nature of the material, the drained shear strength is appropriate for general characterization and analyses. As discussed in Section 8.1.2, the shear strength, τ_{ff}, can be represented by several different equations for different applications, or a linear Mohr-Coulomb strength envelope can be developed for a particular range of normal stress.

The nonlinear failure envelope given by equation 7A and 7B is recommended for general use (see Section I.2.2.1 of Attachment I for supporting details):

$$\tau_{ff} = \sigma'_{ff} \tan\phi' \qquad \text{(Eq. 7A)}$$

where:

$$\phi' = \phi'_1 - \Delta\phi' \log\left(\frac{\sigma'_{ff}}{p_a}\right) = 39° - 3° \log\left(\frac{\sigma'_{ff}}{p_a}\right) \qquad \text{(Eq. 7B)}$$

where: τ_{ff} = shear strength (shear stress on the failure plane at failure)
σ'_{ff} = normal effective stress acting on the failure plane at failure
ϕ' = the effective friction angle at a particular value of σ'_{ff}
ϕ'_1 = the effective friction angle for σ'_{ff} = 1 atmosphere
$\Delta\phi'$ = the decrease in ϕ' per log cycle change in σ'_{ff}
p_a = 1 atmosphere (approximately 2.11622 kips/ft^2).

Note that equations 7A and 7B are based on triaxial compression tests. Experimental studies have shown that triaxial compression tests yield lower strength than such other shear strength tests as triaxial extension, plane strain compression and plane strain extension.

For use in calculating the passive pressure that a 55-foot deep below-grade wall (for the pool in the potential wet-process building) could potentially develop, the overall range of stresses in the alluvium was estimated and a linear failure envelope was fit to equations 7A and 7B, yielding (see Section I.2.2.2 of Attachment I for supporting details):

$$\tau_{ff} = 169 \text{ lbf/ft}^2 + \sigma'_{ff} \tan 36.5° \qquad \text{(Eq. 8)}$$

For use in calculating the passive pressure that soldier piles (for the 55-foot deep excavation for the pool in the potential wet-process building) embedded in alluvium (valid for embedment depths up to 20 feet) could potentially develop, the overall range of stresses in the alluvium was estimated and a linear failure envelope was fit to equations 7A and 7B, yielding (see Section I.2.2.3 of Attachment I for supporting details):

$$\tau_{ff} = 78 \text{ lbf/ft}^2 + \sigma'_{ff} \tan 42.4° \qquad \text{(Eq. 9)}$$

For use in calculating bearing capacity by the method developed by Ueno et al (1998), the following should be used (see Section I.2.2.4 of Attachment I for supporting details):

$$\tau_{ff} = \sigma'_{ff} \tan \phi' = 0.8299\, \sigma'_{ff} (\sigma_a/\sigma'_m)^{0.0486} \qquad \text{(Eq. 10)}$$

where: ϕ' = the effective friction angle as a function of σ'_m
σ_a = 1 atmosphere pressure, expressed in the same units as σ'_m
σ'_m = $\frac{1}{2}(\sigma'_1+\sigma'_3)$.

For use in calculating slope stability by the method developed by Charles and Soares (1984), the following should be used (see Section I.2.2.5 of Attachment I for supporting details):

$$\tau_{ff} = 0.98808555\, (\sigma'_{ff})^{0.95450603} \qquad \text{(Eq. 11)}$$

where: τ_{ff} = shear stress on the failure plane in kips/ft^2
σ'_{ff} = effective normal stress on the failure plane in kips/ft^2.

Note that the value of the constant multiplier in equation 11 depends on the system of units being used, while the exponent is independent of the system of units.

8.2.3 Young's Modulus

Secant Young's modulus is needed for analyses of coefficient of subgrade reaction. The following equation is recommended:

$$E = 777.37(\varepsilon)^{-0.6505}\sigma^{0.5} \qquad \text{(Eq. 12)}$$

where: E = secant Young's modulus in kips/ft^2
ε = axial strain in percent
σ = initial overburden stress in kips/ft^2.

Note that units of kips and feet should be used with equation 12. Supporting details are provided in Section I.2.3 of Attachment I.

8.2.4 Interface Friction

Interface friction is needed for calculation of sliding resistance to horizontal loading. Based on values recommended in the literature for sands and gravels, an ultimate interface friction coefficient of 0.55 is recommended for concrete cast-in-place on undisturbed alluvium (if the alluvium is disturbed by construction operations, it should be recompacted to the same relative density and water content as required for engineered fill). Refer to Section 10.2 for a discussion about the use of this parameter. Supporting details are provided in Section I.2.4 of Attachment I.

8.3 BEDROCK

For the calculations to be performed in this report, the moist unit weight, shear strength, and compressibility characteristics of the bedrock are required.

8.3.1 Moist Unit Weight

Moist unit weight of bedrock may be required for bearing capacity calculations. The moist and dry densities of the various bedrock units vary considerably (BSC 2002, Tables 12 and 34, Figures 101 and 235). The lower values of unit weight are associated with units that also have lower shear-wave velocity and which are considered to have lower shear strength. Hence, for bearing capacity calculations, it is recommended to use a moist unit weight of 100 lbf/ft^3, which corresponds to an average value for Tuff unit "x" (Tpki), the unit with the lowest unit weight (Section I.3.1 of Attachment I). Note, however, that the approach taken in Section 9.2 (bearing capacity) is to use a simplified subsurface representation wherein the bedrock is assumed to have the same properties as the overlying alluvium. This simplified representation can easily address the general issue of bearing capacity without specific foundation locations having been identified. For settlement calculations, it may be desirable to use the unit weight of whatever bedrock units are involved at a specific building location (not known at this time), but use of 100 lbf/ft^3 should be conservative for an elastic analysis.

8.3.2 Shear Strength

Shear strength of bedrock may be required for bearing capacity calculations. However, without a layout of the structures, it is not clear that bedrock needs to be considered in these types of analyses. Therefore, the approach taken in Section I.3.2 of Attachment I is to show that the bedrock can be expected to have greater strength than the overlying material, which would be either engineered fill or alluvium.

8.3.3 Young's Modulus

Young's modulus of bedrock is required for consideration in developing an approach to estimating a coefficient of subgrade reaction. Based on shear-wave velocity data, a secant

Young's modulus of 55,000 kips/ft^2 is recommended. Section I.3.3 of Attachment I provides supporting details.

9. BUILDING FOUNDATIONS, SLABS-ON-GRADE AND VAPOR BARRIERS

9.1 GENERAL ASPECTS OF FOUNDATIONS

Two principles that control the allowable bearing pressure for mat and spread footing foundations are (1) the stresses imparted to the soil by the foundation should not exceed the strength of the soil, and (2) the settlement of the foundation should be within limits tolerable to the structure. Settlement analysis was removed from the scope of work for this report; settlement of the foundations will be calculated at a later time to determine whether settlement controls the allowable bearing pressure.

9.1.1 Minimum Embedment

An additional principle for the design of shallow foundations is that the base of the foundation should be below the depth of potential frost penetration. The depth of frost penetration for foundation design was estimated based on a contour map of extreme frost penetration (USN 1986, Figure 7). Based on this map, it is interpreted that the potential depth of frost penetration is 10 inches. See Attachment III for supporting details.

Based on the types of foundation and structures, it is recommended that the mats and spread footings be embedded at least 24 inches below final grade. For footings on the structure perimeter, the embedment is measured with respect to the lower of the adjacent permanent exterior grade and the adjacent interior slab-on-grade (or if there is no interior slab-on-grade, then the interior ground surface). For interior footings, the embedment is measured with respect to the lowest adjacent permanent interior slab-on-grade (or adjacent interior ground surface if some of the area adjacent to the footing is not confined by a slab-on-grade).

9.1.2 Minimum Footing Width

Based on the type of foundation and structures, it is recommended that continuous wall and spread footings have a minimum width of at least 24 inches.

9.1.3 Uniform Bearing Material

At the present time, the locations of structures and their foundations are assumed (Section 5, Assumptions 1 and 2). Some of the footings may be founded on engineered fill and some on alluvium. For some structures, one or more of the footings might bear on both engineered fill and alluvium, or foundations in one part of the structure might bear on engineered fill while foundations in another part of the structure may bear on alluvium. In order to provide for more uniform bearing conditions in cases where different materials are encountered at the bottom of different footings within a particular structure or over the bottom of a mat foundation or individual footing, any alluvium within 3 feet of the bottom of such foundations should be removed and replaced with engineered fill. Such removal and replacement should extend laterally beyond the edges of the foundation a distance equal to the depth of removal below the foundation.

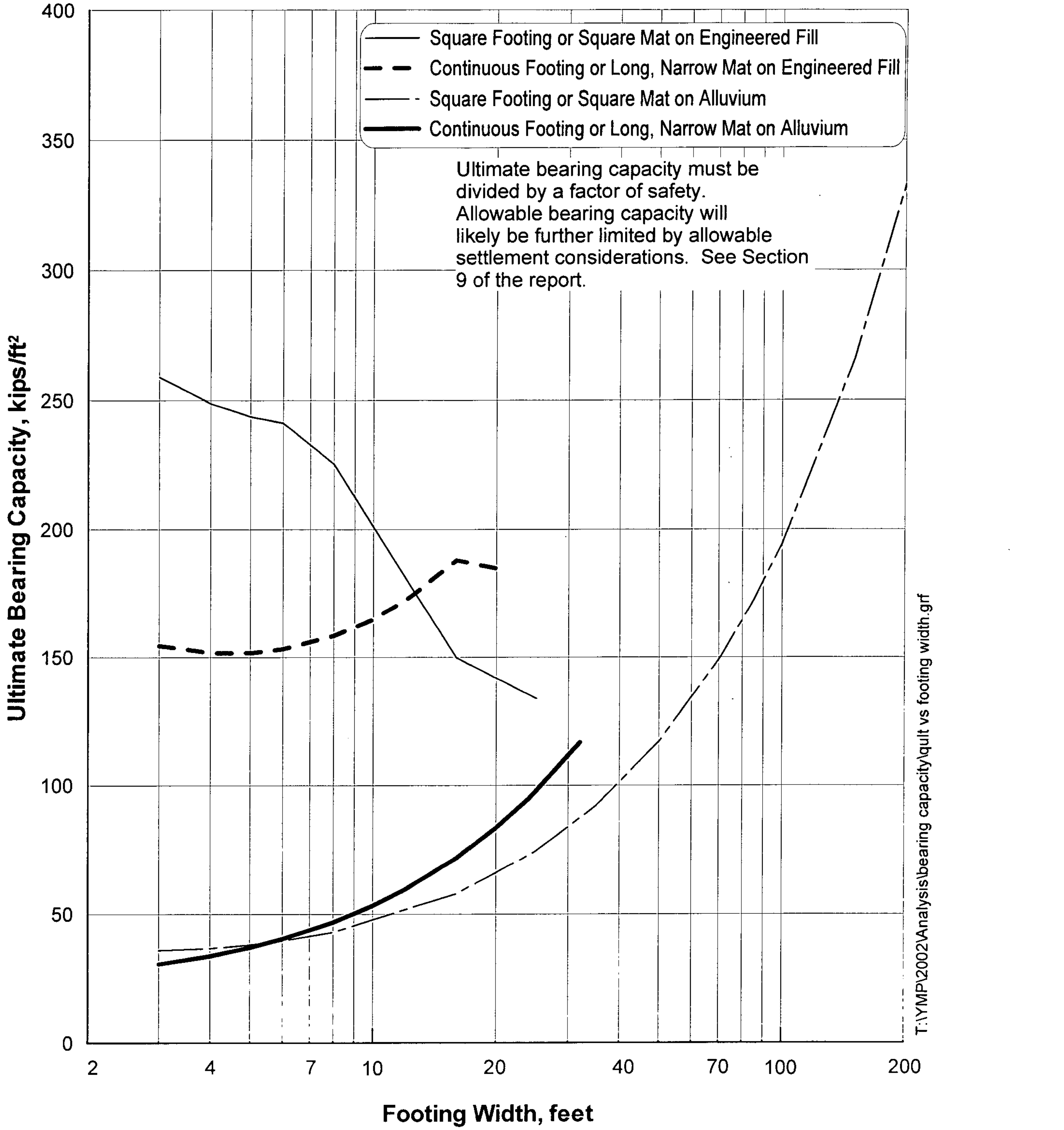
9.2 ULTIMATE BEARING CAPACITY

Figure 2 shows the ultimate bearing capacity under static conditions without consideration of settlement for square and strip foundations of various widths for a 2-foot embedment (see Attachment IV for calculation details). For embedment depths greater than 2 feet, these values are conservative. Separate curves are provided for the ultimate bearing capacity of foundations bearing on engineered fill and on alluvium. Because the ultimate bearing capacity of a foundation founded on engineered fill may be influenced by the underlying alluvium, the ultimate bearing capacity for a foundation bearing on engineered fill should be taken as the lesser of the ultimate bearing capacities obtained using the alluvium and engineered fill curves on Figure 2 if the top of alluvium is located at a depth below the bottom of the foundation that is less than 1.5 times the foundation width. Foundations bearing directly on bedrock are not anticipated. However, if a foundation will bear directly on bedrock, the ultimate bearing capacity for foundations bearing on alluvium may be used because the bedrock is at least as strong as the alluvium.

The ultimate bearing capacity must be divided by a factor of safety to determine an allowable bearing pressure. Based on the degree of site exploration data available and the consequences of failure, it is recommended taking a factor of safety of at least 5 for Quality Level 1 and 2 structures and at least 3.5 for other structures. These factors of safety can be reduced when confirmatory data are acquired (see Section 16.2). In addition, the allowable bearing pressure should not exceed a value that would cause the estimated settlement to exceed the allowable values of total and differential settlement. Settlement calculations are beyond the scope of this report and will be performed by others at a later time when the design details for the structures and their foundations are known.

The allowable bearing pressure can be increased by one-third for the seismic case. This is based on the International Building Code (International Code Council 2000, Table 1804.2) and on practice in southern California, which is a highly seismic area. For more elaborate analysis of seismic bearing pressure, a non-linear finite difference analysis is recommended.[7] Although simple to use, pseudostatic methods are not recommended because there is no basis for choosing an appropriate pseudostatic coefficient and the methods fail to incorporate the beneficial effect on soil resistance that results from the transitory, cyclic nature of the loading.

[7] Such a non-linear finite difference analysis can be performed using a program such as FLAC or FLAC3D. Use of the current version of the software is recommended.



Source: Attachment IV

Figure 2. Ultimate Bearing Capacity

9.3 SLABS-ON-GRADE AND VAPOR BARRIERS

Conventional reinforced concrete slab-on-grade floors may be used for the potential structures. The slab thickness and reinforcement should be designed by the structural engineer for the anticipated floor loads and other structural considerations. It is recommended that these floors be supported on a pad of compacted Engineered Fill. The Engineered Fill pad should extend at least 1 foot below the bottom of floor slabs and any vapor barrier sand. Thickened slab edges should be treated as a foundation (Section 9.1.3).

Any materials disturbed during construction, including during rebar placement, should be removed and replaced with Engineered Fill, properly moisture-conditioned to at least 95 percent relative compaction. The water content of subgrade soil should be maintained at a level slightly over its optimum water content until the slab is poured. At the time of concrete placement, the subgrade soil should be firm and relatively unyielding.

If a moisture-sensitive floor covering (such as tile) is planned on a slab-on-grade, it is recommended that the floor slab be underlain by a vapor barrier, such as an impermeable polyethylene membrane, at least ten mils thick. It may also be desirable to install an impermeable membrane under the building floor. Even at locations where the water table is very deep, as it is at the waste handling facilities site, water has been observed to collect under impervious surfaces, such as roadway pavements in desert environments in the southwestern United States. This water may originate as water vapor migrating upward, which is unable to escape to the atmosphere due to the presence of an impervious surface. This process may be exacerbated by the use of air conditioning in buildings. The purpose of a vapor barrier would be to prevent migration of water into and through the mats. Such water vapor may contain soluble salts, such as sulfates, leached from the soil. Some of these salts may affect the reinforced concrete mat or slab-on-grade. In addition, when the migrating water evaporates inside the building, the salts remain as an encrustation (efflorescence) that can affect floor coverings. This condition can occur even when standing water is not observed on the floor. Vapor barriers can also be helpful in reducing the entry of gases, such as radon, into buildings.

If an impermeable membrane is used, it should be placed on and covered by 2-inch thick layers of moistened (not saturated), clean sand to protect the membrane and promote concrete curing. The sand should have fewer than 5 percent of particles passing a U.S. Standard 75 micron sieve (U.S. Alternative No. 200 sieve), and no particles retained on a U.S. Standard 425 micron sieve (U.S. Alternative No. 4 sieve). The particle-size distribution curve of the sand should be smooth, with no gap grading. The particles should be hard and durable. It may be necessary to import clean sand meeting these requirements from off-site or to process on-site material.

Care should be taken not to puncture the impermeable membrane during construction. Any punctures, whether accidental or intentional, should be repaired before casting the mat. Particular attention should be paid as the mat rebar is being placed to ensure that the membrane is not punctured. The layer of sand beneath the membrane can be compacted before the membrane is placed, but the upper layer should be only lightly rolled by hand with equipment and methods that will not damage the membrane.

10. LATERAL EARTH PRESSURES AND RESISTANCE TO LATERAL LOADS

10.1 LATERAL EARTH PRESSURES ON BELOW-GRADE WALLS

At this time, the only identified potential below-grade wall is for the pool in the potential wet-process building, which will extend approximately 50 to 55 feet below the main floor level, and thus about 50 to 55 feet below final grade. The below-grade wall is not expected to be free to rotate about the base of the wall or to translate laterally during or after compaction of the wall backfill, so the wall will be considered restrained. If the below-grade wall were constructed directly against the natural alluvial deposits, the lateral earth pressures would correspond to the at-rest pressures in the alluvium. However, it is not practical to construct the below-grade wall directly against the natural alluvial deposits; backfill will be placed against the wall.[8] Consequently, the lateral earth pressures acting on the below-grade walls will be the at-rest lateral earth pressures augmented by the lateral earth pressures induced by the compaction equipment. Because the choice of compaction equipment should be left, within limits, to the contractor, the compactor-induced stresses on the below-grade wall were chosen to cover the range of compaction equipment and operational parameters that is consistent with the recommendations in Section 7 and Attachment II.

As discussed in Section 6.4 and BSC (2002, Section 6.6.3), the water table is deep. Consequently, no hydrostatic pressures will act on the wall, provided that adequate drainage is provided, as discussed in Section 13.

For the ranges of compaction equipment and operational parameters that are consistent with the recommendations in Section 7 and Attachment II, the recommended distribution of lateral earth pressure, $p_h(z)$, with depth, z, can be described by a series of line segments with the endpoints in Table 4:

Table 4. Lateral Earth Pressures Acting on Restrained Below-Grade Walls, Including Effects of Compactor-Induced Stresses

Depth (feet)	Pressure on Wall (lbf/ft^2)
0	0
0.5	610
14	760
60	2,850

Attachment V presents supporting details for this analysis and a plot of the pressure distribution in Table 4.

The permanent static lateral earth pressure at any depth can be linearly interpolated between the depths given in Table 4. In the depth interval from 0 to 0.5 feet, the pressure is limited by the strength of the backfill in passive failure. In the depth interval from 0.5 to 14 feet, the pressure is controlled by the stress induced by the compactor. At depths greater than 14 feet, the pressure is

[8] The pool wall could be constructed directly against the alluvium if the slurry trench technique is used to construct a diaphragm wall or if excavation shoring is used and the wall is constructed directly against the shoring, using the shoring as a form for the concrete. Neither possibility is considered likely.

controlled by the at-rest earth pressure and is not affected by the stress induced by the compactor. Note that the exact depth of the below-grade walls is not known, but is expected to be about 50 to 55 feet, including the foundation mat.

For walls that are under the interior of the building, as is expected for the pool walls, the "depth" in Table 4 is relative to the base of the mat and, as discussed below, structure loads, such as the weight of the mat and interior loads, should be treated as surcharge loads. If there are any below-grade walls at the perimeter of a building, the "depth" in Table 4 would be relative to the permanent ground surface, and any surcharge loads would be due to sources external to the structure, such as adjacent structures or parking. There will be a zone of transition between these two cases for walls that are not precisely at the building perimeter, but are close. These can be considered on a case-by-case basis during final design.

Lateral earth pressures will also result from surcharge loads placed near the walls. If a surcharge load, q_s, acts over a large area, the corresponding lateral earth pressure distribution can be estimated as a uniform horizontal pressure of magnitude $0.37 \cdot q_s$ acting on the subsurface wall and sides of the mat. However, to the extent that the lateral earth pressure due to the surcharge is less than the excess of the compactor-induced lateral earth pressure over the at-rest lateral earth pressure, the lateral earth pressure due to the surcharge is reduced. Thus, at any depth, z, the static lateral earth pressure distribution $p_h(z)$ would be the greater of the distribution given in Table 4 or by the following equation:

$$p_h \text{ (in lbf/ft}^2\text{)} = (47.5 \text{ lbf/ft}^3) \cdot z + 0.37 \cdot q_s \qquad \text{(Eq. 13)}$$

where q_s is the surcharge load in lbf/ft^2; and z is the depth in feet below the elevation at which the surcharge load is applied (DON 1986, Sections 4b and 5e of Chapter 3). Equation 13 is based on Duncan and Seed (1986, equation 3) and the value of K_0 shown in Step 12 in Attachment V.

If there is a temporary or permanent point load(s) placed behind the wall, the pressure induced on the wall can be evaluated using the charts in Design Manual 7.02 (DON 1986, p. 7.2-74). If there is a temporary or permanent distributed load(s) acting on a limited area behind the wall, the pressure induced on the wall can be evaluated using the charts in Design Manual 7.02 (DON 1986, p. 7.2-75). The principle of superposition can be used for multiple loads on the backfill surface.

Dynamic lateral pressures will also be imposed on the pool walls due to seismic shaking. At this time the seismic shaking level has not been determined. The dynamic lateral pressures are to be calculated at a later time. For that calculation, the pressure distribution described in this section and Attachment V will be the initial static condition.

At present, there is no information indicating that there will be any below-grade walls or retaining walls that are expected to be free to rotate about the base of the wall or to translate laterally during or after compaction of the wall backfill, such that active pressures would develop on the wall. However, this was considered in case it is needed. The active pressures on below-grade walls and retaining walls that are expected to be free to rotate about the base of the wall or to translate laterally during and after construction may be taken as the pressure exerted by an equivalent fluid with a unit weight of 27 lbf/ft^3. Supporting details are provided in Attachment V.

10.2 RESISTANCE TO LATERAL LOADS

Lateral forces applied to the structure will cause it to move laterally unless resisted. Some lateral forces are sustained, such as the lateral earth pressures discussed in Section 10.1. Other lateral forces are transient, such as the horizontal components of seismic forces. Resistance to lateral movement may be provided by friction between the base of the foundation and the subgrade or by passive pressure developed on below-grade elements of the structure. Passive resistance may develop against the sides of a mat, as well as against the basement walls for the pool in the potential wet-process building.

It is not anticipated that the calculations of resistance to lateral loads will include friction between the slab-on-grade and the subgrade. In any case, for either slabs-on-grade or mats, if an impermeable membrane is present, sliding resistance will almost certainly be controlled by the interface friction coefficient between the membrane and the adjacent materials (i.e., above and below the membrane), in which case the values recommended in this report would not apply. The actual interface friction coefficient will depend on both the choice of membrane and the adjacent materials. Laboratory testing of the combination of materials may be required to define the interface friction coefficient.

The friction coefficient acting at the interface between cast-in-place concrete foundations and an alluvial or engineered fill subgrade is discussed in Sections 8.1.4 and 8.2.4. The friction coefficient is the same for static and dynamic loading; however, the normal force may be reduced by the vertical component of the ground motion that acts coincidentally with the horizontal ground acceleration.

Unlike the friction coefficient, the passive pressure that can potentially develop under conditions of seismic shaking may be different than the passive pressure for static conditions (the same is true of active pressures acting on walls that are free to rotate or displace). It has been known for several decades that, when there is interface friction between the wall and the soil, the triangular failure wedge assumed by Coulomb for calculation of passive pressure under static conditions is unconservative relative to results using failure blocks with curved failure surfaces (see, e.g., Lambe and Whitman 1969, Section 13.4). Morrison and Ebeling (1995) showed that the same is true for the seismic case as solved by Mononobe-Okabe (for a review of the Mononobe-Okabe procedure, see Seed and Whitman (1970), Morrison and Ebeling (1995)). However, as the pseudostatic horizontal acceleration coefficient increases, the agreement between the Mononobe-Okabe procedure and more accurate procedures based on log spiral failure surface improves, and at some point they yield essentially the same result. Consequently, the Mononobe-Okabe procedure can be used if the pseudostatic horizontal acceleration coefficient is large.

The value of the pseudostatic horizontal acceleration coefficient used in the Mononobe-Okabe procedure depends on whether the wall is able and permitted to displace laterally or rotate about its base into the adjacent soil ("unrestrained" wall or foundation) or whether it is unable to freely displace or the designer wishes to limit these displacements ("restrained" wall or foundation).

For unrestrained walls, the American Association of State Highway and Transportation Officials (Barker et al. 1991, Section 4.7.4) recommends:

$$k_h = 0.5 \cdot a_h \qquad \text{(Eq. 14A)}$$

where: k_h = the dimensionless horizontal acceleration coefficient for use in pseudostatic analysis

a_h = the peak horizontal ground acceleration divided by one gravity

and for restrained walls:

$$k_h = 1.5 \cdot a_h \qquad \text{(Eq. 14B)}$$

If the passive pressure for resistance to sliding due to seismic loading is needed, the value of passive pressure may need to be reduced to reflect vertical ground acceleration, which would reduce the effective unit weight of the soil and also the dynamic passive earth pressure coefficient. For typical industrial and commercial projects, it is common practice to assume that the peak vertical acceleration will not occur at the same instant as the peak horizontal acceleration. This practice is based on the asynchrony of peak horizontal and vertical accelerations in recorded acceleration-time histories for historic earthquakes. In practice, the vertical acceleration is generally taken as zero. However, some projects may decide to incorporate additional conservatism in the design by including some level of vertical ground acceleration coincident with the peak horizontal ground acceleration.

As was discussed in Section 9.2 with respect to seismic bearing pressure, pseudostatic methods are simple to use, but have several shortcomings. In particular, pseudostatic methods do not incorporate the beneficial effect on soil resistance that results from the transitory, cyclic nature of the loading. In addition, although there is more precedent for selecting the pseudostatic coefficient for the earth pressure analysis than for the bearing capacity analysis, there is still a good deal of uncertainty in its choice. Consequently, if additional lateral resistance is required, performance of a non-linear finite difference analysis is recommended.

Recommended passive resistance factors are:

Table 5. Ultimate Passive Resistance Factors

Material	Case	Equivalent Fluid Unit weight lbf/ft^3	Minimum Extent of Flat Backfill ft
Engineered fill	Static	2,000	2.4 times foundation embedment
	Seismic	Use Morrison and Ebeling (1995), Eq. 1	Use Morrison and Ebeling (1995), Eq. 4
Alluvium	Static	850	2.0 times foundation/wall embedment
	Seismic	Use Morrison and Ebeling (1995), Eq. 1	Use Morrison and Ebeling (1995), Eq. 4

Note: See Attachment VI for supporting details.

Supporting details for this analysis are described in Attachment VI. For the passive pressure calculations, the presence of the upper foot of soil (either fill or native soil) should be considered nonexistent unless it is protected by pavement or concrete flatwork.

The movement required to develop full passive resistance has been studied experimentally and by analytic methods, such as the Finite Element Method. Fang (1991, Section 6.6) states that the movement required to develop full passive pressure are proportional to the height of the wall, at least as a first approximation. The movements required to reach full passive pressure are larger in loose, compressible soil than in denser, less compressible soil. The movement required to reach full passive pressure in dense to medium dense sand requires that the wall displace laterally into the soil or that the wall rotate about its base into the soil by 1 to 2 percent of the wall height. For a wall that is 50 feet high, very large movements (0.5 to 1 foot) would be required to develop the full passive pressure on the full height of the wall. However, when backfill is compacted directly against the sides of a foundation or wall, it is "prestressed" by the compactive effort, particularly at shallow depth, and can require little wall movement to yield pressures approaching the full passive pressure. Further, considering the magnitude of the passive pressure at depths greater than 10 to 20 feet, it is possible that the wall would bend when subjected to the passive pressures, allowing passive pressure to develop on the upper part of the wall, but not on the lower part.

Appropriate factors of safety should be applied to the ultimate values given for friction coefficient and passive pressure. In the calculation of sliding resistance, it is preferred practice to provide sliding resistance by base friction alone, without counting on passive resistance. If base friction alone is used, a factor of safety of at least 1.5 is recommended. If both base friction and passive resistance are utilized, a larger factor of safety (at least 2.0) should be used (DON 1986, p. 7.2-83). In any event, the structure should be designed structurally for the stresses that would result from base friction and passive resistance acting either separately or in combination. For example, one design case should assume that all the sliding resistance develops as passive pressure near the ground surface, and a second case should assume that all the sliding resistance develops as base friction. This will allow various possible pressure distributions on the structure to be considered, which may affect the design of the walls or interior members.

11. COEFFICIENT OF SUBGRADE REACTION FOR LOCALIZED LOADS

11.1 DEFINITION AND APPROACHES

The coefficient of vertical subgrade reaction, k or k_s, is defined as the ratio of the vertical pressure, q, acting at the foundation/subgrade contact at a point to the settlement, s, at the same point:

$$k = \frac{q}{s} \qquad \text{(Eq. 15)}$$

and has units of force per length cubed. This parameter is sometimes referred to (as in Misiak 2001) as the modulus of subgrade reaction, even though the units of k are not those of a modulus.

The value of k is often estimated by using published "guide" values of k (e.g., Terzaghi 1955, page 314; Scott 1981, Section 7.4.1). The published values offer only the most rudimentary discriminatory factors based on soil type (sand or clay) and soil density (loose or dense) and ignore other potentially significant factors such as foundation dimensions, depth of embedment, subsurface layering, and load level. Published "guide" values are generally only suitable for the least important foundations or for planning or preliminary calculations for more important foundations.

The value of k is occasionally estimated by extrapolating the results of plate load tests. This approach is unsatisfactory when the results of tests on 1-foot plates must be extrapolated to much larger foundations and when the foundation is embedded (but the plate is not).

Values of k for small foundations have been estimated based on a relationship developed by Vesíc in 1961 (Scott 1981, Section 5.2.4) between the secant Young's modulus, E, and k:

$$kB = 0.65 \cdot \sqrt[12]{\frac{EB^4}{E_f I}}\left(\frac{E}{1-\nu^2}\right) \qquad \text{(Eq. 16)}$$

where: E = secant Young's modulus of soil (force per length squared)
B = width of footing (length)
ν = Poisson's ratio (dimensionless)
E_f = stiffness of footing (force per length squared)
I = moment of inertia of footing (length to the fourth power).

One method addressing some of the complexities of subgrade reaction requires iterative calculations on the part of both the geotechnical engineer and the structural engineer. Initially, the structural engineer makes an estimate of the contact pressures acting at the base (not the top) of the mat and provides it to the geotechnical engineer. The geotechnical engineer then estimates a value of k using, for example, guide tables or compression test results. Note that the value of k is expected to vary under the mat. The geotechnical engineer also computes mat settlement. He provides this information to the structural engineer. The structural engineer uses the value of k to compute mat deflection using, for example, a finite element analysis. The structural engineer also computes the contact pressure at the base of the mat. If the mat deflection computed by the structural engineer does not match the settlement calculated by the geotechnical engineer, then either the contact pressures used by the geotechnical engineer are incorrect or the distribution of k is incorrect, or both. The structural engineer provides his contact pressure distribution results to the geotechnical engineer who performs a new settlement analysis. The geotechnical engineer and structural engineer together modify the values of k and the structural engineer again computes the mat deflections and contact pressure distribution. This process iterates until agreement is reached. With the advent of nonlinear finite difference software running on personal computers, this approach, like the coefficient of subgrade reaction, now seems cumbersome and dated.

More recently, Daloglu and Vallabhan (2000) presented a method that takes into account foundation stiffness, foundation size, and depth of soil. In this procedure, the parameter r, representing the effective length of the slab, is developed based on the flexural rigidity of the slab, D, the depth of the soil layer, H, and secant Young's modulus of the soil, E_s (Daloglu and Vallabhan 2000, equation 3):

$$r = \sqrt[4]{\frac{DH}{E_s}} \qquad \text{(Eq. 17)}$$

where: $D = \dfrac{E_f h^3}{12(1-\nu_f^{\,2})}$ (Scott 1981, equation 5.97)

h = thickness of the foundation slab

E_f = Young's modulus of the foundation slab

ν_f = Poisson's ratio of the foundation slab

Using the calculated value of r, along with the depth of soil and location within the slab, a nondimensional value of Winkler's coefficient, K_{nw}, is obtained from the charts provided by Daloglu and Vallabhan (2000). The coefficient of vertical subgrade reaction, k, can then be calculated using:

$$k = \frac{K_{nw} D}{r^4} \qquad \text{(Eq. 18)}$$

11.2 RECOMMENDATIONS

To take into account the depth of fill and footing size and stiffness, it is recommended that the method proposed by Daloglu and Vallabhan (2000) and described in Section 11.1 be used for all large mats and strip loads. For smaller foundations with simple loading, the method proposed by Vesíc in 1961 (Scott 1981, Section 5.2.4), also described in Section 11.1, may be used.

To determine the value of Young's modulus, E, it is recommended to use equation 6 in Section 8.1.3 for engineered fill and equation 12 in Section 8.2.3 for alluvium. The value of E should be evaluated at a depth below ground surface of $0.83 \cdot B + D_f$ (where B is the footing width and D_f is the embedment), but not greater than the combined depth of engineered fill, alluvium, and bedded tuff under the foundation.

The depth of $0.83 \cdot B + D_f$ at which Young's modulus, E, should be calculated, was estimated based on the Schmertmann settlement analysis method (Bowles 1996, page 323). In this method, the strain influence factor varies with normalized depth beneath the footing. For cohesionless soils, this factor is zero directly beneath the footing, increases linearly to 0.6 at a normalized depth (Z/B) of 0.5, and decreases linearly to zero at a normalized depth (Z/B) of 2. The depth of the center of mass of this curve is computed to be at a normalized depth (Z/B) of 0.83 beneath the footing, that is, at a depth of 0.83B. Adding to this the depth of footing embedment yields $0.83 \cdot B + D_f$.

For alluvium, the value of E will need to be estimated by an iterative process because the strain is determined by the analysis using k. For use with equation 16 or 17, strain can be estimated as the beam deflection divided by 1.66B.[9] The depth of the soil layer, H, should be taken as the depth to the top of Tiva Canyon Tuff (not including bedded tuffs or Tuff unit "x" that may overlie the Tiva Canyon Tuff, particularly in areas located east of the Exile Hill fault splay). The depth to Tiva Canyon Tuff may be estimated using Figures 224 to 232 in BSC (2002).

The depth of fill should be taken as the depth to the top of alluvium plus five feet (to allow for possible removal of topsoil and loose materials that may have been encountered during preparation for placement of the existing fill pad). The depth to the top of alluvium may be estimated from topographic maps developed before placement of the existing fill.

