BSC

Design Calculation or Analysis Cover Sheet

Complete only applicable items.

1. QA: QA

2. Page 1

3. System	4. Document Identifier
Subsurface Investigations	100-S0C-CY00-00100-000-00E

5. Title

Supplemental Soils Report

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Geotechnical (C/S/A)

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8. Notes/Comments

Attachments	Total Number of Pages
Appendix A–Not Used	1
Appendix B–Bearing Capacity and Settlement	56
Appendix C–Lateral Earth Pressures and Resistance to Lateral Loads	26

RECORD OF REVISIONS

9. No.	10. Reason For Revision	11. Total # of Pgs.	12. Last Pg. #	13. Originator (Print/Sign/Date)	14. Checker (Print/Sign/Date)	15. EGS (Print/Sign/Date)	16. Approved/Accepted (Print/Sign/Date)
00A	Initial Issue	243	C-26	Peter Chiu 10/27/04	James T. Cameron 10/27/04	----	Farhang Ostadan 11/1/04 R. E. Pernisi 11/4/04
00B	Complete Revision; revised format to conform to EG-DSK-3003; revised contents of 6.4.2.3 (formerly 10.2.3) and 7.1.11 (formerly 11.11) and replaced figure 7-1 (formerly 11-1) to resolve CR5565 concerns. Also addresses issues raised in CR8288 regarding consistency with Ground Motion Report (Section 6.4.2). Note: the layout of the facilities is currently under revision. Even though the exact number and configuration of facilities will change, the design philosophy will continue to use multiple facilities of different sizes located within the North Portal area similar to that shown on Figure 1-1. The calculation will be updated when the layout for the facilities is revised.	255	C-26	James T. Cameron 8/15/06	Nan Deng 8/15/06	------	Farhang Ostadan 8/16/06 Raj S. Rajagopal 8/21/06
00C	Complete Revision; replaced Figs. 1-1 and 6-7; modified Section 7.1.7 and title to Fig 7-1; revised organization of Section 2.	265	C-26	James T. Cameron 2/21/07	Nan Deng 2/21/07	Richard Kotas 2/22/07	Farhang Ostadan 2/22/07 Raj S. Rajagopal 2/22/07
00D	Complete Revision: Replaced Figure 1-1. Added discussion in Section 1.3 regarding revised building layout. Added references, tables and figures in Section 6 regarding additional field and laboratory studies done in 2005 and 2006. Modified App. C to reflect revised building dimensions. Added to Section 7.1.9, Percolation Rates. Modified Tables 2-1 and 2-2 to reflect revised design parameters. Modified Section 3.2 to alert to additional assumptions not requiring verification in Appendices B and C.	272	C-26	James T. Cameron 12/17/07	Nan Deng 12/18/07	James Dockery 12/20/07	Farhang Ostadan 12/18/07 Raj Rajagopal 12/22/07

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00E	Complete Revision: Addresses CR12357 to incorporate information contained in the Supplemental Ground Motion Report (MDL-MGR-GS-000007), issued Feb. 2008. Addresses CR12206, which involved misnumbered figures. Addresses DOE/RW-0585 comments of March 2008. Includes CACN001 and CACN002 into main report. Incorporates 2008 geotechnical data and seismic reports. Replaces Section 6.4.2. Removes Figs. 6-18 through 6.39, and renumbers Figs 6-40 and 6-41 to 6-18 and 6-19, respectively. Removes App. A and its associated references from main text and App. B. Modifies Table 2-1 to reflect changes in references. Updates various DTN's to latest revisions.	192	C-26	James T. Cameron 8/28/08	Nan Deng 8/28/08	Tom Frankert 8/28/08	Farhang Ostadan F. Ostadan 8/28/08 Raj S. Rajagopal MDS 8/28/08

MAA 8/28/08

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DISCLAIMER

The calculations contained in this document were developed by Bechtel SAIC Company, LLC (BSC) and are intended solely for the use of BSC in its work for the Yucca Mountain Project.

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APPENDICES:

Appendix A. Not used
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Appendix C. Lateral Earth Pressures and Resistance to Lateral Loads

LIST OF FIGURES

LIST OF TABLES

ACRONYMS AND ABBREVIATIONS

ACC	Accession Number
ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BLM	U.S. Bureau of Land Management
BSC	Bechtel SAIC Company
c	cohesion
CBR	California Bearing Ratio
C_c	coefficient of curvature
CCCF	Central Control Center Facility
CF	fines content
CPT	Cone penetrometer test
CRCF	Canister Receipt and Closure Facility
CRWMS	Civilian Radioactive Waste Management System
C_u	coefficient of uniformity
D_{10}	grain diameter (in mm) corresponding to 10% passing, by weight (or mass)
D_{30}	grain diameter (in mm) corresponding to 30% passing, by weight (or mass)
D_{60}	grain diameter (in mm) corresponding to 60% passing, by weight (or mass)
d_f	depth of footing
DH	downhole
DIRS	Document Input Reference System
DOE	U.S. Department of Energy
DTN	Data Tracking Number
E	Young's modulus or secant Young's modulus
elev.	elevation
EMWB	Equipment Maintenance/Warehouse Building
EPRI	Electrical Power Research Institute
Eq.	equation
ESF	Exploratory Studies Facility
Fpm	feet per minute
fps	feet per second
ft	foot, feet (unit of measurement)
ft/s	feet per second
ft^2	feet squared
ft^3	feet cubed
G	shear modulus
GM	silty gravel
G_{max}	small-strain (maximum) shear modulus
GP	poorly-graded gravels or gravel-sand mixtures, little or no fines

ACRONYMS AND ABBREVIATIONS (CONTINUED)

GSF	Ground Surface Facility
GW	well-graded gravels or gravel-sand mixtures, little or no fines
H:V	Horizontal:Vertical
HEMF	Heavy Equipment Maintenance Facility
IBC	International Building Code
ICC	International Code Council
ID	identification
IHF	Initial Handling Facility
in.	inch, inches
K_A	coefficient of active earth pressure
K_P	coefficient of passive earth pressure
kip	1,000 pounds (kilopound)
kips/ft^2	kips per square foot
kips/ft^3	kips per cubic foot
K_o	coefficient of at-rest soil pressure
kcf	kips per cubic foot
ksf	kips per square foot
lb/ft^2	pounds per square foot
lb/ft^3	pounds per cubic foot
lbf	pounds force
lb	pounds (usually pounds-force)
LL	liquid limit
MC	moisture content
mm	millimeter
M&O	Management and Operating Contractor
MWV	Midway Valley
N	SPT penetration resistance (blow count)
N_{60}	SPT penetration resistance corrected to 60% efficiency
NNWSI	Nevada Nuclear Waste Site Investigation
NRG	North Ramp Geotechnical
NRSF	North Ramp Surface Facilities
NTS	Nevada Test Site
ORD	Office of Repository Development
p	page
PC	personal computer
pcf	pounds per cubic foot

ACRONYMS AND ABBREVIATIONS (CONTINUED)

pci	pounds per cubic inch
PGA	peak ground acceleration
PI	plasticity index
pp	pages
psf	pounds per square foot
psi	pounds per square inch
"Q"	"quality"
QA	quality assurance
Qal	Quaternary alluvium
RCSC	roller compacted soil cement
RCTS	Resonant Column & Torsional Shear
Rev.	revision
RF	Receipt Facility
SASW	spectral analysis of surface waves
SFS	Surface Facility System
SM	silty sands, sand-silt mixtures
SN	Scientific Notebook
SNL	Sandia National Laboratory
SP	poorly-graded sand or gravelly sands, little or no fines
SPT	Standard Penetration Test
SSI	soil structure interaction
SW	well-graded sand or gravelly sands, little or no fines
tcf	tons (American) per cubic foot
TIC	Technical Information Center
Tmbtl	pre-Rainier Mesa Tuff bedded tuff
Tmr	Rainer Mesa Tuff of the Timber Mountain Group
tons/ft^3	tons (American) per cubic foot
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
Tpcpun	Tiva Canyon Tuff: crystal-poor member, upper nonlithophysal zone
Tpcrn	Tiva Canyon Tuff: crystal-rich member, nonlithophysal zone, but used in BSC (2002) to mean the Tpcr member
Tpki	Tuff unit "x"
tsf	tons (American) per square foot
URS	URS Corporation
USBR	U.S. Bureau of Reclamation

ACRONYMS AND ABBREVIATIONS (CONTINUED)

USN	U.S. Department of the Navy
USS	United States Steel
UTA	University of Texas, Austin
V_p	compression-wave seismic velocity
V_s	shear-wave seismic velocity
WHB	waste handling surface facilities formally designated as WHB or Waste Handling Building
WHF	Wet Handling Facility
WNNRF	Warehouse and Non-Nuclear Receipt Facility
YMP	Yucca Mountain 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.

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.

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

engineered fill–a fill placed by man that meets several criteria, 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 materials' particles, on a dry weight basis, that pass through a U.S. Standard 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.

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, groundwater is not a consideration, so effective overburden pressure is taken to be the same as total overburden pressure.

percent core recovery–in a given cored interval, the ratio of the length of core recovered to the length of the interval, expressed as a percentage.

Poisson's ratio, υ–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, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000ft-lbf/ft^3 (2,700kN-m/m^3)).*

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.

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. 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 of 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 adsorbed water is not considered part of the water in the pore spaces but as water bound to the solid particles–synonym of 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

This report is written as a companion report to *Soils Report for North Portal Area, Yucca Mountain Project,* BSC (2002b). The primary purpose of the current report is to adopt, clarify, and summarize the findings and recommendations of BSC (2002a) and BSC (2002b) into design charts and tables to be used for the design of surface facilities at the Yucca Mountain Project Site (YMP). The surface facilities include all associated surface structures for the nuclear waste handling and support facilities. This report also recommends additional soils investigation and testing for the non-waste handling facilities. These recommendations have been developed for use in design of the surface facilities to a level suitable to support the License Application process.

Subsequent to the issuance of Revision 00A of this calculation additional field and laboratory studies were performed (SNL 2008) and ground motion reports for the site were written (BSC 2004a and BSC 2008a) that more thoroughly address dynamic properties and other seismic considerations, including shear and compression wave velocities and material degradation relationships. This current calculation revision includes consideration of these additional studies and analyses.

1.2 SCOPE

The scope of this report is to provide simplified charts and recommendations of geotechnical parameters to be used for the design and analysis of the surface facilities. Where pertinent, the recommendations provided in BSC (2002b) are used. The current report summarizes the pertinent field and laboratory investigations, the results of material property tests, and provides engineering design parameters including allowable bearing capacity, settlement, lateral earth pressures on retaining walls, and slope evaluation based on site-specific subsurface soil information. Additional recommendations provided include pavement design parameters, percolation rates, and frost penetration. Construction considerations and additional investigations and testing are also discussed.

1.3 PROJECT DESCRIPTION

The configuration of the nuclear waste handling surface facilities area has changed over much iteration from a single building encompassing all aspects of the waste handling process to the configuration used herein, which consists of several major storage and process facilities. The facility layout is shown in Figure 1-1 (Figure 6.2-1 in SNL 2008, see also BSC 2007f). The largest structures are the two aging pads to the north of the building cluster (see also BSC 2007h). The largest buildings are the Canister Receipt and Closure Facilities (Building Nos. 080, 070, and 060). Other major structures include the Wet Handling Facility (050); Initial Handling Facility (51A); Receipt Facility (200); and the Emergency Diesel Generator Facility (26D). The southeast portion of the site area contains an evaporation pond and a stormwater/retention pond. Several smaller facilities (administration, fire rescue, medical, storage, etc.) are located in the southern portion of the site. The nuclear handling surface facilities are typically constructed with heavy reinforced concrete walls, floor and roof slabs, and heavy structural steel framing systems. Foundation pressures are expected to be on the order of 3 to 5 ksf (static) and 10 ksf (dynamic) under the planned structures. A summary of the building dimensions, weights, elevations and reference sources for the larger buildings is provided in Table 1-1 below.

Table 1-1 Summary of Planned Buildings

						References for Dimensions and Loads	
Building	**Plan Dimensions (ft)**	**Slab Thickness (ft)**	**Bottom of Slab Elev (ft)**	**Load (1000 kips)**	**Pressure (ksf)**	**Drawing Reference**	**Calculation**
Receipt Facility, RF	284 x 242	7	3651	189.7	3.45	200-DB0-RF00-00101-000, Rev. 00B	200-SYC-RF00-00100-000-00A
Emergency Diesel Generator Facility	98 x 174	4	3663	28.7	1.68	26D-P10-EG00-00102-000, Rev. 00A	26D-SYC-EG00-00300-000-00A
Canister Receipt and Closure Facility , CRCF #1	262 x 421	6	3656	314.2	3.33	060-DB0-CR00-00101-000, Rev. 00B	060-DBC-CR00-00200-000-00A
Canister Receipt and Closure Facility , CRCF #2			3657				
Canister Receipt and Closure Facility , CRCF #3			3660				
Initial Handling Facility, IHF Large Structure (IHF Cask Process Area Main Structure)	170 x 196.5	6	3672	56.7	1.70	51A-P10-IH00-00102-000, Rev. 00C	51A-DBC-IH00-00200-000-00B
Initial Handling Facility, IHF Small Structure (IHF Loadout Area Concrete Structure)	141.5 x 75		3665	32.3	3.05		
Wet Handling Facility, WHF (pool)	114 x 116	8	3607	42.6	3.22	050-DB0-WH00-00101-000, Rev. 00A	050-SYC-WH00-00300-000-00B
Wet Handling Facility, WHF (building)	270 x 214	6	3661	269.7	4.67	050-DB0-WH00-00102-000, Rev. 00B	

Note: Building dimensions, weights, elevations, and associated drawings and references are all subject to change as design progresses. This table is provided as an approximation of the current building design information. The most up to date information should be used for final design. However, it is not expected that future changes in the building designs will have any significant impact on the conclusions of this report.

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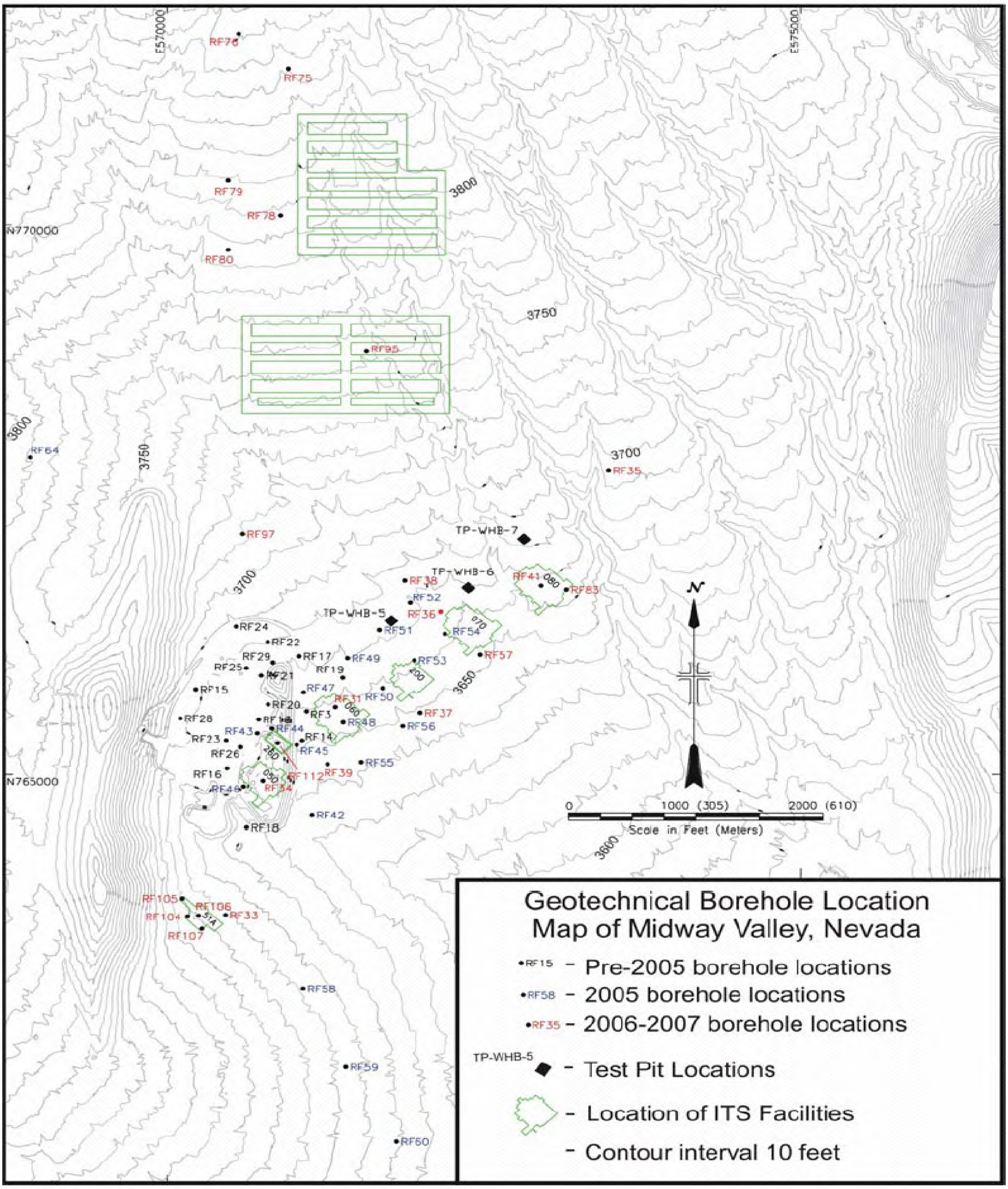

Source: DTNs: GS030783114233.001, GS070683114233.005, GS931008314211.036, MO0707RFGNPMV1.000, MO0706ABRTP567.000, MO0612SMFGLGIB.000, for boreholes, test pits; BSC 2007e, BSC 2007g for ITS facilities.