For unimportant foundations or for planning or preliminary calculations for more important foundations, a coefficient of subgrade reaction for a one-foot square rigid plate, k_{s1}, of 500 tons per cubic foot, the value recommended by Terzaghi (1955, page 314) for dense, dry or moist sand (similar to the site alluvium and engineered fill), may be used. For foundations that are larger than 12 inches square, the value of k should be derived from k_{s1} as described by Terzaghi (1955, page 314-315). For concrete slabs and mats subjected to a concentrated vertical load, the effective foundation size should be determined by a method such as proposed by Terzaghi (1955, pages 303-304). The effective foundation size and the value of k should also be adjusted for the effect of other nearby foundations and loads, if any (Terzaghi 1955, page 305-306).

12. CORROSION POTENTIAL

12.1 FERROUS METALS

12.1.1 Indices for the Potential for Corrosion of Ferrous Metals

The most common index for evaluating a soil's potential to corrode ferrous metals is the soil's electrical resistivity. However, other factors also influence corrosion, including water content, degree of aeration, pH, redox potential, chloride content, sulfate content, stray electrical currents in the soil, and harmful bacteria, fungi and other microorganisms (e.g., sulfate-reducing bacteria). Thus, there is no unique correlation between resistance and corrosion potential with respect to ferrous metals, although the soil resistivity parameter is very widely used in practice and generally considered to be the dominant variable in the absence of microbial activity. One generally adopted corrosion severity rating system for ferrous metals (Corrosion Source 2002) is:

Table 6. Corrosion Severity Ratings

Soil Resistivity (ohm cm)	Corrosivity Rating
>20,000	Essentially non-corrosive
10,000 to 20,000	Mildly corrosive
5,000 to 10,000	Moderately corrosive
3,000 to 5,000	Corrosive
1,000 to 3,000	Highly corrosive
<1,000	Extremely corrosive

9 The depth 1.66B is twice the depth below the bottom of the footing at which Young's modulus is evaluated (0.83B).

Sandy soils in an unsaturated regime typically have high resistivity and are therefore considered the least corrosive type of soil. Clay soils, especially those contaminated with saline water, are on the opposite end of the spectrum.

Another common corrosion potential classification system is described in AWWA (1989, Table 10-2). This system, based on soil classification and soil aeration, defines four soil groups; the members of each soil group have similar corrosion potential. Group I represents the least corrosion potential; it includes soils with good aeration marked by a deep water table and includes soil types such as: sands; sandy loams; light, textured silt loams; and porous loams or clay loams thoroughly oxidized to great depths. At the other end of the corrosion potential spectrum, Group IV represents "unusually corrosive" soils; it includes soils with very poor aeration marked by a water table at the ground surface and includes soils types such as muck, peat, tidal marsh deposits, clays and organic soils, and adobe clay.

Although the details of construction are not known at this time, it is anticipated that the alluvium and engineered fill may come in contact with ferrous metals. At present, few data have been collected concerning either material due to the lack of an identified Q vendor to perform testing.

12.1.2 Alluvium

As discussed in Section 6.2, the alluvium is coarse-grained (sands and gravels). Lesser amounts of poorly graded sand with silt (SP-SM), well-graded sand with silt (SW-SM), silty sand (SM) and silty gravel (GM) were also observed. The fines content (percent passing the 75-micron [No. 200] sieve) of soil samples from the test pits and boreholes are generally low (between 3 and 20 percent). The water content of in situ alluvial deposits appears to be low to moderate. Data from water content tests performed on samples from TP-WHB-1 through -4 (BSC 2002, Table 6) tend to confirm this, particularly considering that the crew performing tests in the pits indicate that they sprayed the tests pits to suppress dust and discharged water from the in situ ring density tests onto the floor of the pits, which could have increased the water content of samples taken at lower elevations. The alluvium will be located over 1,000 feet above the water table, which is located at a typical depth of 1270 feet below the present ground surface (BSC 2002, Section 6.6.3).

Based on this description of the alluvium and groundwater conditions, the alluvium can be classified as belonging in "Group I – Lightly Corrosive" using the corrosion potential classification system described in AWWA (1989, Table 10-2).

Field electrical resistivity testing was performed along eight lines (ER-1 to ER-8) in the North Portal area on May 28 and 29, 1992, before construction of the North Portal construction-support pad. The measurement of electrical resistivity was made using an ABEM® Terrameter in accordance with IEEE STD 81-1983, *IEEE Guide for Measuring Earth Resistivity, Ground Impedance, and Earth Surface Potentials of a Ground System*. Several methods are described in IEEE 81-1983; the Wenner arrangement with four equally spaced electrodes was used for all these measurements. Measurements were made at electrode spacings of 1, 2, 3, 4, 5, 6, 8, 10, 12 and 15 meters. The calculated values of apparent resistivity ranged from 60 to 540 ohm-meters (6,000 to 54,000 ohm-centimeters) (DTN: GS930283114233.001). The temperature at the time of the measurements was not reported; however, resistivity changes only slightly with temperature at temperatures above the freezing point of water (0° C). At the time of year that the fieldwork was performed (May), the soil temperature was almost certainly above 0° C. No interpretations (i.e., conclusion with respect to corrosion) of the field data were found.

Based on this information, the potential for the alluvium to corrode ferrous metals is judged to be moderate to insignificant.

12.1.3 Engineered Fill

As discussed in Section 6.5 of BSC (2002), the potential borrow source for engineered fill at the base of Fran Ridge consists of coarse-grained (gravel and sand) alluvial deposits. The fines content of the single composite sample from the borrow pit was low (5 percent). The fill will be located high above the water table, which is located at a typical depth of 1270 feet below the present ground surface (BSC 2002, Section 6.6.3).

Based on this description of the alluvium and groundwater conditions, the alluvium can be classified as belonging in "Group I – Lightly Corrosive" using the common corrosion potential classification system discussed in Section 12.1.1.

Based on this information, the potential for the fill to corrode ferrous metals is judged to be low.

12.2 CONCRETE

The properties of concrete can be degraded by exposure to an aggressive chemical environment. Among the most aggressive chemicals identified are various salt solutions (aluminum chloride, ammonium nitrate, ammonium sulfate, sodium sulfate, magnesium sulfate and calcium sulfate) and various organic acids, inorganic acids, alkaline solutions and other substances (ACI 201.2R-92 (Reapproved 1997), Table 2.1). At present, there is not enough data to evaluate the degree of soil aggressivity to concrete. As a minimum, laboratory tests to determine sulfate content, chloride content, and pH should be performed on samples of the potential borrow material and the alluvium to provide data to evaluate the degree of soil aggressivity to concrete.

13. SUBSURFACE DRAINAGE

Due to the absence of a groundwater table in the depths of interest for the waste handling facilities structures, subsurface drainage is required only to protect against surface water infiltration and as a precautionary measure against the unexpected, such as water line breaks or irrigation. As noted in Section 4.2, project criteria require that the ground surface slope between 2 and 3 percent. Although not stated, the purpose of this criterion is undoubtedly to provide conditions in which surface runoff will effectively drain off the waste handling facilities area. However, this criterion could be met and drainage could still be poor if the grading plan does not take into account buildings and other obstacles that could retard drainage. Thus, if all the ground surface contours are parallel, as shown on Figure 1, and one side of a building is parallel to the contours, surface water will flow from Exile Hill to the building and will have to flow around the side of the building along a flat gradient. Thus, the first step in addressing the issue of subsurface drainage is to provide an efficacious surface drainage scheme.

The finished grade adjacent to the potential structures should be sloped down and away from the structures, to reduce the potential for water infiltration beneath the structures. Roof drainage should be collected in roof gutters and safely conveyed by downspouts to a storm drain or lined drainage channel/ditch. Wherever possible, a ditch constructed of erosion-resistant materials (or equivalent measures) should be constructed at the tops of slopes. Drainage should be directed to appropriate discharge areas or pipes via non-erosive devices. A regular maintenance program should be implemented to keep drainage in good working condition.

Landscaping and irrigation should be kept to a minimum level to reduce the potential for water infiltration. Only drought-tolerant xerophytes should be considered for landscaping. If irrigation piping is used, a means of detecting leaks should be provided or the piping should be placed above ground. Automatic sprinkler controls should not be used unless there is a means of detecting system deficiencies such as broken sprinkler heads, leaks, and failure to turn off.

In addition, drainage should be provided adjacent to all below-grade building walls and retaining walls and under all building basement floors. A minimum thickness (measured horizontally) of 3 feet of Permeable Material should be placed adjacent to below-grade building walls and retaining walls, and a minimum thickness (measured vertically) of 12 inches of Permeable Material should be placed below all building basement floors. A means of removing water (discharge line) from the zone of Permeable Material must be provided. Buildings that have no basement areas do not require Permeable Material.

Wall backfill should be protected from surface water infiltration with a layer of relatively impermeable material (such as the onsite clayey sand/sandy clay) at least 18 inches thick placed at the ground surface. Alternatively, the area may be paved with portland cement concrete or asphaltic concrete.

If the Permeable Material is manufactured onsite, a suggested gradation is given in Table 7, based on the gradation given in the table in Section G.33 of Duncan et al. (1987):

Table 7. Potential Gradation for Permeable Material

Sieve Size	Percentage Passing
37.5 mm (1½ inches)	100
19.0 mm (¾ inch)	75-100
9.5 mm (3/8 inch)	50-100
4.75 mm (No. 4)	25-60
2.36 mm (No. 8)	0-30
1.18 mm (No. 16)	0

If the Permeable Material is imported from offsite, other gradations may be considered to take advantage of standard gradations available locally. The basic criteria for the Permeable Material are that the material be highly pervious and that it be filter-compatible with the surrounding soil.

As noted in BSC (2002, Section 6.5.2), the Fran Ridge Borrow Area material exhibited significant breakdown (particle size reduction) during laboratory compaction. While this is not a concern relative to using the Fran Ridge Borrow Area material to construct Engineered Fill, it is a concern for using the material to construct Permeable Material, where the increase in the fines content will cause a reduction in the material's hydraulic conductivity. A testing program should be performed on whatever material might be proposed for use as Permeable Material (currently, no source has been discussed).

14. PERMANENT SLOPES

Although a detailed grading plan is not yet available, based on criteria and assumptions (Section 5, Assumption 2) the final grade contours have been estimated and are shown on Figure 1. Figure 1 indicates that some permanent slopes, up to about 25 feet high, may be required. The permanent slopes would be constructed of engineered fill on the native alluvial deposits. Attachment II recommends that any engineered fill slopes be keyed into the underlying material. No permanent slopes in the native alluvial deposits are expected, so these have not been considered in this report.

The maximum allowable inclination for permanent engineered fill slopes under static conditions was examined by using slope stability charts and requiring a factor of safety of 1.5. Attachment VII presents details of this calculation. This calculation indicates that the slope inclination can theoretically be very steep (steeper than 0.5:1 [horizontal to vertical] for slopes up to 25 feet high). However, the outer part of a slope can be difficult to compact. Further, a project criterion (Section 4.2) stipulates that permanent slopes be no steeper than 2:1 (horizontal to vertical). In addition, design slopes may need to be flatter to accommodate seismic forces (the seismic calculation is not part of the scope of this report). Therefore, it is recommended that the design slopes be 2:1 (horizontal to vertical) or flatter.

15. TEMPORARY EXCAVATIONS

Currently, the only required construction excavation that has been identified is for the pool in the potential wet-process building. As mentioned in Section 5, the pool excavation is expected to reach a depth of about 55 feet below the finish floor elevation. Although it is assumed (Section 5, Assumption 7) that the pool will be constructed after the existing fill is removed, the pool excavation could be made before or after the engineered fill is constructed. Further, the pool excavation could be made in an unshored excavation with sloping sideslopes or in a shored excavation. If the excavation is made before the engineered fill is constructed, it is assumed that the excavation will involve only alluvium, whereas if the engineered fill is constructed before the excavation is made, it is assumed that the excavation will involve engineered fill and alluvium. The unshored excavation alternative is discussed in Section 15.1, and the shored excavation alternative is discussed in Section 15.2.

Groundwater at the site is deep; consequently a need for dewatering is not expected. The contractor must, of course, erect efficacious barriers and diversions to prevent surface runoff from entering the excavations.

15.1 TEMPORARY SLOPES

Where space permits and provided that adjacent structures, utility lines, etc. are adequately supported, an unshored excavation with sloping sideslopes may be considered for construction of the pool. Based on the slope stability analysis in Attachment VII and engineering judgement, sideslopes for the pool excavation should be constructed at an inclination no steeper than 1¼:1 (horizontal to vertical) in alluvium and 1:1 in engineered fill. An 8-foot wide (minimum) bench should be constructed at the mid-depth of the excavation. Supporting details are presented in Attachment VII. Flatter slopes should be provided during construction if field conditions so dictate. Although these slopes should be generally stable under temporary conditions, it is possible that some cobbles and boulders may work out of the slope face and plummet down the slope to the bottom of the excavation. The contractor should carefully examine the slopes on a periodic basis and correct any potential safety hazard.

No surcharge loads should be imposed within a horizontal distance of the top of the temporary excavation slope that is equal to the depth of excavation. Surcharge loads include loads from buildings, equipment, construction materials, stockpiled excavated materials, vehicle parking, and traffic. If it is found desirable to impose surcharge loads within this distance, then additional slope stability calculations should be performed. Note that stability calculations including surcharge loads cannot be performed with chart solutions; consequently, appropriate software would need to be qualified for use.

It is not expected that groundwater will be encountered in the excavations. Surface drainage should be controlled by berms or other measures along the top of temporary excavations to prevent surface runoff from entering the excavation. Even with the implementation of these recommendations, some surface sloughing of the temporary excavation slopes may still occur, and workers should be adequately protected. Construction contract documents should make it clear that the Contractor is solely responsible for all aspects of safety on the construction site, including excavation safety. In addition, excavations should be performed in accordance with applicable local, state, and federal regulations and in such manner that excessive ground movement will not occur.

15.2 TEMPORARY SHORING

Temporary shoring should be installed where adequate space is not available for a sloped excavation. Design and installation of any shoring system should be made the sole responsibility of the Contractor.

Settlement and horizontal movement of a structure or facility located near the shoring will occur in proportion to both the distance between the shoring and the facility, and the amount of horizontal deflection of the shoring system. The movements will be greatest near the shoring face and decrease with horizontal distance from the shoring.

Prior to excavation, structures and utilities located within a horizontal distance equal to twice the depth of excavation should be observed to evaluate their pre-construction conditions. During the course of construction, deflection of the shoring system should be measured on a frequent (daily) basis. In addition, the shoring system and adjacent structures should be periodically inspected for signs of distress. Floors and pavements should be surveyed before and during construction to determine the amount of movement, if any, resulting from construction activities. In the event that distress or settlement is noted, an investigation should be performed and corrective measures taken so that continued or worsened distress or settlement is mitigated.

Temporary shoring, such as soldier piles and lagging, can be used to provide support for vertical excavations. Considering the coarse and dense nature of the alluvium, driven sheet piles and piles are not considered suitable at this site.

For design of soldier piles spaced at least 3 diameters on center, the ultimate passive resistance, $P_{p,ult}$, to substantial movement of a soldier pile is approximately (see Attachment VIII for details):

$$P_{p,ult} = 3\ B\ K_p\ (\tfrac{1}{2}\,\gamma\,(H\text{-}B)^2) \qquad \text{(Eq. 19)}$$

where: B = width of the soldier pile
H = depth over which the soldier pile moves enough to develop passive resistance, measured from the bottom of excavation
γ = moist unit weight of the soil developing passive pressure (117 lbf/ft^3 for the alluvium)
K_p = $\tan^2(45° + \phi/2)$
ϕ = internal friction angle of soil (42° for the alluvium for embedments up to 20 feet – see Section I.2.2.3 of Attachment I)

The ultimate passive force should be divided by an appropriate factor of safety selected as a function of the movement expected over the pile length where the passive pressure is developed, since substantial pile movement is required to develop full passive pressure and catastrophic failure may result if the passive resistance is exceeded.

The passive resistance is developed over the cross-sectional area of the soldier piles; for cylindrical piles, this will be the nominal pile diameter times the depth over which passive resistance is developed. Equation 19 effectively lowers the ground surface elevation by one soldier pile diameter for the purposes of computing passive resistance. Supporting details for this analysis are described in Attachment VIII.

The portion of the soldier piles below the adjacent excavation bottom should be concreted to assure firm contact between the pile and supporting soils. To develop firm contact between the upper portion of the shoring and the retained soils, the upper portion of the soldier pile excavation should be filled with concrete, a lean mix of concrete, or cement slurry. To limit sloughing and caving, continuous lagging should be used between soldier piles from the ground surface to the bottom of excavation. Timber lagging should be pressure-treated if it is to remain in place. The use of shotcrete in place of lagging is not recommended. If there is any void space behind the lagging, it should be filled with pea gravel as the lagging is placed.

Given that the shoring system will support a 55-foot high excavation face (or a 35- to 55-foot high excavation face if the excavation is made and the building completed after the existing fill is removed and before the engineered fill is placed), cantilever shoring is not likely to be feasible, and a tied-back or braced excavation will likely be required. If tiebacks are used, it is recommended that the tieback excavations be cased to avoid caving and that the tiebacks be pressure-grouted. Pressure-grouted anchors are sensitive to the details of construction employed by the contractor; hence, the contractor should select the bond strength and this should be demonstrated by load testing. All anchors should be proof-tested, and a percentage should be subject to a longer-duration, higher-load performance test.

The Contractor should design the temporary shoring. Construction documents should make it clear that the Contractor is solely responsible for all aspects of safety on the construction site, including excavation and shoring.

16. RECOMMENDATIONS

16.1 RECOMMENDATIONS

Section 16.1 compiles the primary recommendations made in Sections 7 through 15 and the attachments. The section from which the recommendation comes is indicated in square brackets at the end of the paragraph. The referenced section should be consulted for necessary details explaining the reasons for the recommendation or the range of application, and to avoid misapplication of the recommendation.

Attachment II provides recommendations for the engineered fill in the form of guideline specifications. As the entire attachment is a recommendation, it is not repeated here.

It is recommended that the engineered fill consist of particles that pass a 37.5 mm U.S. Standard Sieve (1½ inch U.S. Alternative Sieve), with no more than 20 percent of particles retained on the 19.0 mm U.S. Standard Sieve (3/4-inch U.S. Alternative Sieve). [Section 7]

It is recommended that a relative compaction criterion linked to ASTM D 1557 be adopted unless counterindicated by future laboratory or field testing of the borrow material. [Section 7]

Heavy equipment, including compactors, should not be allowed close to structures or utilities in a lateral sense in order to limit stresses that could cause damage to the structures or utilities (see Attachment II for distance restrictions). Heavy equipment, including compactors, should not be allowed to operate or park above structures or utilities without the authorization of the Engineer, in order to limit stresses that could cause damage to the structures or utilities. [Section 7]

The geotechnical parameters summarized in Table 3 and explained in Section 8 are recommended for use in analyses. [Section 8]

Based on the types of foundation and structures, it is recommended that the mats and spread footings be embedded at least 24 inches below final grade. For footings on the structure perimeter, the embedment is measured with respect to the lower of the adjacent permanent exterior grade and the adjacent interior slab-on-grade (or if there is no interior slab-on-grade, then the interior ground surface). For interior footings, the embedment is measured with respect to the lowest adjacent permanent interior slab-on-grade (or adjacent interior ground surface if some of the area adjacent to the footing is not confined by a slab-on-grade). [Section 9.1.1]

Based on the type of foundation and structures, it is recommended that continuous wall and spread footings have a minimum width of at least 24 inches. [Section 9.1.2]

For some structures, one or more of the footings might bear on both engineered fill and alluvium, or foundations in one part of the structure might bear on engineered fill while foundations in another part of the structure may bear on alluvium. In order to provide for more uniform bearing conditions in cases where different materials are encountered at the bottom of different footings within a particular structure or over the bottom of a mat foundation or individual footing, any alluvium within 3 feet of the bottom of such foundations should be removed and replaced with engineered fill. Such removal and replacement should extend laterally beyond the edges of the foundation a distance equal to the depth of removal below the foundation. [Section 9.1.3]

For embedment depths of 2 feet or greater, the ultimate bearing capacity under static conditions without consideration of settlement may be taken as shown on Figure 2 for square and strip foundations of various widths. Separate curves are provided for the ultimate bearing capacity of foundations bearing on engineered fill and on alluvium. Because the ultimate bearing capacity of a foundation founded on engineered fill may be influenced by the underlying alluvium, the ultimate bearing capacity for a foundation bearing on engineered fill should be taken as the lesser of the ultimate bearing capacities obtained using the alluvium and engineered fill curves on Figure 2 if the top of alluvium is located at a depth below the bottom of the foundation that is less than 1.5 times the foundation width. Foundations bearing directly on bedrock are not anticipated. However, if a foundation will bear directly on bedrock, the ultimate bearing capacity for foundations bearing on alluvium may be used because the bedrock is at least as strong as the alluvium. [Section 9.2]

It is recommended that the ultimate bearing capacity be divided by a factor of safety of at least 5 for Quality Level 1 and 2 structures and at least 3.5 for other structures. In addition, the allowable bearing pressure should not exceed a value that would cause the estimated settlement to exceed the allowable values of total and differential settlement. [Section 9.2]

The allowable bearing pressure can be increased by one-third for the seismic case. For more elaborate analysis of seismic bearing pressure, a non-linear finite difference analysis is recommended. [Section 9.2]

Conventional reinforced concrete slab-on-grade floors may be used for the potential structures. The slab thickness and reinforcement should be designed by the structural engineer for the anticipated floor loads and other structural considerations. It is recommended that these floors be supported on a pad of compacted Engineered Fill. The Engineered Fill pad should extend at least 1 foot below the bottom of floor slabs and any vapor barrier sand. Thickened slab edges should be treated as a foundation (Section 9.1.3). [Section 9.3]

Any materials disturbed during construction, including during rebar placement, should be removed and replaced with Engineered Fill, properly moisture conditioned to at least 95 percent relative compaction. The water content of subgrade soil should be maintained at a level slightly over its optimum water content until the slab is poured. At the time of concrete placement, the subgrade soil should be firm and relatively unyielding. [Section 9.3]

If a moisture-sensitive floor covering (such as tile) is planned on a slab-on-grade, it is recommended that the floor slab be underlain by an impermeable polyethylene membrane, at least ten-mils thick. It may also be desirable to install impermeable membrane under the buildings to act as a vapor barrier. [Section 9.3]

If an impermeable membrane is used, it should be placed on and covered by 2-inch thick layers of moistened (not saturated), clean sand to protect the membrane and promote concrete curing. The sand should have fewer than 5 percent of particles passing a U.S. Standard 75 micron sieve (U.S. Alternative No. 200 sieve), and no particles retained on a U.S. Standard 425 micron sieve (U.S. Alternative No. 4 sieve). The particle-size distribution curve of the sand should be smooth, with no gap grading. The particles should be hard and durable. [Section 9.3]

Care should be taken not to puncture the impermeable membrane during construction. Any punctures, whether accidental or intentional, should be repaired before casting the mat. Particular attention should be paid as the mat rebar is being placed to ensure that the membrane is not punctured. The layer of sand beneath the membrane can be compacted before the membrane is placed, but the upper layer should be only lightly rolled by hand with equipment and methods that will not damage the membrane. [Section 9.3]

For below-grade walls and retaining walls that are not expected to be free to rotate about the base of the wall or to translate laterally during and after construction, and for the ranges of compaction equipment and operational parameters that are consistent with the recommendations in Section 7 and Attachment II, the recommended distribution of lateral earth pressure, $p_h(z)$, with depth, z, can be described by a series of line segments with the endpoints in Table 4, if there are no surcharge loads acting on the backfill. The meaning of "depth" as used in Table 4 is described in Section 10.1. If there are surcharge loads acting on the backfill, the lateral earth pressure that should be used at any depth is the greater of the pressure from Table 4 or equation 13. For surcharge pressures other than uniform areal loads, standard charts (e.g., DON 1986, page 7.2-73 to 7.2-75) and the principle of superposition can be used. These lateral earth pressures should be used as the initial static condition for dynamic analyses. [Section 10.1]

The active pressures on below-grade walls and retaining walls that are expected to be free to rotate about the base of the wall or to translate laterally during and after construction may be taken as the pressure exerted by an equivalent fluid with a unit weight of 27 lbf/ft^3. [Section 10.1]

Resistance to unlimited lateral movement of structures in response to lateral loading may be provided by friction between the base of the foundation and the subgrade or by passive pressure developed on below-grade elements of the structure. The ultimate friction coefficient acting at the interface between cast-in-place concrete and an alluvial or engineered fill subgrade may be taken as 0.55. It is not anticipated that the calculations of resistance to lateral loads will include friction between the slab-on-grade and the subgrade. In any case, for either slabs-on-grade or mats, if an impermeable membrane is present, sliding resistance will almost certainly be controlled by the interface friction coefficient between the membrane and the adjacent materials (i.e., above and below the membrane), in which case the values recommended in this report would not apply. The actual interface friction coefficient will depend on both the choice of membrane and the adjacent materials. Laboratory testing of the combination of materials may be required to define the interface friction coefficient. [Section 10.2]

Recommended ultimate passive resistance factors are provided in Table 5. For unrestrained and restrained walls, the pseudostatic coefficient may be taken as 0.5 a_h and 1.5 a_h, respectively, where a_h is the peak horizontal ground acceleration for design. If additional lateral resistance is required for the seismic case, performance of a non-linear finite difference analysis is recommended. [Section 10.2]

The values of ultimate passive resistance are based on a backfill that is horizontal to a significant distance from the surface where passive resistance is developed. The movement required to reach full ultimate passive pressure in dense to medium dense sand requires that the wall displace laterally into the soil or that the wall rotate about its base into the soil by 1 to 2 percent of the wall height. If the wall is not capable of such displacement, or if displacements of this magnitude are not desirable, then the passive resistance should be appropriately reduced. For design, appropriate factors of safety should be applied to the ultimate values given for interface friction coefficient and passive pressure. The structure should be designed structurally for the stresses that would result from base friction and passive resistance acting either separately or in combination. [Section 10.2]

For unimportant foundations or for planning or preliminary calculations for more important foundations, a coefficient of subgrade reaction, k, of 500 tons per cubic foot, the value recommended by Terzaghi (1955, page 314) for dense, dry or moist sand (similar to the site alluvium and engineered fill), may be used for one foot by one foot foundations. For foundations larger than 12 inches square, the value of k should be adjusted as described by Terzaghi (1955, page 314-315). For concrete slabs and mats subjected to a concentrated vertical load, the effective foundation size should be determined by a method such as proposed by Terzaghi (1955, pages 303-304). The effective foundation size and the value of k should also be adjusted for the effect of other nearby foundations and loads, if any (Terzaghi 1955, page 305-306). [Section 11.2]

To take into account the depth of fill and footing size and stiffness, it is recommended that the method proposed by Daloglu and Vallabhan (2000) and described in Section 11.1 be used for all large mats and strip loads. For smaller foundations with simple loading, the method proposed by Vesíc in 1961 (Scott 1981, Section 5.2.4), also described in Section 11.1, may be used. [Section 11.2]

To determine the value of Young's modulus, E, for use in coefficient of subgrade reaction calculation, it is recommended that equation 6 in Section 8.1.3 be used for engineered fill and equation 12 in Section 8.2.3 for alluvium. The value should be evaluated at a depth of $0.83 \cdot B+D_f$ (where B is the footing width and D_f is the embedment), but not greater than the combined depth of engineered fill, alluvium and bedded tuff under the foundation. [Section 11.2]

Provide an efficacious surface drainage scheme. [Section 13]

The finished grade adjacent to the potential structures should be sloped down and away from the structures, to reduce the potential for water infiltration beneath the structures. Roof drainage should be collected in roof gutters and safely conveyed by downspouts to a storm drain or lined drainage channel/ditch. Wherever possible, a ditch constructed of erosion-resistant materials (or equivalent measures) should be constructed at the tops of slopes. Drainage should be directed to appropriate discharge areas or pipes via non-erosive devices. A regular maintenance program should be implemented to keep drainage in good working condition. [Section 13]

Landscaping and irrigation should be kept to a minimum level to reduce the potential for water infiltration. Only drought-tolerant xerophytes should be considered for landscaping. If irrigation piping is used, a means of detecting leaks should be provided or the piping should be placed above ground. Automatic sprinkler controls should not be used unless there is a means of detecting system deficiencies such as broken sprinkler heads and leaks, and failure to turn off. [Section 13]

In addition, drainage should be provided adjacent to all below-grade building walls and retaining walls and under all building basement floors. A minimum thickness (measured horizontally) of 3 feet of Permeable Material should be placed adjacent to below-grade building walls and retaining walls, and a minimum thickness (measured vertically) of 12 inches of Permeable Material should be placed below all building basement floors. A means of removing water from the zone of Permeable Material must be provided. Buildings that have no basement areas do not require Permeable Material. [Section 13]

Wall backfill should be protected from surface water infiltration with a layer of relatively impermeable material (such as the onsite clayey sand/sandy clay) at least 18 inches thick placed at the ground surface. Alternatively, the area may be paved with portland cement concrete or asphaltic concrete. [Section 13]

If the Permeable Material is manufactured onsite, a suggested gradation is given in Table 7. If the Permeable Material is imported from offsite, other gradations may be considered to take advantage of standard gradations available locally. [Section 13]

Permanent slopes for engineered fills should designed and constructed at 2:1 (horizontal to vertical) or flatter. [Section 14]

Sideslopes for the pool excavation should be constructed at an inclination no steeper than 1¼:1 (horizontal to vertical) in alluvium and 1:1 in engineered fill. An 8-foot wide (minimum) bench should be constructed at the mid-depth of the excavation. Flatter slopes should be provided during construction if field conditions so dictate. [Section 15.1]

No surcharge loads should be imposed within a horizontal distance of the top of the temporary excavation slope that is equal to the depth of excavation. Surcharge loads include loads from buildings, equipment, construction materials, stockpiled excavated materials, vehicle parking, and traffic. If it is found desirable to impose surcharge loads within this distance, then additional slope stability calculations should be performed. [Section 15.1]

Surface drainage should be controlled by berms or other measures along the top of temporary excavations to prevent surface runoff from entering the excavation. Even with the implementation of these recommendations, some surface sloughing of the temporary excavation slopes may still occur, and workers should be adequately protected. Construction contract documents should make it clear that the Contractor is solely responsible for all aspects of safety on the construction site, including excavation safety. In addition, excavations should be performed in accordance with applicable local, state, and federal regulations and in such manner that excessive ground movement will not occur. [Section 15.1]

Design and installation of any shoring system should be made the sole responsibility of the Contractor. [Section 15.2]

Prior to excavation, structures and utilities located within a horizontal distance equal to twice the depth of excavation should be observed to evaluate their pre-construction conditions. During the course of construction, deflection of the shoring system should be measured on a frequent (daily) basis. In addition, the shoring system and adjacent structures should be periodically inspected for signs of distress. Floors and pavements should be surveyed before and during construction to determine the amount of movement, if any, resulting from construction activities. In the event that distress or settlement is noted, an investigation should be performed and corrective measures taken so that continued or worsened distress or settlement is mitigated. [Section 15.2]

For design of soldier piles spaced at least 3 diameters on center, the ultimate passive resistance to substantial movement of a soldier pile is given approximately by equation 19. [Section 15.2]

The ultimate passive force should be divided by an appropriate factor of safety selected as a function of the movement expected over the pile length where the passive pressure is developed, since substantial pile movement is required to develop full passive pressure and catastrophic failure may result if the passive resistance is exceeded. [Section 15.2]

The portion of the soldier piles below the adjacent excavation bottom should be concreted to assure firm contact between the pile and supporting soils. To develop firm contact between the upper portion of the shoring and the retained soils, the upper portion of the soldier pile excavation should be filled with a lean mix of concrete or cement slurry. To limit sloughing and caving, continuous lagging should be used between soldier piles from the ground surface to the bottom of excavation. Timber lagging should be pressure-treated if it is to remain in place. The use of shotcrete in place of lagging is not recommended. If there is any void space behind the lagging, it should be filled with pea gravel as the lagging is placed. [Section 15.2]

If tiebacks are used, it is recommended that they be cased to avoid caving and that the tiebacks be pressure-grouted. Pressure-grouted anchors are sensitive to the details of construction employed by the contractor; hence, the contractor should select the bond strength and this should be demonstrated by load testing. All anchors should be proof-tested, and a percentage should be subject to a longer-duration, higher-load performance test. [Section 15.2]

The Contractor should design the temporary shoring. Construction documents should make it clear that the Contractor is solely responsible for all aspects of safety on the construction site, including excavation and shoring. [Section 15.2]

16.2 RESTRICTIONS

This report is intended to provide geotechnical input for foundations for the waste handling facilities to support License Application. The locations of individual structures and the site grading plan were not defined at the time the work described in this report was performed. The input was developed in this report to cover a variety of potential layouts. When the borrow area is identified and the locations of individual structures and the site grading plan become known, the data and interpretations in this report should be reviewed to evaluate whether any changes are required and some confirmatory boreholes and velocity measurements may be required. This report may not contain sufficient information for purposes other than those for which it has been prepared.

Only a very small part of the subsurface conditions at the project site has been observed. In view of the general geology of the project area and the presence of non-engineered fill, the possibility of different subsurface conditions cannot be discounted. Conclusions and recommendations presented in this report are based upon the current understanding of the project and the assumption (Section 5, Assumption 5) that the subsurface conditions do not deviate appreciably from those disclosed by the site subsurface exploration and the assumption (Section 5, Assumption 9) that alluvium logged in borehole UE-25 RF#21 between about 70 and 115 feet is in fact bedrock.

The bearing capacity calculation in this report is based on level ground conditions. According to Section 4.2, the final grade over the pad area shall have a nominal slope between 2 and 3 percent (CRWMS M&O 1999b, Section 1.2.1.7). Slopes of 2 to 3 percent are considered sufficiently horizontal for the values of ultimate bearing capacity in this report to be used (subject to consideration of settlement). However, if a foundation is located near a slope, the allowable bearing capacity should be reviewed. For the purpose of triggering a review, "near" may be taken to mean within four times the footing width.

For the lateral earth pressures (including passive, active, at-rest, and compactor-induced) developed in this report to be valid, the ground surface in the zone behind the wall must be horizontal or slope downhill away from the wall for the active, at-rest, and compactor-induced conditions or slope uphill away from the wall for the passive condition. According to Section 4.2, the final grade over the pad area shall have a nominal slope between 2 and 3 percent (CRWMS M&O 1999b, Section 1.2.1.7). Slopes of 2 to 3 percent are considered sufficiently horizontal for the values in this report to be used. In addition, if a slope or grade change (retaining wall) occurs within or at the edges of the engineered fill pad, there must be sufficient distance between the wall/foundation where the passive resistance develops and the slope or grade change (retaining wall) (see Table 5).

Any persons using this report for bidding or construction purposes should perform such independent investigations as they deem necessary to satisfy themselves as to the surface and subsurface conditions to be encountered and the construction procedures and methods to be used in the performance of work on this project.

16.3 RECOMMENDATIONS FOR FUTURE INVESTIGATIONS

16.3.1 Site Field and Laboratory Investigations

Section 8.2.1 of this report notes that the moist unit weight of the alluvium is anomalously low at borehole UE-25 RF#21 between depths of about 70 and 115 feet. This could indicate that the engineering properties of the deep alluvium are different and less favorable than those of the shallower alluvium. This anomaly could also be explained by misidentification of the drill cuttings in borehole UE-25 RF#21, which was not cored. For this calculation, it is assumed (Section 5, Assumption 9) that the drill cuttings in borehole UE-25 RF#21 were misidentified. Based on the pattern of top-of-bedrock contours on Figure 1 and the gamma-gamma density logs (DTN: MO0112GPLOGWHB.001), it is interpreted that the material is actually bedrock. If a more firm understanding of the subsurface materials and their engineering properties is required, the borehole cuttings should be reexamined.

Section 6.2 of this report notes that the existing fill extends from the ground surface to a depth of 28 feet BGS in borehole UE-25 RF#20. Based on the pre-fill ground surface contours (Figure 1), there should be only about 9 feet of fill at that location. Unless there are utilities or structures that were buried at this location, this 19-foot discrepancy suggests that the alluvium in this vicinity was identified during fill construction as being unsuitable to support the fill and was removed. However, this could also be explained by misidentification of the drill cuttings in borehole UE-25 RF#20, which was not cored. For this calculation, it is assumed that the drill cuttings in borehole UE-25 RF#20 were misidentified – it is interpreted that the material is actually alluvium (Section 5, Assumption 10). During removal of the existing fill, this material can be examined and its nature determined. The recommendations in Attachment II provide for examination of the subgrade after removal of the existing fill so that any loose material can be removed. However, the specific issue at borehole UE-25 RF#20 should be resolved before construction to eliminate a potential surprise that could have a significant adverse effect on the construction schedule and budget.

To resolve the issues about stratigraphy raised in the previous two paragraphs and to identify any similar issues, it is suggested that a review be conducted of the cuttings from all the rotary wash boreholes (UE-25 RF#18 through #29), using appropriate methods, such as microscopic examination.

Beyond resolving these issues, additional site subsurface exploration should be conducted to obtain additional soil and rock samples, and in situ and laboratory testing should be conducted to obtain additional data to provide better insight into the variability in subsurface conditions and engineering properties. The data that should be obtained include:

- In situ testing to determine engineering properties of the alluvium. These tests may include static or dynamic plate load tests in a horizontal or vertical orientation or dilatometer/pressuremeter tests.

- Laboratory test data to determine engineering properties of the alluvial deposits. At present, the shear strength has been evaluated based primarily on correlations with shear-wave velocity. The compressibility has been calculated based on low-strain shear modulus derived from shear-wave velocity measurements and the normalized shear modulus curves from dynamic laboratory tests. Better estimates of shear strength should be possible by performing laboratory shear tests on reconstituted samples. Measurement of compressibility by a direct static laboratory method should also performed as an alternative to the method used in this report. Samples of the alluvial material should be scalped on an appropriate sieve and recompacted to the in situ void ratio of the fine fraction. Using 9-inch diameter triaxial specimens, a maximum particle size on the order of 1 to 1.5 inches can be used. The dimensions of available apparatus for compressibility testing of large-diameter specimens will need to be investigated. However, it should be recognized that the natural cementation of the alluvium will not be reproduced in the laboratory tests.

- Stratigraphic data (from boreholes). Additional boreholes should be advanced within the footprint of the structures, once their locations are fixed. To provide better insight into the variability in subsurface conditions, boreholes should be spaced no farther apart than 200 feet within the footprint of each structure. The existing qualified boreholes may be considered if they are in appropriate locations with respect to the structures. For the types of geotechnical calculations performed in this report, it is sufficient to penetrate the alluvium and bedded tuff (Tmbt1, Tpbt5), if present, and about 15 feet into the underlying Tiva Canyon Tuff. Note that some boreholes will likely have to be placed where the muck pile is currently located; it is recommended that the muck pile be removed down to the level of the existing fill pad proper to expedite drilling operations and to avoid the effects on engineering parameters that would be caused by the higher in situ stresses induced by the muck pile. ESF construction-supports installations should be relocated temporarily or permanently to allow optimum location of explorations.

- In situ shear-wave velocity profiles (using cased boreholes).

- In situ unit weight in alluvial deposits using USBR 7221-89, *Determining Unit Weight of Soils In-Place by the Water Replacement Method in a Test Pit*, or similar method in test pits; and borehole geophysical methods, such as gamma-gamma, in boreholes.

- Degree of cementation of alluvial material (from test pits).

- Potential for corrosion of metals and concrete, including laboratory electrical resistivity (on soil at natural water content and under saturated conditions), sulfate content, chloride content and pH. The project corrosion engineer should indicate what types of tests, including any biological tests, are needed for specific types of metal and concrete. Testing procedures acceptable to the project need to be identified or written, if necessary.

- In situ testing of potential for hydroconsolidation (collapse) of alluvial material to provide input requested in Misiak (2001).

- Unconfined or triaxial compression tests and shear-wave velocity measurements on weak rock (Tmbt1, Tpki, Tpbt5), to confirm that its shear strength and compressibility values are significantly better than the corresponding values of the overlying alluvium and potential engineered fill.

16.3.2 Borrow Investigation

Engineered fill is required for the potential facility. Although the preliminary evaluation of the material from the Fran Ridge Borrow Area suggests that this borrow area can provide material that can be used to construct a fill with good engineering properties, the borrow area will require further confirmation as the borrow source. Any restrictions (for example, for environmental reasons) that will be imposed on the areal extent, depth, and slope steepness of the borrow area should be identified. The volume of engineered fill required for construction should be determined and geologic/geotechnical investigation should be performed to prove out a volume of suitable material at least twice the required volume. The geologic/geotechnical investigation should:

- Identify the thickness of any overburden that must be disposed of

- Identify the types of borrow material

- Sample the borrow material. Approximately one bulk sample should be taken for each 10,000 yd^3 of material to be proven

- Establish relevant geotechnical characteristics of the material passing the 37.5 mm sieve:

    - the quantity of oversize (material retained on the 37.5 mm sieve)
    - in-place density and water content of complete material and the minus 37.5 mm fraction

- compaction characteristics
- compressibility
- shear strength
- particle breakdown due to handling and compaction
- potential for corrosion of ferric metals and concrete
- expansion (heave) potential

- Identify factors that may be important for construction schedule or budget, such as excavatability

- Identify the source of construction water and determine its engineering characteristics

- Construct a test fill, if desired. A test fill could be used to:

  - make in situ measurements of engineering properties, including shear-wave velocity and damping
  - examine the effect of construction equipment on the material (as requested in Misiak (2001))
  - verify that ASTM D 1556-00/ASTM D 2216-98 and ASTM D 2922-01/ASTM D 3017-01 yield consistent results (ASTM D 1556-00, *Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method*; ASTM D 2216-98, *Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*; ASTM D 2922-01, *Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)*; and ASTM D 3017-01, *Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth)*. (Note: the versions of the ASTM standards that are current at the time of construction should be used).)

If the borrow is located at depths of 15 to 20 feet or less below existing grade, it may be possible to perform the subsurface investigation using primarily test excavations using a backhoe. If the borrow extends to deeper depths, it may be necessary to supplement the backhoe pits with bucket auger borings.

Geological exploration should be performed to seek potential sources for concrete aggregate, if desired for on-site concrete batching or for Permeable Material. The material from the Fran Ridge Borrow Area is judged to break down too much to provide a good source of Permeable Material.

16.3.3 Analysis

Additional calculations will be required to consider geotechnical issues that were not within the scope of this report. As discussed in Sections 1.2 and 9.1, settlement analyses have not been performed, consequently, the ultimate bearing capacity is based on shear strength alone. Therefore, settlement analyses will have to be performed and the ultimate bearing capacity modified accordingly in order to arrive at an allowable bearing pressure.

As discussed in Sections 1.2 and 5 (Assumption 8), it is assumed for this calculation that the waste handling facilities can be supported on shallow spread footings and mat foundations. The findings of this report support this assumption for static conditions, but no dynamic calculations were performed. Additional calculations are required to establish the suitability of mat and spread footing foundations for seismic conditions.

Because few grading, layout, and design details were available at the time the calculations reported herein were performed, the recommendations in this report should be reviewed as these details become available so that any revised or supplemental recommendations can be made in a timely manner. Although it is expected that the final grading plan will vary from the one on Figure 1, particularly at greater distances from the North Portal, the impacts of these variations on the calculation should be reviewed if the variation exceeds 5 feet at any point.

An investigation should be conducted to ascertain that System Performance Criterion 1.2.1.15 (CRWMS M&O 1999b) is met. This criterion states: "The layout shall locate all surface waste handling facilities away from faults which have 2 in. (5 cm) or more displacement over the past 100,000 years". This may be as simple as comparing the location, orientation, and rupture style of the faults identified in the WHB Area with the assumptions made in Swan et al. (2001) to determine whether these faults come within the range of faulting considered by Swan et al. (2001). (Note: Swan et al. (2001, page 37) concluded on the basis of their geologic mapping and trench investigations that there is not any measurable Quaternary faulting activity in the vicinity of the site, at least since the deposition of middle Pleistocene deposits estimated to be between 350,000 and 76,000 years old.)

To provide a more robust interpretation of subsurface conditions, and for use in determining compliance with System Performance Criterion 1.2.1.15, the stratigraphic and structural data should be interpreted with the assistance of a three-dimensional modeling program such as EARTHVISION. The potential variation in fault locations should be quantified. This same methodology could also be applied to develop a three-dimensional model of shear-wave velocity, which would provide more insight into site response to ground motions and the variation in compressibility of subsurface materials.

Construction sequence is very important to geotechnical recommendations. An example of this is the relative order of constructing structures or utilities that would be adjacent to other potential structures, constructing the pool in the potential wet-process building, removing the non-engineered fill, and placing the engineered fill. The planners, civil, structural, and geotechnical engineers should collaborate to work out a construction sequence on which to base design and construction documents.

As discussed in Section 9.2 and 10.2, pseudostatic methods for calculating bearing capacity and dynamic pressures on retaining walls are simple to use, but have several shortcomings. In particular, pseudostatic methods do not incorporate the beneficial effect on soil resistance that results from the transitory, cyclic nature of the loading. In addition, there is a good deal of uncertainty in the choice of a pseudostatic coefficient. Consequently, if a more refined analysis is required or if additional lateral resistance is required, performance of a non-linear finite difference analysis is recommended.

17. CONCLUSION

This Calculation documents geotechnical evaluations of, and geotechnical recommendations for, foundations for potential waste handling facilities near the Exploratory Studies Facility North Portal of the potential managed geologic repository. These recommendations have been developed for use in design of the potential waste handling facilities to a level suitable to support License Application. The interpretations, findings and recommendations in this Calculation supercede any conflicting interpretations, findings and recommendations given in CRWMS M&O (1999a). Limitations on the use of information and recommendations contained in this report are discussed in Section 1 and restrictions on the use of information and recommendations contained in this report are discussed in Section 16.2.

This calculation should be reviewed after details of the waste handling facilities become available and/or confirmatory geotechnical data are acquired. This report should then be revised as necessary.

The results reported herein are summarized in Table 8:

Table 8. Results

Result	Where Reported
Requirements for engineered fill and earthwork	Section 7, Attachment II
Effect of weather on soil	Section 7
Effect of construction equipment on soil	Section 7
Material parameters	Section 8
Extreme frost penetration depth	Attachment III
Minimum embedment of shallow footings and mats	Section 9.1
Ultimate bearing capacity of shallow footings and mats	Section 9, Figure 2
Requirements for slabs-on-grade and vapor barriers	Section 9.3
Lateral earth pressure on below-grade walls	Section 10.1
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Attachment I
Parameters for Subsurface Materials

This attachment presents the development of geotechnical parameters required for the evaluations in this report. The outline below provides a guide to the contents.

I.1 Engineered Fill

Ref: *Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project* (BSC 2002).

Use the data presented in Section 6.5 of BSC (2002).

Premises:

- The Fran Ridge material will be used for Engineered Fill (see Section 5, Assumption 3).
- The sample of material tested from the Fran Ridge Borrow area is representative of the borrow source (see Section 5, Assumption 3).
- The fill material will be required to have a maximum particle size of either 1-1/2 inches or 3 inches, which is a recommendation of this report (Section 7, Attachment II).
- The fill control will be done using ASTM D 1557 as the laboratory maximum dry unit weight (maximum size 3/4 inch using Method C), which is a recommendation of this report (Attachment II).
- The fill control will use either nuclear or sand-cone testing and use the rock correction by ASTM D 4718-87 (Reapproved 2001). *Standard Practice for Correction of Unit Weight and Water Content for Soils Containing Oversize Particles*, which is a recommendation of this report (Attachment II).
- The specifications will require that the finer fraction (minus 3/4 inch) be compacted at or above optimum water content to at least 95 percent relative compaction, which is a recommendation of this report (Attachment II).
- The water content of the coarse fraction will be at saturated surface dry (SSD) conditions.

I.1.1 Moist Unit Weight of Engineered Fill

Calculate the moist unit weight of the fill using the premises above and the corrections of ASTM D 4718.

The equation for corrected dry unit weight is:

$$C\gamma_D = 100\gamma_F G_C \gamma_w / (\gamma_F P_C + G_C \gamma_w P_F) \qquad \text{(Eq. I-1)}$$

where: $C\gamma_D$ = corrected unit dry weight of the total material (combined finer and oversize fractions)
G_C = bulk specific gravity of the plus ¾-inch particles
γ_F = dry unit weight of the finer fraction
γ_w = unit weight of water (62.428 lbf/ft^3)
P_F = percent of finer fraction by dry weight
P_C = percent of oversize fraction by dry weight = 100-P_F

The equation for corrected water content is:

$$Cw = w_F P_F + w_C P_C \qquad \text{(Eq. I-2)}$$

where: Cw = corrected water content of combined finer and oversize fractions (in percent)
w_F = water content of finer fraction expressed as a decimal
w_C = water content of oversize fraction expressed as a decimal

The project data from BSC (2002) are:

G_C =	2.24	bulk saturated surface dry specific gravity (Table 27)
γ_F =	109.92 lbf/ft^3	96 percent of max dry unit weight of 114.5 lbf/ft^3 (Figure 215)
γ_w =	62.428 lbf/ft^3	see above
w_F =	0.12 decimal	optimum plus one percent (Figure 215)
w_C =	0.053 decimal	absorption (water content at SSD, Table 27)
P_F =	75 percent	percent passing 1/2-inch sieve, interpolated on Figure 214
P_C =	25 percent	percent retained on 1/2-inch sieve =100 minus P_f

Based on the recommended compaction specifications in Attachment II, it is estimated that the fill will be compacted to approximately 96 percent relative compaction and one percent above optimum water content. This corresponds to:

$$C\gamma_D = 116.13 \text{ lbf/ft}^3 \qquad \text{(Eq. I-3)}$$

$$Cw = 10.33 \text{ percent} \qquad \text{(Eq. I-4)}$$

The moist unit weight would then be:

$$C\gamma_m = C\gamma_D \cdot (1 + Cw/100) \qquad \text{(Eq. I-5)}$$

$$C\gamma_m = 128.1 \text{ lbf/ft}^3 \qquad \text{(Eq. I-6)}$$

This unit weight is approximate for fill with a maximum particle size of either 3 inches or 1-1/2 inches. It is noted that the compaction test performed on the sample of Fran Ridge material was scalped on the 1/2 inch sieve. That is why the correction above used the percent passing the 1/2 inch to determine the P_F and P_C. The unit weight above would be applicable to the fill if the maximum particle size in the fill is 3 inches (i.e. the same as the Fran Ridge sample). If the contract were to require 1-1/2 inch maximum size, then the Contractor would need to process the borrow material to meet this requirement. If the Contractor were to crush the borrow material and the material between 1-1/2 inch and 3 inch sizes were reduced to gravel-size pieces between 1-1/2 and 1/2 inch, then the density above would still be appropriate because the P_F and P_C would be correct. Since these are the two most likely scenarios, it is judged that the above unit weight would be applicable to the future construction.

Based on this, it is recommended that a moist unit weight of 128 lbf/ft^3 be used for engineered fill made from material from the Fran Ridge Borrow Area.

I.1.2 Shear Strength of Engineered Fill

I.1.2.1 General

The Fran Ridge borrow sample contained about 25 percent by dry weight of plus 1/2 inch material (BSC 2002, Figure 214). For this amount of oversize, the finer fraction should control the material properties, including shear strength. For this reason the strength testing performed on the scalped sample should be appropriate. The effects of gravel content on the strength of soils have been investigated by several researchers. A summary of the conclusions for testing performed by various researchers on cohesionless gradations of material containing gravel is presented in Donaghe and Torrey (1985). A summary is provided as follows (see reference list in Donaghe and Torrey (1985) for information about the four reports reviewed below):

Holtz and Gibbs (as summarized by Donaghe and Torrey 1985, pages 7-8):

> Found for sand and gravel mixtures that maximum particle size from 3/4 to 3 inch had little effect on shearing resistance because the larger particles were actually so relatively few in number.
>
> They varied gravel content of test specimens and concluded that the effective angle of internal friction, ϕ', increased with gravel content up to 50 to 60 percent gravel depending on maximum particle size.

Leslie (1963) (also summarized by Donaghe and Torrey (1985, page 8)):

> One test series reflected increasing strength with increasing coefficient of uniformity, C_u, and density, with maximum strength developed for the 1-in. maximum particle-size specimens. In another series, Leslie saw little effect from removing oversize particles (+3 inch, +1-1/2 inch, +1 inch).

Marachi, Chan, and Seed (1972) (also summarized by Donaghe and Torrey (1985, page 8)):

> Concluded that the effective angle of internal friction, ϕ', was affected to some extent by the size of particles in the test specimen and by gradation. They tested samples of parallel-graded materials in three sizes of test (36 inch, 12 inch and 2.8 inch diameter.) The gradations were modeled as parallel, but also approximate a successively scalped gradation.
>
> The angle of internal friction for the 36-inch diameter specimens (6 inch maximum particle) was about 1 to 1.5 degrees lower than that of the 12-inch diameter specimens (2-inch maximum size) and 3 to 4 degrees lower than that of 2.8-inch diameter specimens (0.45 inch maximum size). The gravel content also changed with specimen size. The 2.8-inch diameter specimens had 27 to 38 percent gravel, the 12-inch diameter specimens from 70 to 84 percent and the 36-inch diameter specimens had 82 to 95 percent gravel.

Donaghe and Cohen (1978) (also summarized by Donaghe and Torrey (1985, page 8)):

> Working with sand-gravel mixtures with up to 60 percent gravel, reported that strength did not change significantly with increasing maximum particle size up to 3 inches for a constant value of coefficient of uniformity.

For increasing values of C_u (coefficient of uniformity), strength was found to increase over a range of maximum particle size up to almost 1 inch and little increase in strength was noted above 1-inch maximum particle size.

Conclusion:

The research of Holtz and Gibbs suggests that using the strength test results from the scalped Fran Ridge sample would either estimate the strength of the engineered fill with larger particles correctly or underestimate the strength.

The research by Leslie suggests that the results of the Fran Ridge sample would either estimate the strength of the engineered fill with larger particles correctly or underestimate the strength.

The Marachi, Chan and Seed research suggests that including larger particles tends to decrease the measured strength. This would suggest that the scalped Fran Ridge sample may overestimate the strength of the engineered fill with larger particles. It is noted, however, that the Marachi, Chan and Seed research tested gravel contents up to 82 to 95 percent. The gravel content of the Fran Ridge material is only about 48 percent.

The research of Donaghe and Cohen suggests that using the strength test results from the scalped Fran Ridge sample would either estimate the strength of the engineered fill with larger particles correctly or underestimate the strength.

It is our conclusion that the research is not consistent but generally indicates that test results on scalped specimens such as the Fran Ridge sample should lead to accurate or conservative estimates.

It is recommended that the drained strength test results be used directly to evaluate the engineered fill shear strength.

I.1.2.2 Shear Strength Envelopes

It is possible to represent the shear strength envelope for the drained tests using different equations. Because the material is granular, it can be represented as a frictional material with no cohesion. The triaxial test results indicate that the friction angle of the material is a nonlinear function of the normal or confining stresses, which is typical for this type of soil. The strength of the material has therefore been modeled with a friction angle that is a function of the confining or normal stress. Three methods were examined to do this. The first is a generalized curved failure envelope. The second and third methods are specific to particular analysis methods, such as bearing capacity or slope stability, and require that the strength be expressed using a specific functional form. The following describes the fit to each method.

I.1.2.3 General-Purpose Curved Strength Envelope

The generalized curved failure envelope has the form:

$$\tau_{ff} = \sigma'_{ff} \tan \phi' \qquad \text{(Eq. I-7A)}$$

where: τ_{ff} = shear stress acting on the failure plane at failure, i.e., effective shear strength
σ'_{ff} = normal effective stress acting on the failure plane at failure
ϕ' = effective friction angle as a function of σ'_{ff}.

The equation for the friction angle as a function of normal stress is:

$$\phi' = \phi'_1 - \Delta\phi' \log(\sigma'_{ff}/p_a) \qquad \text{(Eq. I-7B)}$$

where: ϕ'_1 = the effective friction angle for σ'_{ff} = 1 atmosphere (p_a=2,116.22 lbf/ft^2)
$\Delta\phi'$ = the decrease in ϕ' per log cycle change in σ'_{ff}
p_a = 1 atmosphere (2,116.22 lbf/ft^2).

The results of the drained triaxial strength test on specimens of scalped Fran Ridge material compacted to 95.4 to 96.4 percent relative compaction indicated the following:

Table I-1. Results of Isotropically Consolidated Drained Triaxial Test on Specimens of Scalped Fran Ridge Material.

p' kips/ft^2	q kips/ft^2	σ_{ff} kips/ft^2	σ_{ff} atm	log (σ_{ff}) log atm	ϕ' for c'=0 degrees
6.180	4.999	2.136	1.0094943	0.0041039	53.988566
10.006	7.687	4.101	1.9376763	0.2872812	50.19602
15.399	10.719	7.938	3.7508764	0.5741327	44.113674
25.505	16.807	14.430	6.81863	0.8336971	41.221277

Note: p' is the mean principal stress = $\sigma_1 + \sigma_3$; q is the principal stress difference or deviator stress = $\sigma_1 - \sigma_3$; and σ'_{ff} is the normal effective stress acting on the failure plane at failure.

A linear regression of the triaxial data (BSC 2002, Figure 217 and Table 28) with failure evaluated at peak deviator stress indicates that ϕ'_1 = 54 degrees and $\Delta\phi'$ = 16 degrees, i.e.:

$$\tau_{ff} = \sigma'_{ff} \tan\phi' = \sigma'_{ff} \tan\left[54° - 16° \log\left(\frac{\sigma'_{ff}}{p_a}\right)\right] \qquad \text{(Eq. I-8)}$$

I.1.2.4 Strength Envelope for Passive Pressure

In the passive pressure case for foundation embedment in engineered fill of less than or equal to 8 feet, the minor principal stress σ'_3 will equal the vertical overburden pressure and will range from zero to (8 ft)(128 lbf/ft^3)/(1000 lbf/kip) = 1.024 kips/ft^2.

Using the following equations and trial and error, the value of σ'_{ff} and ϕ' corresponding to σ'_{3f} = 1.024 kips/ft^2 can be found:

$$\phi' = 54° - 16° \cdot \log(\sigma'_{ff}/2.11622) \qquad \text{(Eq. I-9)}$$

$$\sigma'_{3f} = \sigma'_{ff} / (1+\sin\phi') \quad \text{(Eq. I-10)}$$

The equation relating σ'_{3f}, σ'_{ff} and ϕ' can be derived using the geometry of the Mohr circle and trigonometry, and σ'_{ff} can be calculated by assuming a trial value of σ'_{ff} and using it in equation I-9 to calculate ϕ' and using these values in equation I-10 to compute by σ'_{3f}. When the value of σ'_{3f} calculated from equation I-10 is equal to the target value (1.024 kips/ft^2 in this case), then the values of σ'_{ff} and ϕ' are those sought. This was done to reach the values in the following table:

Table I-2. σ'_{ff} Calculated by Successive Trials

σ'_{ff} kips/ft^2	ϕ' degrees	σ'_{3f} kips/ft^2
1.86	54.897	1.023

Therefore, the effective friction angle for the case of a foundation embedded up to 8-foot depth can be conservatively estimated at 55 degrees.

I.1.2.5 Strength Envelope for Bearing Capacity

An alternative method of modeling the curved failure envelope is presented in Ueno et al. (1998, page 167). The basic equation is:

$$\tan\phi' = \tan(\phi'_a)(\sigma_a/\sigma'_m)^{\alpha} \quad \text{(Eq. I-11)}$$

where: ϕ' is the effective friction angle corresponding to σ'_m
σ_a is 1 atmosphere (2.11622 kips/ft^2)
ϕ'_a is the friction angle at one atmosphere
σ'_m is equal to $\frac{1}{2}(\sigma'_1+\sigma'_3)$ (also denoted by the symbol p')
α is a constant

From the triaxial test performed on the Fran Ridge Borrow material (BSC 2002, Table 28; DTN: MO0203EBSCTCTS.016), the following values are obtained:

Table I-3. Results of Isotropically Consolidated Drained Triaxial Test on Specimens of Scalped Fran Ridge Material, Including Values of $\log(\sigma_a/\sigma'_m)$ and $\log(\tan\phi')$

σ'_m = p' kips/ft^2	q kips/ft^2	σ_a/σ'_m	ϕ' degrees	$\tan\phi'$	$\log(\sigma_a/\sigma'_m)$	$\log(\tan\phi')$
6.18	4.999	0.34243042	54.0	1.375804	-0.46543	0.138557
10.006	7.687	0.2114951	50.2	1.200068	-0.6747	0.079206
15.399	10.719	0.13742581	44.1	0.96953	-0.86193	-0.01344
25.505	16.807	0.08297275	41.2	0.87609	-1.08106	-0.05745

where: $\sigma'_m = p' = \dfrac{\sigma'_1 + \sigma'_3}{2}$ (Table 28 of BSC 2002)

$q = \dfrac{\sigma_1 - \sigma_3}{2}$ (Table 28 of BSC 2002)

$\phi' = \sin^{-1}\left(\dfrac{q}{p'}\right)$

A linear regression (LINEST in Excel) with log(tanϕ') as the x-variable and log(σ_a/σ'_m) as the y-variable yields a slope of 0.3330976 and an intercept of 0.2934634. This means that α = 0.3330976 and that tan $\phi'_a = 10^{0.2934634} = 1.9654564$, yielding (with rounding):

$$\tan\phi' = 1.9655(\sigma_a/\sigma'_m)^{0.3331} \quad \text{(Eq. I-12A)}$$

$$\tau_{ff} = \sigma'_{ff}\tan\phi' = \sigma'_{ff}\cdot 1.9655\,(\sigma_a/\sigma'_m)^{0.3331} \quad \text{(Eq. I-12B)}$$

where: ϕ' = the effective friction angle as a function of σ'_m
σ_a = 1 atmosphere pressure, expressed in the same units as σ'_m
σ'_m = ½ ($\sigma'_1+\sigma'_3$).

Check fit:

Table I-4. Check Fit of Equation I-12B to Results of Isotropically Consolidated Drained Triaxial Test on Specimens of Scalped Fran Ridge Material

σ'_m kips/ft^2	σ_a/σ'_m	ϕ' measured	ϕ' from equation
6.18	0.3424304	54.0	53.980651
10.006	0.2114951	50.2	49.514428
15.399	0.1374258	44.1	45.419255
25.505	0.0829728	41.2	40.621517

The fit is good. Use of equations I-12A and I-12B is recommended for bearing capacity calculation.

I.1.2.6 Strength Envelope for Slope Stability

Another method of modeling the strength is used by Charles and Soares (1984, page 62) for slope stability analysis. The form of the equation is:

$$\tau_{ff} = A(\sigma'_{ff})^b \quad \text{(Eq. I-13)}$$

where: τ_{ff} = shear stress on the failure plane
σ'_{ff} = effective normal stress on the failure plane
A and b are constants.

From the triaxial test performed on the Fran Ridge Borrow material (BSC 2002, Table 28; DTN: MO0203EBSCTCTS.016) the following values are obtained:

Table I-5. Results of Isotropically Consolidated Drained Triaxial Test on Specimens of Scalped Fran Ridge Material, Including Values of σ'_{ff} and τ_{ff}

p'_f kips/ft^2	q_f kips/ft^2	ϕ'	σ'_{ff} - kips/ft^2	τ_{ff} - kips/ft^2
6.18	4.999	54.0	2.136	2.939
10.006	7.687	50.2	4.101	4.921
15.399	10.719	44.1	7.938	7.696
25.505	16.807	41.2	14.430	12.642

Note: $p'_f = \dfrac{\sigma'_1 + \sigma'_3}{2}$ and $q_f = \dfrac{\sigma_1 - \sigma_3}{2}$

The values of p'_f and q_f are from Table 28 (BSC 2002) and the other parameters are calculated as follows:

$$\phi' = \arcsin\left(q_f / p_f'\right) \qquad \text{(Eq. I-14)}$$

$$\sigma'_{ff} = p_f' - q_f \sin\phi' \qquad \text{(Eq. I-15)}$$

$$\tau_{ff} = q_f \cos\phi' \qquad \text{(Eq. I-16)}$$

which can be derived from the Mohr circle at failure.

To determine A and b, perform a linear regression of the log (σ'_{ff}) and log(τ_{ff}) data:

Table I-6. Values of Results of Isotropically Consolidated Drained Triaxial Test on Specimens of Scalped Fran Ridge Material, Including Values of log(σ'_{ff}) and log(τ_{ff})

σ'_{ff} - kips/ft^2	τ_{ff} - kips/ft^2	log(σ'_{ff})	log(τ_{ff})
2.1363105	2.939146	0.3296644	0.46822109
4.1005464	4.920933	0.6128417	0.6920475
7.9376739	7.695815	0.8996933	0.88625464
14.429711	12.64173	1.1592576	1.10180635

The linear regression (LINEST in Excel) of this data yields a slope of 0.7543251 and an intercept of 0.2210638, which gives A = 10^0.2210638 = 1.6636572 and b = 0.7543251:

With these constants, equation I-13 becomes:

$$\tau_{ff} = 1.6636572\,(\sigma'_{ff})^{0.7543251} \qquad \text{(Eq. I-17)}$$

where: τ_{ff} = shear stress on the failure plane - kips/ft^2
σ'_{ff} = effective normal stress on the failure plane - kips/ft^2

Note that the value of the constant multiplier in equation I-17 depends on the system of units being used, while the exponent is independent of the system of units.

Check the fit of equation I-17 against the test results:

Table I-7. Check Fit of Equation I-17 to Results of Isotropically Consolidated Drained Triaxial Test on Specimens of Scalped Fran Ridge Material

σ'_{ff} - kips/ft^2	τ_{ff} - kips/ft^2 measured	τ_{ff} - kips/ft^2 from Eq. I-17
2.136311	2.9391455	2.94943015
4.100546	4.9209335	4.82331802
7.937674	7.6958153	7.93825787
14.42971	12.641725	12.4600885

The fit is good. Equation I-17 is recommended in conjunction with the slope stability chart method of Charles and Soares (1984, page 62) for slope stability analyses.

I.1.3 Compressibility of Engineered Fill

Ref: Section 6.5 of BSC (2002).

Obtain secant Young's modulus from the drained triaxial test (DTN: MO0203EBSCTCTS.016).

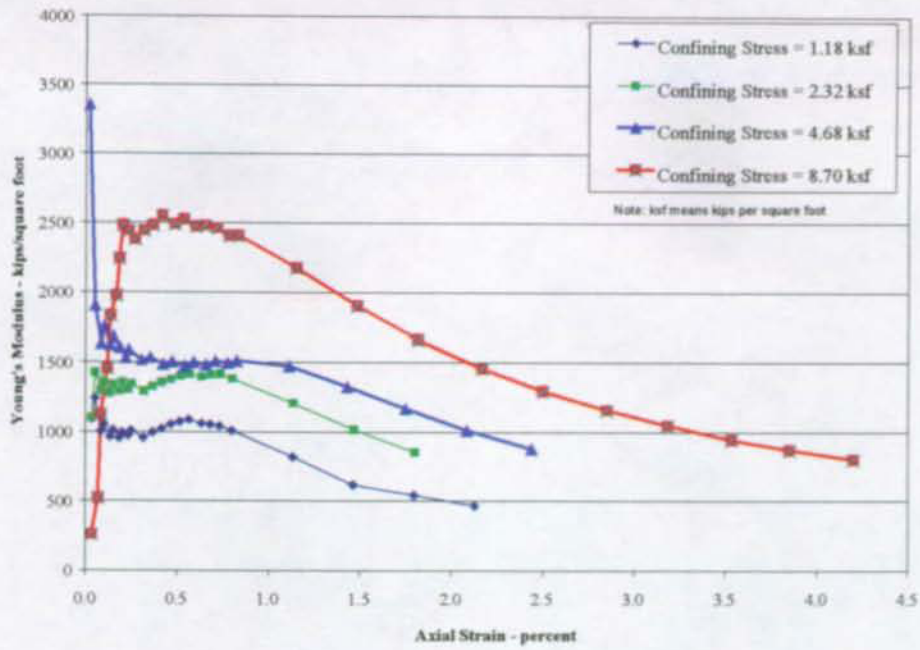
Secant Young's modulus, E is given by:

$$E = \sigma_a/\varepsilon_a \qquad \text{(Eq. I-18)}$$

where: σ_a is the axial stress
ε_a is the axial strain

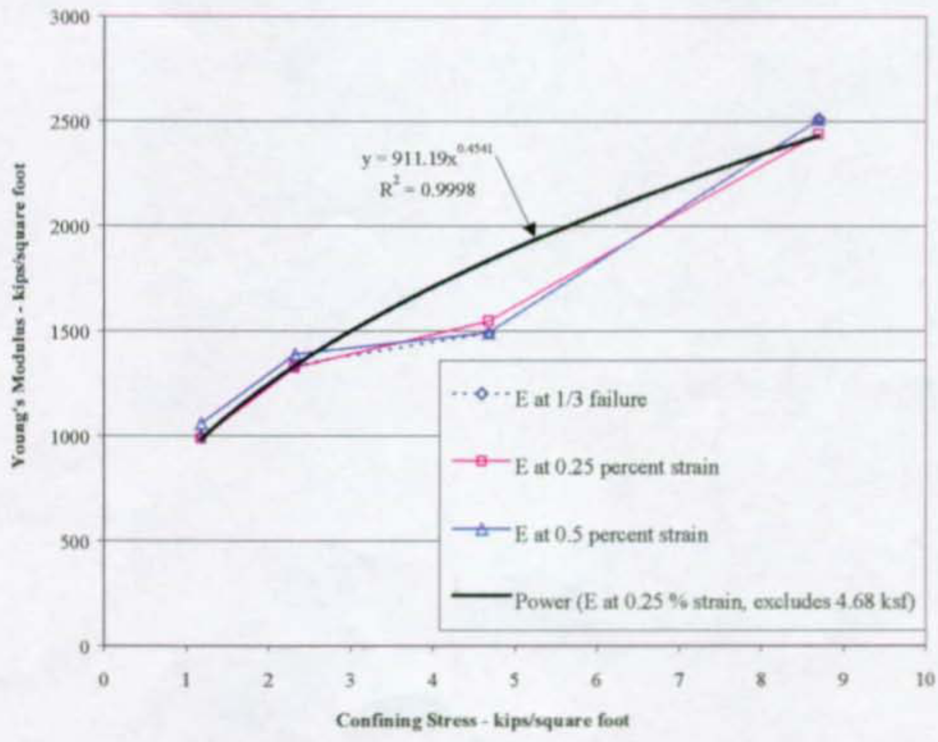
Use the general procedure outlined in Lambe and Whitman (1969, pages 158 and 159).