Figure 1-1. Location Map Showing Geotechnical Boreholes from pre-2005, 2005, and 2006 to 2007 Drilling Programs (SNL 2008, Figure 6.2-1)

1.4 LIMITATIONS

Limitations stated in Section 1.3 of BSC (2002b) apply to this report and are briefly summarized below (refer to BSC 2002b for full descriptions):

1. These recommendations are intended to provide geotechnical input for the surface facilities to support the License Application process.

2. When the final building configuration and borrow source are defined the recommendations should be reviewed to evaluate whether any changes or additional confirmatory borings or field tests are needed (These items are addressed in Section 7.3 of this report.).

3. The bases for the recommendations are limited to the borings, field tests, and laboratory tests performed in the vicinity of the site to date. Although not likely, unanticipated subsurface conditions may be present. The recommendations provided in this report are based on no major deviations occurring from what was observed in the studies to date.

4. The recommended bearing capacities and lateral earth pressures are for near horizontal ground conditions (i.e., less than or equal to a 3% slope). However, modifications to the recommendations can be made on a case-by-case basis for any specific conditions that vary appreciably from the near horizontal ground condition.

5. Any person using this report for bidding purposes should perform independent investigations, as they deem necessary to satisfy themselves that the surface and subsurface conditions are suitably accurate to determine construction procedures and methods.

2 REFERENCES

2.1 PROCEDURES/DIRECTIVES

EG-PRO-3DP-G04B-00037, Rev. 12. *Calculations and Analyses.* Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080519.0005.

IT-PRO-0011, Rev. 9. *Software Management.* Las Vegas, Nevada: Bechtel SAIC Company. ACC: DOC.20080416.0010.

ORD (Office of Repository Development) 2007. *Repository Project Management Automation Plan*. 000-PLN-MGR0-00200-000, Rev. 00E. Las Vegas, Nevada: U.S. Department of Energy, Office of Repository Development. ACC: ENG.20070326.0019.

2.2 DESIGN INPUTS

The input data used or considered in the calculation herein are primarily adopted from the following references (for the surface facilities area):

- Geotechnical Data for Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project, BSC (2002a)
- Soils Report for North Portal Area, Yucca Mountain Project, BSC (2002b)
- Geotechnical Data for a Geological Repository at Yucca Mountain, SNL (2008)
- Development of Earthquake Ground Motion Input for Preclosure Seismic Design and Postclosure Performance Assessment of a Geologic Repository at Yucca Mountain, BSC (2004a)
- Supplemental Earthquake Ground Motion Input for a Geologic Repository at Yucca Mountain, BSC (2008a)

Input data taken from other sources are indicated where they are used.

2.2.1 Input Documents

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ASCE 4-98. 2000. *Seismic Analysis of Safety-Related Nuclear Structures and Commentary.* Reston, Virginia: American Society of Civil Engineers. TIC: 253158. (DIRS 159618)

Bowles, J.E. 1996. *Foundation Analysis and Design.* 5th Edition. New York, New York: McGraw-Hill. TIC: 247039. (DIRS 157929)

BSC (Bechtel SAIC Company) 2002a. *Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project.* ANL-MGR-GE-000003 REV 00. Las Vegas, Nevada: Bechtel SAIC Company. ACC: MOL.20021004.0078. (DIRS 157829)

BSC (Bechtel SAIC Company) 2002b. *Soils Report for North Portal Area, Yucca Mountain Project.* 100-00C-WRP0-00100-000-000. Las Vegas, Nevada: Bechtel SAIC Company. ACC: MOL.20021015.0323; ENG.20050823.0018. (DIRS 159262)

BSC (Bechtel SAIC Company) 2002c. Preliminary Hydrologic Engineering Studies for the North Portal Pad and Vicinity. ANL-EBS-MD-000060 REV 00. Las Vegas, Nevada: Bechtel SAIC Company. ACC: MOL.20021028.0123. (DIRS 157928)

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Yucca Mountain, NV. MDL-MGR-GS-000003 REV 01. Las Vegas, Nevada: Bechtel SAIC Company. ACC: DOC.20041111.0006. (DIRS 170027)

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BSC (Bechtel SAIC Company) 2006. Receipt Facility (RF) Mass Properties. 200-SYC-RF00-00100-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20061206.0001. (DIRS 178795)

BSC (Bechtel SAIC Company) 2007a. Project Design Criteria Document. 000-3DR-MGR0-00100-000-007. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071016.0005. (DIRS 179641)

BSC (Bechtel SAIC Company) 2007b. CRCF Foundation Design. 060-DBC-CR00-00200-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070322.0005. (DIRS 184027).

BSC (Bechtel SAIC Company) 2007c. Emergency Diesel Generator Facility – Mass Analysis & Moments of Inertia. 26D-SYC-EG00-00300-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070323.0003. (DIRS 184115)

BSC (Bechtel SAIC Company) 2007d. Wet Handling Facility (WHF) Mass Properties. 050-SYC-WH00-00300-000-00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070326.0001. (DIRS 184008)

BSC (Bechtel SAIC Company) 2008a. Supplemental Earthquake Ground Motion Input for a Geologic Repository at Yucca Mountain, NV. MDL-MGR-GS-000007 REV 00. Las Vegas, Nevada: Bechtel SAIC Company. ACC: DOC.20080221.0001. (DIRS 183776)

BSC (Bechtel SAIC Company) 2008b. Basis of Design for the TAD Canister-Based Repository Design Concept. 000-3DR-MGR0-00300-000-002. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080229.0007. (DIRS 185025)

BSC (Bechtel SAIC Company) 2008c. Initial Handling Facility (IHF) Foundation Design. 51A-DBC-IH00-00200-000-00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080221.0005. (DIRS 185085)

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2.2.2 Standards

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2.2.3 Data Tracking Numbers (DTNs)

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GS020383114233.003. Geotechnical Borehole Logs for the Waste Handling Building, Yucca Mountain Project, Nevada Test Site, Nevada. Submittal date: 03/28/2002. (DIRS 157980)

GS020483114233.004. Geotechnical Field and Laboratory Test Results from Waste Handling Building Foundation Investigation. Submittal date: 04/15/2002. (DIRS 158242)

GS020783114233.005. Gradation Analysis Test Results and Graphical Plots from Tests Performed on Materials Excavated from In-Situ Density Test Locations in Test Pits from the Waste Handling Building Foundation Investigations. Submittal date: 07/23/2002. (DIRS 159542)

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GS070583114233.002. Geologic Descriptive Logs of Fill and Quaternary Alluvium Material in 19 Repository Facilities Geotechnical Investigations Boreholes for the Yucca Mountain Waste Handling Building, 04/12/2005 - 09/12/2005. Submittal date: 05/22/2007. (DIRS 183302)

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GS070683114233.005. Geotechnical Borehole Logs of 18 Repository Facilities Geotechnical Investigations Boreholes for the Yucca Mountain Waste Handling Building, 05/18/2007 - 06/20/2007. Submittal date: 06/20/2007. (DIRS 182109)

GS080183114233.001. Index Properties of Alluvium Soils from Two Sonic Drill Core Holes Obtained at Yucca Mountain Project, 07/20/2006 to 09/28/2006. Submittal date: 01/08/2008. (DIRS 184671)

GS950308312213.004. Cumulative Infiltration and Surface Flux Rates Conducted in Fortymile Wash and Near UE-25 UZN#7. Submittal date: 03/27/1995. (DIRS 158474)

GS960908312212.009. Cumulative Infiltration and Surface Flux Rates Calculated on Raw Millivolt Readings for FY95. Submittal date: 09/12/1996. (DIRS 158473)

GS931008314211.036. Graphical Lithologic Log of Borehole RF-3 (UE-25 RF#3), Version 1.0. Submittal date: 10/07/1993. (DIRS 150006)

MO0008GSC00286.000. Exploratory Studies Facility (ESF) North Portal Pad, Waste Handling Building (WHB) Profile Sections #3, #4, #5, #6, #7, and #8. Submittal date: 08/17/2000. (DIRS 157306)

MO0112GSC01170.000. Borrow Pit #1 (Fran Ridge), USBR Sample Locations, for WHB Investigations. Submittal date: 12/04/2001. (DIRS 157302)

MO0203DHRSSWHB.001. Dynamic Laboratory Test Data for Rock and Soil Samples from the Waste Handling Building Site Characterization Area. Submittal date: 03/19/2002. (DIRS 158082)

MO0203EBSCTCTS.016. Compaction and Triaxial Compression Tests of Soil Sample. Submittal date: 04/01/2002. (DIRS 157970)

MO0206EBSFRBLT.018. Fran Ridge Borrow Lab Testing. Submittal date: 06/10/2002. (DIRS 158767)

MO0609SASWSEDC.001, Surface Spectral Analysis of Surface Waves (SASW) Experimental Dispersion Curves for FY04 and FY05 for YMP. Submittal Date: 09/19/2006. (DIRS 183293)

MO0609SASWSTDC.003, Surface Spectral Analysis of Surface Waves (SASW) Theoretical Dispersion Curves and VS Profiles for FY04 and FY05 for YMP. Submittal Date: 09/19/2006. (DIRS 182125)

MO0612SMFGLGIB.000. Sample Management Facility Geologic Logs for the Repository Facilities Geotechnical Investigations Boreholes. Submittal date: 12/18/2006. (DIRS 183648)

MO0701ABSRFLL2.000. SASW Investigations for Repository Facilities, As-Built SASW RF Line Locations-2. Submittal date: 01/11/2007. (DIRS 182483)

MO0706ABRTP567.000. As-Built Proposed Repository Facility Test Pits 5, 6 & 7. Submittal date: 07/10/2007. (DIRS 183301)

MO0707RFGNPMV1.000. Repository Facility (RF) Geotechnical Investigations North Portal & Midway Valley - Part 1. Submittal date: 07/24/2007. (DIRS 183189)

MO0708SMFGLGIB.000. Sample Management Facility Geologic Logs for the Repository Facilities Geotechnical Investigations Boreholes. Submittal date: 08/10/2007. (DIRS 183304)

SNF29041993001.002. Percolation Test Data, EFS Muck Storage Area. Submittal date: 12/21/1994. (DIRS 156087)

2.2.4 Drawings

BSC (Bechtel SAIC Company) 2007e. Geologic Repository Operations Area North Portal Site Plan. 100-C00-MGR0-00501-000 REV 00D. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070925.0003. (DIRS 183259) – Historic basis for Figure 1-1.

BSC (Bechtel SAIC Company) 2007f. Geologic Repository Operations Area North Portal Site Plan. 100-C00-MGR0-00501-000 REV 00F. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071116.0004. (DIRS 184014)

BSC (Bechtel SAIC Company) 2007g. Geologic Repository Operations Area Aging Pad Site Plan. 170-C00-AP00-00101-000 REV 00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070926.0008. (DIRS 180072) - Historic basis for Figure 1-1.

BSC (Bechtel SAIC Company) 2007h. Geologic Repository Operations Area Aging Pad Site Plan. 170-C00-AP00-00101-000 REV 00C. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071116.0005. (DIRS 184057)

BSC (Bechtel SAIC Company) 2007i. Emergency Diesel Generator Facility General Arrangement Ground Floor Plan. 26D-P10-EG00-00102-000, Rev. 00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071026.0009. (DIRS 183585)

BSC (Bechtel SAIC Company) 2007j. Initial Handling Facility General Arrangement Ground Floor Plan. 51A-P10-IH00-00102-000 REV 00C. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071226.0017. (DIRS 184529)

BSC (Bechtel SAIC Company) 2007k. Nuclear Facilities Buildings, Wet Handling Facility, Forming Plan at TOC El. (-)34'-0" and (-)52'-0", 050-DB0-WH00-00101-000, Rev 00A, Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070731.0003.

BSC (Bechtel SAIC Company) 2008d. Nuclear Facilities Buildings Canister Receipt and Closure Facility #1 Forming Plan at TOC EL 0'-0", 060-DB0-CR00-00101-000, Rev. 00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080117.0025.

BSC (Bechtel SAIC Company) 2008e. Nuclear Facilities Buildings Receipt Facility Forming Plan at TOC EL 0'-0", 200-DB0-RF00-00101-000, Rev. 00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080205.0002.

BSC (Bechtel SAIC Company) 2008f. Nuclear Facilities Buildings Wet Handling Facility Forming Plan at TOC EL 0'-0", 050-DB0-WH00-00102-000, Rev. 00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080107.0004.

2.3 DESIGN CONSTRAINTS

None.

2.4 DESIGN OUTPUTS

This calculation will be used as input for other calculations. Summaries of material properties and design parameters derived from this calculation are provided in Tables 2-1 and 2-2.

2.5 INPUTS FROM CANCELLED/SUPERCEDED DOCUMENTS

The following cancelled and superseded documents were used in the preparation of this report and are identified herein as historical information. At the time of use, the cancelled and superseded procedures were appropriate procedures to follow for the work performed. In addition, the quality of the work, the results, recommendations and conclusions were not impacted by the cancellation of these procedures.

DTN:GS020383114233.003 was superseded by DTN:GS030783114233.001 to account for bedrock corrections. However, the differences were relatively minor and did not affect the conclusions and recommendations of this report.

Therefore, these documents remain in this calculation for historical traceability purposes.

2.5.1 Procedures/Directives

PA-PRO-0310, Rev 0. *Laboratory Dynamic Rock/Soil Testing*, Las Vegas, Nevada: Bechtel SAIC Company. Effective date: 03/31/2006 (Cancelled: 10/02/2006).

PA-PRO-0312, Rev. 0. *The Preparation, Planning, and Field Verification of Surface-Based Geophysical Logging Operations*, Las Vegas, Nevada: Bechtel SAIC Company. Effective date: 02/28/2006 (Cancelled: 10/02/2006).

AP-SIII.6Q, Rev. 0, ICN 0. *Geophysical Logging Programs for Surface-Based Testing Program Boreholes*, Las Vegas, Nevada: Department of Energy, Office of Civilian Radioactive Waste Management (OCRWM). Effective date: 05/23/2001. Superseded by AP-SIII-8Q, Effective date: 10/26/2001 (Cancelled: 01/27/2005).

2.5.2 Data Tracking Numbers (DTNs)

GS020383114233.003. Geotechnical Borehole Logs for the Waste Handling Building, Yucca Mountain Project, Nevada Test Site, Nevada. Submittal date: 03/28/2002 (DIRS 157980). Superseded by GS030783114233.001 (DIRS 164561) on 07/24/2003.

Table 2-1. Recommended Material Parameters

Design Parameter[a]	Layer			
	Engineered Fill	Roller Compacted Cement [b]	Alluvium	Bedrock
Moist Density, γ (pcf)	127 pcf	130–140 pcf	114–117 pcf	100 pcf
Specific Gravity, G_s	2.5	Not Applicable	2.5	Not Applicable
Shear Strength Parameters	$\phi = 42°$ $c = 0$	$\phi = 0$ $c = 400$ psi (unconf. comp.)	$\phi = 39°$ $c = 0$	Not Applicable
At-Rest Earth Pressure Coefficient, K_o	0.33	Not Applicable	0.37	Not Applicable
Active Earth Pressure Coefficient, K_A	0.20	Not Applicable	0.23	Not Applicable
Passive Earth Pressure Coefficient, K_P	5.0	Not Applicable	4.4	Not Applicable
Static Elastic Modulus, E (ksi)	14–28	Not Available	30–75	Not Applicable
Poisson's Ratio, υ	0.3–0.4	0.3	0.25–0.34[f]	0.29 [f]
Shear Wave Velocity, Vs (fps)	630–1,500	2,000–3,000	BSC (2008a), Section 6.4.2	BSC (2008a), Section 6.4.2
Compression Wave Velocity, Vp (fps)	1,500–3,700 [d]	3,700–5,600[d]		
Low-Strain Shear Modulus, G (ksi) [e]	10–60	100–270	40–200	150–1,000
Low-Strain Elastic Modulus, E (ksi) [e]	30–170	260–700	100–500	400–2,500
Shear Modulus Reduction, G/G_{max}	Same as alluvium	Figure 6-18	BSC (2008a), Section 6.4.4	BSC (2008a), Section 6.4.4
Material Damping Ratio, D%	Same as alluvium	Figure 6-19		
Resistivity (ohm-m)	To Be Determined	To Be Determined	To Be Determined	Not Applicable
CBR[e]	20-60	Not Applicable	20–60	Not Applicable
Soil Profile Type (ICC 2000)	S_D (stiff soil)	S_C (very dense soil and soft rock) to S_B (rock)	S_C (very dense soil and soft rock)	S_B (rock) to S_A (hard rock)

[a] see applicable sections in calculation or appendices for basis of parameters
[b] additional testing required for verification (see Section 7.3.12)
[c] additional testing required (see Section 7.3.4)
[d] determined using V_S , υ and elastic theory
[e] determined using V_S, γ, υ and elastic theory
[f] BSC 2008a, Figure 6.2-9

Table 2-2. Summary of Recommended Surface Facilities Foundation Design Parameters

Design Parameter	Results / Recommendations	Section
Soil Material Properties	Table 2-1	7.1.2
Foundation Pressure	Settlement controls design Square and Strip footings: Figure 7-2 and Figure 7-3 Mat Foundations: Maximum Allowable Soil Bearing = 50 ksf for extreme loads = 10 ksf for normal loads	7.1.2
Estimated Settlements	<u>Square and strip footings</u> Figure 7-4 through Figure 7-6 <u>Mat foundation (400’ × 300’)</u> Load (ksf) Center (in) Corner (in) Differential (in) 3 0.2–0.4 negligible 0.4 5 0.5–1.6 < 0.1 1.5 7 1.3–2.9 < 0.1 2.9	7.1.3 App. B. Table B7-1
Lateral Pressures	<u>Yielding walls</u> Static and seismic pressures: Figure 7-7 Surcharge loads: Figure 7-8 and Figure 7-9 <u>Non-yielding walls</u> Static and seismic pressures: Figure 7-10 Compactor-induced pressures: Figure 7-11 thru Figure 7-15	7.1.5
Lateral Load Resistance	Ultimate Friction Coefficient, tan ϕ for alluvium: 0.81 for engineered fill: 0.90 Passive resistance: 515 pcf equivalent fluid	7.1.6
Temporary Shoring	For braced excavation Lateral pressure: 17H psf	7.1.5.1
Temporary Slopes	1.5H:1V	7.2.5
Permanent Slopes	2H:1V	7.2.5
Modulus of Subgrade **Reaction (these static loading** ranges may be doubled for short-term loading)	<u>Alluvium</u> / <u>Engineered Fill</u> Horizontal: 104-120 kcf (60-70 pci) / 60-96 kcf (35-55 pci) Vertical: 1ft ×1ft footing 1000 kcf (580 pci) / 600 kcf (350 pci) Large mats 155-520 kcf (90-300 pci) / 75-250 kcf (45-145 pci)	7.1.4
Saturated Permeability	5×10^{-5} to 5×10^{-4} fpm	7.1.9
Percolation Rate	1.8 in/hr	7.1.9
Depth of Frost Penetration	10 inches: see Figure 7-16	7.1.11
Minimum Footing Depth	2 feet	7.1.11

3 ASSUMPTIONS

This calculation is a compilation of available geotechnical information for use in design of surface facilities. It is written to adopt, clarify, and summarize findings and recommendations of BSC (2002a) and BSC (2002b) into design charts and tables. The same assumptions as listed in Section 5 of BSC (2002b) have been used in this calculation. There were no assumptions requiring verification in BSC, 2002b.

3.1 ASSUMPTIONS REQUIRING VERIFICATION

There are no assumptions used in this calculation requiring verification.

3.2 ASSUMPTIONS NOT REQUIRING VERIFICATION

Appendices B (Section B5) and C (Section C5) include additional assumptions not requiring verification specific to their subject matter. There are no additional assumptions (other than those listed in BSC, 2002b) used in this calculation.

4 METHODOLOGY

4.1 QUALITY ASSURANCE

This calculation was prepared in accordance with procedure EG-PRO-3DP-G04B-00037, *Calculation and Analyses*. *The Basis of Design for the TAD Canister-Based Repository Design Concept* (BSC 2008b) classifies the nuclear waste handling surface facilities as Important to Safety. Hence, the approved version of this document is designed as QA:QA.

4.2 USE OF SOFTWARE

Excel 2003 and Word 2003, which are part of this Microsoft Office 2003 suite of programs, were used in this report. Microsoft Office 2003 as used in this calculation is classified as Level 2 software usage as defined in IT-PRO-0011, *Software Management* and is listed on ORD (2007), *Repository Project Management Automation Plan*.

Mathcad, Version 13 was utilized in this calculation. Mathcad was operated on a PC system running the Window 2003 operating system. Mathcad as used in this calculation is considered as Level 2 software usage as defined in IT-PRO-0011, *Software Management*. Mathcad, Version 13 is listed on the ORD (2007), *Repository Project Management Automation Plan*.

4.3 CALCULATION APPROACH

This calculation reviews existing analyses, reports, drawings, and other documents to determine relevant aspects that have the potential to contribute to and enhance the evaluation of soil materials present at the site. Analytical methods of relevant engineering concepts with arithmetic computation and logic are used.

4.4 DESIGN CRITERIA

The criteria itemized in Section 4.2 of BSC (2002b) are, in general, applicable for this calculation. The current project design criteria are contained in BSC (2007a).

5 LIST OF ATTACHMENTS

5.1 APPENDICES

Analyses performed for use in the study herein are documented in the following attached appendices:

Appendix A: Not Used
Appendix B: Bearing Capacity and Settlement
Appendix C: Lateral Earth Pressures and Resistance to Lateral Loads

6 BODY OF CALCULATION

6.1 SITE DESCRIPTION

6.1.1 Location

The YMP site is situated in the southwestern part of U.S. Department of Energy (DOE) Nevada Test Site (NTS), and on parts of adjacent Nellis Air Force Range and U.S. Bureau of Land Management (BLM) lands (See Section 1.2.2 of CRWMS M&O 1999). The site of the potential surface facilities is totally within Area 25 of the NTS. The surface facilities site extends east from the North Portal Pad, which is the fill pad that was constructed for the Exploratory Studies Facility (ESF). A small portion of the site in the northwest corner lies within undocumented fill. The site is approximately 27 miles west-northwest of Mercury, Nevada (Figure 6-1) and is located in Nye County, Nevada approximately 100 miles northwest of the city of Las Vegas.

The approximate northing and easting coordinate ranges of the proposed site are N760,000 to N772,000 and E570,000 to E574,000, respectively (Nevada State Plane). The latitudinal and longitudinal coordinates are 36° 50’ and 116° 26.5’, respectively.

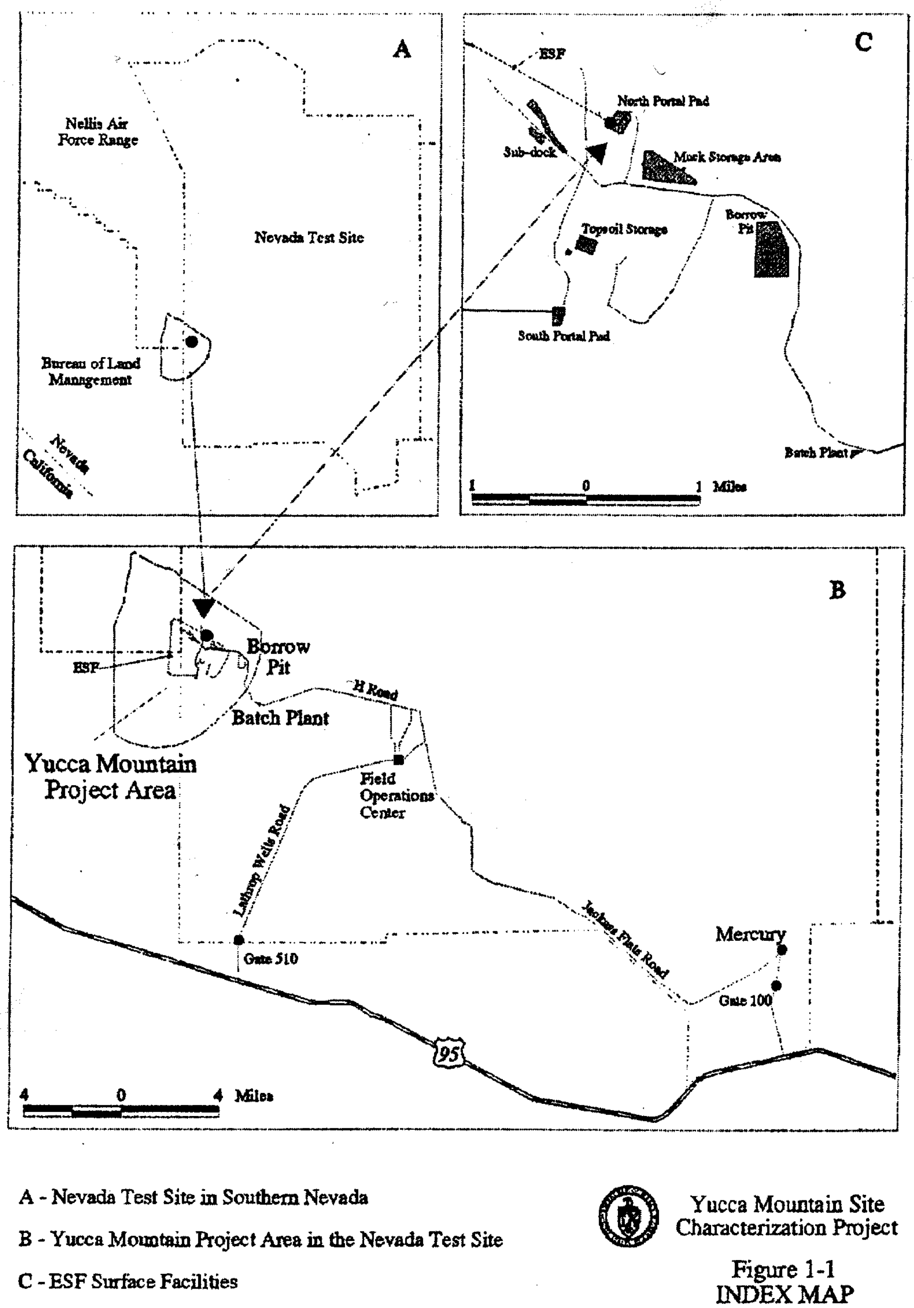


Figure 6-1. Site Vicinity Map (Figure 1-1 from CRWMS M&O 1999).

6.1.2 Summary of Site Geology

The surface facilities site lies on the western edge of the central portion of the Midway Valley at the eastern toe of Exile Hill. Yucca Mountain lies about 2 miles west of the surface facilities site. Figure 6-2 shows the general geologic features in the vicinity of the site, with the surface facilities area indicated near the center of this figure.

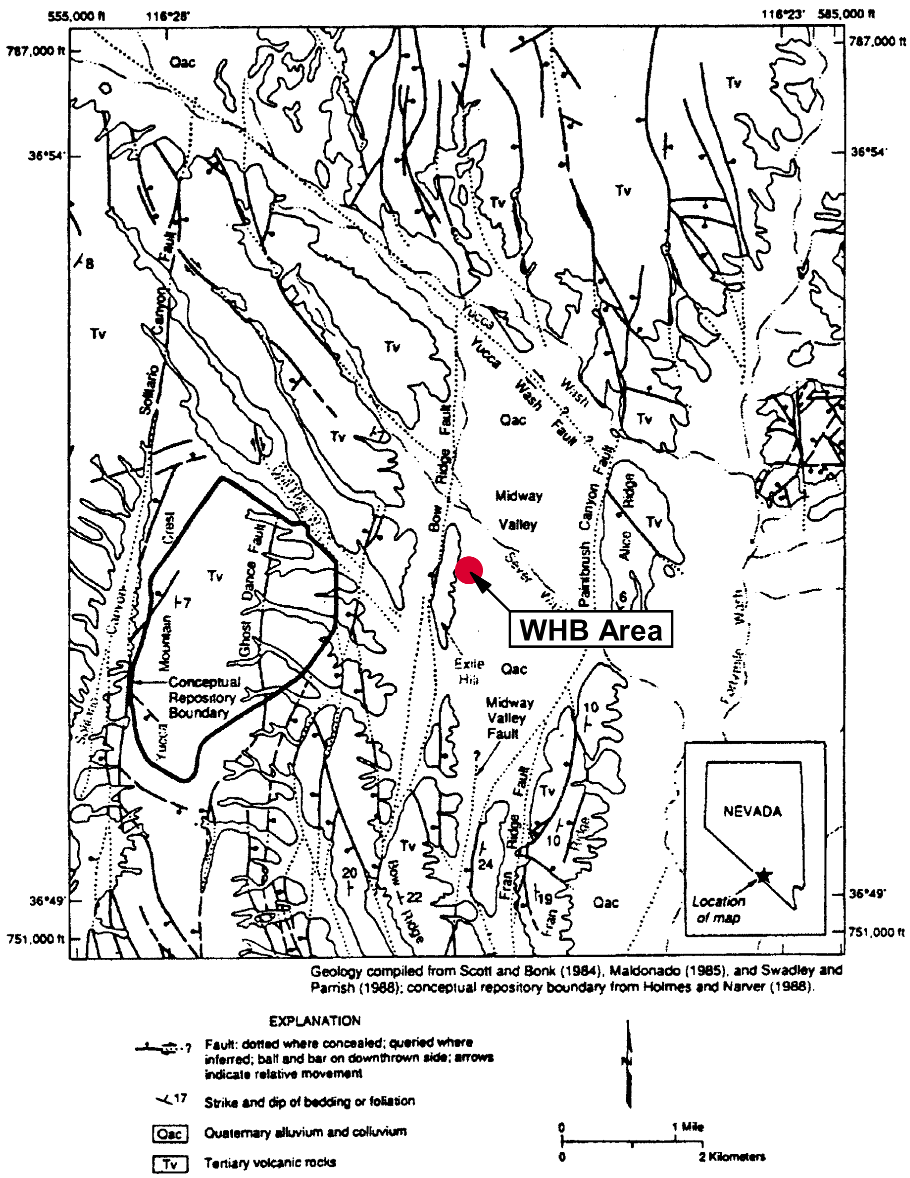


Figure 6-2. Generalized Map of the Midway Valley area. (Fig. 1-1 from Gibson et al. 1992).

The generalized geology of the site consists of alluvial and colluvial deposits overlying tuff bedrock. Volcanic rocks of Miocene age dominate the area. Small, intermittent flood-type drainage deposits cross the site area from west to east. The alluvial and colluvial deposits, which originated from Yucca Mountain on the west, vary from about 60 to 190 feet thick under the current building layout and deepen to several hundred feet in the center of the Midway Valley. Thorough descriptions of the geologic settings in the area can be found in Section 2 of CRWMS M&O (1999) and Section 6.6 of BSC (2002a) and their corresponding references.

6.1.3 Existing Conditions and Surface Features

The existing surface conditions and features are succinctly summarized in the following paragraphs, which were excerpted from Section 1.2.2 of CRWMS M&O (1999):

> The ground surface elevation in the vicinity of the WHB [surface facilities] site ranges from about 3,000 feet in the lower reaches of Forty Mile Wash, southeast of the site, to over 6,000 feet in the closer areas of Timber Mountain Caldera, about 4 miles to the north.
>
> The crest of Yucca Mountain averages roughly 4,900 feet in elevation. Relief near the site of the WHB [surface facilities] site is approximately 250 feet, from roughly 3,850 feet elevation at the crest of Exile Hill, immediately west of the site, to roughly 3,600 feet elevation at the center of Midway Valley, east of the site.
>
> The North Portal Pad is located along the western margin of Midway Valley, at the eastern base of Exile Hill. It is an area of approximately 800 to 1,200 feet by 600 to 700 feet of man-made fill sloping roughly 2 degrees to the east, and is situated at approximately 3,670 to 3,683 feet elevation. Muck piles along the eastern side of the North Portal Pad rise to approximately 3,700 feet elevation. The eastern part of the surface facilities footprint is in the area of the present muck piles.
>
> Beneath fill placed for the North Portal Pad is a variable thickness of colluvial and alluvial material overlying Tertiary volcanic bedrock units. The North Portal Pad is the surface at which the ESF tunnel portal was constructed. The pad supports the muck-handling facilities for the tunnel excavation, as well as offices, shops and rail equipment supporting the boring of the ESF tunnel, and facilities for engineering and scientific testing in the ESF.

6.1.4 Subsurface Conditions

This section provides a general description of some of the subsurface conditions at the surface facilities area. The descriptions of the subsurface conditions are based on information obtained from existing boreholes in the area. Refer to BSC (2002a) Section 6.6.2 and BSC (2002b) Section 6 for more detail. Figure 6-4 and Figure 6-5 show existing geologic cross-sections near the site. The cross-sections are taken from Figures 225, cross-section A-A’ and 226, cross-section B-B’ of BSC (2002a) and span in the NW-SE and NE-SW directions, respectively. Figure 6-7 shows the cross-section and pre-2005 boring locations. Although these cross-sections do not span through the area of the current layout of the proposed facilities, they present a general summary of the expected subsurface conditions. It should be noted that these cross-sections were based on data tracking numbers GS020383114233.003 and MO0008GSC00286.000. GS020383114233.003 is historical information (see Section 2.5.2), and has been superseded by GS030783114233.001 to account for bedrock depth corrections. The revisions in GS030783114233.001 are not reflected in Figure 6-4 and Figure 6-5. However, the differences are relatively minor and will not affect the recommendations of this calculation. A sketch of the stratigraphy beneath a typical surface facility is shown in Figure 6-6.

6.1.4.1 Existing Fill

Non-engineered fill, varying in thickness from 5 to 50 feet (refer to DTNs - GS030783114233.001, GS070683114233.005, MO0707RFGNPMV1.000, and MO0708SMFGLGIB.000, and Figure 6-10 for fill contact depths), covers the surface of the western edge of the proposed structures at the site. The existing fill is planned to be removed prior to the construction of the surface facilities (see BSC 2002b, Section 6.1) and be replaced by an engineered fill. Section 3.7 of CRWMS M&O (1999) provides more information about the existing fill. It is understood that the fill consists of tunnel muck material from the Exile Hill, and from borrow areas of Fran Ridge and Forty-mile Wash. Note that Section 5, Assumption 10 of BSC (2002b) states that 28 feet of existing fill was initially logged in one of the borings at the surface facilities area (UE-25 RF#20) and may have been misidentified during field exploration. For that location, the existing fill may, instead, have been only 9 feet thick.

6.1.4.2 Alluvium

Beneath the existing fill there is a layer of alluvial material, consisting of interbedded calcite-cemented (caliche) and non-cemented poorly sorted, coarse-grained gravel with sand and some fines, cobbles, and boulders (refer to Tables 4 and 5 of BSC 2002a and Table 6-1, for alluvium contact depths). Available information indicates that the thickness of the alluvium is likely to vary considerably at some locations due to irregular erosion of the current surface and the depth to bedrock. Furthermore, cemented and un-cemented soil layers appear randomly within this soil unit. The alluvium generally ranges in thickness from 2 ft to 192 ft over the site (see Figure 6.2-4 of SNL 2008). Under the major building footprints, the alluvium thickness ranges from about 57 feet to 192 feet with the thickness increasing eastward from Exile Hill (see Figure 6-9). Note that Section 5, Assumption 9 of BSC (2002b) states that alluvium logged in borehole UE-25 RF#21 between 70 and 115 feet may have been misidentified and may, in fact, be bedrock.