The secant Young's modulus was calculated from the triaxial compression test data and plotted on Figure I-1. Representative values of E at two strain levels equal to 0.25 and 0.5 percent strain, and E at the stress level of 1/3 the maximum deviator stress, were chosen. It is judged that these levels of strain and stress are consistent with the levels anticipated beneath foundations. For example, if the foundation were designed for a factor of safety of 3 against bearing capacity failure, the stress level of 1/3 deviator stress would be appropriate. If, for example, a 50-foot mat foundation were to be designed for 1.5 inches of settlement, the average strain beneath the foundation would be on the order of 1.5/50/12 or 0.25 percent. The modulus values are plotted versus stress on Figure I-2.



DTN: MO0203EBSCTCTS.016

Figure I-1. Young's Modulus from Drained Triaxial



Source: Figure I-1

Figure I-2. Young's Modulus vs. Confining Stress

The results indicate that the values of modulus are very similar regardless of which of the three criteria is selected. For the purposes of this analysis the secant Young's modulus corresponding to 0.25 percent strain is chosen. The power fit equation to the data excluding the point at 4.68 kips/ft^2 is:

$$E = 911.19(\sigma')^{0.4541} \quad \text{(Eq. I-19)}$$

where: σ' is the initial isotropic consolidation stress prior to loading in kips/ft^2. The data point at 4.68 kips/ft^2 was not included. It is suspected that this point is low. The general trend of E with σ should be a power function of the form similar to Equation I-19 (Lambe and Whitman 1969, page 158).

Another source of data is the resonant column and torsional shear tests performed on compacted specimens. The data were taken from BSC (2002, Attachment XVII (they are also in DTN: MO0203DHRSSWHB.001)). The data were chosen for specimens compacted to at least 95 percent relative compaction. The G_{max} values (based on resonant column tests) were plotted versus consolidation pressure and a power equation was fit to the data as shown on Figure I-3. The fitted equation is:

$$G_{max}\ (\text{kips/ft}^2) = 1996.7(\sigma')^{0.5861} \quad \text{(Eq. I-20)}$$

where: σ' is the initial isotropic consolidation stress prior to loading in kips/ft^2.

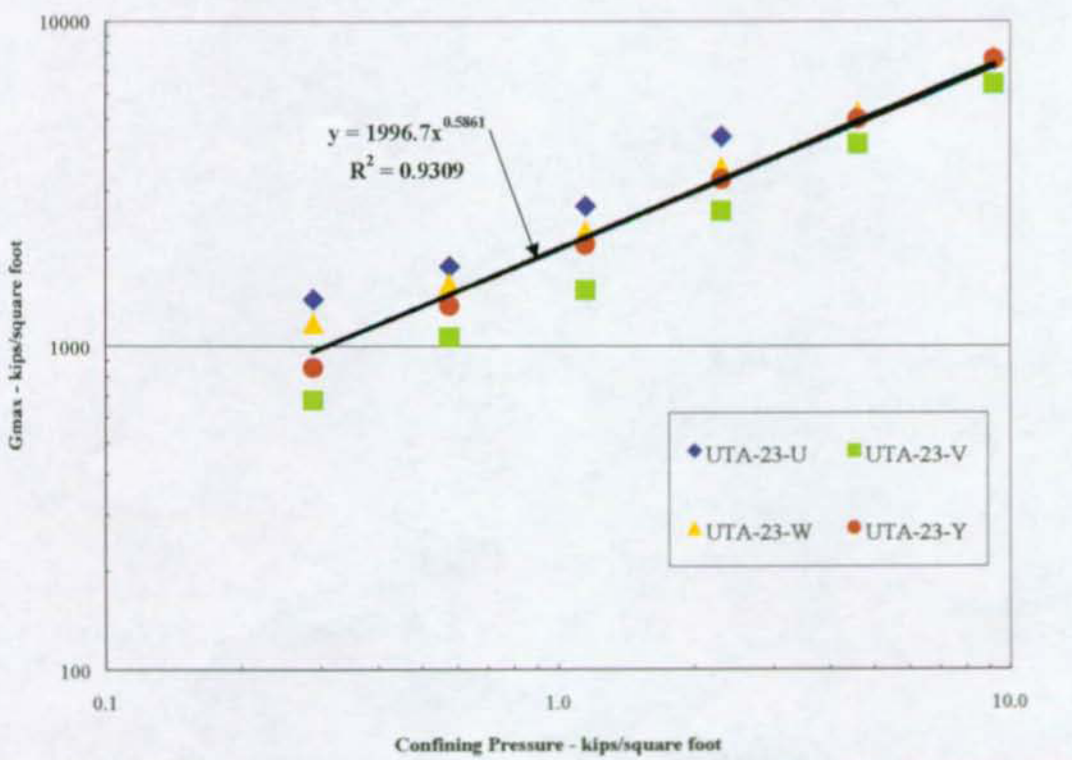
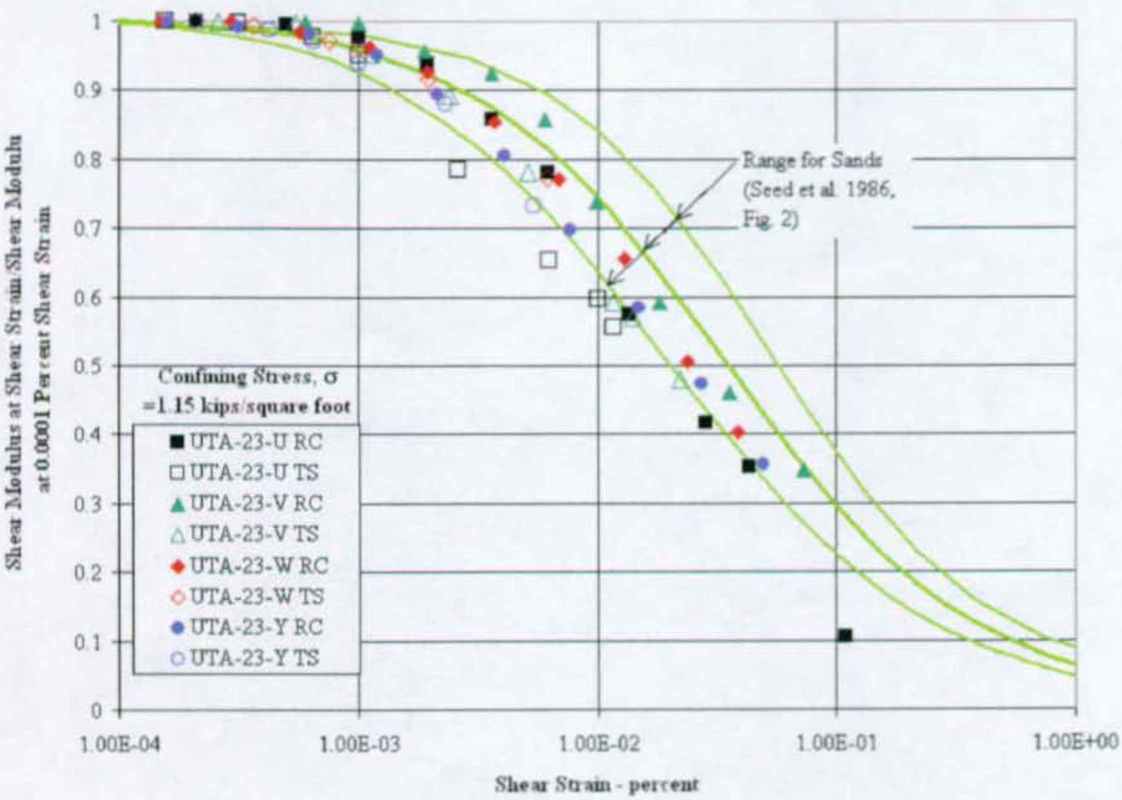


Figure I-3. Gmax vs. Confining Pressure

The data for resonant column/torsional shear specimens (BSC 2002, Attachment XVII) that were compacted to at least 95 percent relative compaction and consolidated to 8 pounds per square inch (psi) (1.15 kips/ft^2) cell pressure were examined next. The data, along with the Seed et al. (1986, Figure 2) relationship developed for sands, are plotted on Figure I-4. The data for the specimens compacted to at least 95 percent relative compaction and consolidated to 32 psi (4.6 kips/ft^2) cell pressure, along with the Seed et al. (1986, Figure 2) sand curves, are plotted on Figure I-5.

The plot of G/G_{max} versus shear strain on Figure I-4 indicates that the specimens consolidated to 8 psi (1.15 kips/ft^2) generally followed the average curve at strains less than 0.003% and the lower-bound relationship for sands proposed by Seed et al. (1986, Figure 2) at higher strain levels. Figure I-5 indicates that the specimens consolidated to 32 psi generally followed the upper-bound curve at strains less than 0.003% and the average curve proposed by Seed et al. (1986, Figure 2) at strains greater than 0.003%.



DTN: MO0203DHRSSWHB.001

Figure I-4. Variation of Shear Modulus with Shear Strain, σ = 1.15 kips/ft^2

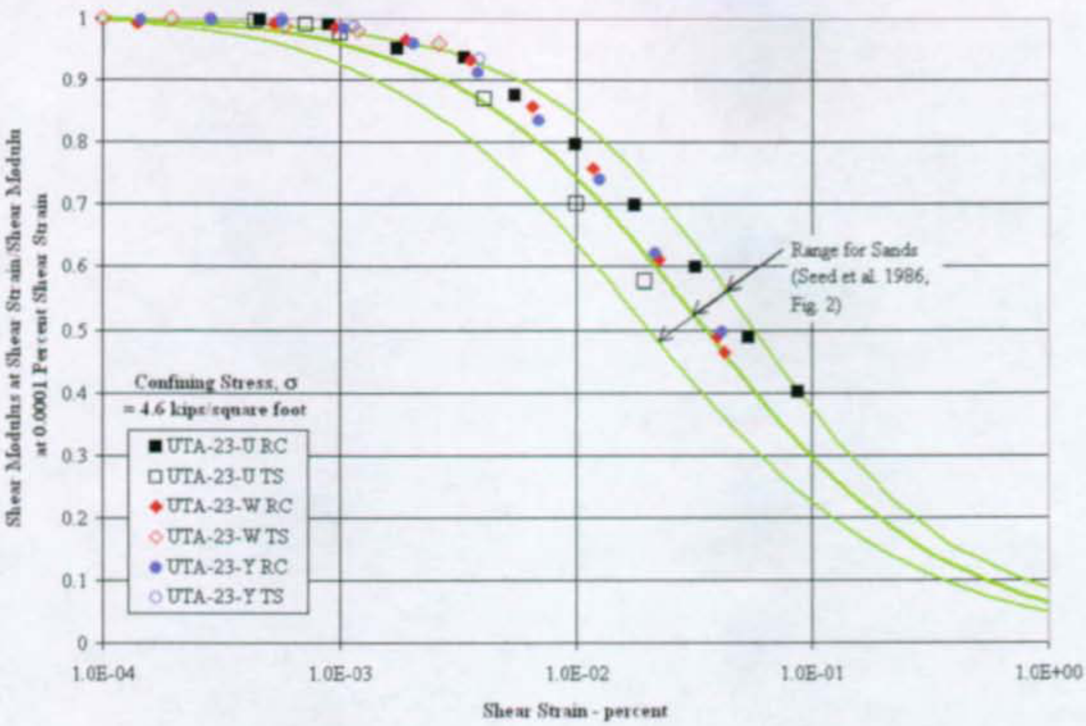


Figure I-5. Variation of Shear Modulus with Shear Strain, σ=4.6 kips/ft^2

<u>For σ' = 8 psi = 1.15 kips/ft^2</u>

Calculate G_{max} using equation I-20:

$$G_{max}\ (\sigma'=1.15\ \text{kips/ft}^2) = 2167\ \text{kips/ft}^2 \qquad \text{(Eq. I-21)}$$

Calculate G at an axial strain of 0.25 percent as follows.

To calculate the maximum shear strain, γ_{max}, corresponding to the major and minor principal normal strains, ε_1 and ε_3, use the relationship from Poulos and Davis (1991, page 6):

$$\gamma_{max} = \varepsilon_1 - \varepsilon_3 \qquad \text{(Eq. I-22)}$$

Given that Poisson's ratio, μ , $= -\varepsilon_3/\varepsilon_1$:

$$\gamma_{max} = \varepsilon_1 + \mu\varepsilon_1 = \varepsilon_1(1+\mu) \qquad \text{(Eq. I-23)}$$

Using the Mohr circle of strain, it can be shown that:

$$\gamma_{ff} = \gamma_{max}\cos\phi \qquad \text{(Eq. I-24)}$$

where: γ_{ff} is the shear stain on the failure plane at failure

γ_{max} is the maximum shear strain.

Substituting equation I-23 into equation I-24, the relationship between γ and ε_{1f} (ε_1 at failure) becomes:

$$\gamma_{ff} = \varepsilon_{1f}(1+\mu)\cos\phi \qquad \text{(Eq. I-25)}$$

Estimate μ as 0.35 from the common values for medium and dense cohesionless soils from Bowles (1996, page 123).

Estimate ϕ as 54 degrees (see shear strength at 1.15 kips/ft^2). Then equation I-25 becomes:

$$\gamma_{ff} = 0.79\ \varepsilon_1 \qquad \text{(Eq. I-26)}$$

For 0.25 percent axial strain, the shear strain from equation I-26 would be:

$$\gamma_{ff} = 0.20 \text{ percent} \qquad \text{(Eq. I-27)}$$

Calculate G at shear strain = 0.20 percent using the lower-bound Seed et al. (1986, Figure 2) curve, and the value of G_{max} from equation I-21 above:

$$G\ (\sigma'=1.15 \text{ kips/ft}^2,\ \gamma_{ff} = 0.20 \text{ percent}) = 319 \text{ kips/ft}^2 \qquad \text{(Eq. I-28)}$$

Calculate the secant Young's modulus for this shear modulus using the following equation (Lambe and Whitman 1969, page 151, equation 12.4):

$$E = 2(1+\mu)G \qquad \text{(Eq. I-29)}$$

where: μ is Poisson's ratio.

Estimate μ as 0.35 from the common values for medium and dense cohesionless soils from Bowles (1996, pages 123). Then, equations I-28 and I-29 yield:

$$E = 861 \text{ kips/ft}^2 \text{ from dynamic tests} \qquad \text{(Eq. I-30)}$$

The value of E calculated from the relationship for E developed from isotropically consolidated, drained triaxial tests for an axial strain of 0.25 percent (equation I-19) is:

$$E\ (\sigma'=1.15 \text{ kips/ft}^2) = 971 \text{ kips/ft}^2 \qquad \text{(Eq. I-31)}$$

Comparing this value of E (equation I-31) with the value derived from the shear modulus data (equation I-30), the percent difference is 11 percent. This is reasonable agreement between the dynamic and static testing for this level of stress and strain.

<u>For σ' = 32 psi = 4.6 kips/ft^2</u>

Calculate G_{max} based on the resonant column and torsional shear test results using equation I-20:

$$G_{max}\ (\sigma' = 4.6\ \text{kips/ft}^2) = 4884\ \text{kips/ft}^2 \qquad \text{(Eq. I-32)}$$

Calculate G at an axial strain of 0.25 percent as follows.

From equation I-25, the relationship between γ and ε_1 is:

$$\gamma = \varepsilon_1\ (1+\mu)\ \cos\phi \qquad \text{(Eq. I-25)}$$

Estimate μ as 0.35 from the common values for medium and dense cohesionless soils from Bowles (1996, page 123). Estimate ϕ as 49 degrees (see shear strength at 4.6 kips/ft^2). Then:

$$\gamma = 0.89\ \varepsilon_1 \qquad \text{(Eq. I-33)}$$

For 0.25 percent axial strain the shear strain would then be:

$$\gamma = 0.22\ \text{percent} \qquad \text{(Eq. I-34)}$$

Calculate G at shear strain = 0.22 percent using the Seed et al. (1986, Figure 2) mid-range curve, and G_{max} above:

$$G\ (\sigma' = 4.6\ \text{kips/ft}^2, \gamma = 0.22\ \text{percent}) = 887\ \text{kips/ft}^2 \qquad \text{(Eq. I-35)}$$

Calculate the secant Young's modulus for this shear modulus using the following (Lambe and Whitman 1969, page 151, equation 12.4):

$$E = 2(1+\mu)G \qquad \text{(Eq. I-36)}$$

where: μ is Poisson's ratio.

Estimate μ as 0.35 from the common values for medium and dense cohesionless soils from Bowles (1996, page 123). Then, the approximate value of E corresponding to the dynamic tests is:

$$E = 2395\ \text{kips/ft}^2 \qquad \text{(Eq. I-37)}$$

The value of E calculated from the relationship for E developed from isotropically consolidated, drained triaxial tests for an axial strain of 0.25 percent (equation I-19) is:

$$E\ (\sigma' = 4.6\ \text{kips/ft}^2) = 1822\ \text{kips/ft}^2 \qquad \text{(Eq. I-38)}$$

Comparing this value of E (equation I-38) with the value derived from the shear modulus data (equation I-37), the percent difference is -31 percent. This is not very good agreement between the dynamic and static testing. In this case the dynamic results are less conservative (they estimate a higher modulus).

Conclusion:

It is recommended to use secant Young's modulus from equation I-19, which is based on isotropically consolidated, drained triaxial test results. It is also recommended to use the initial overburden pressure, σ'_v, for σ' in equation I-19.

I.1.4 Interface Friction between Engineered Fill and Cast-in-Place Concrete

For resistance to lateral loads the interface friction between cast-in-place concrete and the foundation soils is needed.

Tests on the borrow material indicate it is a poorly graded sand with gravel (SP) (BSC 2002, Figure 214).

Table 1 on page 7.2-63 of Design Manual 7.02 (DON 1986) gives ultimate friction factors between mass concrete and various materials.

The factors for clean gravel, gravel-sand mixtures, and coarse sand are:

Table I-8. Ultimate Interface Friction Coefficient Between Mass Concrete and Clean Gravel, Gravel-Sand Mixtures, and Coarse Sand

Interface Friction Coefficient, tan δ	Interface Friction Angle, δ, degrees
0.55 to 0.60	29 to 31

The factors for clean fine to medium sand, silty medium to coarse sand, and silty or clayey gravel are:

Table I-9. Ultimate Interface Friction Coefficient Between Mass Concrete and Clean Fine to Medium Sand, Silty Medium to Coarse Sand, and Silty or Clayey Gravel

Interface Friction Coefficient, tan δ	Interface Friction Angle, δ, degrees
0.45 to 0.55	24 to 29

Based on the description of the Fran Ridge Borrow material the recommended friction factor is:

Table I-10. Recommended Ultimate Interface Friction Coefficient Between Mass Concrete and Engineered Fill

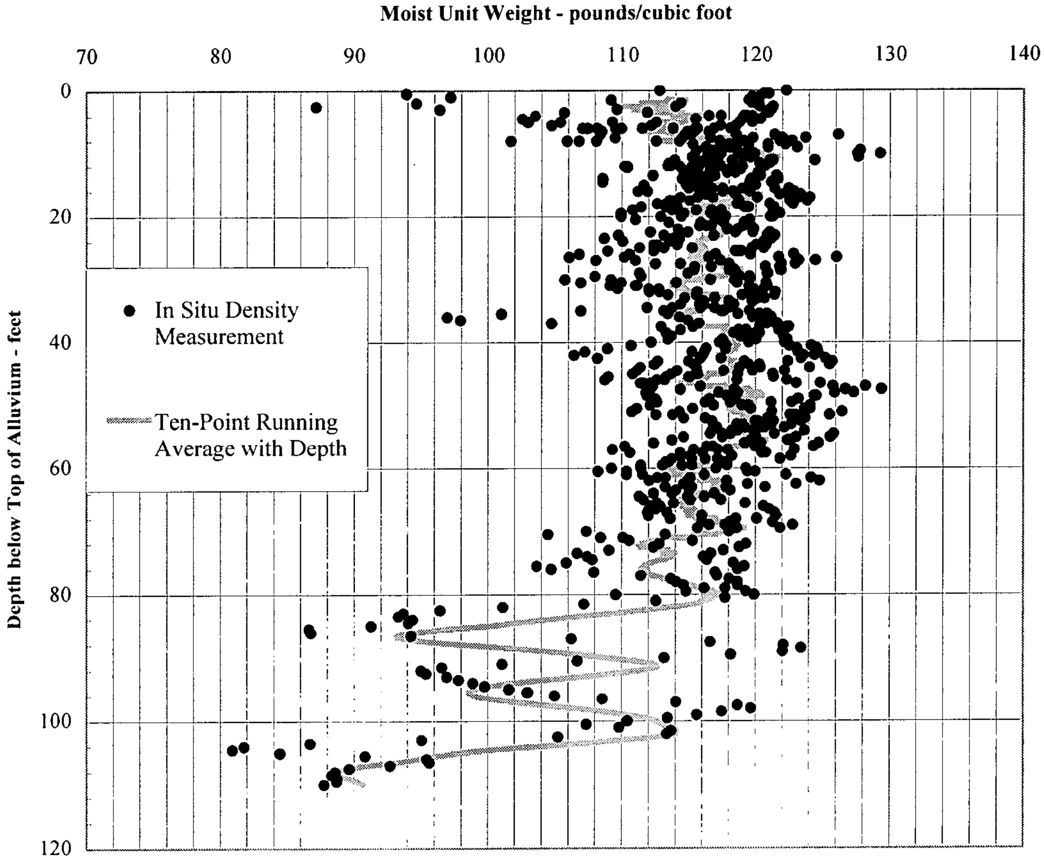
Interface Friction Coefficient, tan δ
0.55

I.2 Alluvium

I.2.1 Moist Unit Weight of Alluvium

Ref: Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project (BSC 2002).

Figure I-6 plots the moist unit weight data for alluvium from the reference report (DTNs: MO0112GPLOGWHB.001, GS020483114233.004, GS920983114220.001; Ho et al. 1986, page 14) versus depth below the top of alluvium. That is, if there was fill overlying the alluvium, then the depth of the fill was subtracted from the depths indicated on Figure 236 of BSC (2002). If the data were reported as density, the data were converted to unit weight. The density data for drive-tube samples taken from borehole UE-25 RF#3B (DTN: SNSAND85081500.000, Table 3) were not included in the analysis because the data are clearly not consistent with that derived by the other methods of obtaining unit weight. The unit weights from the drive tube specimens are generally lower and the likely reason for this is that the coarse granular soils are being loosened by the driving process (BSC 2002, Section 6.8.3).



DTNs: MO0112GPLOGWHB.001, GS020483114233.004, GS920983114220.001; Ho et al. 1986, page 14

Figure I-6. Moist Unit Weight vs Depth below top of Qal Contact

Based on the trends of moist unit weight versus depth on Figure I-6, the alluvium was divided into two units. The first is from 0 to 8 foot depth and the second from 8 to 70 foot depth. The average values from these depth ranges were 114 and 117 lbf/ft^3, respectively. The data plotted on Figure I-6 indicate that the moist unit weight decreases below about 70 foot depth. The lower unit weight values come from the data obtained in borehole RF#21 which was logged only from cuttings. It has been assumed (Section 5, Assumption 9) for the purposes of this calculation that the drill cuttings were misidentified, and that the unit weights are from bedrock.

Recommend using a moist unit weight for alluvium of 114 lbf/ft^3 in upper 8 feet and 117 lbf/ft^3 below 8 feet depth, where the depth is measured from the top of the alluvium contact.

I.2.2 Shear Strength of Alluvium

I.2.2.1 General

Ref: Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project (BSC 2002).

Two approaches can be taken to estimate the shear strength of the alluvium at the site: (1) correlations between measured shear-wave velocity and blow count (and/or other parameters) coupled with correlations between blow count (and/or other parameters) and shear strength, and (2) correlations between relative density and shear strength. Given the high shear-wave velocities measured in the alluvium (BSC 2002, Figures 4 to 5, 7 to 19, 32, 54, 54 to 55, 57, 61, 76, 82, 85 to 87, VII-1 to VII-2, and VII-4 to VII-16), the first approach (using shear-wave velocity) will yield the greater shear strength values. For conservatism, the second approach (using relative density) is taken.

Although, as will be seen below, the shear strength developed from the relative density approach is adequate for anticipated project needs (particularly bearing capacity) and it appears appropriate to adopt a conservative approach, it should be pointed out that the relative density values are lower than would be expected based on the high shear-wave velocity of the alluvium. The calculated values of relative density from the tests may be lower than actual values for several reasons:

1. The laboratory values of maximum and minimum density could have been affected by gravel interference. Gravel interference occurs when the larger soil particles situated at the rigid walls and base of the mold prevent finer particles from filling void space in the same manner as would occur in the field. This can result in falsely low values of maximum and minimum density of the overall material and of the "fine fraction" for cases where a fine fraction is being considered in the calculation.

2. The laboratory values of maximum and minimum densities could have been affected by layer mixing. Exposures in test pits TP-WHB-1 to -4 indicate that the alluvium generally includes thin layers of granular soil of recognizably different particle-size distribution. These layers were often only 4 to 6 inches thick. When layers of different gradation are involved in the in-place density test and are mixed together and submitted to the laboratory for determination

of maximum and minimum density and other characteristics, the particle-size distribution of the tested material spans a greater range than the distribution measured for the individual layers. The mixture could be a more well-graded soil, but it could be gap-graded. The mixing effect will lead to the measurement of maximum and minimum densities that are different than the values for any of the individual layers or for the weighted average of these layers. Often, relative density values less than zero are calculated if layers are mixed.

Despite these potential problems, the shear strength (friction angle) of alluvium has been estimated using the relative density data obtained on the site alluvium and correlations of friction angle with relative density, and expressed using four methods previously used in Section I.1.2. Relative density results for alluvium from Table 6 of BSC (2002) are summarized in Table I-11:

Table I-11. Summary of and Statistics for Relative Density Results

USCS Group Symbol	Depth (feet)	Relative Density (%)
GW-GM	4	74
GW-GM	12	74
GP-GM	12	41
SP-SM	12.3	48
GP	20	55
GP	8	86
GP-GM	12	68
GW-GM	16	70
SW-SM	16	54
SP-SM	16	81
GW	19	66
GP-GM	8	51
GP-GM	8	25
GP-GM	8	65
GP	12	84
GP	12	90
GP-GM	12	72
GW-GM	19	120
GW	4	75
GP-GM	8	36
GP	12	80
GW-GM	16	72

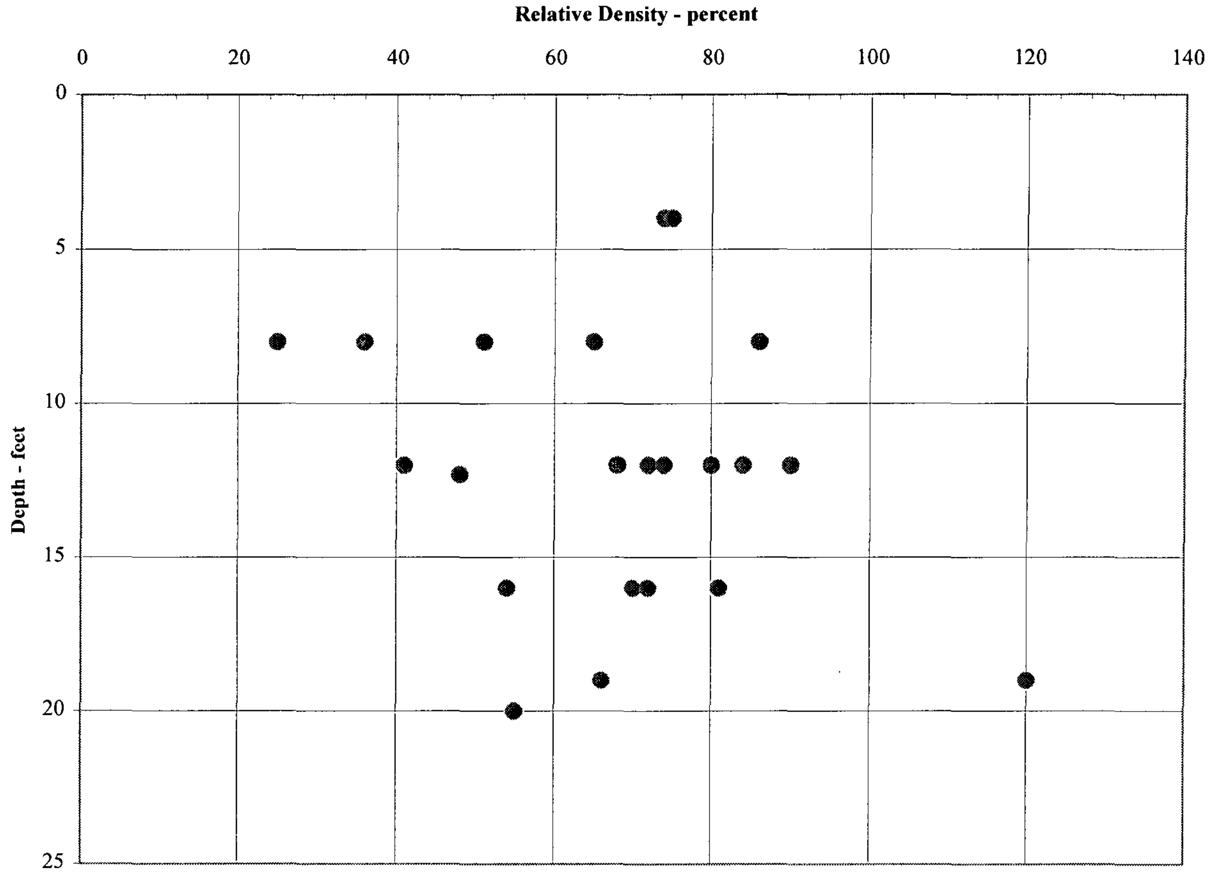
Relative Density Statistics	
Mean	68
Median	71
Mode	74
Standard Deviation	21
Range	95
Minimum	25
Maximum	120
Count	22

Source: BSC (2002, Table 6)

Relative density is plotted versus depth on Figure I-7. There is no clear correlation of relative density with depth. Use correlations of friction angle with relative density to evaluate the friction angle, enter the correlation using the mean value, 68, and mean plus and minus one standard deviation (47 and 89 percent, respectively).

Schmertmann's correlation (Bowles 1996, page 100). (Correlation 1):Entering Schmertmann's correlation chart with mean and mean plus and minus one standard deviation[1] values of relative density and using the correlations for uniform gravel/well-graded gravel-sand-silt and uniform coarse sand well-graded medium sand the following is evaluated in Table I-12

[1] In this attachment, "sigma" means standard deviation.



Source: Table I-11

Figure I-7. Relative Density Versus Depth

Table I-12. Friction Angle Estimated from Schmertmann's Correlation

	Relative Density %	Friction Angle degrees (Uniform Coarse Sand Line)	Friction Angle degrees (Uniform Gravel Line)
Mean minus 1 sigma	47	39.4	41.8
Mean	68	41.4	43.3
Mean plus 1 sigma	89	43.6	45.0

Design Manual 7.01 (USN 1986, page 7.1-149). (Correlation 2):

Entering this chart with mean and mean plus and minus one standard deviation values of relative density and using the correlations for SW and GW yields:

Table I-13. Friction Angle Estimated from Design Manual 7.01 Correlation

	Relative Density %	**Friction Angle degrees (Using SW line)**	**Friction Angle degrees (Using GW line)**
Mean minus 1 sigma	47	33.6	35.9
Mean	68	36.1	39
Mean plus 1 sigma	89	39.1	43

Meyerhof's correlation (Duncan, Horz and Yang 1989, page 18). (Correlation 3)

Entering table with mean and mean plus and minus one standard deviation values of relative density and using the correlations for clean sand and gravelly sand yields:

Table I-14. Friction Angle Estimated from Meyerhof's 1956 Correlation

	Relative Density %	Friction Angle degrees (Using Sand)	Friction Angle degrees (Using Gravelly Sand)
Mean minus 1 sigma	47	37.0	42.0
Mean	68	42.3	47.3
Mean plus 1 sigma	89	47.5	52.5

Mitchell and Katti's correlation (Duncan, Horz and Yang 1989, page 19). (Correlation 4)

Entering the table (Duncan, Horz and Yang 1989, page 19) with mean and mean plus and minus one standard deviation values of relative density yields:

Table I-15. Friction Angle Estimated from Mitchell and Katti's 1981 Correlation

	Relative Density %	Friction Angle degrees
Mean minus 1 sigma	47	33.4
Mean	68	35.8
Mean plus 1 sigma	89	38.1

Hatanaka and Uchida (1996, Equation 8) proposed a relationship between penetration resistance and drained internal friction angle: (Correlation 5)

$$\phi_d = 20 + (20N_1)^{0.5} \qquad \text{(Eq. I-39)}$$

where: ϕ_d = friction angle from drained triaxial tests
N_1 = SPT blow count normalized to 1 atmosphere confining stress

A relationship between SPT N value and relative density is also contained in Hatanaka and Uchida (1996, Equation 5):

$$D_r = 21(N/(\sigma_v'/98+0.7))^{0.5} \qquad \text{(Eq. I-40)}$$

where: D_r = relative density in percent
N = SPT blow count
σ'_v= effective overburden pressure in kPa

For an overburden pressure of one atmosphere (98 kPa) (where $N = N_1$), the previous equation is solved for N (N_1) as follows:

$$N_1 = 1.7(D_r/21)^2 \qquad \text{(Eq. I-41)}$$

Entering this into the correlation between ϕ_d and N_1 proposed by Hatanaka and Uchida (1996, equation 8), equation I-39, yields (for σ'_v = 1 atmosphere):

$$\phi_d = 20 + \sqrt{34}\left(\frac{D_r}{21}\right) \qquad \text{(Eq. I-42)}$$

Using equation I-42, the following values of ϕ_d are calculated:

Table I-16. Friction Angle Calculated from Equation I-42

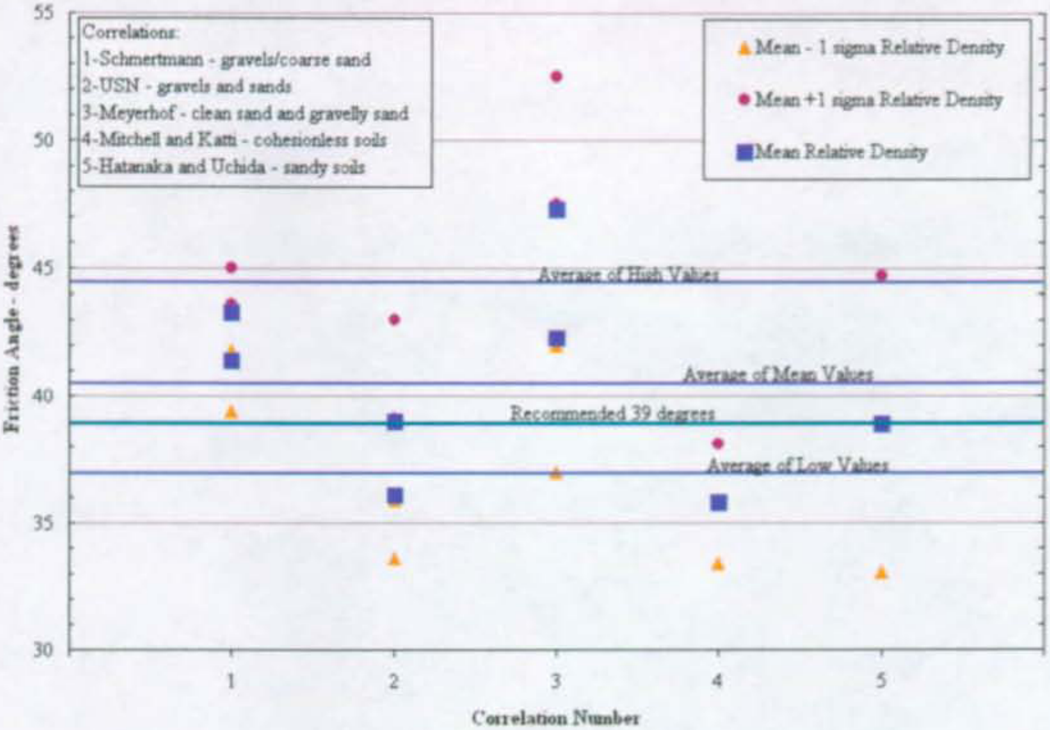
Relative Density %	Relative Density %	Friction Angle degrees
Mean minus 1 sigma	47	33.1
Mean	68	38.9
Mean plus 1 sigma	89	44.7

Summary Table for Plot (Figure I-8):

Table I-17. Summary of Friction Angles from Various Correlations

Correlation No.	low	mean	high
(1) Schmertmann/sand	39.4	41.4	43.6
(1) Schmertmann/gravelly sand	41.8	43.3	45.0
(2) USN/SW	33.6	36.1	39.1
(2) USN/GW	35.9	39.0	43.0
(3) Meyerhof/clean sand	37.0	42.3	47.5
(3) Meyerhof/gravelly sand	42.0	47.3	52.5
(4) Mitchell and Katti	33.4	35.8	38.1
(5) Hatanaka and Uchida/sandy soils	33.1	38.9	44.7
Average	37.0	40.5	44.2

Based on the correlations, an effective friction angle of 39 degrees corresponding to approximately halfway between the low and mean is recommended for an effective confining pressure of 1 atmosphere.