6.1.4.3 Bedrock

As Section 6.3 of BSC (2002b) asserts, there are non-welded and welded tuffs from the units of Timber Mountain and Paintbrush groups underlying the surface deposits of fill and alluvium.

The non-welded units include the following:

- Pre-Rainier Mesa Tuff bedded tuffs (Tmbt1) of the Timber Mountain Group

- Tuff unit “x” (Tpki) of the Paintbrush Group
- Pre-Tuff unit “x” bedded tuffs (Tpbt5) of the Paintbrush Group

Beneath the non-welded units is the densely welded Tiva Canyon Tuff consisting of the following:

- Younger crystal-rich member (Tpcr)
- Older crystal-poor member (Tpcp)

Both of the Tiva Canyon Tuff members are further divided into zones. Refer to Section 6.6.2 and Attachments I and II of BSC (2002a) for a detailed geologic description of the bedrock. Figure 6-3 shows elevation contours for the top-of-bedrock (Figure 232 of BSC 2002a).

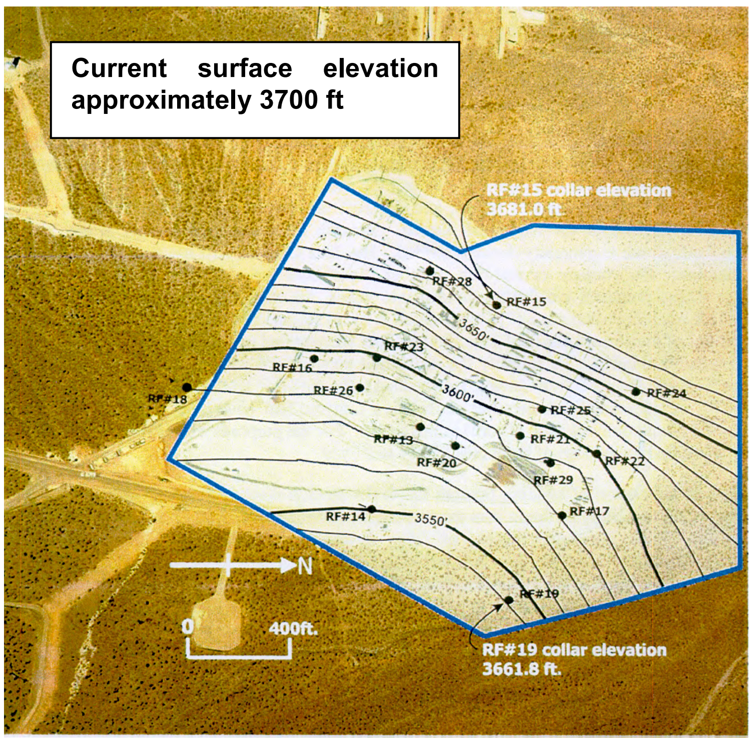


Figure 6-3. Elevation Contours for Top-of-Bedrock Encountered in Boreholes (Figure 232 of BSC 2002a)

6.1.4.4 Groundwater

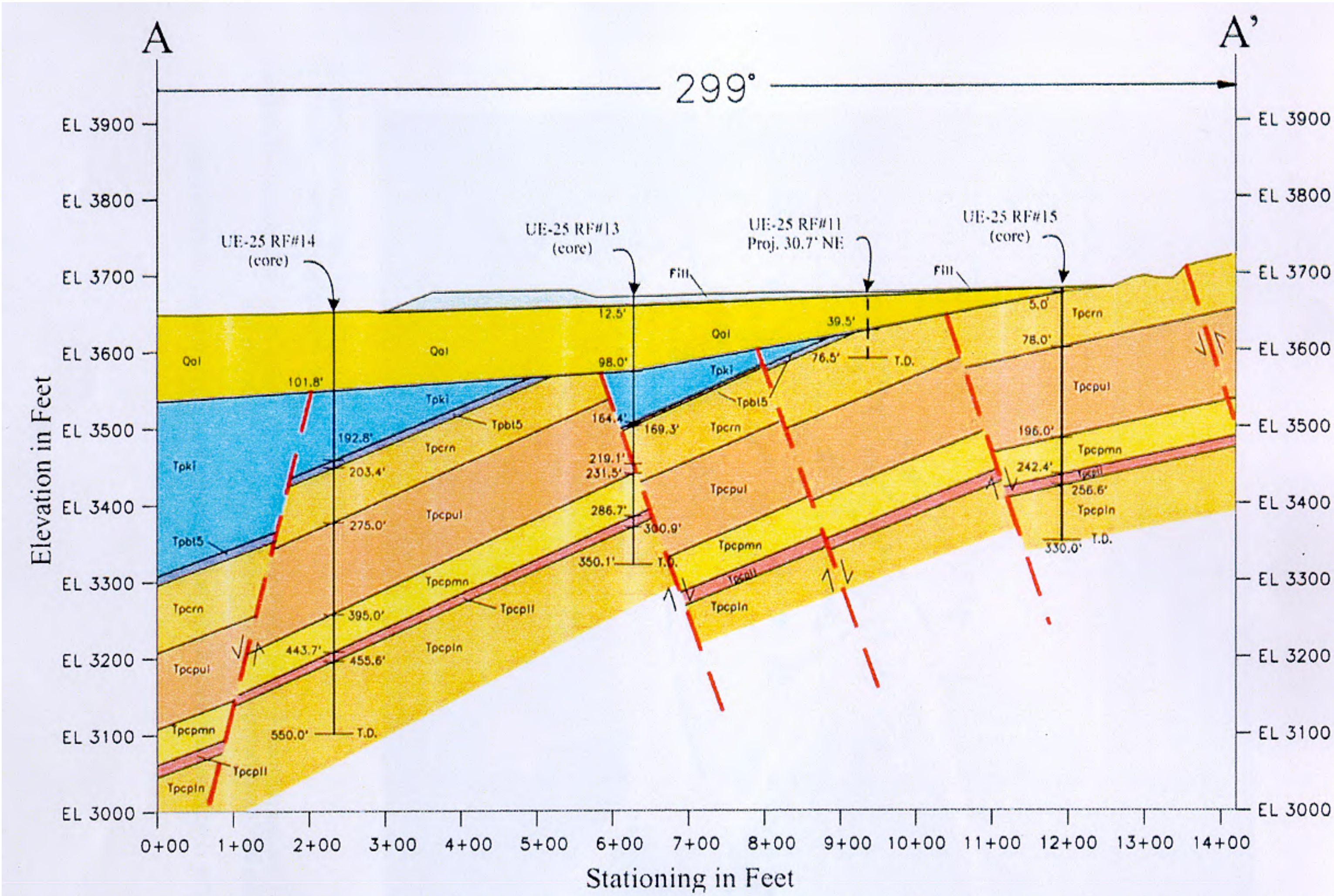
Groundwater data relevant to the area is summarized in Section 6.6.3 of BSC (2002a). The groundwater table is located at a typical depth of 1,270 feet below the present ground surface, and is over 1,000 feet below the top of bedrock in the North Portal Area. Hence, groundwater does not affect the geotechnical calculations presented in this study.

6.1.4.5 Proposed Engineered Fill

It is assumed (BSC 2002b, Section 5, Assumption 1) that the existing fill will be removed and that the surface facilities will be founded on the alluvium soil or engineered fill. Any required engineered fill will likely be obtained from the alluvial sand and gravel deposits at the Fran Ridge Borrow Area, which is located approximately 1.5 miles southeast of the surface facilities. However, due to the large design lateral and vertical accelerations, alternative measures are being considered to anchor the structures to the ground by a more positive means, such as roller-compacted soil cement (RCSC). Section 6.1.4.6 below discusses estimated properties of RCSC for design evaluation purposes.

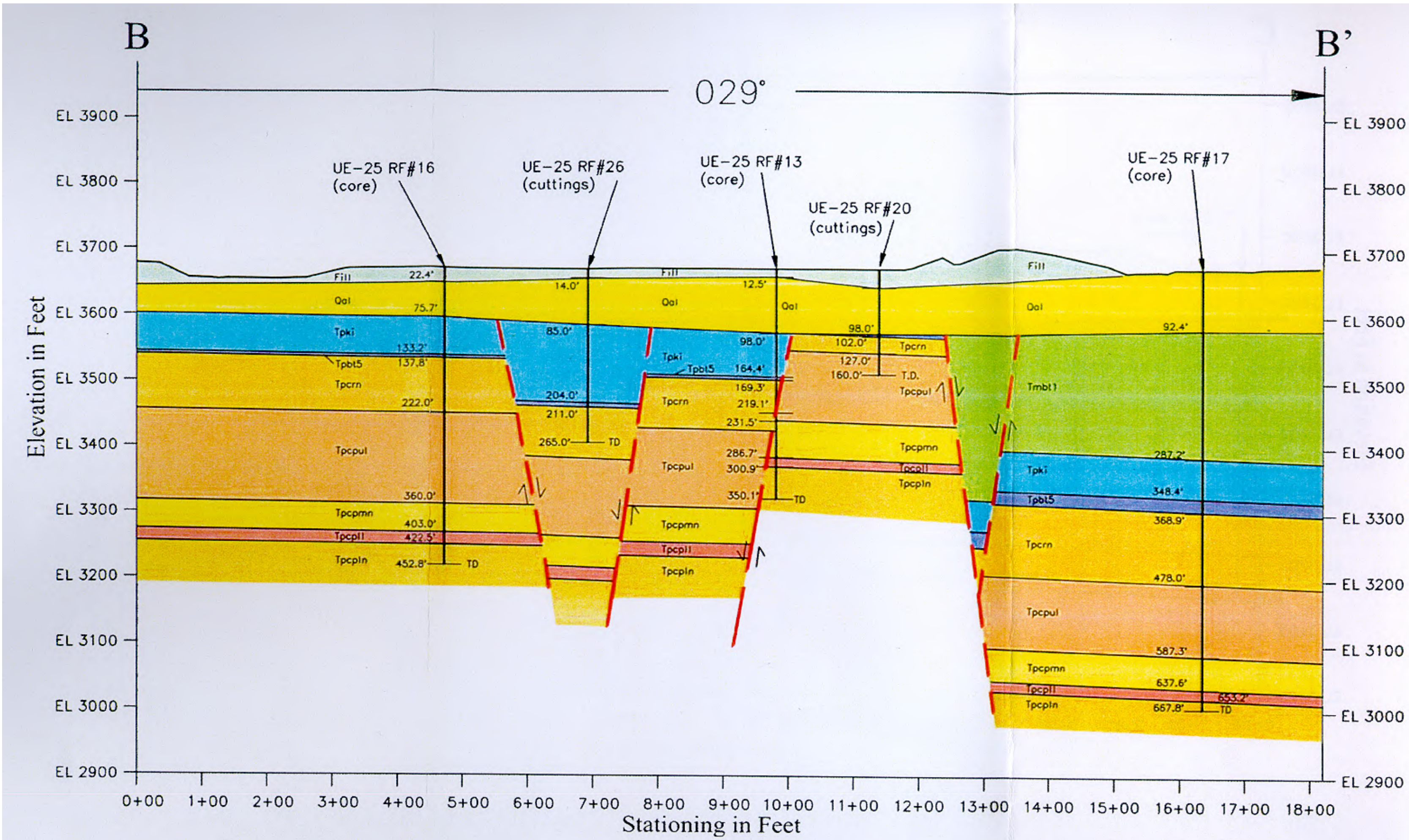
6.1.4.6 Roller-Compacted Soil Cement

A literature review was performed to define typical soil properties for use in evaluating potential benefits of using roller-compacted soil cement to replace the tunnel muck that currently underlies the surface facilities site (see BSC 2004b). Papers regarding properties of roller-compacted concrete as well as deep soil mix technologies were reviewed. It is anticipated that a soil-concrete mixture could provide the desired soil response properties for seismic design of the structures and simultaneously provide a high quality control in the field. The report resulting from the literature review is provided in BSC (2004b).



Sources: Figure 225 of BSC 2002a and Assumption 6 of BSC 2002a, DTN:MO0008GSC00286.000

Figure 6-4. Surface Facilities Area Geologic Cross Section A-A' (see Figure 6-7 for location of cross-section).



Sources: Figure 226 of BSC 2002a, DTN:MO0008GSC00286.000

Figure 6-5. Surface Facilities Area Geologic Cross Section B-B' (see Figure 6-7 for location of cross-section)

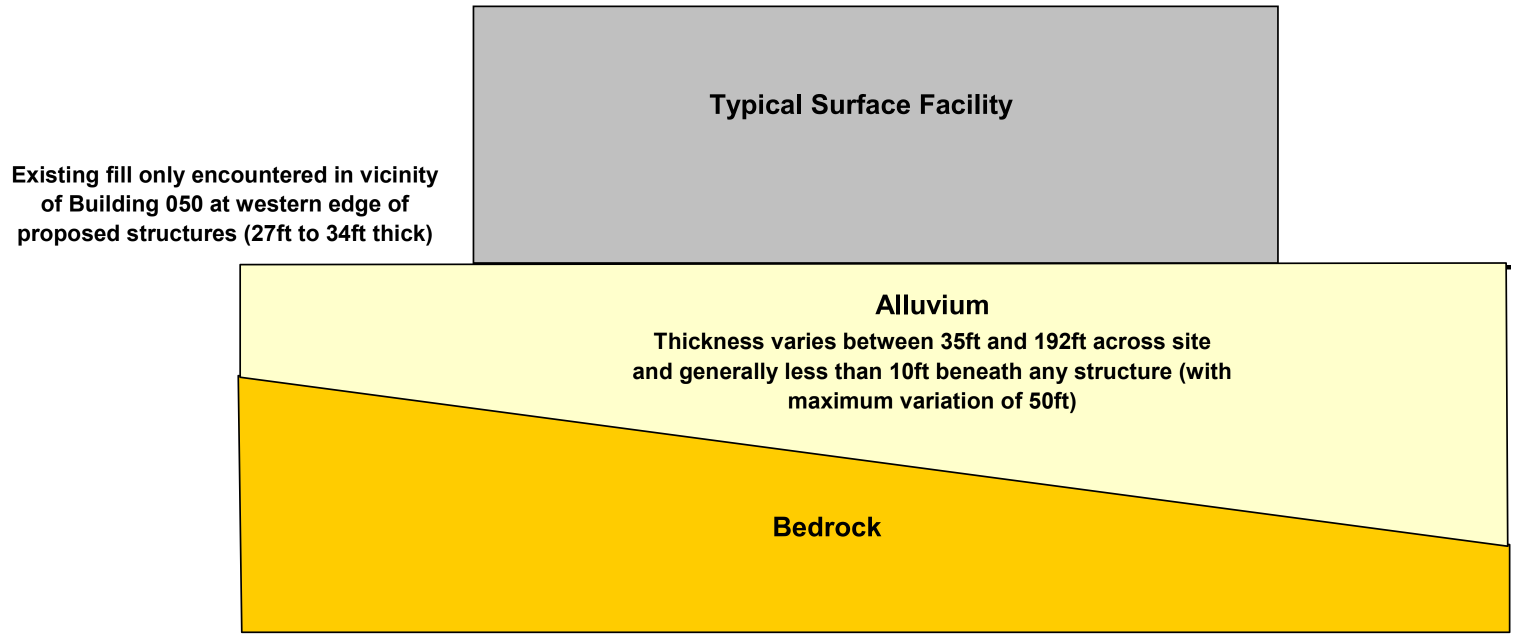


Figure 6-6. Sketch of Stratigraphy Underlying Typical Surface Facility (not to scale).

6.2 FIELD EXPLORATION AND TESTING

The following sections summarize the soil investigations and field tests performed in the surface facilities area. Soil investigations in the surface facilities area have been conducted since the mid-1980s. Data obtained at the site (as presented in the BSC 2002a and BSC 2002b references) is primarily relied upon as the direct input for the analyses contained in this report. The subsurface investigations for BSC (2002a) and BSC (2002b) were performed in 2000 and 2001. Additional investigations were performed in 2005 and 2006 (SNL 2008) that included borings and test pits with associated field and laboratory testing, and SASW measurements. These latter investigations were performed to augment and confirm the previously existing information and to investigate subsurface conditions in new areas resulting from building layout reconfigurations. Other data acquired from explorations prior to those covered in the BSC (2002a) and BSC (2002b) investigations are used as corroborative information.

6.2.1 Field Exploration

6.2.1.1 Borings

Within the surface facilities area, 15 total boreholes (UE-25 RF#14 to RF#26, RF#28, and RF#29) were drilled in 2000 using core hole and mud rotary drilling techniques. Depths of the borings ranged from approximately 100 to 670 feet below top of bedrock (Table 4, BSC 2002a). A previous boring (UE-25 RF#13) was cored in 1998 to a depth of approximately 350 feet (Table 5, BSC 2002a).

Nineteen additional shallow borings were drilled in 2005 using the sonic coring technique to depths varying between 104 feet and 417 feet, primarily to determine the depth of alluvium in the surface facilities area and to augment the geologic understanding of the area. In addition, a series of laboratory gradation tests were performed on selected samples of the alluvial material obtained from two of the sonic cores. This information is provided in SNL (2008). SNL (2008) also includes the depth of alluvium from twenty three additional boreholes drilled in 2006 and the early part of 2007.

A boring designated as NRG#1 was drilled at the top of the nearby Exile Hill in 1992 (McKeown 1992). Studies performed between 1984 and 1985 (Neal 1985, and Neal 1986) produced 4 borings located within the surface facilities area (UE-25 RF#3, RF#3b, RF#9, and RF#11).

The coordinates of all borings drilled in the surface facilities area are provided in Table 6-1. Their locations are shown in Figure 1-1. Figure 6-8 shows nearby boring locations, including those drilled in 2005 and in 2006-2007, with respect to the planned building footprints. Note that the full depth of some of the borings drilled in the 2006-2007 timeframe are not shown, as only the upper portion of the boring logs was fully reviewed and qualified for use in this report. However, the omission of the data from the lower portions of the boring logs has no effect on the results, conclusions or recommendations of this report. As seen in the figure, numerous borings were drilled northwest of the most recent proposed building layout due to changes in the building arrangement during the course of the investigative studies. The depth of rock encountered in each of the borings is indicated in Figure 6-9. In general, the rock depth increases from about 35 ft at the southwest end of the building area to as much as 192 ft at the northeast end. The depth of fill encountered at each of the boring locations is depicted in Figure 6-10.

6.2.1.2 Test Pits and Trenches

Previous investigations in the surface facilities area during the 1980s and 1990s included numerous excavations of shallow test pits (designated as NNWSI, SFS, NRSF, GSF and MWV-P) and trenches (MWV-T). Documentation of these test pits is provided in Holmes & Narver 1983, Ho et al. 1986, McKeown 1992, and Map ID SA95-9-15 of DOE 1995.

Investigations performed from 2000 to 2001 included four test pits (TP-WHB-1 to –4) excavated in the surface facilities area. The test pits were each excavated to a depth of approximately 20 feet into the alluvial material. No fill was encountered in these test pits. A total of 22 samples of the alluvium were obtained from the four test pits for laboratory testing. The locations of these 4 test pits are shown in Figure 1 in BSC (2002b).

In 2006 three additional test pits (TP-WHB-5, TP-WHB-6, and TP-WHB-7) were excavated in the surface facilities area to observe subsurface conditions, to perform in situ density tests, and to obtain both disturbed and undisturbed samples for laboratory testing. Each excavation extended to about a 20-foot depth entirely within alluvium material well above bedrock. The 2006 test pit locations are shown in Figure 1-1.

All test pit locations excavated within the site vicinity are shown in Figure 6-11 in relation to the planned building footprints. Table 6-2 provides a summary of all known test pits and trenches excavated in the surface facilities area.

Four disturbed samples of material to be potentially used as engineered fill were obtained from the existing borrow area (Fran Ridge Borrow Area) at widely spaced locations. The location of Fran Ridge is shown in Figure 6-12. Figure 6-12 (taken from Figure 213 of BSC 2002a) also shows the sampling locations taken from the Fran Ridge Borrow Area. These samples were combined into a composite sample and taken to offsite laboratory facilities for testing.

Table 6-1. Boring Information in Surface Facilities Area.

Date	Identification	Coordinates (Nevada State Plane), ft		Surface Elevation (ft)	Total Depth (ft)	Fill Thickness (feet)	Depth to Rock (ft)	Source
		Northing	Easting					
March 1984 - July 1985	UE-25 RF#3	765,575	571,100	3,657.7	301		91	Neal (1985) & Neal (1986)
	UE-25 RF#3B	765,695	571,066		111			
	UE-25 RF#9	765,945	570,643		105		105	
	UE-25 RF#11	765,622	570,435		77		39.5	
November 1992	UE-25 NRG#1	765,359	569,803		150.1			McKeown (1992)
October 1998	UE-25 RF#13	765,500	570,720	3,671.1	350.1	12.5	98	BSC (2002a) DTN:GS030783114233.001
June - November 2000	UE-25 RF#14	765,309	571,066	3,651.4	550		101.8	
	UE-25 RF#15	765,774	570,225	3,680.8	330	5.0	6.5	
	UE-25 RF#16	765,056	570,473	3,672.0	452.8	22.4	75.7	
	UE-25 RF#17	766,076	571,042	3,673.4	667.8		96.1	
	UE-25 RF#18	764,522	570,627	3,640.3	493.6		65	
	UE-25 RF#19	765,880	571,384	3,661.6	645.2		120	
	UE-25 RF#20	765,637	570,797	3,671.1	160	28	98	
	UE-25 RF#21	765,899	570,739	3,672.9	192.2		115	
	UE-25 RF#22	766,206	570,793	3,680.3	540.6		80	
	UE-25 RF#23	765,311	570,465	3,673.9	159.1	12	76	
	UE-25 RF#24	766,345	570,543	3,685.8	268	10	30	
	UE-25 RF#25	765,968	570,627	3,676.4	159	10	70	
	UE-25 RF#26	765,248	570,580	3,670.8	264.9	14	85	
	UE-25 RF#28	765,510	570,105	3,680.2	99.8	5	15	
	UE-25 RF#29	766,018	570,836	3,672.6	430		85	
April - June 2005	UE25 RF-42	764,633	571,142	3,634.9	118.9		84	DTN:GS070683114233.005
	UE25 RF-43	765,376	570,709	3,669.9	110.1	19.4	90.5	
	UE25 RF-44	765,419	570,829	3,676.3	143.5	26.8	108	
	UE25 RF-45	765,268	571,022	3,650.0	125.5		93	
	UE25 RF-46	764,890	570,603	3,669.2	103.5	27.2	84.2	
	UE25 RF-47	765,747	571,077	3,663.9	122.3		97	
	UE25 RF-48	765,474	571,387	3,653.6	159.3		113.3	
	UE25 RF-49	766,059	571,421	3,668.8	142.9		112.9	
	UE25 RF-50	765,785	571,698	3,656.3	155.5		123.2	
	UE25 RF-51	766,314	571,672	3,672.0	156.7		128.4	
	UE25 RF-52	766,557	571,915	3,672.4	184.7		164.7	
	UE25 RF-53	766,040	571,948	3,661.3	160.6		138	
	UE25 RF-54	766,279	572,190	3,661.6	196.7		183.1	
	UE25 RF-55	765,112	571,531	3,642.2	154.2		113	
	UE25 RF-56	765,439	571,857	3,646.8	416.9		129.7	
	UE25 RF-57	766,089	572,470	3,651.9			173.6	
	UE25 RF-58	763,061	571,073	3,667.7	150.7		134.2	
	UE25 RF-59	762,347	571,407	3,664.6	179		155.3	
	UE25 RF-60	761,667	571,809	3,650.1	195.6		144.5	

Note: RF–Repository Facility

Table 6 1 (cont'd). Boring Information in Surface Facilities Area

Date	Identification	Coordinates (Nevada State Plane), ft		Surface Elevation (ft)	Total Depth (ft)	Fill Thickness (feet)	Depth to Rock (ft)	Source
		Northing	Easting					
2006-2007	UE-25 RF#31	765,614	571,327	3,657.1			105.5	DTNs: GS030783114233.001, MO0707RFGNPMV1.000, MO0708SMFGLGIB.000
	UE-25 RF#33	763,730	570,460	3,671.3			87.6	
	UE-25 RF#34	764,942	570,753	3,684.1		50.4	115.4	
	UE-25 RF#35	767,763	573,480	3,693.8			110.7	
	UE-25 RF#36	766,480	572,155	3,664.6			171.8	
	UE-25 RF#37	765,562	571,996	3,647.6			130.1	
	UE-25 RF#38	766,760	571,874	3,673.5			148.7	
	UE-25 RF#39	765,095	571,264	3,644.6			100.4	
	UE-25 RF#41	766,715	572,950	3,666.1			192.4	
	UE-RF#64	767,880	568,919	3,787.6			69.5	
	UE-RF#75	771,417	570,954	3,851.4			60.4	
	UE-RF#76	771,732	570,564	3,870.9			132	
	UE-RF#78	770,082	570,895	3,806.0			135.5	
	UE-RF#79	770,399	570,480	3,818.2			132.3	
	UE-RF#80	769,769	570,480	3,796.1			127.9	
	UE-RF#83	766,679	573,151	3,662.8			142.2	
	UE-RF#95	768,844	571,573	3,753.0			182.1	
	UE-RF#97	767,184	570,596	3,711.0			79.1	
	UE-RF#104	763,719	570,163	3,682.6			50.3	
	UE-RF#105	763,877	570,121	3,679.5			35.4	
	UE-RF#106	763,725	570,246	3,677.6			62.6	
	UE-RF#107	763,608	570,272	3,681.0			72.2	
	UE-RF#112	765,284	570,872	3,688.8		34.8	121	

Note: RF–Repository Facility

Table 6-2. Test Pit and Trench Information in Surface Facilities Area.

Seq. No.	Date	Identification	Coordinates (Nevada State Plane), ft		Source
			Northing	Easting	
1	May 1983	NNWSI 2	764,850	570,941	Holmes & Narver (1983)
2	May 1984	SFS-3	764,850	570,941	Ho et al (1986)
3	Spring 1992	NRSF-TP-1	765,193	569,828	McKeown (1992) & DOE (1995)
4		NRSF-TP-2	765,313	569,892	
5		NRSF-TP-3	765,359	569,946	
6		NRSF-TP-4	765,383	569,998	
7		NRSF-TP-5	765,430	569,977	
8		NRSF-TP-6	765,510	570,002	
9		NRSF-TP-7	765,463	570,093	
10		NRSF-TP-8	765,506	570,101	
11		NRSF-TP-9	765,571	570,029	
12		NRSF-TP-10	765,669	570,015	
13		NRSF-TP-11	765,638	570,206	
14		NRSF-TP-12	765,641	570,035	
15		NRSF-TP-13	765,798	570,140	
16		NRSF-TP-14	765,700	570,244	
17		NRSF-TP-15	765,837	570,228	
18		NRSF-TP-16	765,790	570,344	
19		NRSF-TP-17	765,916	570,277	
20		NRSF-TP-18	765,860	570,382	
21		NRSF-TP-19	765,621	570,511	
22		NRSF-TP-20	765,541	570,436	
23		NRSF-TP-21	765,599	570,346	
24		NRSF-TP-22	765,521	570,313	
25		NRSF-TP-23	765,462	570,390	
26		NRSF-TP-24	765,218	570,255	
27		NRSF-TP-25	765,113	570,360	
28		NRSF-TP-26	765,016	570,036	
29		NRSF-TP-27	765,256	570,246	
30		NRSF-TP-27a	765,259	570,330	
31		NRSF-TP-28	765,093	570,256	
32		NRSF-TP-29	765,107	570,201	
33		NRSF-TP-30	765,127	570,156	
34		NRSF-TP-31	764,987	570,135	
35		NRSF-TP-32	765,084	569,969	

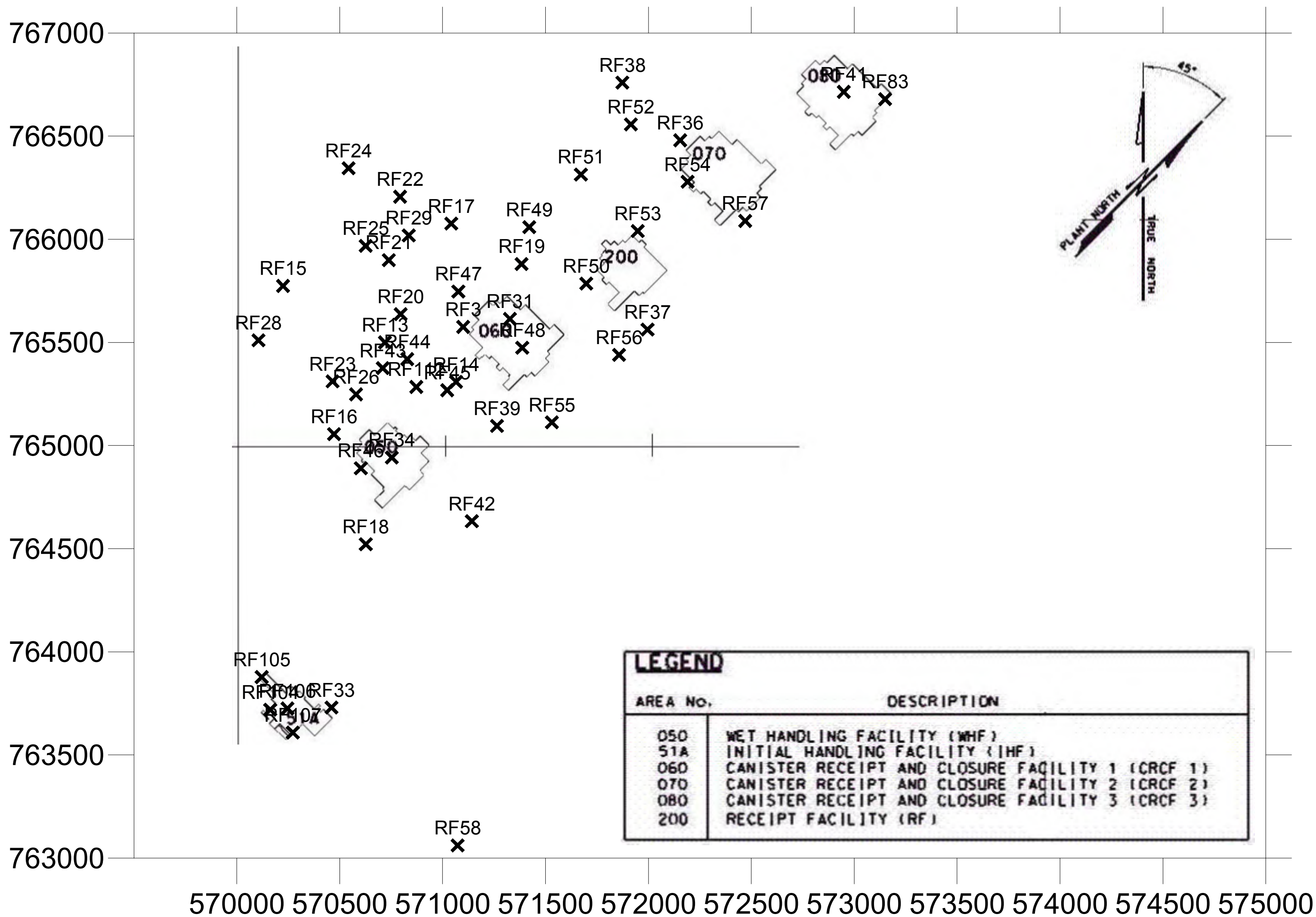
Notes:
1. NNWSI–Nevada Nuclear Waste Site Investigation
2. SFS–Surface Facility System
1. NRSF–North Ramp Surface Facilities
2. SFS-3 was deepened from pre-existing NNWSI 2

Table 6-2. Test Pit and Trench Information in Surface Facilities Area (continued)

Seq. No.	Date	Identification	Coordinates (Nevada State Plane), ft		Source
			Northing	Easting	
36	September 1992	GSF-TP-1	765,966	570,884	USBR (1992)
37		GSF-TP-2	765,539	571,110	
38		GSF-TP-3	765,040	571,110	
39		GSF-TP-4	764,519	571,040	
40		GSF-TP-5	764,000	570,935	
41	1992	MWV-P1	765,405	570,849	DOE (1995)
42		MWV-P2	765,259	571,652	
43		MWV-P3	764,148	570,845	
44		MWV-P9	762,931	572,751	
45		MWV-P28	765,178	571,005	
46		MWV-P29	765,147	570,387	
47		MWV-P30	765,149	570,599	
48		MWV-P31	765,150	570,717	
49		MWV-P32	765,189	571,029	
50		MWV-P32a	765,144	571,028	
51	1992	MWV-T5A	765,212	570,501	DOE (1995)
52		MWV-T6	765,173	569,987	
53		MWV-T7	765,482	570,059	
54	July 2000	TP-WHB-1	766,304	570,772	BSC (2002a)
55		TP-WHB-2	765,595	571,106	
56		TP-WHB-3	765,306	571,161	
57		TP-WHB-4	765,950	571,453	
58	August-September 2006	TP-WHB-5	766,398	571,766	DTN: GS070583114233.003
59		TP-WHB-6	766,696	572,372	
60		TP-WHB-7	767,137	572,812	

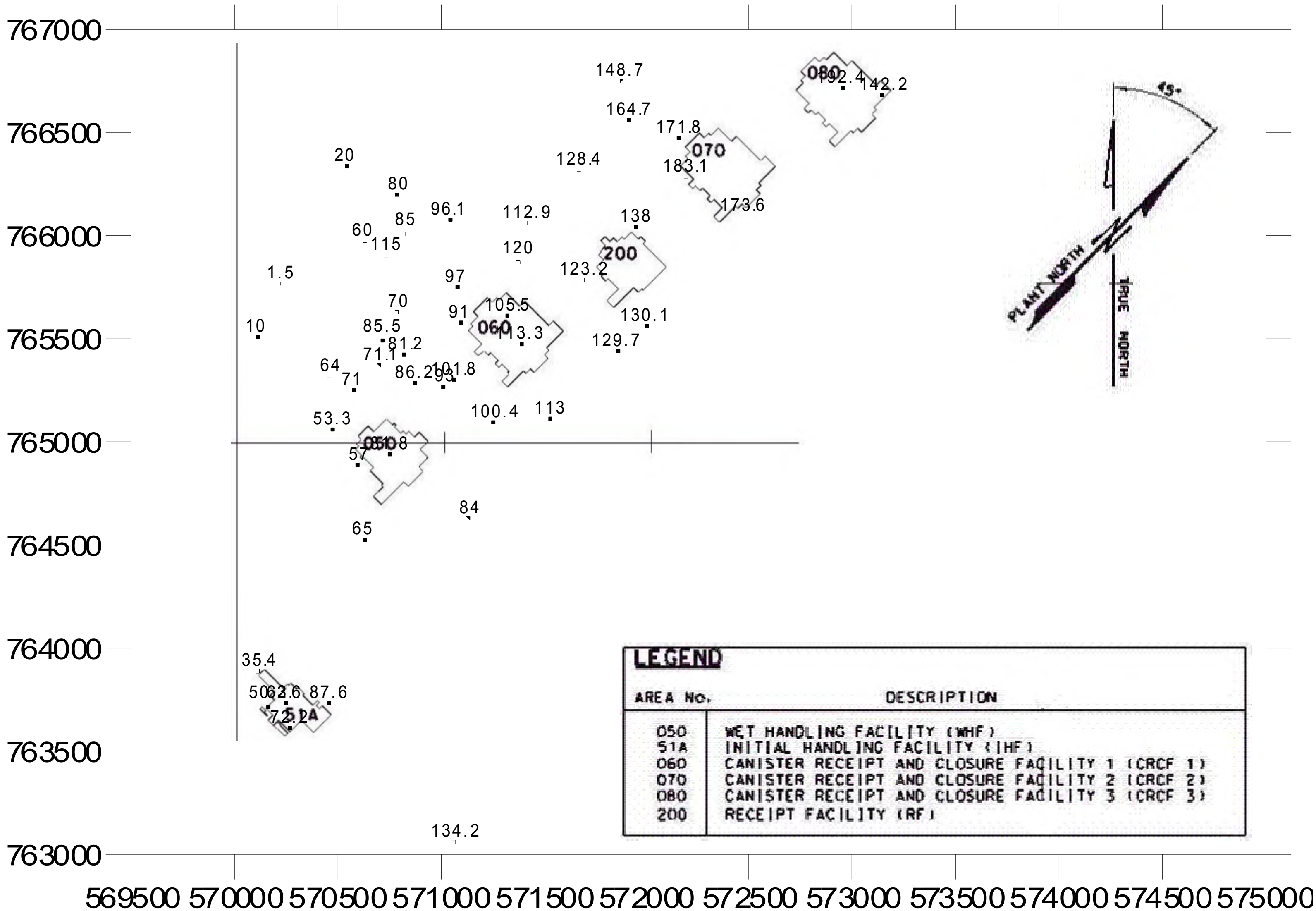
Notes:
1. GSF–Ground Surface Facility
2. MWV–Midway Valley
3. WHB–Waste Handling Building

Figure 6-7. Locations of Soil Exploration in the Surface Facilities Area (only pre-2005 borings shown). Cross-Sections shown in Figure 6-4, Figure 6-5, and Figure 7-1. Excerpted from Figure 224, BSC (2002a).