Source: Table I-17

Figure I-8. Friction Angle Estimated from Relative Density

The effects of confining pressure on the friction angle of the alluvium were evaluated using the data from Maeda and Miura (1999). They performed a number of triaxial compression tests on sands at varying confining pressures. They plotted the results as internal friction coefficient, tan ϕ_d versus confining pressure on Figure 20 of their paper. The tests were performed on "UNIFORM SAMPLE SO-SAND, D_r=70%." To evaluate the decrease in friction angle with a 10-fold increase in confining pressure ($\Delta\phi'$ in the equation for a generalized curved failure envelope), the values of tan ϕ_d were read off Figure 20 of Maeda and Miura (1999) for confining pressures of 50 and 500 kPa for each test series. The values of ϕ were calculated as the arctangent of tan ϕ_d and subtracted to get the value of $\Delta\phi'$. The values were then averaged. The results indicate that the change in friction angle ranged from 2.0 to 3.8 degrees and averaged 3 degrees. Using the average of 3 degrees, the curved failure envelope for the alluvium would be modeled as:

Generalized Curved Failure Envelope:

$$\tau_{ff} = \sigma'_{ff} \tan \phi' \qquad \text{(Eq. I-43)}$$

where: τ_{ff} = shear stress acting on the failure plane at failure, i.e., effective shear strength

σ'_{ff} = normal effective stress acting the failure plane at failure

ϕ' = effective friction angle as function of normal stress.

Equation I-7B gives the friction angle as a function of normal stress:

$$\phi' = \phi'_1 - \Delta\phi' \log(\sigma'_{ff}/p_a) \qquad \text{(Eq. I-7B)}$$

where: ϕ'_1 = the effective friction angle for σ'_{ff} = 1 atmosphere (p_a=2,116.22 lbf/ft^2)
$\Delta\phi'$ = the decrease in ϕ' per log cycle change in σ'_{ff}
p_a = 1 atmosphere (2,116.22 lbf/ft^2).

The values for triaxial compression selected are ϕ'_1 = 39 degrees and $\Delta\phi'$ = 3 degrees; that is:

$$\tau_{ff} = \sigma'_{ff} \tan\left[\phi'_1 - \Delta\phi' \log\left(\frac{\sigma'_{ff}}{p_a}\right)\right] = \sigma'_{ff} \tan\left[39° - 3° \log\left(\frac{\sigma'_{ff}}{p_a}\right)\right] \qquad \text{(Eq. I-44)}$$

I.2.2.2 Strength Envelope for Passive Pressure on Deep Below-Grade Wall

A 55-foot deep excavation is planned at the site for a pool in the wet-process facility. To develop the passive pressures for the walls, the general curved failure envelope above (Equation I-44) is used to calculate an equivalent linear Mohr-Coulomb failure envelope for an appropriate range of stress. This was done by selecting varying values of σ'_{ff} at regular stress increments and calculating the resulting friction angle and minor principal effective stress σ'_3 such that σ'_3 varied in regular increments up to the value of overburden pressure at a depth of 55 feet, (8 ft x 114 lbf/ft^2 + 47 ft x 117 lbf/ft^2)/ 1000 pounds/kip = 6.4 kips/ft^2. For the passive pressure calculation, the overburden pressure is taken as the minor principal stress. Finally, the values of p' and q were calculated and an equivalent linear Mohr-Coulomb failure envelope c' and ϕ' were fit. The calculated values are as follows:

Table I-18. Calculated Values of p' and q for Use in Estimating Strength to Use in Evaluating Passive Pressure on Deep Below-Grade Wall

σ'_{ff} kips/ft^2	ϕ' deg	σ'_3 kips/ft^2	p' kips/ft^2	q kips/ft^2
1.47	39.5	0.9	2.47	1.57
2.94	38.6	1.8	4.81	3.00
4.41	38.0	2.7	7.12	4.39
5.89	37.7	3.7	9.39	5.74
7.36	37.4	4.6	11.65	7.07
8.83	37.1	5.5	13.89	8.39
10.30	36.9	6.4	16.12	9.69

A linear regression of the values of q and p' in the table above yields a slope of 0.5942 and an intercept of 0.1361 kips/ft^2, which yields ϕ' = 36.5 degrees and c' = 0.169 kips/ft^2. Therefore, the generalize Mohr-Coulomb failure envelope would be:

$$\tau_{ff} = c' + \sigma'_{ff} \tan\phi' = 169 \text{ lbf/ft}^2 + \sigma'_{ff} \tan 36.5° \qquad \text{(Eq. I-45)}$$

where: σ'_{ff} = effective normal stress on the failure plane at failure.

This equivalent linear Mohr-Coulomb failure envelope is recommended for the calculation of passive earth pressures on a rigid structure embedded approximately 55 feet below grade. There maybe engineered fill located at the structure, however, it is conservative to use the unit weight and shear strength of alluvium. Because the unit weight and shear strength of the alluvium are lower than engineered fill the passive pressures will be lower.

I.2.2.3 Strength Envelope for Passive Pressure on Soldier Piles

Another case requiring an equivalent linear Mohr-Coulomb failure envelope is the passive pressure on soldier piles embedded in the alluvium. This was done by selecting varying values of σ'_{ff} at regular stress increments and calculating the resulting friction angle and minor principal effective stress σ'_3 such that σ'_3 varied in regular increments up to the value of overburden pressure at a depth of 20 feet: (20 ft x 117 lbf/ft^2)/1000 pounds/kip = 2.34 kips/ft^2. This depth was selected to represent the approximate embedment depth of a soldier pile below excavation grade. For the passive pressure calculation the minor principal effective stress is taken as the overburden pressure. Finally, the values of p' and q were calculated and an equivalent linear Mohr-Coulomb failure envelope c' and ϕ' were fit. The friction angle corresponding to triaxial extension is appropriate for this analysis (Kulhawy and Mayne 1990, Figure 4-6). Therefore, the basic curved failure envelope is modified by multiplying ϕ_{tc} by 1.12 (Kulhawy and Mayne 1990, page 4-14).

The values for triaxial extension selected are

$$\phi'_1 = 43.68 \text{ degrees } (1.12 \text{ times } 39°) \quad \text{(Eq. I-46)}$$

$$\Delta\phi' = 3 \text{ degrees} \quad \text{(Eq. I-47)}$$

Table I-19. Calculated Values of p' and q for Use in Estimating Strength to Use in Evaluating Passive Pressure on Soldier Piles

σ'_{ff} kips/ft^2	ϕ' degrees	σ'_3 kips/ft^2	p' kips/ft^2	q kips/ft^2
0.56	45.4	0.33	1.14	0.81
1.12	44.5	0.66	2.21	1.55
1.68	44.0	0.99	3.25	2.26
2.25	43.6	1.33	4.28	2.95
2.81	43.3	1.67	5.30	3.64
3.37	43.1	2.00	6.31	4.31
3.93	42.9	2.34	7.32	4.98

A linear regression of p' and q yields a slope of 0.674 and an intercept of 0.0573 kips/ft^2, which yields ϕ' = 42.4 degrees and c' = 0.078 kips/ft^2. Therefore, the generalize Mohr-Coulomb failure envelope would be:

$$\tau_{ff} = 78 \text{ lbf/ft}^2 + \sigma'_{ff} \tan 42.4° \qquad \text{(Eq. I-48)}$$

This equivalent linear Mohr-Coulomb failure envelope is recommended for the calculation of passive earth pressures on soldier beams embedded into alluvium.

I.2.2.4 Strength Envelope for Bearing Capacity

A method of modeling the curved failure envelope is presented in Ueno et al. (1998, page 167), and is given by equation I-11:

$$\tan \phi' = \tan(\phi'_a)(\sigma_a/\sigma'_m)^{\alpha} \qquad \text{(Eq. I-11)}$$

where: ϕ' is the effective friction angle at the mean effective stress of σ'_m
σ_a is 1 atmosphere (2.11622 kips/ft^2)
ϕ'_a is the friction angle at one atmosphere
σ'_m is equal to $(\sigma_1+\sigma_3)/2$

Calculating the values using ϕ'_a = 39 degrees and $\Delta\phi'$ = 3 degrees, the following is obtained:

Table I-20. Generated Mohr Circles and Resultant Values of $\log(\sigma_a/\sigma'_m)$ and log $\tan\phi'$ for Use in Evaluating Shear Strength of Alluvium for Bearing Capacity Calculations

σ'_{ff} kips/ft^2	ϕ'	σ'_3	p' kips/ft^2	q kips/ft^2	σ'_m kips/ft^2	σ_a/σ'_m	tan ϕ'	$\log(\sigma_a/\sigma'_m)$	log tanϕ'
2.000	39.1	1.227	3.318	2.092	3.32	0.63772	0.81191	-0.19537	-0.09049
4.000	38.2	2.472	6.472	4.000	6.47	0.32699	0.78609	-0.48546	-0.10453
6.000	37.6	3.725	9.569	5.844	9.57	0.22115	0.77128	-0.65532	-0.11279
8.000	37.3	4.983	12.632	7.649	12.63	0.16753	0.76090	-0.77590	-0.11867
10.000	37.0	6.244	15.669	9.425	15.67	0.13506	0.75292	-0.86948	-0.12325

A linear regression (using LINEST in Excel) of this $\log(\sigma_a/\sigma'_m)$ versus log $\tan\phi'$ data yields a slope of 0.04859411 and an intercept of -0.0809701. This means that α = 0.04859411 and $\tan \phi'_a = 10^{-0.0809701}$ = 0.82990795, that is (with rounding):

$$\tan \phi' = 0.8299\,(\sigma_a/\sigma'_m)^{0.0486} \qquad \text{(Eq. I-49A)}$$

$$\tau_{ff} = \sigma'_{ff} \tan \phi' = 0.8299\,\sigma'_{ff}\,(\sigma_a/\sigma'_m)^{0.0486} \qquad \text{(Eq. I-49B)}$$

where: ϕ' = the effective friction angle as a function of σ'_m
σ_a = 1 atmosphere expressed in the same units as σ'_m
σ'_m = $\frac{1}{2}(\sigma'_1+\sigma'_3)$

Check fit (Equation I-49A) against test results:

Table I-21. Check Fit of Equation I-49A Against the Mohr Circles Generated in Table I-20

σ'_m kips/ft^2	σ_a/σ'_m	ϕ' curved	ϕ' from equation
3.32	0.63772199	39.1	39.08
6.47	0.32699376	38.2	38.17
9.57	0.22114857	37.6	37.64
12.63	0.16753223	37.3	37.27
15.67	0.13505924	37.0	36.98

The fit is good. Suggest using the following in conjunction with the method of evaluating bearing capacity by Ueno et al. (1998):

$$\tau_{ff} = \sigma'_{ff} \tan \phi' = 0.8299\, \sigma'_{ff} (\sigma_a/\sigma'_m)^{0.0486} \qquad \text{(Eq. I-49B)}$$

where: ϕ' = the effective friction angle as a function of σ'_m
σ_a = 1 atmosphere expressed in the same units as σ'_m (e.g. 2.11622 kips/ft^2)
$\sigma'_m = \frac{1}{2}(\sigma'_1+\sigma'_3)$

I.2.2.5 Strength Envelope for Slope Stability

A method of modeling the strength is used by Charles and Soares (1984) for slope stability analysis.

The form of the equation is given by equation I-13:

$$\tau_{ff} = A(\sigma')^b \qquad \text{(Eq. I-13)}$$

where: τ_{ff} = shear stress on the failure plane
σ'_{ff} = effective normal stress on the failure plane
A and b are constants

For slope stability analyses, plane strain conditions are applicable based on the summary provided in Kulhawy and Mayne (1990, Figure 4-6). Table 4-2 of Kulhawy and Mayne (1990) indicates that the friction angle in plane strain compression is approximately equal to:

$$\phi_{psc} = 1.12\phi_{tc} \qquad \text{(Eq. I-50)}$$

where: ϕ_{psc} is the plane strain compression friction angle in degrees
ϕ_{tc} is the triaxial compression friction angle in degrees

The general curved failure envelope above is based on triaxial compression testing, therefore, the curved failure envelope for plane strain compression conditions would be (1.12)(39 degrees), or 43.68 degrees.

From the curved failure envelope, with ϕ'_a of 44 degrees and $\Delta\phi'$ of 3 degrees, the following is calculated:

Table I-22. Values of σ'_{ff} and τ_{ff} for Use in Evaluating Shear Strength of Alluvium for Use in Slope Stability Analysis

σ'_{ff} kips/ft^2	ϕ' degrees	σ'_{3f} kips/ft^2	p' kips/ft^2	q kips/ft^2	τ_{ff} kips/ft^2	$\log(\sigma'_{ff})$	$\log(\tau_{ff})$
0.25	46.5	0.1	0.53	0.38	0.263	-0.60206	-0.5798737
0.50	45.6	0.3	1.02	0.73	0.510	-0.30103	-0.2925435
1.00	44.7	0.6	1.98	1.39	0.988	0	-0.0052047
1.50	44.1	0.9	2.91	2.03	1.455	0.17609126	0.16287612
2.00	43.8	1.2	3.83	2.65	1.915	0.30103	0.28212882

To determine A and b, perform a linear regression of log σ'_{ff} and log τ_{ff}.

Using the linear regression function (LINEST) in Excel, the slope is 0.95450603 and the intercept is -0.0052055, yielding A = 0.98808555 and B = $10^{-0.005255}$ = 0.95450603, or:

$$\tau_{ff} = 0.98808555\,(\sigma'_{ff})^{0.95450603} \qquad \text{(Eq. I-51)}$$

where: τ_{ff} = shear stress on the failure plane in kips/ft^2
σ'_{ff} = effective normal stress on the failure plane in kips/ft^2

Note that the value of the constant multiplier in equation I-51 depends on the system of units being used, while the exponent is independent of the system of units.

Check the fit of equation I-51 against the test results:

Table I-23. Check Fit of Equation I-51 Against the Mohr Circles Generated in Table I-22

σ'_{ff} kips/ft^2	τ_{ff} kips/ft^2	τ_{ff} from eq. kips/ft^2
0.25	0.26	0.26
0.50	0.51	0.51
1.00	0.99	0.99
1.50	1.46	1.46
2.00	1.91	1.91

The fit is good. This model (equation I-51) is recommended in conjunction with the slope stability chart method of Charles and Soares (1984).

I.2.3 Compressibility of Alluvium

Ref: *Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project* (BSC 2002).

Using the shear-wave velocity data from BSC (2002) (DTNs: MO0111DVDWHBSC.001, MO0110DVDBOREH.000, MO0110SASWWHBS.000), obtain the small-strain shear modulus G_{max}. All of the alluvial shear-wave velocity data from the database of SASW surveys and downhole surveys were put into one spreadsheet. A G_{max}, σ_v ordered pair was created at one-foot depth intervals for each survey. G_{max} and σ_v at each point was calculated using the following equations (the first of which is from Bowles 1996, page 1108):

$$G_{max} = (\gamma/32.174)v_s^2/1000 \qquad \text{(Eq. I-52)}$$

where: γ = 117 lbf/ft^3 (typical density of alluvium from Section I.2.1)
v_s = the shear wave velocity in ft/s at depth d
G_{max} = small-strain shear modulus in kips/ft^2

and:

$$\sigma_v = \gamma d \qquad \text{(Eq. I-53)}$$

where: γ = 117 lbf/ft^3 or 0.117 kips/ft^3 (typical density of alluvium from Section I.2.1)
d = the depth of the measurement in feet.
σ_v is the overburden pressure in kips/ft^2

The values of G_{max} were then plotted versus σ_v on Figure I-9. The data were then clipped by removing values of G_{max} greater than 30,000 kips/ft^2, which were considered "outliers". The mean and standard deviation of the G_{max} values were then calculated.

The statistical analysis indicated that the mean value of G_{max} is 16326 kips/ft^2 and the standard deviation is 6115 kips/ft^2.

A linear regression was performed on the clipped data using the "slope" and "intercept" functions of Excel. The linear regression equation is:

$$\text{mean } G_{max}\ (\text{kips/ft}^2) = 8755 + 1513\sigma_v \qquad \text{(Eq. I-54)}$$

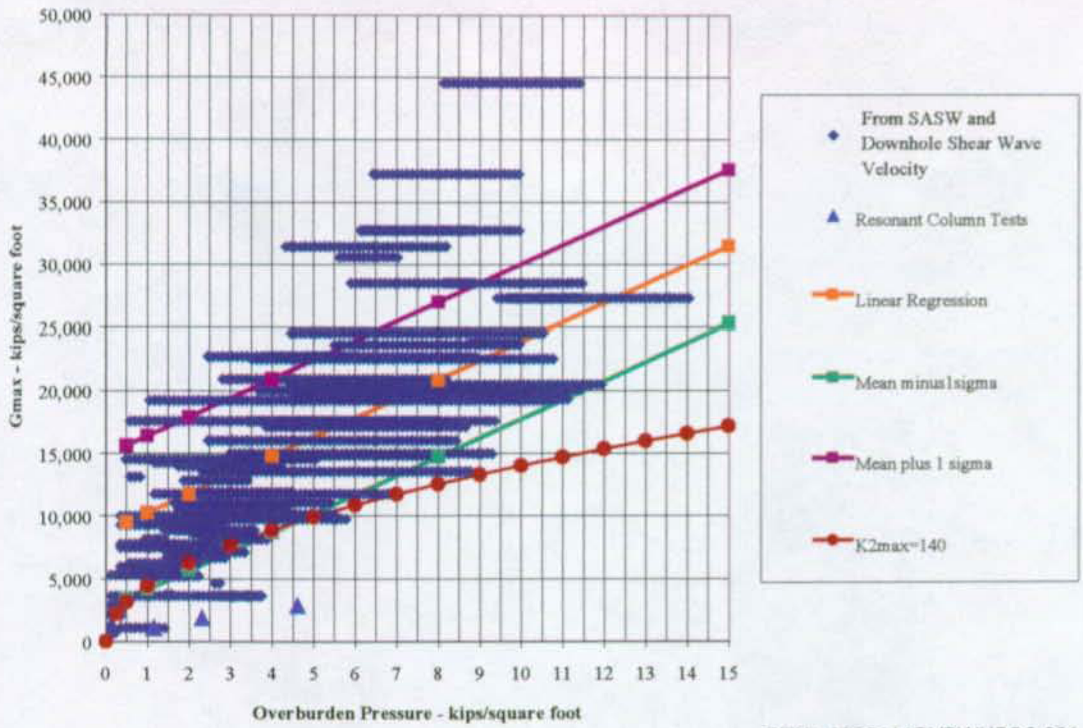
where: σ_v is the overburden pressure in kips/ft^2.

The mean minus one standard deviation (mean-1σ or mean $-$ 1 sigma) relationship would be:

$$G_{max}\ (\text{kips/ft}^2) = 2640 + 1513\sigma_v \qquad \text{(Eq. I-55)}$$

The mean plus one standard deviation (mean$+1\sigma$ or mean + 1 sigma) relationship would be:

$$G_{max}\ (\text{kips/ft}^2) = 14870 + 1513\sigma_v \qquad \text{(Eq. I-56)}$$



DTNs: MO0111DVDWHBSC.001, MO0110DVDBOREH.000, MO0110SASWWHBS.000

Figure I-9. Gmax vs. Overburden Pressure for Qal

Therefore, for plotting lines on Figure I-9:

Table I-24. Statistical Values of G_{max} Versus Overburden Pressure

σ_v kips/ft^2	G_{max}, kips/ft^2		
	Mean	Mean -1 sigma	Mean +1 sigma
0.5	9512	3397	15627
1	10268	4153	16383
2	11781	5666	17896
4	14807	8692	20922
8	20859	14744	26974
15	31450	25335	37565

The results of resonant column testing on reconstituted alluvial specimens (BSC 2002, Table XII-19a in Attachment XII) indicated:

Table I-25. Results of Resonant Column Tests on Reconstituted Alluvial Specimens

σ'_0 kips/ft^2	G_{max} kips/ft^2
1.152	1098
2.304	1875
4.608	2854

These values are also plotted on Figure I-9 and are low compared to the values derived from the shear-wave velocity measurements. It is noted that the dry density of the resonant column specimens was much lower than typical in situ values. This could explain the difference in the G_{max} values. Studies have shown that the value of G_{max} is sensitive to relative density. Because of this and because the shear-wave velocities that were converted to G_{max} values and plotted on Figure I-9 were measured in situ and reflect significant in situ conditions such as stress state, density, age, stress history, structure and cementation (if present), it is concluded that the G_{max} values from shear-wave velocity are the more representative values.

A method of correlating G_{max} with stress is described by Seed et al. (1986, page 1017, Equation 1). The form of the equation is:

$$G_{max} = 1000K_{2max}\,(\sigma'_m)^{1/2} \qquad \text{(Eq. I-57)}$$

where: σ'_m is confining stress in lbf/ft^2
K_{2max} is a coefficient
G_{max} is the low-strain shear modulus in lbf/ft^2.

Seed et al. (1986, page 1023, Table 4) quoted values of K_{2max} for gravelly soils that ranged from 90 to 188 based on shear-wave velocity measurements. The mid-range of these values would be about 140. This value was used in the equation above with adjustment of units (lbf/ft^2 to kips/ft^2) to derive the following equation:

$$G_{max} = 140(1000\sigma')^{1/2} \qquad \text{(Eq. I-58)}$$

where: σ' is stress in kips/ft^2
G_{max} is low-strain shear modulus in kips/ft^2.

Values of G_{max} calculated using equation I-58 are presented on Figure I-9 as the curve identified as K_{2max}=140.

The results of these analyses plotted on Figure I-9 indicate that equation I-58 is nearly a lower bound of the results of the shear-wave velocity derived results. Equation I-58 nearly follows the mean minus one standard deviation curve in the stress range of 0.5 to 6 kips/ft^2, which is generally the stress range of interest in the foundation analyses.

The variation of normalized shear modulus, G/G_{max}, presented on Figure 142 (BSC 2002) indicate that the variation follows closely the "average" curve for sands from Seed et al. (1986,

Figure 2). As noted previously the density of the dynamic test specimen was low. It has been reported (Goto et al. 1994, pages 154 and 156; Konno et al. 1994, pages 188 and 199), though, that the variation of G/G_{max} with strain is relatively insensitive to relative density and it is therefore recommended that Seed et al.'s (1986, Figure 2) "average" relationship for sands be used.

Based on the above relationships and for σ=1.15 kips/ft^2:

Calculate G_{max} using equation I-58:

$$G_{max}\ (\sigma'=1.15\ \text{kips/ft}^2) = 4748\ \text{kips/ft}^2 \qquad \text{(Eq. I-59)}$$

Calculate G at an axial strain of 0.25 percent, which corresponds to $\gamma_{ff} = 1.049147\varepsilon_1$ based on equation I-25 with $\mu = 0.35$ (Bowles 1996, page 123).

Estimate ϕ as 39 degrees from strength correlations (Section I.2.2.1).

For axial strain of 0.25 percent, the shear strain would then be

$$\gamma_{ff} = 1.049147\ \varepsilon_1 = 0.262287 \qquad \text{(Eq. I-60)}$$

Calculate G at shear strain = 0.262287 percent using the Seed et al. (1986, Figure 2) "average" sand curve and G_{max} above:

$$G\ (\sigma'=1.15\ \text{kips/ft}^2, \gamma=0.262\ \text{percent}) = 771\ \text{kips/ft}^2 \qquad \text{(Eq. I-61)}$$

Calculate secant Young's modulus from this shear modulus using the following (Lambe and Whitman 1969, page 151, equation 12.4):

$$E = 2(1+\mu)G \qquad \text{(Eq. I-62)}$$

Estimate Poisson's ratio, μ, as 0.35 from the common values for medium and dense cohesionless soils from Bowles (1996, page 123).

Therefore:

$$E = 2082\ \text{kips/ft}^2 \qquad \text{(Eq. I-63)}$$

Table I-26 gives values of E for various stress and strain conditions following this approach.

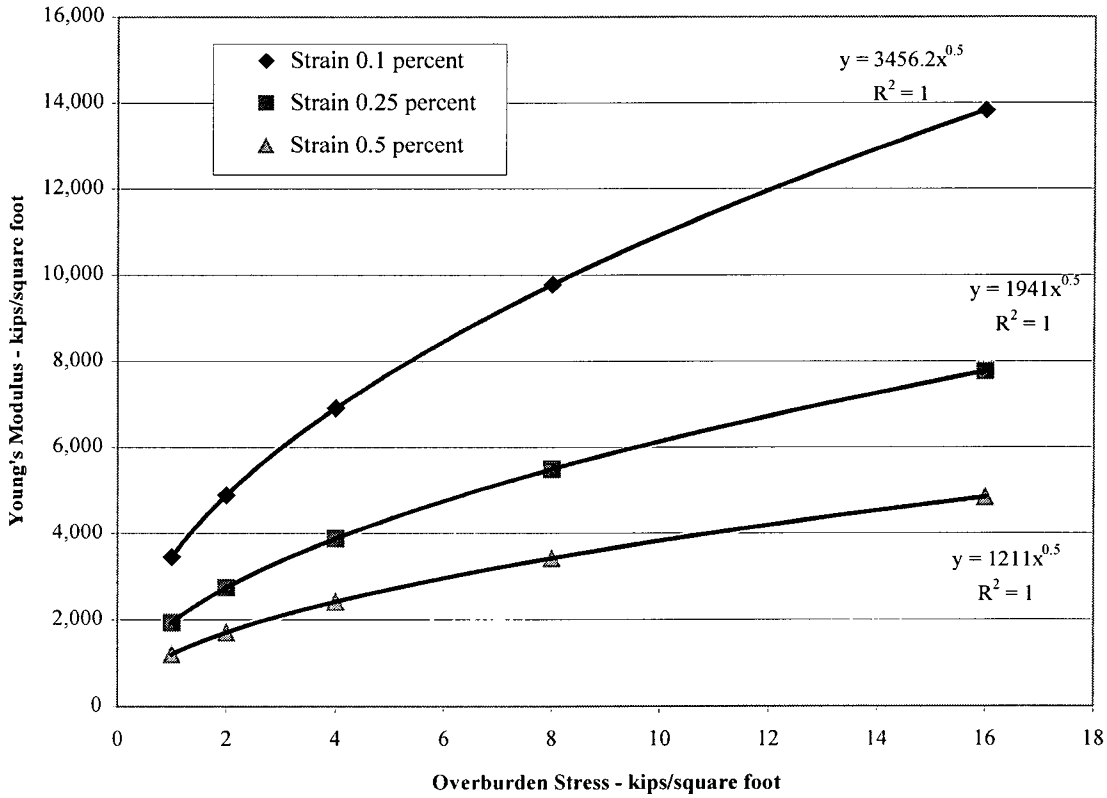
Figure I-10 plots E versus overburden for the three strain levels in Table I-26 and the power fit (equation I-64) to each.

$$E = a\sigma^{1/2} \qquad \text{(Eq. I-64)}$$

where: σ and E are in kips/ft^2.

Table I-26. Modulus E for Various Stress and Strain Conditions

Stress kips/ft^2	Axial Strain %	G_{max} kips/ft^2	γ_f percent	G/G_{max}	G kips/ft^2	E kips/ft^2
1	0.1	4427.19	0.104915	0.2891	1280	3456
2	0.1	6260.99	0.104915	0.2891	1810	4888
4	0.1	8854.38	0.104915	0.2891	2560	6912
8	0.1	12521.98	0.104915	0.2891	3621	9776
16	0.1	17708.75	0.104915	0.2891	5120	13825
1	0.25	4427.19	0.262287	0.1624	719	1941
2	0.25	6260.99	0.262287	0.1624	1017	2745
4	0.25	8854.38	0.262287	0.1624	1438	3882
8	0.25	12521.98	0.262287	0.1624	2033	5490
16	0.25	17708.75	0.262287	0.1624	2876	7764
1	0.5	4427.19	0.524574	0.1013	449	1211
2	0.5	6260.99	0.524574	0.1013	634	1713
4	0.5	8854.38	0.524574	0.1013	897	2422
8	0.5	12521.98	0.524574	0.1013	1269	3425
16	0.5	17708.75	0.524574	0.1013	1794	4844



Source: Table I-26

Figure I-10. Young's Modulus Estimated from Shear Wave Velocity

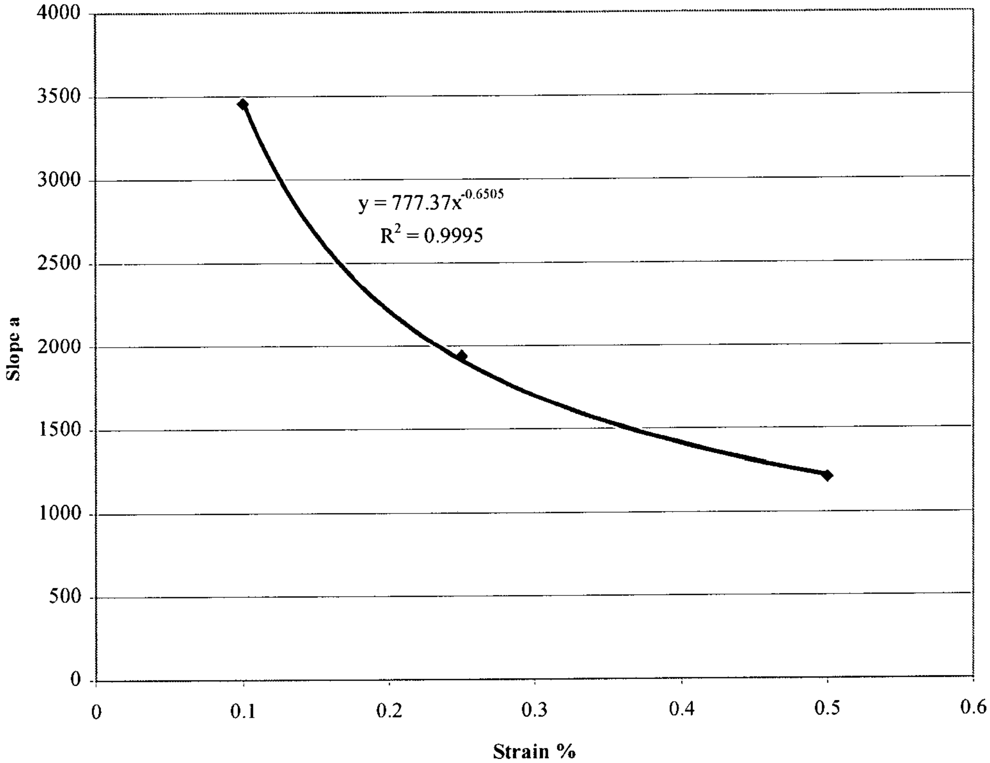
The values of coefficient a are:

Table I-27. Values of Coefficient a Versus Strain

Axial Strain, %	a
0.1	3456
0.25	1941
0.5	1211

Figure I-11 plots "a" versus strain and indicates the following fit:

$$a = 777.37(\varepsilon)^{-0.6505} \quad \text{(Eq. I-65)}$$



Source: Figure I-10.

Figure I-11. Slope of E vs Sigma versus Strain

This would mean that the secant Young's modulus, E, in kips/ft^2 is:

$$E = 777.37(\varepsilon)^{-0.6505}\sigma^{0.5} \quad \text{(Eq. I-66)}$$

where: ε is axial strain in percent
σ is vertical overburden stress in kips/ft^2.

Equation I-66 is applicable for the strain range of 0.1 to 0.5 percent. It is also applicable for the stress range of 0 to 6 kips/ft^2 and would be very conservative for stresses greater than 6 kips/ft^2.

It is recommended that this relationship (Eq. I-66) be used for secant Young's modulus within the strain range of 0.1 to 0.5 percent and within the stress range from 0 to 6 kips/ft^2.

I.2.4 Interface Friction

For resistance to lateral loads, the interface friction between cast-in-place concrete and the foundation soils is needed.

Tests on the alluvial material indicate it consists mainly of GP, GP-GM, GW-GM, and lesser amounts of SP-SM, SW-SM, SM and GM material.

Design Manual 7.02 (DON 1986, Table 1 on page 7.2-63) gives ultimate friction factors between mass concrete and various materials.

The factors for clean gravel, gravel-sand mixtures, and coarse sand are:

Table I-28. Ultimate Interface Friction Coefficient Between Mass Concrete and Clean Gravel, Gravel-Sand Mixtures, and Coarse Sand

Interface Friction Coefficient, tan δ	Interface Friction Angle, δ, degrees
0.55 to 0.60	29 to 31

The factors for clean fine to medium sand, silty medium to coarse sand, and silty or clayey gravel are:

Table I-29. Ultimate Interface Friction Coefficient Between Mass Concrete and Clean Fine to Medium Sand, Silty Medium to Coarse Sand, and Silty or Clayey Gravel

Interface Friction Coefficient, tan δ	Interface Friction Angle, δ, degrees
0.45 to 0.55	24 to 29

Based on the description of the alluvial material, the recommended friction factor is:

Table I-30. Recommended Ultimate Interface Friction Coefficient Between Mass Concrete and Engineered Fill

Interface Friction Coefficient, tan δ
0.55

I.3 Bedrock

I.3.1 Moist Unit Weight of Bedrock

Ref: *Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project* (BSC 2002).

Use the lower unit weight of rock as this is conservative in bearing capacity calculations.

Base the moist unit weight of the rock on the results for Tpki (the lower unit weight rock unit). From Table 12 of BSC (2002), the average unit weight of Tpki is 98.25 lbf/ft^3 based on the results of gamma-gamma density measurements. From Table 34 of BSC (2002), the average saturated unit weight of Tpki is 103.5 lbf/ft^3 based on the results of laboratory testing. It is recommended that a moist unit weight of 100 lbf/ft^3 be used for rock.

I.3.2 Shear Strength of Bedrock

Ref: *Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project* (BSC 2002).

There is no shear strength data for the rock.

Because it is anticipated that foundations will be constructed on either fill or alluvium but not on rock, the shear strength of rock is not a critical property and a conservative value may be used in calculations.

The rock is considered at least as strong as the alluvium based on the following:

1) The shear-wave velocity of the rock is consistently greater than that of the alluvium (BSC 2002, Table VII-1).

2) The unconfined compressive strength of the rock from measurements at the North Ramp and repository block (DTN: SNL01A05059301.005, SNL02030193001.001 through SNL02030193001.024, SNL02030193001.026, SNL02030193001.027) indicate a minimum unconfined compressive strength of 0.8 MPa (16.75 kips/ft^2), and this was measured on a sample of the weak bedded tuffs that are below the Tiva Canyon Tuff and hence would not even be of consequence for bearing capacity for the surface facilities (lithostratigraphic unit contact depths were based on DTNs: GS940308314211.009, GS940708314211.032, GS940908314211.045, and GS941108314211.052). Using the curved failure envelope developed for alluvium (I.2.2.1), ϕ'_1, = 39 degrees, $\Delta\phi'$ = 3 degrees, 0.8 MPa would correspond to the strength of the alluvium at about 198 feet depth.

It is recommended that the same strength parameters be used for rock as are used for the alluvium.

I.3.3 Compressibility of Bedrock

Ref: *Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project* (BSC 2002).

In reviewing the summaries of shear-wave velocity measurements in the rock it is estimated that the lower end of the shear-wave velocities generally occur in the Tpcr[2] and are about 2,700 ft/s. See Figures 32 and 33 of BSC (2002) for a summary of the data from the suspension method.

Calculate the G_{max} for this shear-wave velocity using equation I-52 (Bowles 1996, page 1108):

$$G_{max} = (\gamma/32.174)v_s^2/1000 \qquad \text{(Eq. I-52)}$$

where: γ is unit weight, 117 lbf/ft^3 (average for Tpcr, see BSC 2002, Table 12)
v_s is shear-wave velocity in ft/s at depth d
G_{max} is small-strain shear modulus in kips/ft^2

$$G_{max} = 26{,}510 \text{ kips/ft}^2 \qquad \text{(Eq. I-67)}$$

Based on the results of the dynamic testing of specimens of Tiva Canyon Tuff, there is little modulus degradation with shear strain (see BSC 2002, Figures 126, 127 and 128). A conservative value of G/G_{max} of 0.8 at a shear strain of 0.1 percent was chosen to get a lower-bound value of G.

This would mean that a secant shear modulus for high strains would be about:

$$G = 21{,}208 \text{ kips/ft}^2 \qquad \text{(Eq. I-68)}$$

Calculate the secant Young's modulus from this secant shear modulus using the following (Lambe and Whitman 1969, page 151 equation 12.4):

$$E = 2(1+\mu)G \qquad \text{(Eq. I-69)}$$

where: μ is Poisson's ratio.

μ is 0.3 from the suspension measurements for Tpcr (BSC 2002, Figure 37, Table VII-4). Then:

$$E = 55{,}141 \text{ kips/ft}^2 \qquad \text{(Eq. I-70)}$$

A secant Young's modulus of 55,000 kips/ft^2 is recommended for rock.

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2 Note: In BSC (2002), Tiva Canyon Tuff crystal-rich member (Tpcr) is referred to as Tpcrn.

Attachment II
Guideline Earthwork Specifications

This section presents recommendations for earthwork in the form of suggested specifications written in Construction Specification Institute format. Reference is made to several other potential specification sections that are not further discussed in this report, though it is expected that they, or some alternative formulation, will be required for the final construction specifications. It is emphasized that these are not actual specifications, but merely a means of providing recommendations for earthwork in a comprehensive, systematic manner.

SECTION 02315
ENGINEERED FILL

PART 1 GENERAL

1.01 DESCRIPTION

A. This Section of the Specifications covers the preparation of subgrade to receive fill; the type of materials suitable for use in fills and backfills; the placement of fills; backfilling around the structures; the compaction standards; and the methods of testing compacted fills and backfills.

B. The CONTRACTOR shall furnish all labor, supervision, equipment, operations, and materials to locate and support existing underground and aboveground facilities, to remove existing unsatisfactory material, excavate to the required grade, dispose of excavated soil and rock, prepare areas to be filled, import fill materials from the Fran Ridge Borrow Site, moisture condition fill materials, spread and compact fill and backfill, fine grade and dispose of excess and unsuitable materials.

C. It shall be the CONTRACTOR's responsibility to place, spread, moisten or dry, and compact the fill in strict accordance with these Specifications to the lines and grades indicated on project Drawings or as directed in writing by the ENGINEER.

D. Deviations from these Specifications will be permitted only upon written authorization from the OWNER or his representative.

1.02 RELATED WORK SPECIFIED ELSEWHERE

A. Section 01300, Submittals

B. Section 02220, Demolition

C. Section 02230, Clearing, Grubbing and Stripping

D. Section 02360, Excavation Support and Protection

1.03 REFERENCE STANDARDS

A. The publications listed below form a part of this section of the Specifications to the extent referenced. The publications are referenced in the text by basic designation only. Where a date is given for reference standards, that edition shall be used. Where no date is given for reference standards, the latest edition available on the date of the mandatory prebid conference stipulated in the Instructions to Bidders shall be used.

B. American Society for Testing and Materials (ASTM):

1. ASTM C 117 - Standard Test Method for Materials Finer than 75-µm (No 200) Sieve in Mineral Aggregates by Washing.
2. ASTM C 127 – Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate.
3. ASTM C 136 - Standard Test Method for Sieve Analysis of Fine and Coarse Grained Aggregates.
4. ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils.
5. ASTM D 1556 - Standard Test Method for Density of Soil in Place by the Sand-Cone Method.
6. ASTM D 1557 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 [2,700 kN-m/m^3]).
7. ASTM D 2216 - Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.
8. ASTM D 2922 - Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth).
9. ASTM D 3017 - Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth).
10. ASTM D 4318 - Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.
11. ASTM D 4718 - Standard Practice for Correction of Unit Weight and Water Content for Soils Containing Oversize Particles.

1.04 DEFINITIONS

A. Maximum dry density or reference maximum dry density - the maximum dry density in pound-mass per cubic foot, obtained in accordance with laboratory compaction standard ASTM D 1557 Method C. The maximum dry density is determined for the minus ¾ -inch fraction of the soil.

B. Optimum water content - the water content at which a material can be compacted to its maximum dry density when compacted in accordance with laboratory compaction standard ASTM D 1557. The optimum water content is determined for the minus ¾-inch fraction of the soil.

C. In-place water content - the water content of the total fill material at the time it is compacted, as determined by ASTM D 2216.

D. Field compaction water content - the water content of the minus ¾-inch fraction of the fill material at the time it is compacted, as determined by ASTM D 2216.

E. Field compaction wet density – The unit weight of the minus ¾-inch fraction of the fill material determined in the field in the compacted fill using either ASTM D 1556 or ASTM D 2922 in conjunction with ASTM D 4718.

F. Percent compaction or relative compaction – the degree to which fill is compacted or is to be compacted. It is the ratio, expressed as a percentage, of the field compaction dry density of the minus ¾-inch material to the maximum dry density of the minus ¾-inch material obtained using the specified laboratory compaction standard.

G. Over-excavation – Unauthorized excavation beyond the limits of excavation as shown on the Drawings or otherwise indicated in writing by the Engineer.

H. Segregate – To decompose or separate a material into two parts with different particle-size distributions.

1.05 SUBMITTALS

A. The CONTRACTOR shall submit in accordance with Section 01300, Submittals.

B. The CONTRACTOR shall submit a detailed work plan describing the proposed methods and equipment and the sequencing of operations. After acceptance of the work plan, no changes shall be made to the work plan without the prior written acceptance of the ENGINEER.

C. The CONTRACTOR shall submit certified laboratory test results performed on samples of the material that he proposes to use for fill a minimum of 14 calendar days before the CONTRACTOR requires approval of the material. The test results shall demonstrate that the proposed material satisfies the requirements of this specification.

D. The CONTRACTOR shall submit a bulk sample of the material that he proposes to use for fill a minimum of 14 calendar days before the CONTRACTOR requires approval of the material. The bulk samples shall weigh at least 100 pounds each. If the material changes or different processing methods are used, then the CONTRACTOR shall submit additional samples.

1.06 QUALITY CONTROL

A. The GEOTECHNICAL ENGINEER will observe and test the work performed under this Section of the Specifications to check conformance with these Specifications. Where any nonconformance is revealed, the CONTRACTOR shall implement corrective measures, including up to complete removal and replacement of placed material, until conformance is achieved as confirmed by further testing by the GEOTECHNICAL ENGINEER. All costs for retests shall be borne by the CONTRACTOR.

B. The CONTRACTOR shall cooperate with the GEOTECHNICAL ENGINEER and allow him unrestricted access to the site as required for the performance of his duties. The CONTRACTOR shall not endanger the GEOTECHNICAL ENGINEER or otherwise attempt to impede him in the course of his duties.

C. The CONTRACTOR shall provide a minimum notice of 48 hours to the GEOTECHNICAL ENGINEER before beginning or restarting earthwork operations that will require the presence of the GEOTECHNICAL ENGINEER on site. Notice provided to answering machines is not valid unless acknowledged.

D. After the completion of grading, the GEOTECHNICAL ENGINEER will prepare a written opinion of grading. Neither the testing performed by the GEOTECHNICAL ENGINEER nor his opinion as to whether or not the fill was constructed in accordance with these Specifications shall relieve the CONTRACTOR of his responsibility to construct the fills in accordance with the Contract Documents. The CONTRACTOR shall perform his own quality control tests to verify compliance with these Specifications. He shall not request the Geotechnical Engineer to perform tests on completed work until he has completed his testing and verified compliance.

PART 2 PRODUCTS

2.01 MATERIALS

A. General

1. During earthwork operations, soil types other than those identified in the geotechnical investigation report for the borrow area may be encountered by the CONTRACTOR. The CONTRACTOR shall contact the GEOTECHNICAL ENGINEER for his evaluation of the suitability of using these soils as fill material prior to placement or disposal.

2. All Engineered Fill materials shall meet the minimum requirements of these specifications.

B. Engineered Fill Material

1. Materials for Engineered Fill shall consist of materials imported from the Fran Ridge Borrow Area that, in the opinion of the GEOTECHNICAL ENGINEER, are suitable for use in constructing Engineered Fill.

2. Engineered Fill material shall not contain any perishable, spongy, organic, hazardous or other undesirable materials, and shall be free of trash and debris.

3. Engineered Fill material shall not contain rocks or hard lumps that will not pass through a 37.5 mm (1-1/2 inches) sieve, and at least 80 percent (by weight) of its particles shall pass through a U.S. Standard 19.0 mm (3/4 inch) sieve. Particles whose greatest dimension is greater than 19 mm (3/4 inch) but less than 37.5 mm (1-1/2 inches) shall be

placed by the CONTRACTOR so that it is completely surrounded by compacted, finer material; no nesting of gravel shall be permitted.

2.02 EQUIPMENT

A. Compaction shall be accomplished by: sheepsfoot rollers; vibratory rollers; multiple-wheel, pneumatic-tired rollers; vibratory plates, or other types of acceptable compacting equipment. Equipment shall be of such design that it is able to compact the fill to at least the minimum specified relative compaction.

B. Three categories of compaction equipment are defined herein and their use shall be as specified in Part 3. No compaction equipment shall be used that does not meet the definition of heavy or lightweight compaction equipment.

1. Heavy compaction equipment shall be limited to compactors with a total static weight plus dynamic force (if any) of 70,600 pounds-force or less for any element in contact with the ground, a total static weight plus dynamic force per lineal inch of width of 840 pounds-force/inch or less for any element in contact with the ground, and a maximum element width of 84 inches. By element is meant a tire, drum, plate or other part of the equipment that comes in physical contact with the ground.

2. Lightweight compaction equipment (roller- or drum-type) shall be limited to roller and drum compactors with a total static weight plus dynamic force of 4,500 pounds-force or less for any element in contact with the ground, a total static weight plus dynamic force per lineal inch of width of 130 pounds-force/inch or less for any element in contact with the ground, and a maximum element width of 35 inches. By element is meant a tire, drum, plate or other part of the equipment that comes in physical contact with the ground.

3. Lightweight compaction equipment (vibratory plates) shall be limited to plate compactors with a total static weight plus dynamic force of 4,875 pounds-force or less. The total static weight plus dynamic force per lineal inch of plate width and per square inch of plate area shall be limited to 9.3 pounds-force/inch^2, respectively. The plate should measure no greater than 21 inches in width and 25 inches in length.

C. Compaction equipment, including hand-operated equipment and tools, shall be of a type suitable for the material being compacted and adequate to obtain the compaction specified. If inadequate compaction is obtained, equipment with more suitable characteristics (such as weight, dimensions, vibration frequency) or different types of equipment shall be used. Jetting, puddling, or other hydroconsolidation techniques shall not be used.

D. The speed, vibratory frequency, ballast load and other operating parameters of the compaction equipment shall be such as to achieve the required compaction.

E. Equipment for applying water shall be of a type and quality adequate for the work, shall be free of leaks or equipment problems, and shall be equipped with distributor bars or other approved device to ensure uniform application.

PART 3 EXECUTION

3.01 DEMOLITION, CLEARING, GRUBBING AND STRIPPING

A. The CONTRACTOR shall demolish structures and utilities as specified in Section 02220, Demolition.

B. Unless otherwise indicated on the Drawings or by the OWNER in writing, the CONTRACTOR shall clear, grub and strip appropriate areas of the site in accordance with Section 02230, Clearing, Grubbing and Stripping.

3.02 SUBGRADE PREPARATION

A. General

1. The CONTRACTOR shall be prepared to handle cobbles, boulders, and other oversized materials encountered during excavation. No payment will be made for over-excavation or for the resultant additional backfill required, its placement, or its compaction. If it is required to remove boulders, the resulting excavation shall be backfilled with Engineered Fill meeting the gradation and compaction requirements of this Specifications Section.

2. The use of methods that result in over-excavation beyond the limits of excavation shown on the Drawings or communicated in writing by the ENGINEER will not be permitted. The CONTRACTOR shall take care to avoid disturbance of adjacent and underlying ground.

3. The material at the bottom of excavation shall be undisturbed by the excavation process. If at any place the natural foundation material is disturbed or loosened during the process of excavation or otherwise, it shall be removed and replaced with Engineered Fill at no cost to the OWNER.

4. All subgrade surfaces shall be firm, unyielding, and free of standing water at the time of placing Engineered Fill on them. The materials shall be excavated to sufficient depths to ensure removal of all loose, soft, weak, unstable, organic, or other materials not suitable as subgrade for Engineered Fill, as determined by the GEOTECHNICAL ENGINEER. The subgrade surface shall be proof-rolled by the CONTRACTOR with a piece of heavy rubber-tired construction equipment to aid the GEOTECHNICAL ENGINEER'S evaluation of the subgrade. If localized areas of removal are required, the areas shall be replaced with Engineered Fill meeting the requirements of this Specification Section.

5. Rocks, stones or other objects shall not project above the subgrade surface on which fill is to be placed. All such protruding materials shall be removed. Any rocks or stones in the prepared subgrade, including those cut back or trimmed, that are, in the GEOTECHNICAL ENGINEER's opinion, loose or otherwise unsuitable in the subgrade shall be removed and disposed of.

6. After the preparation has been completed and the subgrade is approved by the GEOTECHNICAL ENGINEER, the CONTRACTOR shall promptly place Engineered Fill on the approved subgrade to prevent deterioration of the surface. If deterioration of

the surface does occur due to slaking, drying, wetting, mechanical breakdown, loosening from traffic, erosion, or other cause, the CONTRACTOR shall perform additional excavation, compaction, subgrade preparation, and cleanup as required, at no cost to the OWNER.

B. Subgrade for Fill Slopes

1. Remove all loose soil, existing fill, slough, and weathered rock to alluvium or bedrock, as approved by the GEOTECHNICAL ENGINEER.

2. A key shall be established at the base of all fill slopes. The key shall have a minimum width of 20 feet. The location of keys shown on grading plans may need adjustment by the GEOTECHNICAL ENGINEER depending on field conditions.

3. Bottom keys shall be excavated a minimum depth of 2 feet below the lowest adjacent pad surface at the toe and shall slope to the heel to 3 feet below the same reference elevation.

3.03 FILL PLACEMENT AND COMPACTION

A. The CONTRACTOR shall obtain approval from the GEOTECHNICAL ENGINEER of stripping and subgrade preparation before the placement of any fill on any fill subgrade begins. Surfaces to receive fill shall then be scarified to a minimum depth of 6 inches until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment used, and shall be brought to at least the minimum specified compaction water content and compacted to at least the minimum relative compaction required for Engineered Fill.

B. Before each fill layer is placed, the fill material shall be moisture conditioned such that the water content of the minus ¾ inch fraction of the material is equal to or above the minimum compaction water content required by this specification. When the water content of the fill material is too high, the CONTRACTOR shall aerate the fill materials by blading, mixing or other satisfactory methods until the water content is as specified. If the water content of the fill is too low, the CONTRACTOR shall scarify, blade, or mix the soil with water to bring the water content within the required range.

C. Fill to be compacted by heavy compaction equipment shall be placed in horizontal layers not exceeding eight inches in thickness, measured before compaction. Fill to be compacted by light weight compaction equipment, such as hand tampers, shall be placed in horizontal layers not exceeding four inches in thickness, measured before compaction.

D. Each layer of fill shall be spread evenly and shall be thoroughly mixed during the spreading to obtain uniformity of material and moisture in each layer. Avoid such operations as spreading piles of material over long distances that cause the material to segregate.

E. After each layer has been placed, mixed and spread evenly, it shall be thoroughly compacted by the CONTRACTOR.

F. Compaction shall be continuous over the entire area, and the equipment shall make sufficient passes to compact the fill uniformly. All fill placed on-site shall be treated in like manner until finished grades are attained. Compaction shall be by mechanical means: jetting, puddling, or hydroconsolidation techniques shall not be used.

G. Heavy compaction equipment shall not be used within two feet of walls or pipes. Heavy compaction equipment shall not be operated above any structure or pipe without the written approval of the Engineer. Only lightweight compaction equipment shall be used within two feet of walls and pipes or above any structure or pipe. In addition, no other equipment weighing in excess of 50,000 pounds shall be operated or parked within 5 feet of walls or pipes. No equipment shall be operated or parked above any structure or pipe without the written approval of the Engineer.

H. The CONTRACTOR shall not backfill against walls or other structures until the concrete has obtained compressive strength equal to the specified 28-day compressive strength or as approved by the ENGINEER.

I. Bench the fill into existing slopes steeper than 6:1 (horizontal:vertical). As placement progresses, benches shall be cut into firm material, as determined by the GEOTECHNICAL ENGINEER during grading. Benches should typically be one equipment-width wide and about 4-feet high measured at the back of the bench. In steeper areas, adjust the width of the bench as needed to maintain the same bench height of 4 feet. Care should be taken to avoid cutting into the vertical backface of the bench during fill placement, which has the undesirable effect of yielding a smoother overall backcut and a reduced bench height. Care should be taken in benching as the fill approaches the top of slope so that vertical cuts will not be exposed in the finished configuration.

J. When backfilling within three feet of pipelines or five feet of structures, special care shall be exercised not to remove lateral support or undermine them in any way that could cause settlement, displacement or deformation of the pipeline or structure.

K. Fill shall be brought up uniformly on all sides of pipelines, walls, or other structures. To safeguard against movement of pipelines or structures, the CONTRACTOR shall place backfill at these locations in layers not exceeding four inches in thickness and shall thoroughly compact each layer with hand-operated, power-driven tampers.

L. Material conditioning, filling, and compacting operations shall be executed in a systematic manner. The CONTRACTOR shall devise whatever safeguards and standards of operation are required to ensure that compacted fill or backfill will meet or exceed the minimum compaction standards. If the CONTRACTOR's methods are not conducive to achieving the desired results, these methods shall be immediately changed so that the desired results are obtained.

M. All material limits shall be constructed within a tolerance of ± 0.1 foot, except where another tolerance is specified herein or shown on the Drawings.

N. Sliver fills and fill-over-cuts are to be avoided. At a minimum, they will require construction of keys with a width of at least one-half the slope height, but not less than 15 feet, founded in firm bearing materials to provide adequate support.

O. To minimize surficial slumps on Engineered Fill slopes, the following grading procedures should be undertaken:

1. Compacted fill slopes shall be backrolled during placement at intervals not exceeding 4 feet in vertical height, and

2. At completion, the fill slope face shall be rolled for the entire height with a smooth-drum vibratory roller. To obtain the required compaction and appearance of the slope face, the soil moisture should be maintained at or above optimum water content from the time of mass filling to the completion of rolling. To be most effective, this equipment should be anchored and manipulated from a side-boom tractor.

3. As an alternative to the above Item 2, the fill slope may be overbuilt with an additional 2 horizontal feet of compacted fill, and final-trimmed to expose the compacted inner core at the final grade elevation.

4. Take care to construct the slope in a workmanlike manner so that the slope is initially positioned at its proper bearing and slope ratio geometry and does not require later "tack-on," lamination," and "wedge add-on" fills. Any add-on correction to a fill slope will require overfilling the affected area in minimum equipment-width-wide compacted lifts that are benched into the existing fill prism. There will be no additional compensation to the CONTRACTOR for this effort. Excess material shall be removed at the completion of rough grading.

P. The CONTRACTOR should be aware that care must be taken to avoid spillage of loose material down the face of the slopes during grading. These materials shall be removed from slope areas if spillage occurs.

3.04 COMPACTION REQUIREMENTS

A. All scarified surfaces and fill materials shall be compacted such that the minus ¾-inch fraction of the material is compacted to at least 95 percent relative compaction at a water content equal to or greater than the optimum water content.

3.05 PROTECTION OF WORK AND ADJACENT PROPERTIES

A. The CONTRACTOR shall grade all excavated surfaces to provide good drainage away from construction slopes and prevent ponding of water. He shall control surface water and the transport of silt and sediment to avoid damage to adjoining properties or to finished work on the site. The CONTRACTOR shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control measures have been installed.

B. The CONTRACTOR shall dispose of all water resulting from dewatering operations legally and in ways that will not cause damage to public or private property, or constitute a nuisance or menace to the public, in accordance with state and local ordinances and requirements.

C. The CONTRACTOR shall make every effort to minimize the amount of dust raised in excavating, on haul roads and access roads, and all other work areas in the course of construction activities.

D. The CONTRACTOR shall protect benchmarks, monuments, and other reference points against displacement or damage. Repair or replace benchmarks, monuments and other permanent survey features that become displaced or damaged due to the performance of this work.

E. After earthwork is completed and the GEOTECHNICAL ENGINEER has finished his observations of the work, no further excavation, filling or backfilling shall be performed.

3.06 FIELD DENSITY TESTING

A. The in-place density shall be obtained following ASTM D 1556 (sand cone method) or ASTM D 2922 (nuclear method-shallow depth) test method. The in-place water content shall be obtained following ASTM D 2216 (oven drying) or ASTM D 3017 (nuclear method-shallow depth).

B. Field compaction water content - the water content of the minus ¾ -inch fraction of the fill material at the time it is compacted will be determined by ASTM D 2216.

C. If the surface is disturbed, the density tests shall be made in the compacted materials below the disturbed zone. When these tests indicate that the density or water content of any layer of fill or portion thereof does not meet the minimum specified relative compaction or the water content is not within the specified range, the particular layer or portions thereof shall be reworked, prior to placement of additional fill, until the specified relative compaction and water content have been obtained.

D. The tests mentioned herein above will be performed by the GEOTECHNICAL ENGINEER during the progress of the work to determine compliance with the compaction requirements specified herein, and the CONTRACTOR shall cooperate in the making of such tests and allow a reasonable time for conducting tests.

E. Number of Field Density Tests:

1. A minimum of one density test will be made on each 500 cubic yards of fill placed, with a minimum of at least one test per lift.

2. A minimum of one test for every 50 lineal feet in each backfill lift in trenches or one test for every lift of fill around structures.

3. At least one test for every full or partial shift of compaction operations on mass earthwork.

4. A minimum of one test whenever there is a change in the quality of moisture control or effectiveness of compaction as determined by the GEOTECHNICAL ENGINEER.

F. The CONTRACTOR shall remove material as required in any areas where compaction of the material does not fully comply with these Specifications. The unsatisfactorily compacted material shall be promptly removed, after notification, and reworked or replaced with material that conforms to the specified requirements at no additional cost to the OWNER.

END OF SECTION

Attachment III
Frost Penetration

The potential for frost penetration at the building sites was estimated based on Figure 7 from USN (1986). An enlarged copy of this figure showing the approximate location of the YMP site is presented on Figure III-1. Based on this map, the potential frost penetration is ten inches.

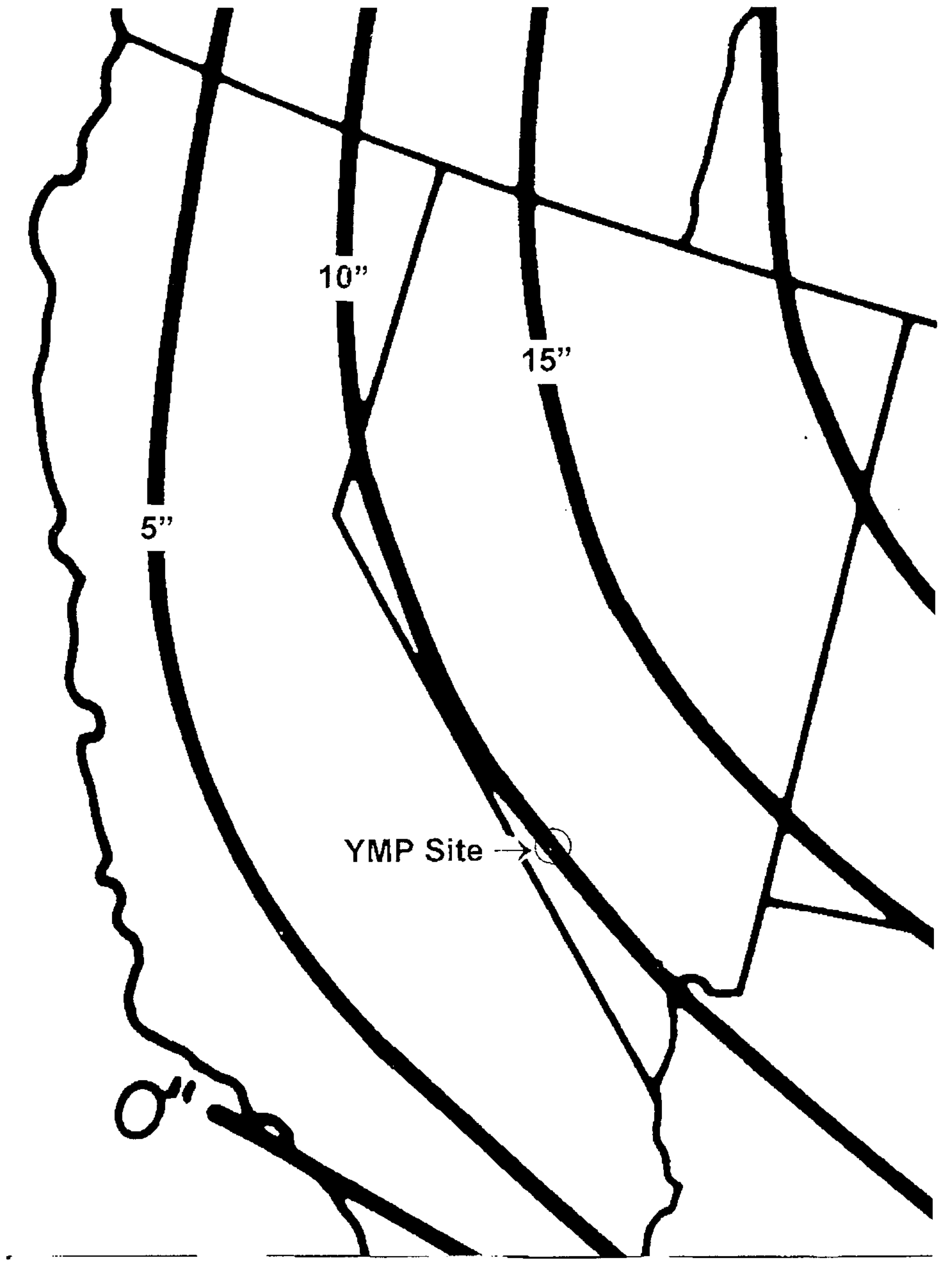

Figure III-1. Extreme Frost Penetration (in inches) at the North Portal Area

Attachment IV
Ultimate Static Bearing Capacity

The ultimate static bearing capacity, q_u, of a foundation embedded beneath a level ground surface and subjected to a symmetrical static vertical load was calculated by equation 3.17 in Das (1995) (note that the inclined load factors are omitted as only vertical loads are considered in this report; also the symbology differs somewhat):

$$q_u = cN_c s_c d_c + qN_q s_q d_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma \quad \text{(Eq. IV-1)}$$

where:
- c = soil's cohesion
- q = effective stress at the level of the bottom of foundation = γD_f (for footings above the water table)
- γ = unit weight of soil
- D_f = depth of embedment of foundation
- B = width of foundation
- N_c, N_q, N_γ = bearing capacity factors
- s_c, s_q, s_γ = shape factors
- d_c, d_q, d_γ = depth factors

For the proper value of γ to use for cases where the groundwater table is located at a depth below ground surface that is less than D_f+B, see Das (1995, Section 3.4). Note that for this report, groundwater is deeper than D_f+B (Section 6.4), so the total unit weight should be used.

Bearing capacity factors are given by equations 3.18, 3.19 and 3.20 in Das (1995):

$$N_q = \tan^2\left(45 + \frac{\phi}{2}\right)e^{\pi \tan\phi} \quad \text{(Eq. IV-2)}$$

$$N_c = (N_q - 1)\cot\phi \quad \text{(Eq. IV-3)}$$

$$N_\gamma = 2(N_q + 1)\tan\phi \quad \text{(Eq. IV-4)}$$

The shape factors are given in Table 3.7 in Das (1995):

$$s_c = 1 + \frac{B}{L}\frac{N_q}{N_c} \quad \text{(Eq. IV-5)}$$

$$s_q = 1 + \frac{B}{L}\tan\phi \quad \text{(Eq. IV-6)}$$

$$s_\gamma = 1 - 0.4\frac{B}{L} \qquad \text{(Eq. IV-7)}$$

where L is the length of the foundation.

The depth factors are given in Table 3.7 in Das (1995):

Condition (a): $D_f/B \leq 1$

$$d_c = 1 + 0.4\frac{D_f}{B} \qquad \text{(Eq. IV-8)}$$

$$d_q = 1 + 2\tan\phi(1-\sin\phi)^2\frac{D_f}{B} \qquad \text{(Eq. IV-9)}$$

$$d_\gamma = 1 \qquad \text{(Eq. IV-10)}$$

Condition (b): $D_f/B > 1$

$$d_c = 1 + 0.4\tan^{-1}\left(\frac{D_f}{B}\right) \qquad \text{(Eq. IV-11)}$$

$$d_q = 1 + 2\tan\phi(1-\sin\phi)^2\tan^{-1}\left(\frac{D_f}{B}\right) \qquad \text{(Eq. IV-12)}$$

$$d_\gamma = 1 \qquad \text{(Eq. IV-13)}$$

The moist unit weight of alluvium was taken as 114 lbf/ft^3 (Section I.2.1) (using the lower value for the unit weight will yield a lower ultimate bearing capacity). The moist unit weight of engineered fill was taken as 128 lbf/ft^3 (Section I.1.1). The shear strength envelopes for engineered fill and alluvium were taken from Sections I.1.2.5 and I.2.2.4. The shear strength, τ_{ff}, of the engineered fill was estimated to be:

$$\tau_{ff} = \sigma'_{ff}\tan\phi' = \sigma'_{ff}\cdot 1.9655\,(\sigma_a/\sigma'_m)^{0.3331} \qquad \text{(Eq. I-12B)}$$

where: ϕ' = the effective friction angle as a function of σ'_m
σ_a = 1 atmosphere pressure using same units as σ'_m
σ'_m = $\frac{1}{2}(\sigma'_1+\sigma'_3)$

The shear strength, τ_{ff}, of the alluvium was estimated to be:

$$\tau_{ff} = \sigma'_{ff}\tan\phi' = 0.8299\,\sigma'_{ff}\,(\sigma_a/\sigma'_m)^{0.0486} \qquad \text{(Eq. I-49B)}$$

The bearing capacity equations are based on a linear Mohr-Coulomb failure criterion. For the engineered fill, the values of ϕ' and c' that should be used to calculate bearing capacity factor N_q

are not straightforward because the shear surface depends on the foundation dimensions, foundation embedment and the subsurface materials, and the values of ϕ' and c′ that are equivalent to equations I-12B and I-49B vary along the shear surface. The selection of appropriate values of c and ϕ for the calculation of N_q was performed following Ueno et al. (1998). According to the Ueno et al. (1998) procedure, the values of equivalent c′ and ϕ' should be calculated over the average stress range from $a\gamma B$ to $b\gamma B$, where a and b equal 2 and 10, respectively, for strip footings and 1 and 15, respectively, for circular footings (Ueno et al. 1998, equation 7 and Table 3). The values for circular footings were considered to be appropriate for the square foundations that are commonly used for to support columns when shallow foundations are possible and deep foundations are not required.

The calculation procedure for the engineered fill and alluvium is as follows (see Ueno et al. (1998) for definitions of symbols and general background):

1. Input values of B, L, D_f, γ, α and tan ϕ_a
2. Note that tensile strength, σ_t, was taken to be zero, so the extended failure criterion denoted in Ueno et al (1998) by the use of "*" in the variable symbology reduces to the regular failure criterion (tensile strength is zero) denoted without use of "*" (see Ueno et al. 1998, equation 6).
3. Calculate B/L and D_f /B
4. Calculate a and b from B/L.
5. Calculate σ'_m corresponding to a and b as $\sigma'_m = a\gamma B + \gamma D_f$ and $b\gamma B + \gamma D_f$, where γD_f is the vertical stress at the depth of the base of the foundation. The Ueno et al. (1998) equations did not include the term γD_f because they only considered foundations with no embedment.
6. Calculate tan ϕ' at $\sigma'_m = a\gamma B$ and $b\gamma B$
7. Calculate ϕ' (in radians) at $\sigma'_m = a\gamma B$ and $b\gamma B$ as the arctan of tan ϕ'
8. Calculate τ_m at $\sigma'_m = a\gamma B$ and $b\gamma B$ as $\tau_m = \sigma'_m \sin \phi'$
9. Calculate σ'_{ff} at $\sigma'_m = a\gamma B$ and $b\gamma B$ as $\sigma'_{ff} = \sigma'_m - \tau_m \sin \phi'$
10. Calculate τ_{ff} at $\sigma'_m = a\gamma B$ and $b\gamma B$ as $\tau_{ff} = \sigma'_{ff} \tan \phi'$
11. Calculate the equivalent constant value of ϕ' over the range $\sigma'_m = a\gamma B$ and $b\gamma B$ as:

$$\phi' = \arcsin(\Delta\tau_m / \Delta\sigma'_m) \qquad \text{(Eq. IV-14)}$$

where $\Delta\tau_m = \tau_m$ at $\sigma'_m = b\gamma B$ minus τ_m at $\sigma'_m = a\gamma B$, and $\Delta\sigma'_m = (b\text{-}a)\gamma B$.