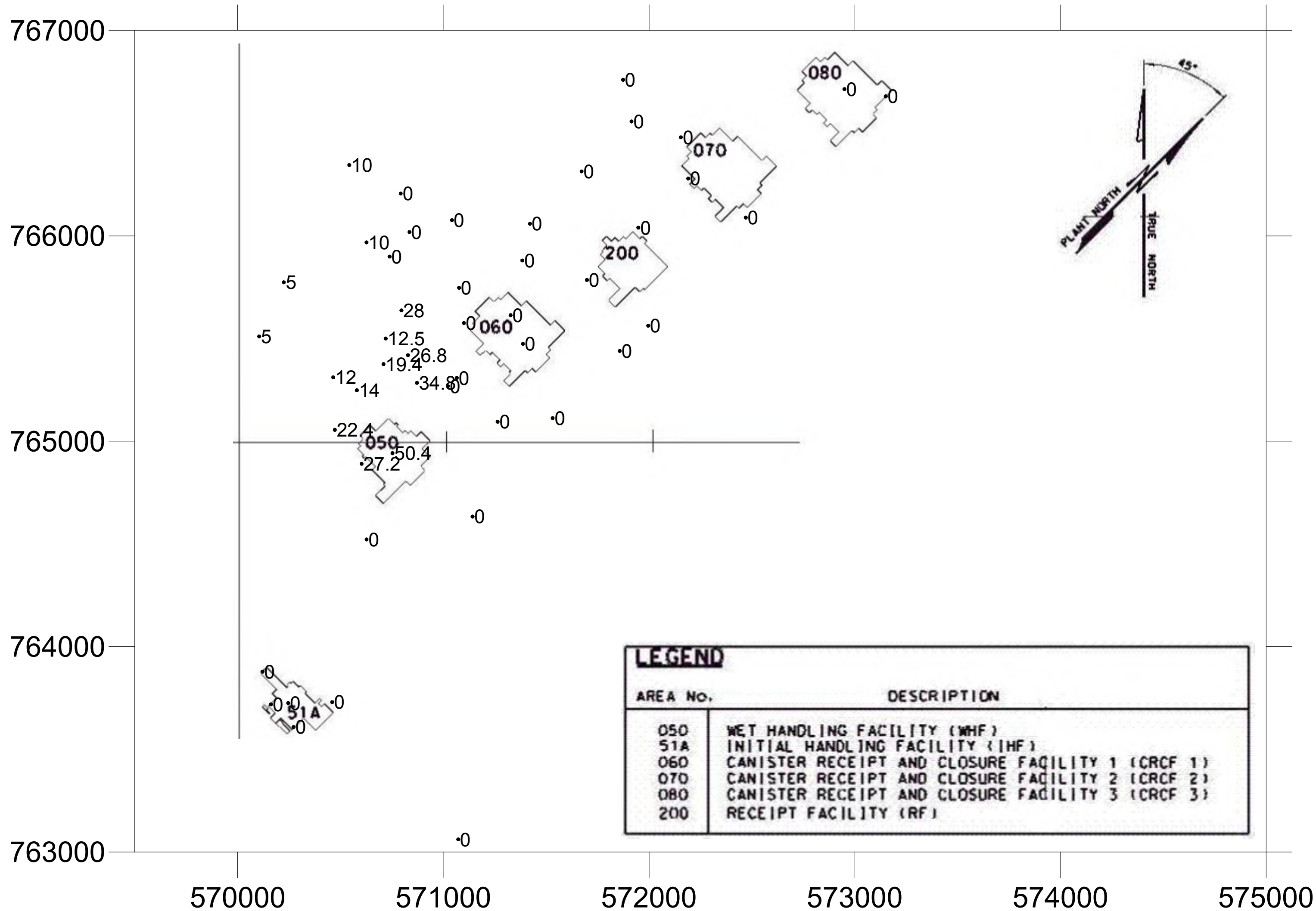
DTNs: GS030783114233.001, GS070583114233.002, GS070683114233.005, MO0707RFGNPMV1.000, MO0708SMFGLGIB.000

Figure 6-8. Locations of Borings in Surface Facilities Area with respect to Building Footprints.



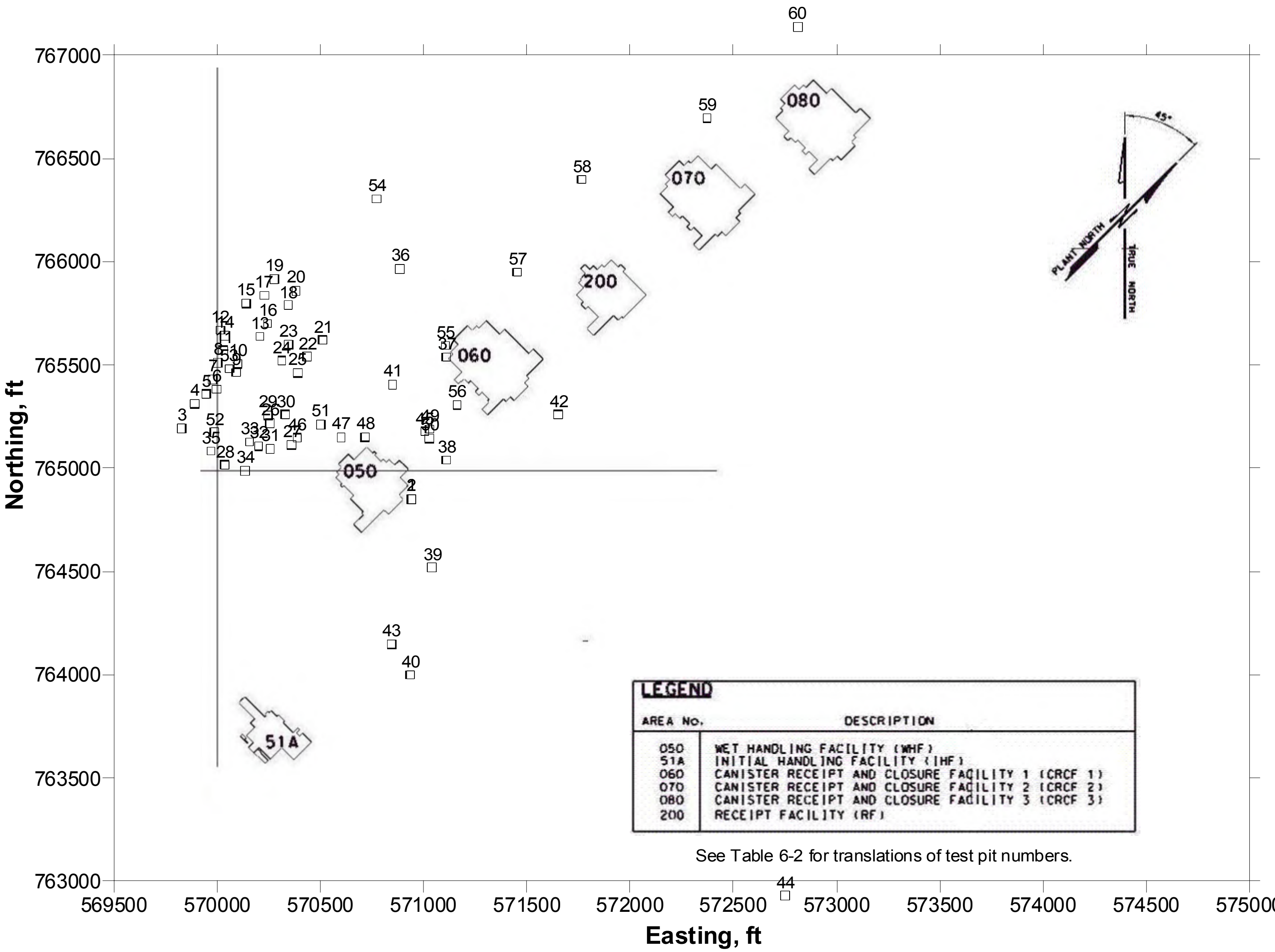
DTNs: GS030783114233.001, GS070583114233.002, GS070683114233.005, MO0707RFGNPMV1.000, MO0708SMFGLGIB.000

Figure 6-9. Depth to Rock in Building Area.



DTNs: GS030783114233.001, GS070583114233.002, GS070683114233.005, MO0707RFGNPMV1.000, MO0708SMFGLGIB.000

Figure 6-10. Depth of Fill Encountered in Building Area.



DTNs: GS020383114233.001, GS070583114233.003

Figure 6-11. Locations of Test Pits in Surface Facilities Area with respect to Building Footprints.

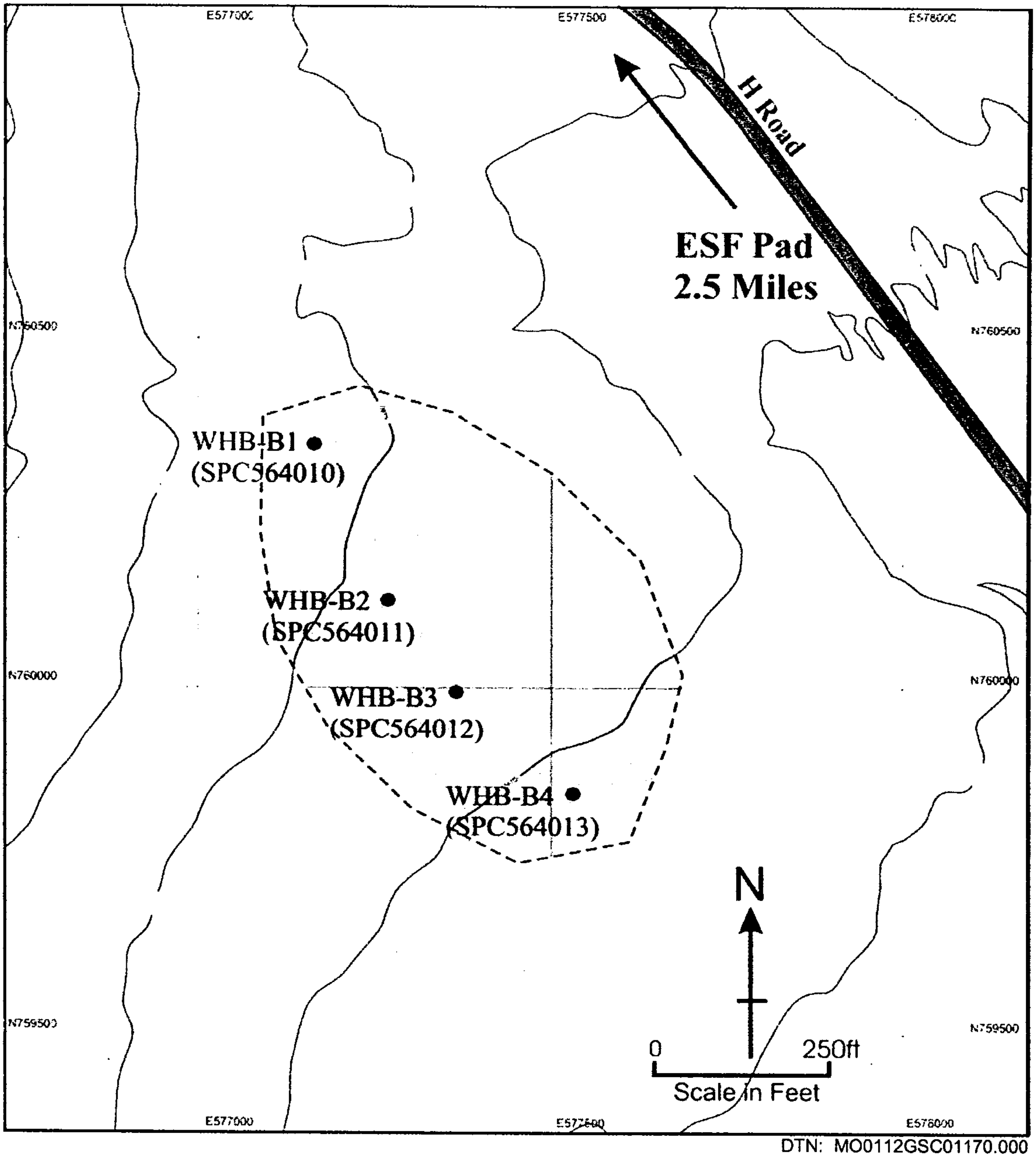


Figure 6-12. Location of Fran Ridge Borrow Pit #1 Samples.

6.2.2 Field Tests

6.2.2.1 In Situ Density Testing

Six 6-foot diameter ring density tests and sixteen 20-inch diameter sand cone density tests were performed on the alluvial material within test pit excavations in the Fran Ridge borrow area (TP-WHB-1 through TP-WHB-4) from depths of 4 to 20 feet. Caliper and gamma-gamma wireline surveys were also performed in some of the borings primarily to determine the density of the subsurface materials. This is discussed in Section 6.2.2.4. Table 6-3 lists the standards used for the testing.

Table 6-3. Test Standards Used for In Situ Density Testing.

Test	Standard
Ring density test	• USBR 7221-89, *Procedure for Determining Unit Weight of Soils In-Place by the Water Replacement Method in a Test Pit*
Sand cone density test	• USBR 7205-89, *Procedure for Determining Unit Weight of Soils In-Place by the Sand-Cone Method*
Gamma-gamma wireline survey	• PA-PRO-0312, Rev. 0, ICN 0, *The Preparation, Planning, and Field Verification of Surface-Based Geophysical Logging Operations* (this information is considered historical, see Section 2.5.1)

Nine 6-foot diameter ring density tests were also conducted in the 3 test pits performed in 2006. Results of the in situ density tests are shown in Table 6 of BSC (2002a) and in DTN:GS070683114233.004 and discussed in the material properties section (Section 6.4) of this report. The materials from these tests were sealed and taken to an offsite geotechnical laboratory for further soil property and classification testing (See Section 6.4).

In-place density tests were also conducted for materials from several test pits and borings performed in the mid-1980s to early 1990s (see Table 6-1 and Table 6-2). Methods used to measure the densities included water replacement tests (McKeown 1992), and sand cone and nuclear densometer tests (Ho, et al. 1986). Data from these tests are compiled and used as corroborative information in the analyses contained herein.

6.2.2.2 Standard Penetration Tests

Standard Penetration Test (SPT) blowcounts were obtained at 5-foot intervals up to 100 feet in depth in RF#13 using a Modified California (MC) sampler (140-pound hammer with a drop of 30 inches). A review of the literature also revealed that SPT blowcounts were performed in TP-NNWSI2 (up to 5 feet depth) in May 1983 (Holmes & Narver 1983).

6.2.2.3 Seismic Velocity Surveys at Surface Facilities Area

Several seismic velocity surveys were conducted in the surface facilities area in order to determine the dynamic characteristics of the subsurface materials. The following 3 methods were used:

1. Downhole (DH)
   - 22 total surveys extending down to approximately 640 feet in depth
2. Suspension logging (DH)
   - 16 receiver-to-receiver surveys extending down to approximately 650 feet in depth
   - 16 source-to-receiver surveys extending down to approximately 650 feet in depth
3. Spectral-analysis-of-surface waves (SASW)
   - 35 survey lines extending down to approximately 500 feet in depth (2000-2001)
   - 18 survey lines in the North Portal Area and 6 lines in the Aging Pad Area, generally extending from 400 feet to approximately 1500 feet in depth (2004-2005)

The results and comparisons of the pre-2005 surveys are documented in BSC (2002a). The 24 additional SASW surveys performed in 2005 are documented in MO0609SASWSEDC.001 and MO0609SASWSTDC.003. Table 6-4 shows a list of the references containing the procedures used to conduct the seismic surveys.