12. Calculate the equivalent constant value of c over the range $\sigma'_m = a\gamma B$ and $b\gamma B$ as:

$$c' = \{\tau_m(a) - \sigma'_m(a) \sin \phi'\} / \cos \phi' \qquad \text{(Eq. IV-15)}$$

where $\sigma'_m(a) = a\gamma B$ and $\tau_m(a)$ is the value of τ_m at $\sigma'_m(a)$.

13. With the calculation of the equivalent constant values of c′ and ϕ′ in steps 11 and 12, the calculation of bearing capacity for soils with a non-linear failure criterion proceeds in the same manner as the calculation for soils with a linear Mohr-Coulomb failure criterion.

14. Calculate the shape bearing capacity factors, factors and depth factors using Das (1995, equations 3.18 to 3.20 and Table 3.7)

15. Calculate the ultimate bearing capacity using Das (1995, equation 3.17)

Using this procedure the ultimate bearing capacity was calculated for various square and continuous strip foundations with a depth of embedment of 2 feet (the minimum recommended embedment). For the engineered fill, the calculated bearing capacity values are high because the shear strength given by equation I-12B is high. A second calculation was performed using the Mohr-Coulomb strength given by equation 1. The calculation results are tabulated in Table IV-4 for the engineered fill using the shear strength given by equation I-12B, in Table IV-5 for the engineered fill using the shear strength given by equation 1, and in Table IV-6 for the alluvium using the shear strength given by equation I-49B. For the engineered fill, the lesser of the results calculated using the two shear strength equations has been retained. The results are plotted in Figure 2 in the main text.

Limitation

For the ultimate bearing capacities developed in this attachment to be valid, the ground surface in the area around the foundation must be horizontal or slope uphill away from the foundation. According to Section 4.2, the final grade over the pad area shall have a nominal slope between 2 and 3 percent (CRWMS M&O 1999b, Section 1.2.1.7), which means that the ground could slope downhill away from the foundation as much as 2 to 3 percent. Downhill slopes of 2 to 3 percent are considered sufficiently horizontal for the values in this attachment to be used. The values in this attachment should not be used where downhill slopes steeper than 2 to 3 percent are involved. If a foundation is located near a slope, the allowable bearing capacity should be reviewed. For the purpose of triggering a review, "near" may be taken to mean within four times the footing width.

Table IV-1. Bearing Capacity of Shallow Foundation on Engineered Fill With Curved Strength Envelope (Equation I-12B)

α = 0.3331 tan ϕ_a = 1.9655 σ_a = 2.11622 kips/ft^2

B	L	D_f	a	b	γ	σ_m' (ksf) at		tan ϕ' at σ_m'=		ϕ' (rad) at σ_m'=		σ_{ff} (ksf) at σ_m=		τ_{ff} (ksf) at σ_m=		τ_m (ksf) at σ_m=		c'	ϕ'	s_q	d_q	N_q	q_q	s_c	d_c	N_c	q_c	s_γ	d_γ	N_γ	q_γ	q_{ult}
feet	feet	feet			kcf	aγB+q	bγB+q	aγB+q	bγB+q	aγB+q	bγB+q	aγB+q	bγB+q	aγB+q	bγB+q	aγB+q	bγB+q	ksf	degr				ksf				ksf				ksf	ksf
2	2	2	1	15	0.128	0.512	4.096	3.1533	1.5774	1.2637	1.0058	0.0468	1.1743	0.1475	1.8523	0.488	3.4594	0.11	56.00	2.48	1.09	1128	779.1	2.48	1.4	760.17	300.5	0.6	1	3348	257.1	1,336.8
3	3	2	1	15	0.128	0.64	6.016	2.9274	1.3878	1.2416	0.9464	0.0669	2.056	0.1958	2.8534	0.6056	4.8809	0.16	52.68	2.31	1.07	540	343.1	2.31	1.27	411.04	192.1	0.6	1	1420	163.5	698.8
4	4	2	1	15	0.128	0.768	7.936	2.7549	1.2655	1.2226	0.9021	0.0894	3.0506	0.2463	3.8605	0.7219	6.2266	0.21	50.17	2.2	1.06	329	197.4	2.2	1.2	273.93	149.4	0.6	1	792	121.7	468.5
5	5	2	1	15	0.128	0.896	9.856	2.617	1.1774	1.2058	0.8667	0.1142	4.1304	0.2988	4.863	0.837	7.5121	0.25	48.16	2.12	1.06	229	131.0	2.12	1.16	203.75	127.4	0.6	1	513	98.4	356.9
6	6	2	1	15	0.128	1.024	11.776	2.5031	1.1096	1.1907	0.8373	0.1409	5.2778	0.3528	5.8563	0.9509	8.7477	0.30	46.48	2.05	1.05	172	95.0	2.06	1.13	162.01	114.4	0.6	1	364	83.8	293.2
8	8	2	1	15	0.128	1.28	15.616	2.3238	1.01	1.1644	0.7904	0.2	7.73	0.4648	7.8076	1.1758	11.097	0.40	43.79	1.96	1.05	112	58.6	1.97	1.1	115.47	100.4	0.6	1	216	66.4	225.3
16	16	2	1	15	0.128	2.304	30.976	1.9106	0.804	1.0886	0.6772	0.4954	18.814	0.9466	15.127	2.0413	19.409	0.81	37.28	1.76	1.03	44.5	20.7	1.78	1.05	57.18	86.7	0.6	1	69.3	42.6	149.9
25	25	2	1	15	0.128	3.456	48.256	1.6692	0.6936	1.0311	0.6064	0.9128	32.581	1.5236	22.599	2.9647	27.503	1.28	33.21	1.65	1.02	26.8	11.6	1.68	1.03	39.35	87.4	0.6	1	36.4	34.9	133.9
35	35	2	1	15	0.128	4.736	67.456	1.5029	0.6204	0.9837	0.5553	1.4533	48.708	2.1842	30.219	3.943	35.562	1.80	30.27	1.58	1.02	19	7.8	1.62	1.02	30.81	91.7	0.6	1	23.3	31.4	130.9
50	50	2	1	15	0.128	6.656	96.256	1.3419	0.5511	0.9303	0.5037	2.3767	73.832	3.1891	40.689	5.337	46.459	2.57	27.32	1.52	1.01	13.7	5.4	1.56	1.02	24.52	99.6	0.6	1	15.2	29.1	134.1
70	70	2	1	15	0.128	9.216	134.66	1.204	0.4928	0.8777	0.4579	3.7622	108.34	4.5297	53.392	7.0896	59.523	3.56	24.71	1.46	1.01	10.3	3.9	1.51	1.01	20.30	110.4	0.6	1	10.4	28.1	142.4
85	85	2	1	15	0.128	11.136	163.46	1.1305	0.462	0.8466	0.4328	4.8887	134.71	5.5264	62.233	8.3409	68.553	4.29	23.28	1.43	1.01	8.92	3.3	1.48	1.01	18.40	118.2	0.6	1	8.54	27.9	149.4
100	100	2	1	15	0.128	13.056	192.26	1.0721	0.4377	0.8202	0.4126	6.0742	161.35	6.5122	70.619	9.5475	77.086	5.00	22.14	1.41	1.01	7.93	2.9	1.47	1.01	17.04	125.8	0.6	1	7.27	27.9	156.5
150	150	2	1	15	0.128	19.456	288.26	0.9387	0.3824	0.7538	0.3653	10.342	251.47	9.7086	96.174	13.316	102.97	7.24	19.48	1.35	1	6.08	2.1	1.42	1.01	14.36	148.8	0.6	1	5.01	28.9	179.8
200	200	2	1	15	0.128	25.856	384.26	0.8539	0.3475	0.7067	0.3345	14.953	342.85	12.768	119.15	16.79	126.14	9.35	17.76	1.32	1	5.14	1.7	1.4	1	12.92	169.4	0.6	1	3.93	30.2	201.4
250	250	2	1	15	0.128	32.256	480.26	0.7932	0.3226	0.6706	0.3121	19.799	434.98	15.705	140.34	20.046	147.46	11.34	16.52	1.3	1	4.56	1.5	1.38	1	11.99	188.3	0.6	1	3.3	31.7	221.5
300	300	2	1	15	0.128	38.656	576.26	0.7468	0.3036	0.6415	0.2948	24.815	527.61	18.533	160.2	23.13	167.43	13.24	15.57	1.28	1	4.16	1.4	1.37	1	11.34	205.8	0.6	1	2.88	33.1	240.3
350	350	2	1	15	0.128	45.056	672.26	0.7097	0.2884	0.6172	0.2808	29.965	620.62	21.265	179.02	26.076	186.31	15.06	14.80	1.26	1	3.87	1.3	1.36	1	10.85	222.3	0.6	1	2.57	34.6	258.1
2	1E+07	2	2	10	0.128	0.768	2.816	2.7549	1.7871	1.2226	1.0606	0.0894	0.6715	0.2463	1.2	0.7219	2.4574	0.13	57.93	1	1.07	1823	501.4	1	1.4	1141.50	214.0	1	1	5823	745.3	1,460.6
3	1E+07	2	2	10	0.128	1.024	4.096	2.5031	1.5774	1.1907	1.0058	0.1409	1.1743	0.3528	1.8523	0.9509	3.4594	0.20	54.74	1	1.06	843	229.5	1	1.27	595.13	149.9	1	1	2387	458.4	837.7
4	1E+07	2	2	10	0.128	1.28	5.376	2.3238	1.4408	1.1644	0.9641	0.2	1.7478	0.4648	2.5182	1.1758	4.4165	0.27	52.30	1	1.06	499	135.1	1	1.2	385.29	123.3	1	1	1295	331.5	589.8
5	1E+07	2	2	10	0.128	1.536	6.656	2.1869	1.3419	1.1419	0.9303	0.2656	2.3767	0.5809	3.1891	1.3969	5.337	0.34	50.31	1	1.05	338	91.1	1	1.16	280.00	109.3	1	1	818	261.8	462.1
6	1E+07	2	2	10	0.128	1.792	7.936	2.0774	1.2655	1.1222	0.9021	0.3371	3.0506	0.7003	3.8605	1.6147	6.2266	0.41	48.65	1	1.05	249	66.8	1	1.13	218.42	101.0	1	1	568	218.3	386.0
8	1E+07	2	2	10	0.128	2.304	10.496	1.9106	1.153	1.0886	0.8563	0.4954	4.506	0.9466	5.1953	2.0413	7.9291	0.55	45.95	1	1.04	157	41.9	1	1.1	151.10	92.1	1	1	327	167.4	301.5
10	1E+07	2	2	10	0.128	2.816	13.056	1.7871	1.0721	1.0606	0.8202	0.6715	6.0742	1.2	6.5122	2.4574	9.5475	0.70	43.82	1	1.04	112	29.8	1	1.08	115.82	88.0	1	1	217	139.0	256.7
12	1E+07	2	2	10	0.128	3.328	15.616	1.6904	1.01	1.0366	0.7904	0.8628	7.73	1.4584	7.8076	2.8643	11.097	0.85	42.07	1	1.03	86.2	22.8	1	1.07	94.42	86.1	1	1	157	120.9	229.8
16	1E+07	2	2	10	0.128	4.352	20.736	1.5459	0.919	0.9966	0.7432	1.2839	11.242	1.9847	10.331	3.6541	14.031	1.16	39.30	1	1.03	58.3	15.3	1	1.05	69.99	85.2	1	1	97	99.4	199.9
20	1E+07	2	2	10	0.128	5.376	25.856	1.4408	0.8539	0.9641	0.7067	1.7478	14.953	2.5182	12.768	4.4165	16.79	1.47	37.17	1	1.02	43.9	11.5	1	1.04	56.54	86.2	1	1	68	87.1	184.8
24	1E+07	2	2	10	0.128	6.4	30.976	1.3595	0.804	0.9366	0.6772	2.247	18.814	3.0548	15.127	5.1555	19.409	1.77	35.45	1	1.02	35.2	9.2	1	1.03	48.06	88.0	1	1	51.6	79.2	176.4
32	1E+07	2	2	10	0.128	8.448	41.216	1.2394	0.731	0.8919	0.6313	3.331	26.861	4.1285	19.636	6.5748	24.324	2.38	32.80	1	1.02	25.5	6.6	1	1.03	37.97	92.5	1	1	34.1	69.9	169.0

Note: Calculations are in units of kips and feet.

Table IV-2. Bearing Capacity of Shallow Foundation on Engineered Fill With Mohr-Coulomb Envelope (Equation 1)

$\alpha = 0$ $\tan \phi_a = 0.7588$ $\sigma_a = 2.11622$ kips/ft^2

B	L	D_f	a	b	γ	σ_m' (ksf) at		$\tan\phi'$ at σ_m'=		ϕ' (rad) at σ_m'=		σ_{ff} (ksf) at σ_m=		τ_{ff} (ksf) at σ_m=		τ_m (ksf) at σ_m=		c'	ϕ'	s_q	d_q	N_q	q_q	s_c	d_c	N_c	q_c	s_γ	d_γ	N_γ	q_γ	q_{ult}
feet	feet	feet			kcf	$a\gamma B+q$	$b\gamma B+q$	$a\gamma B+q$	$b\gamma B+q$	$a\gamma B+q$	$b\gamma B+q$	$a\gamma B+q$	$b\gamma B+q$	$a\gamma B+q$	$b\gamma B+q$	$a\gamma B+q$	$b\gamma B+q$	ksf	degr				ksf				ksf				ksf	ksf
2	2	2	1	15	0.128	0.512	4.096	0.7588	0.7588	0.6491	0.6491	0.3249	2.59935	0.2465	1.972388	0.3095	2.4759	1.79	37.19	1.76	1.24	43.99	24.5	1.78	1.40	56.66	252.2	0.6	1	68.28	5.2	282.0
3	3	2	1	15	0.128	0.64	6.016	0.7588	0.7588	0.6491	0.6491	0.4061	3.8178	0.3082	2.896945	0.3869	3.6365	1.79	37.19	1.76	1.16	43.99	22.9	1.78	1.27	56.66	228.2	0.6	1	68.28	7.9	259.0
4	4	2	1	15	0.128	0.768	7.936	0.7588	0.7588	0.6491	0.6491	0.4874	5.03624	0.3698	3.821502	0.4642	4.7971	1.79	37.19	1.76	1.12	43.99	22.2	1.78	1.20	56.66	216.2	0.6	1	68.28	10.5	248.9
5	5	2	1	15	0.128	0.896	9.856	0.7588	0.7588	0.6491	0.6491	0.5686	6.25469	0.4315	4.746059	0.5416	5.9577	1.79	37.19	1.76	1.09	43.99	21.7	1.78	1.16	56.66	209.0	0.6	1	68.28	13.1	243.8
6	6	2	1	15	0.128	1.024	11.776	0.7588	0.7588	0.6491	0.6491	0.6498	7.47314	0.4931	5.670616	0.619	7.1183	1.79	37.19	1.76	1.08	43.99	21.4	1.78	1.13	56.66	204.2	0.6	1	68.28	15.7	241.3
8	8	2	1	15	0.128	1.28	15.616	0.7588	0.7588	0.6491	0.6491	0.8123	9.91003	0.6164	7.51973	0.7737	9.4395	1.79	37.19	1.76	1.06	43.99	21.0	1.78	1.10	56.66	198.2	0.6	1	68.28	21.0	240.1
16	16	2	1	15	0.128	2.304	30.976	0.7588	0.7588	0.6491	0.6491	1.4621	19.6576	1.1095	14.91619	1.3927	18.724	1.79	37.19	1.76	1.03	43.99	20.4	1.78	1.05	56.66	189.2	0.6	1	68.28	42.0	251.5
25	25	2	1	15	0.128	3.456	48.256	0.7588	0.7588	0.6491	0.6491	2.1932	30.6236	1.6642	23.2372	2.0891	29.17	1.79	37.19	1.76	1.02	43.99	20.2	1.78	1.03	56.66	185.9	0.6	1	68.28	65.5	271.7
35	35	2	1	15	0.128	4.736	67.456	0.7588	0.7588	0.6491	0.6491	3.0055	42.8081	2.2806	32.48277	2.8628	40.776	1.79	37.19	1.76	1.01	43.99	20.1	1.78	1.02	56.66	184.3	0.6	1	68.28	91.8	296.1
50	50	2	1	15	0.128	6.656	96.256	0.7588	0.7588	0.6491	0.6491	4.2239	61.0848	3.2051	46.35112	4.0234	58.185	1.79	37.19	1.76	1.01	43.99	20.0	1.78	1.02	56.66	183.1	0.6	1	68.28	131.1	334.1
70	70	2	1	15	0.128	9.216	134.66	0.7588	0.7588	0.6491	0.6491	5.8485	85.4537	4.4379	64.84226	5.5709	81.396	1.79	37.19	1.76	1.01	43.99	19.9	1.78	1.01	56.66	182.2	0.6	1	68.28	183.5	385.7
85	85	2	1	15	0.128	11.136	163.46	0.7588	0.7588	0.6491	0.6491	7.067	103.73	5.3624	78.71062	6.7315	98.805	1.79	37.19	1.76	1.01	43.99	19.9	1.78	1.01	56.66	181.9	0.6	1	68.28	222.9	424.6
100	100	2	1	15	0.128	13.056	192.26	0.7588	0.7588	0.6491	0.6491	8.2854	122.007	6.287	92.57897	7.8921	116.21	1.79	37.19	1.76	1.00	43.99	19.9	1.78	1.01	56.66	181.6	0.6	1	68.28	262.2	463.7
150	150	2	1	15	0.128	19.456	288.26	0.7588	0.7588	0.6491	0.6491	12.347	182.929	9.3688	138.8068	11.761	174.24	1.79	37.19	1.76	1.00	43.99	19.9	1.78	1.01	56.66	181.1	0.6	1	68.28	393.3	594.3
200	200	2	1	15	0.128	25.856	384.26	0.7588	0.7588	0.6491	0.6491	16.408	243.852	12.451	185.0347	15.629	232.27	1.79	37.19	1.76	1.00	43.99	19.9	1.78	1.00	56.66	180.9	0.6	1	68.28	524.4	725.1
250	250	2	1	15	0.128	32.256	480.26	0.7588	0.7588	0.6491	0.6491	20.47	304.774	15.533	231.2625	19.498	290.3	1.79	37.19	1.76	1.00	43.99	19.8	1.78	1.00	56.66	180.7	0.6	1	68.28	655.5	856.1
300	300	2	1	15	0.128	38.656	576.26	0.7588	0.7588	0.6491	0.6491	24.531	365.696	18.614	277.4904	23.367	348.33	1.79	37.19	1.76	1.00	43.99	19.8	1.78	1.00	56.66	180.7	0.6	1	68.28	786.6	987.1
350	350	2	1	15	0.128	45.056	672.26	0.7588	0.7588	0.6491	0.6491	28.593	426.619	21.696	323.7182	27.235	406.36	1.79	37.19	1.76	1.00	43.99	19.8	1.78	1.00	56.66	180.6	0.6	1	68.28	917.7	1,118.1
2	1E+07	2	2	10	0.128	0.768	2.816	0.7588	0.7588	0.6491	0.6491	0.4874	1.78705	0.3698	1.356017	0.4642	1.7022	1.79	37.19	1	1.24	43.99	13.9	1	1.40	56.66	142.0	1	1	68.28	8.7	164.7
3	1E+07	2	2	10	0.128	1.024	4.096	0.7588	0.7588	0.6491	0.6491	0.6498	2.59935	0.4931	1.972388	0.619	2.4759	1.79	37.19	1	1.16	43.99	13.0	1	1.27	56.66	128.5	1	1	68.28	13.1	154.6
4	1E+07	2	2	10	0.128	1.28	5.376	0.7588	0.7588	0.6491	0.6491	0.8123	3.41165	0.6164	2.58876	0.7737	3.2497	1.79	37.19	1	1.12	43.99	12.6	1	1.20	56.66	121.7	1	1	68.28	17.5	151.8
5	1E+07	2	2	10	0.128	1.536	6.656	0.7588	0.7588	0.6491	0.6491	0.9748	4.22395	0.7396	3.205132	0.9285	4.0234	1.79	37.19	1	1.09	43.99	12.3	1	1.16	56.66	117.7	1	1	68.28	21.8	151.8
6	1E+07	2	2	10	0.128	1.792	7.936	0.7588	0.7588	0.6491	0.6491	1.1372	5.03625	0.8629	3.821503	1.0832	4.7971	1.79	37.19	1	1.08	43.99	12.2	1	1.13	56.66	114.9	1	1	68.28	26.2	153.3
8	1E+07	2	2	10	0.128	2.304	10.496	0.7588	0.7588	0.6491	0.6491	1.4621	6.66084	1.1095	5.054247	1.3927	6.3446	1.79	37.19	1	1.06	43.99	11.9	1	1.10	56.66	111.6	1	1	68.28	35.0	158.5
10	1E+07	2	2	10	0.128	2.816	13.056	0.7588	0.7588	0.6491	0.6491	1.7871	8.28544	1.356	6.28699	1.7022	7.8921	1.79	37.19	1	1.05	43.99	11.8	1	1.08	56.66	109.5	1	1	68.28	43.7	165.0
12	1E+07	2	2	10	0.128	3.328	15.616	0.7588	0.7588	0.6491	0.6491	2.112	9.91003	1.6026	7.519734	2.0117	9.4395	1.79	37.19	1	1.04	43.99	11.7	1	1.07	56.66	108.2	1	1	68.28	52.4	172.3
16	1E+07	2	2	10	0.128	4.352	20.736	0.7588	0.7588	0.6491	0.6491	2.7618	13.1592	2.0957	9.985223	2.6307	12.534	1.79	37.19	1	1.03	43.99	11.6	1	1.05	56.66	106.5	1	1	68.28	69.9	188.0
20	1E+07	2	2	10	0.128	5.376	25.856	0.7588	0.7588	0.6491	0.6491	3.4116	16.4084	2.5888	12.45071	3.2497	15.629	1.79	37.19	1	1.02	43.99	11.5	1	1.04	56.66	105.5	1	1	68.28	87.4	204.4
24	1E+07	2	2	10	0.128	6.4	30.976	0.7588	0.7588	0.6491	0.6491	4.0615	19.6576	3.0819	14.9162	3.8686	18.724	1.79	37.19	1	1.02	43.99	11.5	1	1.03	56.66	104.8	1	1	68.28	104.9	221.2
32	1E+07	2	2	10	0.128	8.448	41.216	0.7588	0.7588	0.6491	0.6491	5.3612	26.156	4.068	19.84719	5.1066	24.914	1.79	37.19	1	1.01	43.99	11.4	1	1.03	56.66	104.0	1	1	68.28	139.8	255.2

Note: Calculations are in units of kips and feet.

Table IV-3. Bearing Capacity of Shallow Foundation on Alluvium With Curved Strength Envelope (Equation I-49B)

α = 0.0486 tan ϕ_a = 0.8299 σ_a = 2.11622 kips/ft^2

B	L	D_f	a	b	γ	σ_m' (ksf) at		tan ϕ' at σ_m'=		ϕ' (rad) at σ_m'=		σ_{ff} (ksf) at σ_m=		τ_{ff} (ksf) at σ_m=		τ_m (ksf) at σ_m=		c'	ϕ'	s_q	d_q	N_q	q_q	s_c	d_c	N_c	q_c	s_γ	d_γ	N_γ	q_γ	q_{ult}
feet	feet	feet			kcf	aγB+q	bγB+q	aγB+q	bγB+q	aγB+q	bγB+q	aγB+q	bγB+q	aγB+q	bγB+q	aγB+q	bγB+q	ksf	degr				ksf				ksf				ksf	ksf
2	2	2	1	15	0.114	0.456	3.648	0.894	0.808	0.73	0.68	0.2534	2.20659	0.2266	1.78342	0.304	2.29309	0.03	38.547	1.797	1.226	52.642	26.4	1.812	1.400	64.812	4.2	0.6	1	85.5	5.8	36.5
3	3	2	1	15	0.114	0.57	5.358	0.885	0.793	0.724	0.671	0.3198	3.28859	0.2829	2.60872	0.3776	3.32985	0.03	38.067	1.783	1.154	49.371	23.2	1.799	1.267	61.763	4.7	0.6	1	78.9	8.1	35.9
4	4	2	1	15	0.114	0.684	7.068	0.877	0.783	0.72	0.664	0.3867	4.38311	0.3391	3.43048	0.4509	4.35624	0.04	37.715	1.773	1.117	47.125	21.3	1.790	1.200	59.646	5.3	0.6	1	74.4	10.2	36.7
5	5	2	1	15	0.114	0.798	8.778	0.87	0.774	0.716	0.659	0.4541	5.48698	0.3952	4.24944	0.5238	5.37481	0.05	37.437	1.766	1.094	45.436	20.0	1.783	1.160	58.041	5.9	0.6	1	71.1	12.2	38.0
6	6	2	1	15	0.114	0.912	10.49	0.865	0.768	0.713	0.655	0.5219	6.5983	0.4512	5.0661	0.5965	6.38711	0.06	37.208	1.759	1.079	44.094	19.1	1.777	1.133	56.758	6.5	0.6	1	68.5	14.1	39.6
8	8	2	1	15	0.114	1.14	13.91	0.855	0.757	0.708	0.648	0.6584	8.83862	0.5631	6.69376	0.7409	8.39672	0.07	36.842	1.749	1.060	42.050	17.8	1.767	1.100	54.789	7.6	0.6	1	64.5	17.6	43.1
16	16	2	1	15	0.114	2.052	27.59	0.831	0.733	0.693	0.632	1.2136	17.9538	1.0087	13.1518	1.3116	16.303	0.13	35.949	1.725	1.031	37.510	15.2	1.745	1.050	50.346	12.2	0.6	1	55.9	30.6	58.0
25	25	2	1	15	0.114	3.078	42.98	0.815	0.717	0.684	0.622	1.8496	28.3875	1.5073	20.3516	1.9444	25.0414	0.20	35.371	1.710	1.020	34.875	13.9	1.731	1.032	47.717	17.0	0.6	1	50.9	43.5	74.4
35	35	2	1	15	0.114	4.218	60.08	0.803	0.705	0.676	0.614	2.5656	40.1185	2.059	28.2974	2.6401	34.6284	0.27	34.935	1.699	1.015	33.030	13.0	1.720	1.023	45.853	22.1	0.6	1	47.5	56.9	92.0
50	50	2	1	15	0.114	5.928	85.73	0.789	0.693	0.668	0.606	3.6522	57.9003	2.883	40.1402	3.673	48.8427	0.39	34.474	1.687	1.010	31.200	12.1	1.709	1.016	43.984	29.4	0.6	1	44.2	75.6	117.2
70	70	2	1	15	0.114	8.208	119.9	0.777	0.682	0.661	0.599	5.1181	81.8518	3.9767	55.8266	5.036	67.5751	0.53	34.041	1.676	1.007	29.587	11.4	1.699	1.011	42.317	38.7	0.6	1	41.3	98.9	149.1
85	85	2	1	15	0.114	9.918	145.6	0.77	0.676	0.656	0.594	6.2271	99.9504	4.7941	67.5315	6.0503	81.5008	0.64	33.792	1.669	1.006	28.703	11.0	1.693	1.009	41.395	45.5	0.6	1	39.8	115.6	172.0
100	100	2	1	15	0.114	11.628	171.2	0.764	0.67	0.652	0.591	7.3427	118.141	5.6094	79.1946	7.059	95.3418	0.75	33.583	1.664	1.005	27.988	10.7	1.689	1.008	40.646	52.1	0.6	1	38.5	131.7	194.4
150	150	2	1	15	0.114	17.328	256.7	0.749	0.657	0.643	0.581	11.098	179.278	8.3152	117.835	10.39	141.009	1.12	33.066	1.651	1.004	26.300	9.9	1.677	1.005	38.860	73.2	0.6	1	35.5	182.4	265.5
200	200	2	1	15	0.114	23.028	342.2	0.739	0.648	0.636	0.575	14.894	240.987	11.007	156.198	13.686	186.138	1.48	32.702	1.642	1.003	25.180	9.5	1.669	1.004	37.662	93.3	0.6	1	33.6	229.9	332.7
250	250	2	1	15	0.114	28.728	427.7	0.731	0.641	0.631	0.57	18.721	303.117	13.687	194.35	16.955	230.867	1.84	32.420	1.635	1.002	24.353	9.1	1.662	1.003	36.770	112.8	0.6	1	32.2	275.3	397.3
300	300	2	1	15	0.114	34.428	513.2	0.725	0.636	0.627	0.566	22.573	365.578	16.359	232.331	20.203	275.278	2.20	32.191	1.630	1.002	23.703	8.8	1.657	1.003	36.064	131.8	0.6	1	31.1	319.1	459.7
350	350	2	1	15	0.114	40.128	598.7	0.719	0.631	0.624	0.563	26.445	428.312	19.022	270.169	23.432	319.426	2.56	31.998	1.625	1.002	23.170	8.6	1.653	1.002	35.483	150.4	0.6	1	30.2	361.5	520.5
2	1E+07	2	2	10	0.114	0.684	2.508	0.877	0.823	0.72	0.689	0.3867	1.49512	0.3391	1.2306	0.4509	1.59383	0.03	38.799	1	1.224	54.459	15.2	1	1.400	66.491	2.7	1	1	89.2	10.2	28.0
3	1E+07	2	2	10	0.114	0.912	3.648	0.865	0.808	0.713	0.68	0.5219	2.20659	0.4512	1.78342	0.5965	2.29309	0.04	38.324	1	1.152	51.091	13.4	1	1.267	63.371	3.2	1	1	82.3	14.1	30.7
4	1E+07	2	2	10	0.114	1.14	4.788	0.855	0.798	0.708	0.673	0.6584	2.92632	0.5631	2.33407	0.7409	2.98559	0.05	37.974	1	1.116	48.767	12.4	1	1.200	61.195	3.7	1	1	77.7	17.7	33.8
5	1E+07	2	2	10	0.114	1.368	5.928	0.848	0.789	0.703	0.668	0.796	3.65223	0.6748	2.88299	0.8846	3.67298	0.06	37.697	1	1.093	47.015	11.7	1	1.160	59.542	4.2	1	1	74.2	21.2	37.1
6	1E+07	2	2	10	0.114	1.596	7.068	0.841	0.783	0.699	0.664	0.9345	4.38311	0.7862	3.43048	1.0275	4.35624	0.07	37.468	1	1.078	45.621	11.2	1	1.133	58.218	4.7	1	1	71.5	24.4	40.4
8	1E+07	2	2	10	0.114	2.052	9.348	0.831	0.772	0.693	0.657	1.2136	5.85666	1.0087	4.5219	1.3116	5.71289	0.09	37.103	1	1.060	43.495	10.5	1	1.100	56.184	5.7	1	1	67.3	30.7	46.9
10	1E+07	2	2	10	0.114	2.508	11.63	0.823	0.764	0.689	0.652	1.4951	7.34268	1.2306	5.60944	1.5938	7.05902	0.11	36.816	1	1.048	41.912	10.0	1	1.080	54.656	6.7	1	1	64.2	36.6	53.3
12	1E+07	2	2	10	0.114	2.964	13.91	0.816	0.757	0.685	0.648	1.7785	8.83862	1.452	6.69376	1.8745	8.39673	0.13	36.581	1	1.040	40.662	9.6	1	1.067	53.441	7.7	1	1	61.8	42.3	59.6
16	1E+07	2	2	10	0.114	3.876	18.47	0.806	0.747	0.678	0.642	2.35	11.854	1.8937	8.85452	2.4321	11.052	0.18	36.209	1	1.031	38.770	9.1	1	1.050	51.589	9.6	1	1	58.2	53.1	71.8
20	1E+07	2	2	10	0.114	4.788	23.03	0.798	0.739	0.673	0.636	2.9263	14.8941	2.3341	11.0067	2.9856	13.686	0.22	35.919	1	1.025	37.369	8.7	1	1.040	50.206	11.4	1	1	55.6	63.4	83.5
24	1E+07	2	2	10	0.114	5.7	27.59	0.791	0.733	0.669	0.632	3.5066	17.9538	2.7733	13.1518	3.5358	16.303	0.26	35.683	1	1.021	36.267	8.4	1	1.033	49.110	13.2	1	1	53.5	73.2	94.8
32	1E+07	2	2	10	0.114	7.524	36.71	0.78	0.722	0.663	0.626	4.6766	24.1196	3.6491	17.4249	4.6285	21.4964	0.34	35.309	1	1.016	34.604	8.0	1	1.025	47.444	16.7	1	1	50.4	92.0	116.7

Note: Calculations are in units of kips and feet.

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Attachment V
Lateral Pressures on Permanent Below-Grade Walls

Objective: Provide lateral pressures acting on permanent below-grade walls. At this time, the only potential below-grade wall is the pool section of the wet-process building, which will extend approximately 55 feet below the main floor level, and thus about 55 feet below final grade.

General: The below-grade wall is not expected to be free to rotate about the base of the wall or to translate laterally during and after compaction or the wall backfill, so the wall will be considered restrained. If the below-grade wall were constructed directly against the natural alluvial deposits, the lateral earth pressures would correspond to the at-rest pressures in the alluvium. However, it is not practical to construct the below-grade wall directly against the natural alluvial deposits; backfill will need to be placed against the wall. Consequently, the stresses induced by compaction equipment over and above the at-rest pressures needs to be calculated. Because the choice of compaction equipment should be left, within limits, to the contractor, an enveloping curve is developed for use in estimating compactor-induced stresses on the below-grade wall.

Because the water table is deep (Section 6), no hydrostatic pressures will act on the wall.

Method: Stresses due to compactors computed based on the simplified hand-calculation procedure proposed in "Compaction-Induced Earth Pressures Under K_0-Conditions," by Duncan and Seed (1986). At-rest (K_0) pressures are also discussed in the same reference.

Steps in Computing Stresses due to Compactors:

1) Compute the profile of peak lateral compaction pressure, $\Delta\sigma'_{h,vc,p}$ versus depth.

For drum-type compactors, use the following equation (see Addendum A for derivation):

$$\Delta\sigma'_{h,vc,p} = \frac{(\mathrm{RF/RW})d}{3}\left[\left(\frac{\mathrm{CHD}}{\mathrm{SLD}^3{}_{\text{near side}}}\right)-\left(\frac{\mathrm{CHD+RW}}{\mathrm{SLD}^3{}_{\text{far side}}}\right)+\left(\frac{\mathrm{CHD+RW}}{\mathrm{d}^2\mathrm{SLD}_{\text{far side}}}\right)-\left(\frac{\mathrm{CHD}}{\mathrm{d}^2\mathrm{SLD}_{\text{near side}}}\right)\right] \tag{Eq. V-1}$$

where: RF = the static weight plus dynamic force, if any, acting on the drum (lbf)
RW = roller width (ft)
d = depth (ft)
CHD = clear horizontal distance (ft)
$\mathrm{SLD}_{\text{near side}} = \sqrt{\mathrm{CHD}^2 + \mathrm{d}^2}$
$\mathrm{SLD}_{\text{far side}} = \sqrt{(\mathrm{CHD+RW})^2 + \mathrm{d}^2}$

Note: For rollers with two or more vibratory drums ("double-drum vibratory rollers"), or for static rollers with two or more drums, the force on each drum should be determined and the engineer should evaluate whether the drums are sufficiently far apart that only one of the drums needs to be considered (this needs to be assessed by the engineer on a case by case basis, as done in Duncan et al. (1991, pages 1835, 1839, 1841)).