Table 6-4. References of Seismic Survey Procedures.

Method	Procedure
Downhole	• Redpath Geophysics: SN-M&O-SCI-030-V1 (Wong 2002b) • GEOVision: SN-M&O-SCI-025-V1 (Luebbers 2002c)
Suspension	• SN-M&O-SCI-024-V1 (Luebbers 2002a) • SN-M&O-SCI-024-V2 (Luebbers 2002b)
SASW	• SN-M&O-SCI-022-V1 (Wong 2002c) • SN-M&O-SCI-040-V1 (Wong 2002a)

Table 6-5 (Table 31 from BSC 2002a) describes and compares the different seismic velocity surveying methods. Table 6-6 lists the borings in which the seismic velocity surveys were performed in the surface facilities area. The locations of the borings in which downhole and

suspension seismic surveys were performed are shown in Figure 1-1. Figure 6-13 (Figure 6.2-7 of SNL 2008) shows the locations of SASW lines at the surface facilities site.

Table 6-5. Comparison of Downhole Seismic, Suspension Seismic and SASW Methods (Table 31 of BSC 2002a)

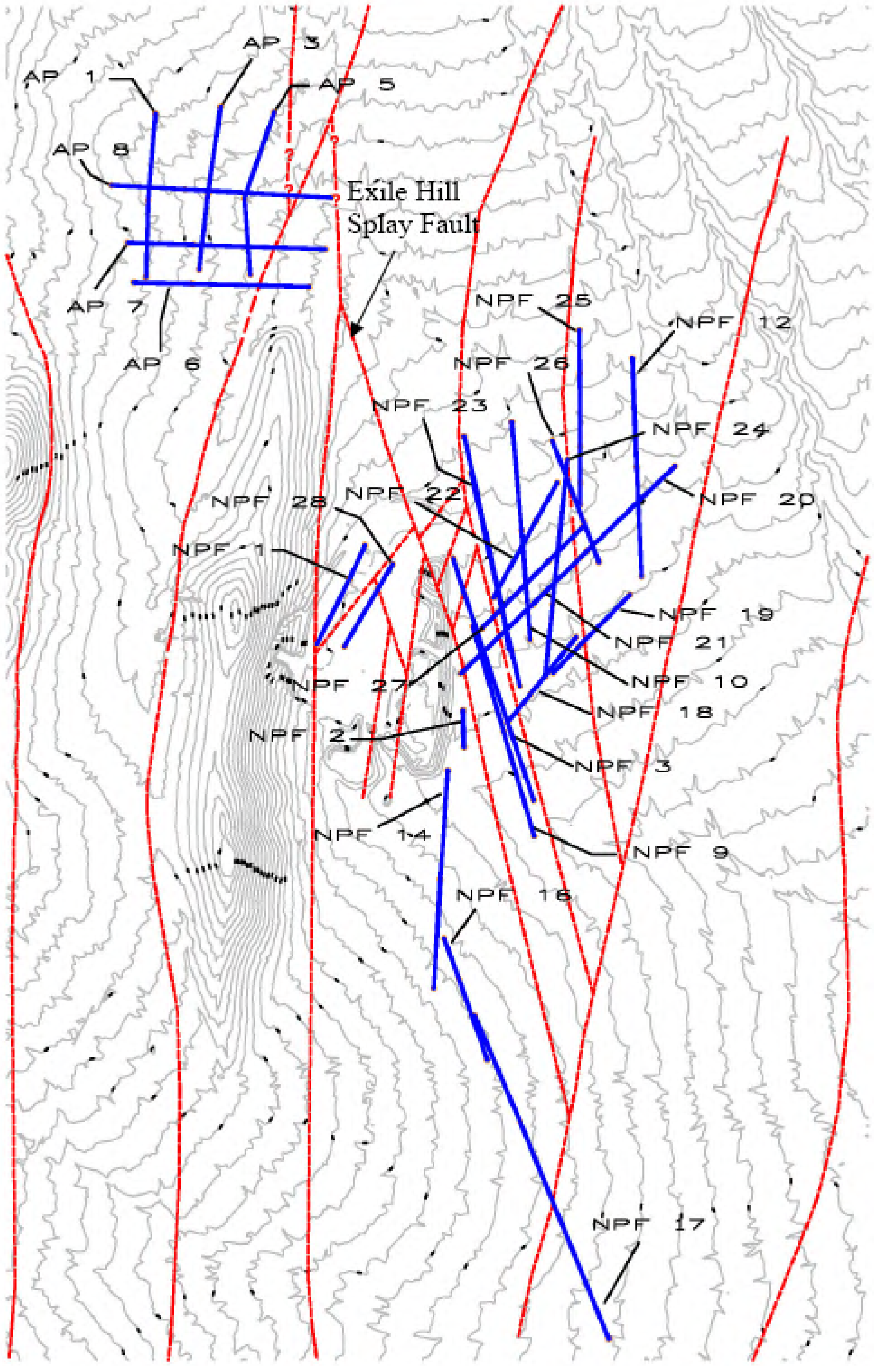
Table 31: Comparison of Downhole Seismic, Suspension Seismic and SASW Methods

Characteristic	Suspension Seismic	Downhole Seismic	SASW
Energy source	Built-in solenoid hammer	Hammer on plank	Hammer at close source-receiver spacings; sledgehammer, dropped weight, bulldozer or vibroseis at longer spacings
Type of wave generated	P and S	P and S	Rayleigh or other surface wave
Ability to reverse polarity	Yes	Yes	No
Primary direction of wave motion	Upward, vertical	Downward, near vertical but becoming more inclined at shallow depth	Horizontal
Wave frequency, Hz	S wave 500 - 1,000 P wave 1,000 – 3,000	S wave 20 - 40 P wave 50 – 200	5 – 500 or more
Boreholes required	One	One	None
Borehole requirements	Liquid-filled; uncased generally preferred; plastic casing is acceptable	Dry preferred; casing optional	Not applicable
Maximum effective depth, ft	1,600	300 to 700	Up to 500
Resolution	Resolution constant with depth	Resolution decreasing with depth	Resolution decreasing with depth
Borehole drift survey	Not required	Not required	Not applicable
Space limitations	Can be performed wherever a borehole can be drilled	Can be performed wherever a borehole can be drilled	Line length is about 2 times the depth surveyed, so on-site and off-site constraints may limit survey depth
Type of wave interpreted	P and S_H	P and S_H	R, converted to S using theory and assumed Poisson's ratio
Interval velocity	Yes	Only with geophones at multiple depths	No
Average velocity	Yes, by accumulation of individual travel times	Yes	Yes

Table 6-6. Seismic Velocity Survey Summary

Borehole ID	Downhole Seismic		Suspension Seismic (source-to-receiver and receiver-to-recevier	SASW[4]
	Redpath Geophysics [1]	**GEOvision Inc. [1]**	**Luebbers M. J. [2]**	**University of Texas at Austin [3]**
UE-25 RF#13	×	×		SASW-1
UE-25 RF#14	×		×	
UE-25 RF#15	×		×	SASW-10+37
UE-25 RF#16 [5]	×		×	SASW-29
UE-25 RF#17		×	×	SASW-34+36 [6]
UE-25 RF#18 [5]	×		×	
UE-25 RF#19	×		×	
UE-25 RF#20 [5]	×		×	
UE-25 RF#21 [5]	×		×	SASW-2
UE-25 RF#22 [5]	×		×	SASW-23
UE-25 RF#23	×		×	SASW-32+35, SASW-33
UE-25 RF#24 [5]	×		×	SASW-4
UE-25 RF#25	×		×	
UE-25 RF#26	×		×	
UE-25 RF#28 [5]	×		×	SASW-8
UE-25 RF#29	×		×	
UE-25 RF#42				SASW-RF42[7]
UE-25 RF#48				SASW-RF48[7]
UE-25 RF#49				SASW-RF49[7]
UE-25 RF#55				SASW-RF55[7]
UE-25 RF#56				SASW-RF56[7]

[1] October through December 2000 surveys (BSC 2002a)
[2] September through December 2000 surveys (Luebbers 2002b)
[3] July through August 2000 surveys unless otherwise noted (BSC 2002a)
[4] A total of 53 SASW surveys were performed in the proposed surface facilities area. A total of 40 shear-wave velocity profiles were developed. Eleven of these profiles were performed between existing boreholes from BSC (2002a) and 5 were performed between new borings drilled in 2005. Refer to Figure 6-13 for SASW line locations.
[5] Caliper and gamma-gamma wireline surveys were performed in these boreholes
[6] 2 velocity profiles measured at SASW line survey
[7] 2005 surveys (MO0609SASWSEDC.001 and MO0609SASWSTDC.003)



Source: DTN: MO0701ABSRFLL2.000 [DIRS 182483]; BSC 2004 [DIRS 170029].

NOTE: Red: mapped and inferred faults and fault splays; blue: SASW lines.

Figure 6-13. Locations of SASW lines at the surface facilities site (Figure 6.2-7 of SNL 2008)

6.2.2.4 Borehole Wireline

Caliper and gamma-gamma wireline surveys were performed in 7 boreholes (RF#16, #18, #20, #21, #22, #24, and #28.). Caliper measurements were performed in order to assess the extent of erosion of the borehole walls by the drilling fluid and its potential effects on the suspension seismic results. The main purpose of performing the gamma-gamma measurements was to evaluate the density of the subsurface materials.

The process established in PA-PRO-0312, Yucca Mountain Site Characterization Project Field Verification of Geophysical Operation, and AP-SIII.6Q, Geophysical Logging Programs for Surface-Based Testing Program Boreholes, were followed for both the caliper and gamma-gamma wireline surveys. The information in these cancelled or superseded documents is considered historical (see Section 2.5.1).

6.3 LABORATORY TESTING

This section discusses laboratory testing conducted on samples taken during 2000 to 2007 from the borings and test pits performed at the surface facilities area.

6.3.1 Static Testing

Documentation of all static laboratory testing prior to 2005 is found in Sections 6.2.9 and 6.5.2 of BSC (2002a) for the alluvial and borrow pit materials, respectively. More recent test results are provided in Section 6.2 of SNL (2008). A summary of the static laboratory test results is presented in Section 6.4 of this report.

6.3.1.1 Alluvium

Static tests were performed on 22 samples of alluvial material obtained from test pits TP-WHB-1 through TP-WHB-4, and on 9 samples from TP-WHB-5 through TP-WHB-7. The tests conducted and the testing standards used are listed in Table 6-7.

Table 6-7. Laboratory Tests and Standards Conducted on Alluvium.

Test	Standard
Atterberg Limits	• USBR 5350-89, *Procedure for Determining the Liquid Limit of Soils by the One-Point Method* • USBR 5360-89, *Procedure for Determining the Plastic Limit and Plasticity Index of Soils.*
Maximum and Minimum Index Unit Weights	For particles passing the 3-inch sieve: • USBR 5525-89, *Procedure for Determining the Minimum Index Unit Weight of Cohesionless Soil* • USBR 5530-89, *Procedure for Determining the Maximum Index Unit Weight of Cohesionless Soils.*
Particle-Size Distribution	• USBR 5325-89, *Procedure for Performing Gradation Analysis of Gravel Size Fraction of Soils* • USBR 5330-89, *Procedure for Performing Gradation Analysis of Fines and Sand Size Fraction of Soils, Including Hydrometer Analysis* • USBR 5335-89, *Procedure for Performing Gradation Analysis of Soils Without Hydrometer–Wet Sieve.*
Specific Gravity	For particles passing the 4.75 mm (No. 4) sieve: • USBR 5320-89, *Procedure for Determining Specific Gravity of Soils (volume method)* For particles retained on the 4.75 mm (No. 4) sieve: • USBR 5320-89, *Procedure for Determining Specific Gravity of Soils (suspension method).*
Unified Soil Classification System	• USBR 5000-86, *Procedure for Determining Unified Soil Classification (Laboratory Method).*
Water Content	• USBR 5300-89, *Procedure for Determining Moisture Content of Soil and Rock by the Oven Method.*

6.3.1.2 Engineered Fill

Disturbed samples of the borrow material were taken from 4 locations (WHB-B1 to WHB-B4) in the Fran Ridge Borrow Area and then combined into one bulk sample. The tests conducted are listed in Table 6-8 below along with the testing standards used (where provided).

Table 6-8. Laboratory Tests and Standards Conducted on Engineered Fill Material.

Test	Standard
Atterberg Limits	• USBR 5350-89, *Procedure for Determining the Liquid Limit of Soils by the One-Point Method.*
Compaction Test	• ASTM D 1557, *Standard Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3)).*
Maximum and Minimum Index Unit Weights	For particles passing the 3-inch sieve: • USBR 5525-89, *Procedure for Determining the Minimum Index Unit Weight of Cohesionless Soils* • USBR 5530-89, *Procedure for Determining the Maximum Index Unit Weight of Cohesionless Soils.*
Particle-Size Distribution	• USBR 5325-89, *Procedure for Performing Gradation Analysis of Gravel Size Fraction of Soils* • USBR 5335-89, *Procedure for Performing Gradation Analysis of Soils Without Hydrometer–Wet Sieve.* • ASTM C 136, *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates*, for 3 conditions: (1) as received; (2) after scalping on the ½-inch sieve and prior to compaction; and (3) after the compaction test on the ½-inch minus material.
Specific Gravity	For particles passing the 4.75 mm (No. 4) sieve: • USBR 5320-89, *Procedure for Determining Specific Gravity of Soils (volume method).* Denver, Colorado laboratory for particles retained on the 4.75 mm (No. 4) sieve: • USBR 5320-89, *Procedure for Determining Specific Gravity of Soils (suspension method).*
Triaxial Test	• Four triaxial tests performed on reconstituted specimens under isotropically consolidated, drained conditions.
Unified Soil Classification System	• USBR 5000-86, *Procedure for Determining Unified Soil Classification (Laboratory Method).*

6.3.2 Dynamic Testing

Dynamic properties of the alluvium, bedrock (tuff), and engineered fill were evaluated using combined resonant column and torsional shear (RCTS) tests. The laboratory dynamic testing was performed in the Geotechnical Engineering Center at the University of Texas at Austin. Testing procedures are presented in Section 6.2.10.1 of BSC (2002a) and SN-M&O SCI-033-V1 (Wong 2002d).

Dynamic properties, including the shear modulus and material damping relative to shearing strain, were determined from the laboratory tests on samples of alluvium, bedrock, and engineered fill. A summary of the results of the dynamic testing is presented in Section 6.4.2 of this report. Table 6-9 lists the testing standard and reference used for the dynamic tests.

Table 6-9. Standard and Reference Used for Dynamic Testing.

Test	Standard and Reference
Resonant column and torsional shear (RCTS)	• PA-PRO-0310, *Laboratory Dynamic Rock/Soil Testing* (this information is considered historical, see Section 2.5.1). • SN-M&O-SCI-033-V1 (Wong 2002d)

6.3.2.1 Alluvium

Due to the granular nature of the alluvial material, undisturbed samples could not be obtained from boreholes or test pits before the summer of 2007. Therefore, one combined alluvial sample was collected from boreholes RF#14 to #17 to provide a general representative sample of the alluvium and to provide sufficient quantities of material to perform the dynamic laboratory tests. The specimen was reconstituted in the laboratory using the standard under-compaction method of Ladd (1978).

Additionally, dynamic testing was also performed on a soil sample taken from borehole RF#13 in 1999. A summary of the test results from this sample is provided in CRWMS M&O (1999, Appendix Q).

6.3.2.2 Engineered Fill

Ten reconstituted specimens taken from the Fran Ridge borrow area were tested, again using the standard under-compaction method of Ladd (1978). Four of the samples were tested in 2 stages to investigate the dynamic property effects of increasing the water content of the granular fill after placement.

6.3.2.3 Bedrock (Tuff)

Eighteen undisturbed specimens taken from boreholes RF#14 to #17 were tested. During testing, the specimens were divided into three groups based on their dry unit weight, γ_d:

- Group 1: γ_d from 133 pcf to 147 pcf
- Group 2: γ_d from 117 pcf to 132 pcf
- Group 3: γ_d from 78 pcf to 94 pcf

6.4 MATERIAL PROPERTIES

This section presents a summary and discussion of the results of both static and dynamic laboratory tests on the soil units at the site. All information presented in the following sections is based on data presented in BSC (2002a), BSC (2002b), SNL (2008), and BSC (2008a). A summary of recommended material properties for design is presented in Table 2-1 and Table 2-2.

6.4.1 Static Soil Properties

6.4.1.1 Alluvium

Results of the in situ density tests and laboratory tests conducted on the alluvial material from TP-WHB-1 to TP-WHB-4 are shown in Tables 6 and 13 of BSC (2002a), respectively. Results of the in situ density tests and laboratory tests conducted on the alluvial material from TP-WHB-5 to TP-WHB-7 are provided in Table 6.2-4 in SNL (2008).

The following sections describe the results of testing on 31 samples obtained at depths ranging from 4 to 20 feet. There were no alluvium samples obtained for depths greater than 20 feet.

6.4.1.1.1 General Characteristics

The alluvium material is generally medium dense to dense, and varies between a 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). Intermittent layers of calcite-cemented material (caliche) are present in the alluvium (BSC 2002b, Section 6.2 and DTNs:GS070583114233.002 and GS070583114233.003). However, the presence (i.e., strengthening effect) of caliche was conservatively ignored in this report. Table 6-10 provides a summary of average soil properties determined from the laboratory testing performed on the 7 test pits.