For plate compactors, which apply the force over a rectangular area, use the following equation (see Addendum B for derivation):

$$\Delta\sigma_h = \frac{2p}{\pi}\left[\arctan\left(\frac{l'b}{zR_3'}\right) - \arctan\left(\frac{Xb}{zR_3}\right) - \frac{l'bz}{(R_1')^2 R_3'} + \frac{Xbz}{R_1^2 R_3}\right] \qquad \text{(Eq. V-2)}$$

where: p = the static weight plus dynamic force, if any, acting on the plate divided by the area of the plate (lbf/ft^2)

b = <u>one-half</u> the length of plate (measured parallel to the wall)

l = width of the plate (measured perpendicular to wall)

z = depth of point below the ground surface

X = horizontal distance between wall and nearest edge of the rectangular loaded area

$R_1 = \sqrt{l^2 + z^2}$

$R_3 = \sqrt{l^2 + b^2 + z^2}$

$l' = l + X$

R_3' and R_1' are calculated using l' rather than l.

2) Compute α (based on Figure 2 in Duncan and Seed (1986)):

$$\text{If } \phi < 30°,\ \alpha \approx 0.4007 - \left(1.2566 - 2.9072\sin^2\phi\right) \qquad \text{(Eq. V-3)}$$

$$\text{If } \phi \geq 30°,\ \alpha \approx 15.1698 * 0.0451^{\left(\frac{1}{\sin\phi}\right)}(\sin\phi)^{-4.0198} \qquad \text{(Eq. V-4)}$$

3) Compute the scaling factor, F, using equation 11 in Duncan and Seed (1986):

$$F = \frac{5^\alpha}{4} - 0.25 \qquad \text{(Eq. V-5)}$$

where α is defined in Step 2.

4) Compute the Scaled Lateral Pressure profile by multiplying the Lateral Pressure by F.

5) Compute K_1 for cohesionless soils = $K_{1,\phi'}$ (Duncan and Seed 1986, Table 1):

$$K_{1,\phi'} = \tan^2\left(45 + \frac{\phi'}{2}\right) \qquad \text{(Eq. V-6)}$$

6) Add the Scaled Peak Lateral Pressure to the at-rest lateral pressure profile, limiting the values to a maximum of $K_1\sigma'_v$ (Duncan and Seed 1986, equation 12 and page 14):

$$\sigma'_{h,r} = K_0\sigma'_v + F \cdot \Delta\sigma'_{h,vc,p} \leq K_1\sigma'_v \qquad \text{(Eq. V-7)}$$

where: $\sigma'_{h,r}$ = the residual lateral stress as a function of depth
σ'_v = the effective vertical stress,
K_0 = coefficient of earth pressure at rest $\approx$ 1 - sinϕ.

7) Define σ'_A as the effective lateral stress at the depth where $K_0\sigma'_v + F \cdot \Delta\sigma'_{h,vc,p_v}$ is equal to $K_1\sigma'_v$:

$$\sigma'_A = K_0\sigma'_v + F \cdot \Delta\sigma'_{h,vc,p} = K_1\sigma'_v \qquad \text{(Eq. V-8)}$$

8) Below the depth at which $K_0\sigma'_v + F \cdot \Delta\sigma'_{h,vc,p} = K_1\sigma'_v$, the effective lateral stress increases linearly (Duncan and Seed 1986, equation 13a):

$$\sigma'_h = \sigma'_A + \Delta\sigma'_h = \sigma'_A + K_2\,\Delta\sigma'_v \qquad \text{(Eq. V-9)}$$

where: $K_2 = K_0(1\text{-}F)$ (Duncan and Seed 1986, equation 13b)

9) However, the previous equation is subject to the restriction that the value of σ'_h is never less than $K_0\sigma'_v$ (Duncan and Seed 1986, page 14):

$$\Delta\sigma'_h \geq K_0\sigma'_v \qquad \text{(Eq. V-10)}$$

<u>Determining the enveloping curve</u>

10) Pick a force and compactor width for a hand roller, and compute the stresses on the wall when the compactor is right against the wall (CHD=0). The calculation in this report was based on a compactor with two 35-inch drums and a total force of 9,000 lbf divided equally between the drums, based on a review of the characteristics of several of the available hand-operated equipment as summarized in Duncan et al. (1991, Table 6). The two drums were taken to be sufficiently far apart that only one need be considered in the calculation (this needs to be assessed by the engineer on a case by case basis, as done in Duncan et al. 1991, pages 1835, 1839, 1841). Thus, the total force/width for the drum was approximately 130 lbf/inch.

11) A friction angle of 54 degrees was selected for calculating the compactor-induced stresses, based on the judgement that the level of stress associated with the critical compaction equipment would be on the order of one atmosphere (approximately 2,116 lbf/ft^2). For this value of σ'_{ff}, equation I-8 gives a friction angle of 54 degrees.

12) For the at-rest line, a friction angle of 39 degrees was used, corresponding to the friction angle of the alluvium at moderate to high stress levels. The alluvium properties were used because the lower shear strength (relative to the engineered fill shear strength) yields a conservative result. The shear strength of the alluvium was evaluated in Attachment I and can be expressed by equation I-44. The friction angle varies with confining pressure.

For simplicity, and considering the approximate nature of the at-rest earth pressure coefficient, the friction angle was evaluated for the deeper end of potential wall depths (about 50 feet, not including the base slab. At this depth the overburden pressure, σ'_v, is approximately 5.85 kips/ft^2, using a unit weight of 117 lbf/ft^3 for the alluvium (see Section I.2.1 of Attachment I). Given $\sigma'_v = \sigma'_{1f} = 5.85$ kips/ft^2, the friction angle ϕ' can be found by trial and error substitution into equation I-44 and a simple geometric relation for the Mohr circle ($\sigma'_{ff} = \sigma'_{1f} \cdot [1\text{-}\sin\phi']$). Following this approach, the friction angle is approximately 39 degrees. Thus, $K_0 \approx 1\text{-}\sin 39° = 0.37$ (Duncan and Seed 1986, Table 1).

13) Pick a force and compactor width for large equipment, and compute the stresses on the wall at a distance such that the forces imposed on the wall are less than those imposed by the hand equipment. The calculation in this report was based on a single-drum compactor with a drum width of 84 inches and a total force of 70,600 lbf on the single drum. These values were selected based on a review of the characteristics of several available models of self-propelled single-drum vibratory rollers, as summarized by Duncan et al. (1991, Table 5). Thus, the total force/width for the drum was approximately 840 lbf/inch. The friction angles used were the same as in Step 1 above. Based on the calculations presented above, at a clear horizontal distance of 3 feet from the wall, the stresses imposed by large equipment will be less than those imposed by hand equipment next to the wall.

14) Pick a force and compactor width for a hand tamper, and compute the stresses on the wall when the tamper is right against the wall (CHD=0). For this purpose, a total force of 4875 lbf, a width of 21 inches, and a length of 25 inches were used, based on a review of the characteristics of several available models of hand tampers as summarized by Duncan et al. (1991, Table 9). Thus, the total force/area was approximately 9.3 psi. The friction angles used were the same as in Step 1 above. Based on the calculations presented above, the stresses imposed by hand tampers will be less than those imposed by hand rollers next to the wall. Figure V-1 shows the pressures resulting from both the hand and the heavy equipment calculations.

15) Envelope the curves. Because the pressures resulting from the hand roller at a distance of 0 feet are always higher than the pressures resulting from the heavy roller at a distance of 2 feet, the hand tamper curve is applicable. The recommended points for this curve are in Table V-1.

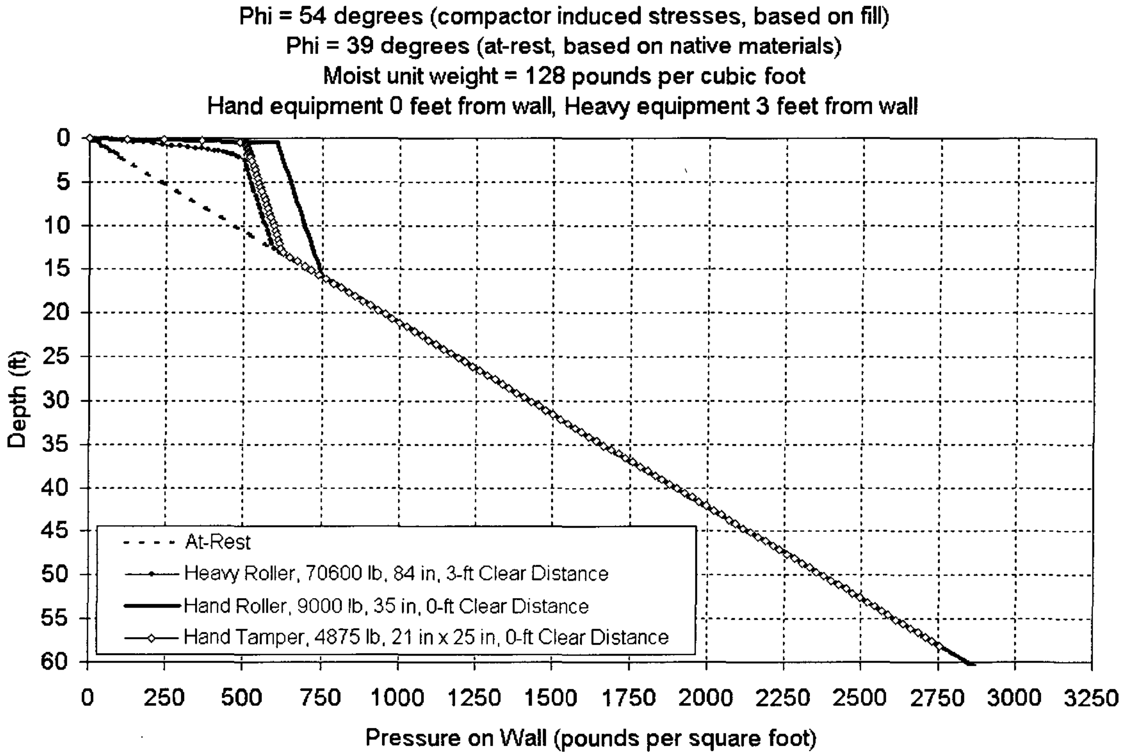

Figure V-1. Lateral Earth Pressure on Restrained Wall

Table V-1. Lateral Earth Pressure Distribution Incorporating the Effects of Compactor-Induced Stresses

Depth (feet)	Pressure on Wall (lbf/ft^2)
0	0
0.5	610
16	760
60	2,850

Active Pressures: At present, there is no information indicating that there will be any below-grade walls or retaining walls that are expected to be free to rotate about the base of the wall or to translate laterally during or after compaction of the wall backfill, such that active pressures would develop on the wall. However, this was considered in case it is needed.

The active pressure coefficient, K_a, can be calculated using (DON 1986, page 7.2-62):

$$K_a = \tan^2\left(45 - \frac{\phi'}{2}\right) \qquad \text{(Eq. V-11)}$$

Because no walls with active conditions have been identified, the active pressure is conservatively evaluated using the properties of the alluvium. The shear strength of the alluvium was evaluated in Attachment I and can be expressed by equation I-44. The friction angle varies with confining pressure. For simplicity, and considering the approximate nature of this active earth pressure coefficient, the friction angle was evaluated for the deeper end of potential wall depths (50 feet, not including the base slab) by assuming that σ'_{ff} in equation I-44 equals one atmosphere, which yields a friction angle of 39 degrees. This value of σ'_{ff} corresponds to a value of $\sigma'_v = \sigma'_{1f}$ of 5.7 kips/ft^2 using simple geometric relations for the Mohr circle ($\sigma'_{ff} = \sigma'_{1f} \cdot [1-\sin \phi']$). This overburden pressure corresponds to a depth of about 49 feet using a unit weight of 117 lbf/ft^3 for the alluvium (see Section I.2.1 of Attachment I).

For a friction angle of 39 degrees, K_a equals approximately 0.23. With this value of K_a and a unit weight of 117 lbf/ft^3, the active earth pressure is zero at the ground surface and increases at the rate of 27 lbf/ft^2 per foot below ground surface. In other words, the active pressures on below-grade walls and retaining walls that are expected to be free to rotate about the base of the wall or to translate laterally during and after construction may be taken as the pressure exerted by an equivalent fluid with a unit weight of 27 lbf/ft^3.

Addendum A to Attachment V

The horizontal earth pressure acting in the ground due to a point load applied at the ground surface may be computed by the equation (Poulos and Davis 1991, equation 2.2b):

$$\Delta\sigma_h = \frac{P}{2\pi}\left[\frac{3r^2z}{R^5} - \frac{1-2\nu}{R^2+zR}\right] \qquad \text{(Eq. V-12)}$$

where: $\Delta\sigma_h$ = horizontal pressure at any point in the ground
P = applied vertical load acting at ground surface
r = horizontal distance from load to point where $\Delta\sigma_h$ is calculated
z = depth of point below ground surface
$R = \sqrt{r^2+z^2}$
ν = Poisson's ratio.

In the case of a point load acting on the backfill adjacent to a wall, the maximum horizontal earth pressures acting on the wall occur along the line that is in the plane that is orthogonal to the wall and passes through the applied load. If the wall is rigid, then the horizontal earth pressures according to equation V-12 should be doubled (Duncan and Seed 1986, page 11). Equation V-12 can be written for the earth pressure acting on a relatively rigid concrete retaining wall due to a point load applied at the surface of the backfill (where the applied load and the point on the wall are in a plane orthogonal to the wall) as:

$$\Delta\sigma_h = \frac{P}{\pi}\left[\frac{3x^2z}{R^5} - \frac{1-2\nu}{R^2+zR}\right] \qquad \text{(Eq. V-13)}$$

where: x = horizontal distance from wall to load measured perpendicular to wall.

Theoretically, $0 \leq \nu \leq 0.5$ for homogeneous, isotropic elastic materials. Over this range, the smaller ν is, the larger the second term of equation V-12 will be, and the smaller $\Delta\sigma_h$ will be. In some cases $\Delta\sigma_h$ will become negative, unless $\nu = 0.5$. Making the conservative simplification that $\nu = 0.5$ and taking $3/\pi = 1$, equation V-13 can be approximated as:

$$\Delta\sigma_h = P\frac{x^2 z}{R^5} \qquad \text{(Eq. V-14)}$$

Then, the pressure acting on a relatively rigid concrete retaining wall due to a line load of finite length acting perpendicular to the wall can be derived by integrating equation V-14 over the length of the line:

$$\Delta\sigma_h = \frac{Pz}{(x_2 - x_1)}\int_{x_1}^{x_2}\frac{x^2 dx}{(x^2+z^2)^{2.5}} \qquad \text{(Eq. V-15)}$$

Standard tables of integrals give:

$$\int\frac{x^2 dx}{X^n\sqrt{X}} = \frac{(2b^2 - 4ac)x + 2ab}{(2n-1)cqX^{n-1}\sqrt{X}} + \frac{4ac + (2n-3)b^2}{(2n-1)cq}\int\frac{dx}{X^{n-1}\sqrt{X}} \qquad \text{(Eq. V-16)}$$

where: $q = 4ac\text{-}b^2$
$k=4c/q$

In equation V-16, let $a = z^2$, $b = 0$, $c = 1$: then, $X = R^2 = x^2+z^2$. Then, equation V-16 (with $n = 2$ and $X = R^2 = x^2+z^2$) can be used to solve equation V-15, as follows:

$$\Delta\sigma_h = \frac{Pz}{x_2 - x_1}\left\{\left.\frac{-4z^2 x}{3(4z^2)X^{1.5}}\right|_{x_1}^{x_2} + \frac{4z^2}{3(4z^2)}\int_{x_1}^{x_2}\frac{dx}{X\sqrt{X}}\right\} \qquad \text{(Eq. V-17)}$$

Standard tables of integrals give:

$$\int\frac{dx}{X\sqrt{X}} = \frac{2(2cx+b)}{q\sqrt{X}} \qquad \text{(Eq. V-18)}$$

where X and q are defined as for equation V-16. Again, let $a = z^2$, $b = 0$, $c = 1$, and apply equation V-18 to equation V-17:

$$\Delta\sigma_h = \frac{Pz}{x_2 - x_1}\left\{\left.\frac{-x}{3X^{1.5}}\right|_{x_1}^{x_2} + \frac{1}{3}\left[\frac{4x}{4z^2\sqrt{X}}\right]_{x_1}^{x_2}\right\} \qquad \text{(Eq. V-19)}$$

$$= \frac{Pz}{3(x_2 - x_1)}\left\{\frac{x_1}{(x_1^2+z^2)^{1.5}} - \frac{x_2}{(x_2^2+z^2)^{1.5}} + \frac{x_2}{z^2(x_2^2+z^2)^{0.5}} - \frac{x_1}{z^2(x_1^2+z^2)^{0.5}}\right\} \qquad \text{(Eq. V-20)}$$

Addendum B to Attachment V

For a uniform vertical pressure applied over a rectangular area at the ground surface, the horizontal earth pressure acting in the ground under any corner of the rectangular area may be computed by the following equation (Poulos and Davis 1991, equation 3.18b):

$$\Delta\sigma_h = \frac{p}{2\pi}\left[\arctan\left(\frac{LB}{zR_3}\right) - \frac{LBz}{R_1^2 R_3}\right] \qquad \text{(Eq. V-21)}$$

where: $\Delta\sigma_h$ = horizontal pressure acting in the direction of the width of the loaded area at depth z under a corner of the loaded area

p = applied vertical pressure acting at ground surface
L = width of loaded area (measured perpendicular to wall)
B = length of loaded area (measured parallel to the wall)
z = depth of point below the ground surface
$R_1 = \sqrt{L^2 + z^2}$
$R_3 = \sqrt{L^2 + B^2 + z^2}$

Consider the case of a uniform vertical pressure applied over a rectangular area on the backfill adjacent to a wall, where one side of the rectangle is parallel to the wall. Then, the maximum horizontal earth pressures acting on the wall will occur along the line that is in the plane that is orthogonal to the wall and passes through the centerline of the loaded area. If the wall is rigid, then the horizontal earth pressures according to equation V-21 should be doubled (Duncan and Seed 1986, page 11). Using the principle of superposition from the theory of elasticity, equation 1 can be written for the earth pressure acting on a relatively rigid concrete retaining wall due to a point load applied at the surface of the backfill (where one edge of the rectangular loaded area touches the wall) as:

$$\Delta\sigma_h = \frac{2p}{\pi}\left[\arctan\left(\frac{LB}{zR_3}\right) - \frac{LBz}{R_1^2 R_3}\right] \qquad \text{(Eq. V-22)}$$

where: B = one-half the length of loaded area (measured parallel to the wall)
L = width of loaded area (measured perpendicular to wall)

If there is a horizontal separation of magnitude *X* between the rectangular loaded area and the wall, the maximum horizontal earth pressures can be calculated using equation V-22 and the principle of superposition from the theory of elasticity:

$$\Delta\sigma_h = \frac{2p}{\pi}\left[\arctan\left(\frac{L'B}{zR_3'}\right) - \arctan\left(\frac{XB}{zR_3}\right) - \frac{L'Bz}{\left(R_1'\right)^2 R_3'} + \frac{XBz}{R_1^2 R_3}\right] \qquad \text{(Eq. V-23)}$$

where: $L' = L + X$

and R_3' and R_1' are calculated using L' rather than L.

Limitation

For the lateral earth pressures developed in this attachment to be valid, the ground surface in the zone behind the wall must be horizontal or slope downhill away from the wall. According to Section 4.2, the final grade over the pad area shall have a nominal slope between 2 and 3 percent (CRWMS M&O 1999b, Section 1.2.1.7). Slopes of 2 to 3 percent are considered sufficiently horizontal for the values in the attachment to be used. The values in this attachment should not be used where slopes steeper than 2 to 3 percent are involved.

Attachment VI
Passive Resistance to Static Lateral Loads

To determine static passive pressure coefficient, K_p, accounting for soil/wall friction, use the passive pressure coefficient chart presented in Design Manual 7.02 (DON 1986, p. 7.2-67), which are based on the "log-spiral" method of calculation.

Engineered Fill

The input values required are:

Angle of internal friction, ϕ, in degrees

The friction angle from equation 2B for the engineered fill is high in the stress range of interest for foundations that are embedded no more than 8 feet below finished grade. Because the engineered fill may soften as resistance is mobilized, this calculation will use a reduced friction angle of 45 degrees.

Slope inclination, β, in degrees.

There is no slope, therefore $\beta = 0$.

Wall friction, δ, in degrees.

Estimate the wall-soil friction angle, δ, as ½ the soil friction angle (ϕ), or about 22.5 degrees. (Various sources recommend different values for δ. Bowles (1996, page 619) states that values of $\delta = 0.6\phi$ to 0.8ϕ are reasonable for concrete walls where forms are used, giving a relatively smooth backface. Lambe and Whitman (1969, page 175) state that the angle of wall friction is usually about equal to ϕ_{cv}, the ultimate (or constant-volume) friction angle of the backfill soil, and typically has a value of about 30°. DON (1986, page 7.2-63) recommends specific ultimate friction factors for different interface (the closest matches to the case in this attachment are formed concrete against clean sand, silty sand-gravel mixture, single size hard rock fill, for which DON (1986, page 7.2-63) recommends an ultimate friction factor of 0.30 to 0.40 and formed concrete against clean gravel, gravel-sand mixture, well-graded rock fill with spalls, for which DON (1986, page 7.2-63) recommends an ultimate friction factor of 0.40 to 0.50). It is judged that an angle of wall friction, δ, of 22.5 degrees is conservative for this calculation.)

Steps to read chart (DON 1986, Figure 6 on p. 7.2-67):

Compute $\beta/\phi = 0/45 = 0$

Find $\phi = 45$ degrees, read up to $\beta/\phi = 0$, read over to get $K_{p(\delta/\phi=-1)} \approx 33.31$.

Use table, with $\delta/\phi = -0.5$, $\phi = 45$, to obtain Reduction Factor, $R \approx 0.500$.

Multiply $K_{p(\delta/\phi=1)} \cdot R = 33.31 \cdot 0.500 = \mathbf{K_p \approx 16.7}$.

Compute the Equivalent Fluid Unit Weight (EFUW)

Using the total unit weight of the engineered fill, γ, equal to 128 lbf/ft^3 (Section I.1.1 of Attachment I), compute the equivalent fluid pressure (EFUW) using:

$$EFUW_{ultimate} = \gamma \cdot K_p = 128 \cdot 16.7 = 2{,}138 \text{ lbf/ft}^3$$

The pressure that would be exerted by this equivalent fluid acts at an angle δ to the vertical wall, hence the horizontal component is:

$$EFUW_{ultimate} \cdot \cos 22.5° \approx 1{,}975 \text{ lbf/ft}^3 \approx 2{,}000 \text{ lbf/ft}^3$$

Limitation

For this EFUW to be valid, the ground surface in the zone where passive resistance develops must be horizontal or slope uphill away from the wall. The distance, d_{min}, to which the ground must be horizontal or slope uphill away from the wall can be taken as the lateral extent of the Coulomb passive wedge:

$$d_{min} = H \cdot \tan(45° + \phi/2) = H \cdot \tan(45° + 45°/2) = 2.4H$$

If the ground surface slopes downward away from the wall within distance d_{min}, additional calculations will be required. Even beyond distance d_{min} the passive resistance may be reduced if there is a steep slope or a retaining wall. Consequently, the geotechnical engineer should review the design after the grading plan has been developed.

Alluvium

The input values required are:

Angle of internal friction, ϕ, in degrees

As discussed in Section 8.2.2 of the main text, the friction angle for the stress range of interest is 36.5 degrees. There is a small amount of cohesion (169 lbf/ft^2), which is ignored for these calculations. The effective friction angle is not decreased as was done for the engineered fill because the alluvium is not nearly so dense and thus does not have the same potential for strain softening.

Slope inclination, β, in degrees.

There is no slope, therefore β = 0.

Wall friction, δ, in degrees.

Estimate the wall-soil friction angle, δ, as ½ the soil friction angle (see above, under the Engineered Fill discussion), or about 18.3 degrees.

Steps to read chart (DON 1986, Figure 6 on p. 7.2-67):

Compute $\beta/\phi = 0/36.5 = 0$

Find 36.5 degrees, read up to $\beta/\phi = 0$, read over to get $K_{p(\delta/\phi=-1)} \approx 12$.

Use table, with $\delta/\phi = -0.5$, $\phi = 36.5$, and interpolate to obtain $R \approx 0.649$.

Multiply $K_{p(\delta/\phi=1)} \cdot R = 12 \cdot 0.649 = \mathbf{K_p \approx 7.8}$.

Compute the Equivalent Fluid Unit Weight (EFUW)

Using the total unit weight of the alluvium for depths greater than eight feet, γ, of 117 lbf/ft^3 (Section I.2.1 of Attachment I), compute the equivalent fluid unit weight (EFUW) using:

$$EFUW_{ultimate} = \gamma \cdot K_p = 117 \cdot 7.8 = 912 \text{ lbf/ft}^3$$

The pressure that would be exerted by this equivalent fluid acts at an angle δ to the vertical wall, hence the horizontal component is

$$EFUW_{ultimate} \cdot \cos 18.3° = 866 \text{ lbf/ft}^3 \approx 850 \text{ lbf/ft}^3$$

Limitation

For this EFUW to be valid, the ground surface in the zone where passive resistance develops must be horizontal or slope uphill away from the wall. The distance, d_{min}, to which the ground must be horizontal or slope uphill away from the wall can be taken as the lateral extent of the Coulomb passive wedge:

$$d_{min} = H \cdot \tan(45° + \phi/2) = H \cdot \tan(45° + 36.5°/2) = 2.0H$$

If the ground surface slopes downward away from the wall within distance d_{min}, a particular calculation will be required. Even beyond distance d_{min} the passive resistance may be reduced if there is a steep slope or a retaining wall. Consequently, the geotechnical engineer should review the design after the grading plan has been developed.

Attachment VII
Slope Stability

This slope stability calculation is performed using charts by Charles and Soares (1984). This method involves computing Γ, a dimensionless stability number, and using the chart of slope inclination as a function of Γ and b (Figure 3 in Charles and Soares (1984)).

Material strength is given by the power function (Charles and Soares 1984, equation 2):

$$\tau_{ff} = A\,(\sigma'_{ff})^{b} \qquad \text{(Eq. VII-1)}$$

where:

b is a dimensionless exponent
A is a constant with dimensions equal to the dimensions of σ_{ff} to the (1-b) power.

Use of Figure 3 in Charles and Soares (1984) requires calculating the value of Γ, a dimensionless stability number:

$$\Gamma = F_s/A \cdot (\gamma\, H)^{(1-b)} \qquad \text{(Eq. VII-2)}$$

where: F_s = required factor of safety
γ = unit weight of soil
H = height of slope

The moist unit weights of engineered fill and alluvium were taken from Sections I.1.1 and I.2.1. The shear strength envelops for engineered fill and alluvium were taken from Sections I.1.2 and I.2.2.

Charles and Soares (1984, Figure 3) presents solutions using the methods of slices of Fellenius and Bishop. A copy of Figure 3 from Charles and Soares (1984) is included as Figure VII-1 and shows interpolated curves for the values of b for the engineered fill and the alluvium (Table 3).

Table VII-1. Analyses for Temporary Slopes with Factor of Safety = 1.25

Material	γ kips/ft^3	**b**	**A** [kips/ft^2]$^{(1-b)}$	**H** feet	Γ	cot β from Fig. VII-1
Alluvium	0.117	0.95450603	0.98808555	11	1.280	1.1
				22	1.321	1.1
				33	1.345	1.1
				44	1.363	1.2
				55	1.377	1.2
Engineered fill	0.128	0.7543251	1.6636572	10	0.798	< 0.5
				15	0.882	< 0.5
				20	0.947	< 0.5
				25	1.000	< 0.5

Note: cot β is the slope inclination in the format cot β horizontal to 1 vertical.

Table VII-2. Analyses for Permanent Slopes with Factor of Safety = 1.5

Material	γ kips/ft^3	b	A [kips/ft^2]$^{(1-b)}$	H feet	Γ	cot β from Fig. VII-1
Engineered fill	0.128	0.7543251	1.6636572	10	0.958	< 0.5
				15	1.058	< 0.5
				20	1.136	< 0.5
				25	1.120	< 0.5

Note: cot β is the slope inclination in the format cot β horizontal to 1 vertical.

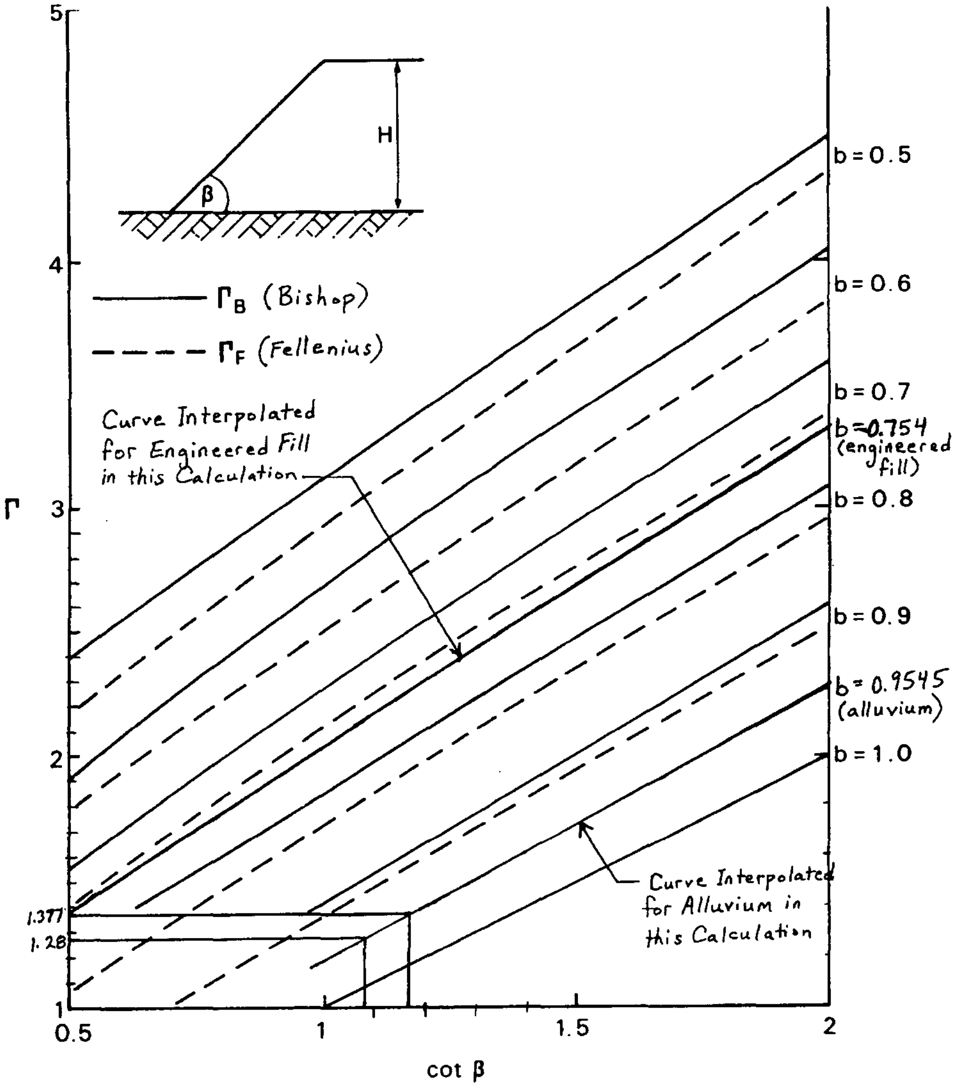

Fig. 3. Stability numbers from circular arc analyses

Modified from Charles and Soares (1984, Figure 3)

Figure VII-1. Stability Numbers from Circular Arc Analyses

Attachment VIII
Lateral Earth Pressures on Temporary Shoring

Lateral Earth Pressures Acting on Tied-back/Braced Shoring

As discussed in Sections 6 and 7, the engineered fill and alluvium are gravels and sands with few fines. For this type of material, Fang (1991, Figure 12.22) recommends a distribution developed by Peck. Peck's distribution is a uniform horizontal pressure, p_h, given by:

$$p_h = 0.65\, K_a\, \gamma\, H \qquad \text{(Eq. VIII-1)}$$

where: γ = moist unit weight of soil
H = the shoring height
K_a = the active earth pressure coefficient

K_a can be calculated by the following equation (DON 1986, p. 7.2-62):

$$K_a = \tan^2\left(45 - \frac{\phi'}{2}\right) \qquad \text{(Eq. VIII-2)}$$

where: ϕ' = internal friction angle of soil (35° for the alluvium for embedments up to 55 feet – see Attachment V, and reflecting some loosening of the soil behind the shoring during installation)

The total horizontal force per lineal foot of shoring, P_h, is the product of p_h and the shoring height.

shoring height (ft)	35	40	45	50	55
p_h (lbf/ft^2)	721.3	824.4	927.4	1030.4	1133.5
P_h (pounds per foot of wall)	25,246	32,974	41,733	51,522	62,342

Relative to the grades existing before the existing fill was constructed (original grade), engineered fill up to 25 feet thick will be required in the southwest part of the pad and excavation up to 11 feet deep will be required at the north end of the pad (Section 7). The area where the thickness of engineered fill exceeds 20 feet is limited, so it is judged that the engineered fill, if there is any at the time the shoring is installed, should not exceed about 20 feet in depth. The alluvium against the shoring may range from about 35 to 55 feet in extent and may or may not be overlain by engineered fill, depending on construction sequence.

Passive Resistance for Soldier Piles in Tiedback/Braced Shoring Systems

For calculation of the ultimate passive resistance for a soldier pile, Design Manual 7.02 (DON 1986, p. 7.2-112) states that the ultimate passive resistance of a soldier pile is approximately 3 times the passive pressure of the soil acting on the embedded area of the soldier pile. Note that the passive force acts horizontally since friction is taken to be zero along the pile-soil interface in the Design Manual 7.02 (DON 1986, p. 7.2-112) method. DON (1986, p. 7.2-112) also states that the soil resistance in front of the wall to a depth of one soldier-pile diameter below the bottom of excavation should be ignored. It is recommended to utilize a more conservative version of this provision and consider the soil in front of the wall to a depth of one soldier-pile diameter below the bottom of excavation to be non-existent.

$$P_{p,ult} = 3\ B\ K_p\ (\tfrac{1}{2}\ \gamma\ (H\text{-}B)^2) \qquad \text{(Eq. VIII-3)}$$

where: B = the width of the soldier pile
H = the depth over which the soldier pile moves enough to develop passive resistance
γ = moist unit weight of the soil developing passive pressure (117 lbf/ft^3 for the alluvium – see Section I.2.1 of Attachment I)
$K_p = \tan^2(45° + \phi'/2)$ (DON 1986, p. 7.2-62)

The ultimate passive force should be divided by an appropriate factor of safety.

As an example, suppose an 18-inch diameter soldier pile is embedded 11.5 feet below the bottom of excavation and it is determined that the entire embedded length can rotate/translate sufficiently to develop passive pressure. Then, the ultimate passive force, $P_{p,ult}$, on one soldier pile is (using equation VIII-3):

$$P_{p,ult} = 3\ (1.5\ ft)\ \tan^2(45° + 42°/2)\ (\tfrac{1}{2})(117\ lbf/ft^3)\ (11.5\ ft\ \text{-}1.5\ ft)^2 = 132{,}800 \text{ pounds}$$

The ultimate passive force should be divided by an appropriate factor of safety selected as a function of the movement expected in over the pile length where the passive pressure is developed, since substantial pile movement is required to develop full passive pressure.

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