Table 6-10. Results from Tests Performed on Alluvial Samples at Surface Facilities Area Test Pits WHB-1 through WHB-3, and WHB-5 through WHB-7 (DTNs: GS020483114233.004, GS070683114233.004).

Test	Results
Particle size distribution	57 ± 13% (gravel & cobbles) 37 ± 12% (sand) 6 ± 2.5% (fines)
Plasticity	Non-plastic
Average Density	116 pcf maximum index (passing 3-inch sieve) 92 pcf minimum index (passing 3-inch sieve) 108 pcf dry in-place 71 ± 20% relative
Average minimum index density	91 pcf (passing 3-inch sieve)
Average specific gravity and absorption (passing 3-inch sieve)	2.37 apparent 2.25 bulk (saturated surface dry) 2.16 bulk (oven dry) 4.0% absorption
Average specific gravity and absorption (retained on No. 4 and passing 3-inch sieve)	2.46 apparent 2.26 bulk (saturated surface dry) 2.12 bulk (oven dry 6.5% absorption
Average specific gravity (passing No. 4)	2.52
Average water content	7.1 % (passing No. 4 sieve) 4.9 % (retained on No. 4 sieve)

A comparison of the data from Table 6-10 with soil data from earlier geotechnical investigations (1980's and early 1990's) shows good corroboration of the soil properties. The specific gravity of the alluvium at the site is less than typically encountered for sand and gravel soils, likely due to the volcanic origin of the Yucca Mountain soils. See, for instance, USN (1986), pp. 7.1-23, which uses a specific gravity of 2.65 for granular soils in their tables of typical values.

Additional tests were performed on samples obtained from two of the borings (RF#47 and RF#52) drilled in 2006. The results of these tests are provided in Table 6-11. Note that the average test values are very similar to those obtained from the test pit samples. The only discernable difference is that the sonic drill samples have an increased amount of finer material due to the breakage of the larger gravels inherent in the sonic coring method.

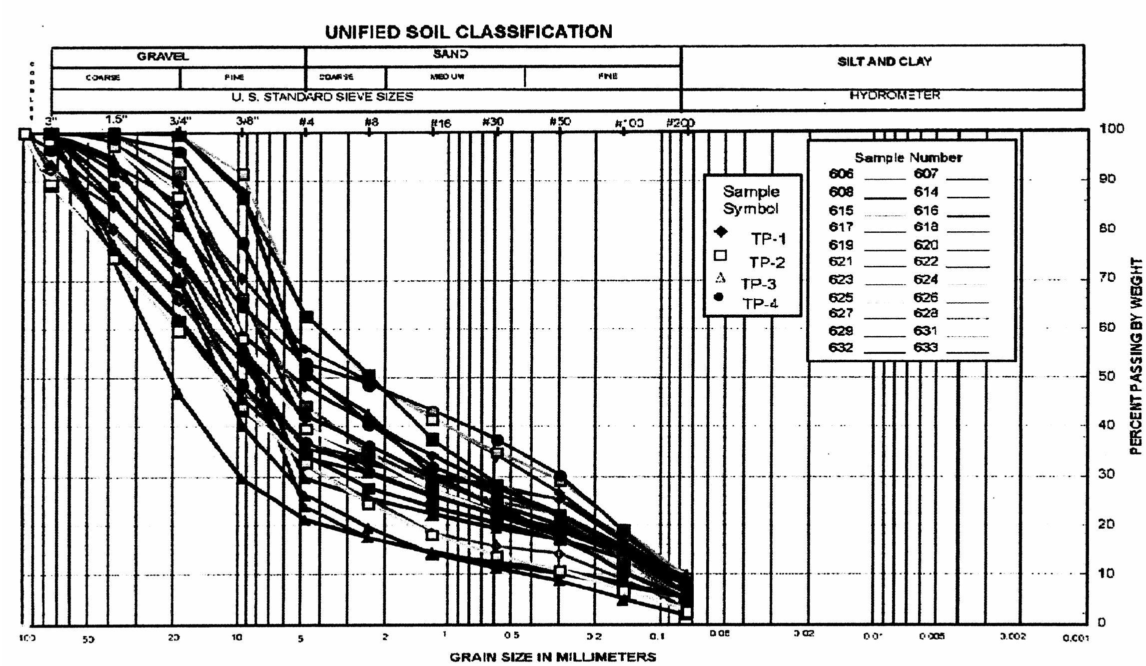
Table 6-11 Results of Laboratory Tests Performed on Alluvium Samples From Borings RF#47 and RF#52 (Table 6.2.3 of SNL 2008)

Sonic Drill Hole Number	Specimen Depth	Gradation Test Results				Minus No. 4 Sieve Fraction	Plus No. 4 Sieve Fraction				USCS Group Symbol
		Cobbles (6 to 3-inch)	Gravel (3-inch to No.4 Sieve)	Sand (No.4 to No. 200 Sieve)	Fines (minus No. 200 Sieve)	Minus No.4 Specific Gravity	Average Apparent Specific Gravity	Average Bulk Specific Gravity (SSD)	Average Bulk Specific Gravity (oven dry)	Average Absorption	
	feet	%	%	%	%					%	
UE-25 RF#47	4.5-8.8	0.0	46.7	41.9	11.4	2.51	2.43	2.26	2.15	5.5	GP-GM
UE-25 RF#47	8.8-13.5	4.0	39.5	43.3	12.2	N/A	2.44	2.25	2.11	6.4	SM
UE-25 RF#47	13.5-15.7	0.0	24.4	56.5	19.1	2.52	2.46	2.26	2.13	6.4	SM
UE-25 RF#47	15.7-18.2	9.7	39.4	40.1	10.8	N/A	2.47	2.31	2.19	5.1	SP-SM
UE-25 RF#47	18.2-20.5	0.0	33.9	48.3	17.8	2.50	2.44	2.26	2.14	5.7	SM
UE-25 RF#47	20.7-21.7	0.0	57.2	34.7	8.1	2.53	2.36	2.20	2.08	5.8	GW-GM
UE-25 RF#47	21.7-26.2	0.0	31.4	51.8	16.8	2.51	2.49	2.29	2.17	6.1	SM
UE-25 RF#47	26.2-28.6	4.0	40.6	44.1	11.3	N/A	2.42	2.23	2.10	6.3	SP-SM
UE-25 RF#47	28.6-31.6	0.0	24.7	53.8	21.5	2.51	2.45	2.25	2.12	6.3	SM
UE-25 RF#47	31.6-34.8	3.6	39.0	43.3	14.1	2.51	2.46	2.27	2.14	6.1	SM
UE-25 RF#47	34.8-36.5	0.0	33.1	47.3	19.6	2.52	2.41	2.22	2.09	6.3	SM
UE-25 RF#47	36.5-40.9	5.6	35.9	44.3	14.2	N/A	2.46	2.30	2.19	5.0	SM
UE-25 RF#47	40.9-43.8	8.3	41.1	38.1	12.5	N/A	2.47	2.30	2.18	5.4	GM
UE-25 RF#47	43.8-51	0.0	43.3	42.0	14.7	2.49	2.44	2.28	2.17	5.1	GM
UE-25 RF#47	51.9-52.7	0.0	35.2	53.9	10.9	2.51	2.46	2.28	2.16	5.6	SP-SM
UE-25 RF#47	54.1-56.8	0.0	36.9	45.4	17.7	2.51	2.48	2.30	2.19	5.3	SM
UE-25 RF#47	56.8-58.2	0.0	21.8	53.1	25.1	2.48	2.49	2.31	2.20	5.3	SM
UE-25 RF#47	59-68.6	2.1	43.2	41.0	13.7	N/A	2.49	2.30	2.18	5.8	GM
UE-25 RF#47	70-87.5	2.6	37.0	46.6	13.8	N/A	2.46	2.30	2.19	4.9	SM
UE-25 RF#47	87.5-88.9	0.0	31.1	41.3	27.6	2.54	2.51	2.29	2.15	6.7	SM
UE-25 RF#52	0-2.9	9.7	38.0	37.0	15.3	N/A	2.43	2.25	2.12	5.9	GM
UE-25 RF#52	2.9-4.8	0.0	40.1	51.5	8.4	2.54	2.50	2.31	2.19	5.6	SP-SM
UE-25 RF#52	5.4-11	1.1	48.4	40.3	10.2	N/A	2.46	2.31	2.20	4.8	GW-GM
UE-25 RF#52	11-24.2	0.0	43.9	41.4	14.7	2.50	2.51	2.31	2.18	6.1	SM
UE-25 RF#52	24.2-30.7	0.0	15.9	68.0	16.1	2.43	2.49	2.27	2.13	6.9	SM
UE-25 RF#52	30.7-35.2	0.0	43.8	45.4	10.8	2.50	2.52	2.35	2.24	5.1	SP-SM
UE-25 RF#52	35.2-40	0.0	37.4	46.8	15.8	2.51	2.50	2.31	2.18	6.0	SM
UE-25 RF#52	40.3-44.5	3.5	46.7	40.2	9.6	N/A	2.50	2.29	2.15	6.5	GW-GM
UE-25 RF#52	44.5-50.3	0.0	34.7	51.5	13.8	2.51	2.51	2.32	2.20	5.8	SM
UE-25 RF#52	50.3-61.8	1.7	44.2	40.6	13.5	N/A	2.46	2.29	2.18	5.4	GM
UE-25 RF#52	61.8-68.4	2.0	35.8	48.4	13.8	N/A	2.45	2.27	2.15	6.0	SM
UE-25 RF#52	68.4-104.5	1.9	41.8	16.9	14.5	N/A	2.44	2.26	2.13	5.9	SM
UE-25 RF#52	104.5-107.9	14.0	36.0	34.1	15.9	N/A	2.42	2.30	2.22	3.7	GM
UE-25 RF#52	109.5-147.9	8.6	42.9	35.7	12.8	N/A	2.86	2.28	1.97	15.8	GM
UE-25 RF#52	149.8-152.5	0.0	38.3	44.5	17.2	2.53	2.44	2.24	2.10	6.7	SM
UE-25 RF#52	152.5-155.7	16.9	35.7	35.1	12.3	N/A	2.44	2.29	2.18	5.0	GM
UE-25 RF#52	157.8-160.7	5.9	44.5	36.6	13.0	N/A	2.45	2.31	2.20	4.7	GM
Average ->		**2.8**	**37.9**	**43.9**	**14.6**	**2.5**	**2.5**	**2.3**	**2.2**	**6.0**	

DTN: GS080183114233.001

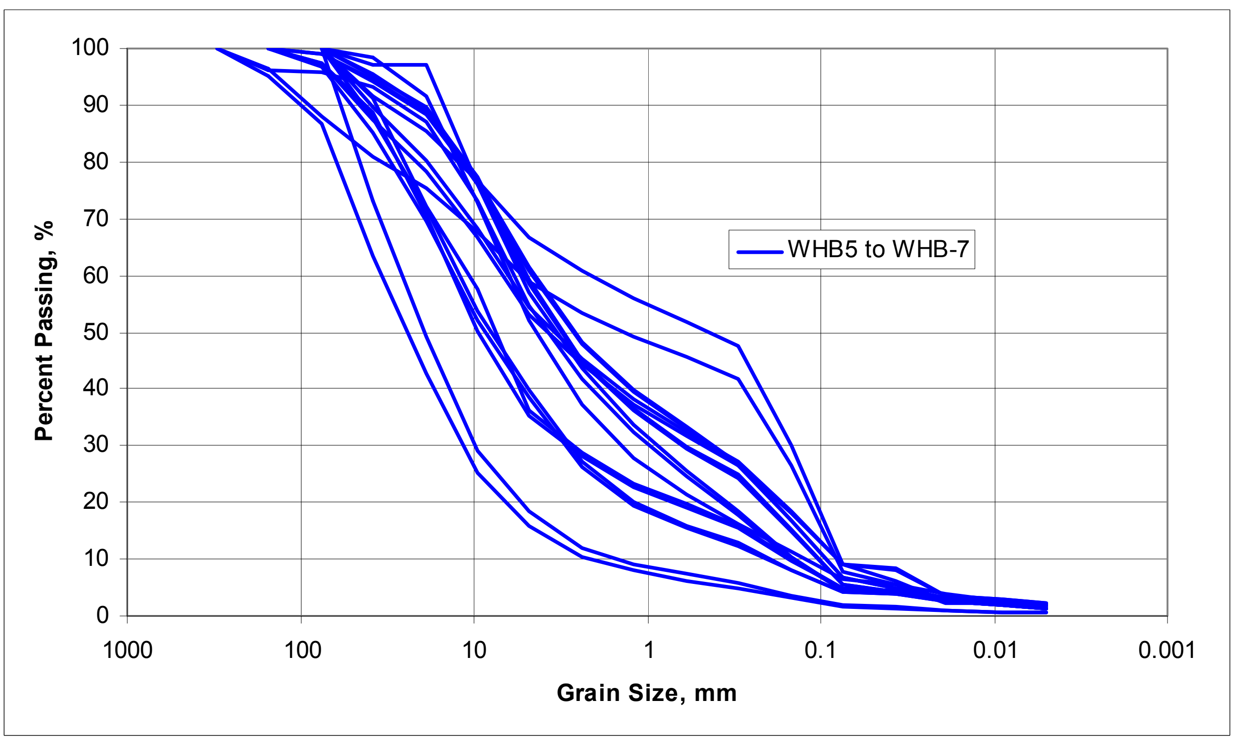
6.4.1.1.2 Gradation

Plots of gradation test results from test pits WHB-1 through WHB-4 (DTN: GS020783114233.005) are provided in Figure 6-14, while those for WHB-5 through WHB-7 (DTN: GS070683114233.004) are provided in Figure 6-15. Gradations for sonic Borings RF-47 and RF-52 are provided in Figure 6-16. As described above, the sonic coring breaks up some of the larger gravels resulting in a shift to finer particles as illustrated by comparison of Figure 6-14 and Figure 6-15 (both from test pits) to Figure 6-16 (from sonic cores).



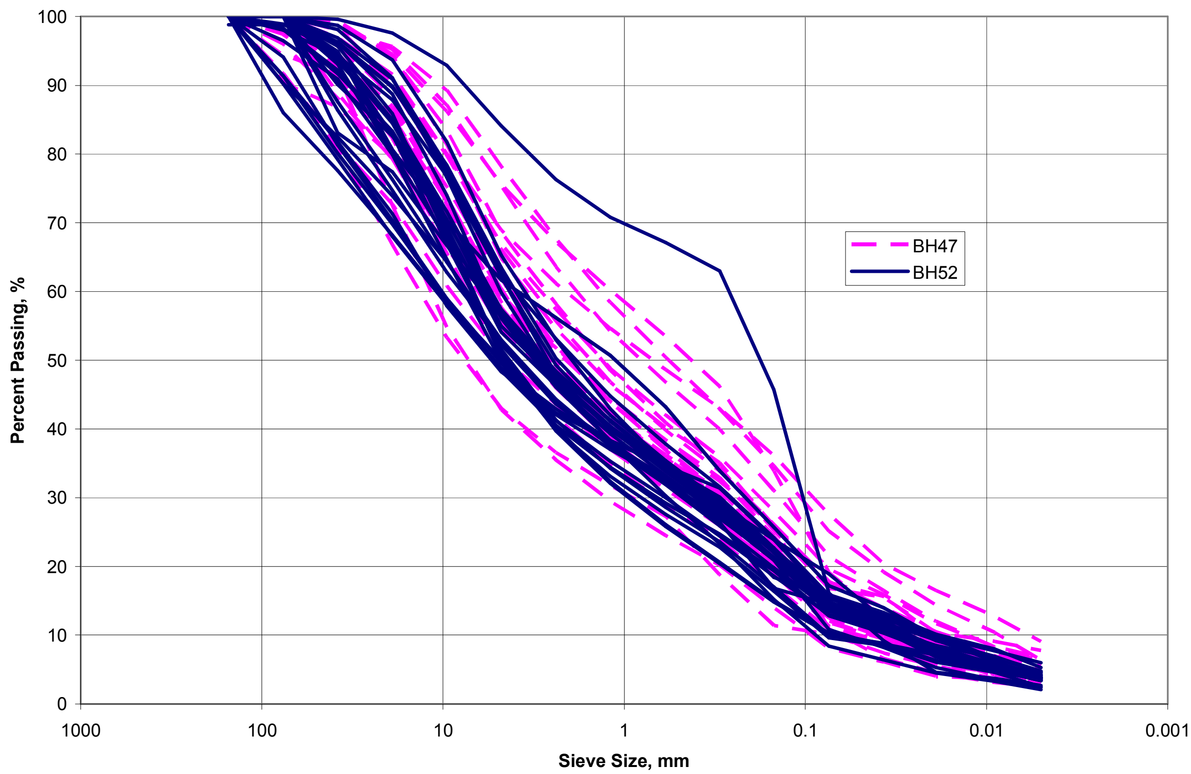
DTNs: GS020483114233.004, GS020783114233.005

Figure 6-14. Particle-Size Distribution Curves for Alluvium for TP-WHB-1 to TP-WHB-4



DTN: GS070683114233.004

Figure 6-15. Particle-Size Distribution Curves for Alluvium for TP-WHB-5 to TP-WHB-7



DTN: GS080183114233.001

Figure 6-16. Particle-Size Distribution Curves for Sonic Borings RF-47 and RF-52

6.4.1.1.3 Density

From the 31 samples taken within the alluvium from the 2002 and 2006 field tests, in-place dry density, and minimum and maximum index density tests were performed (see Table 6-10). An average relative density of 71% was determined from these tests.

Density testing included 15 ring density and 16 sand cone tests conducted down to 20 feet in depth into the alluvium, and gamma-gamma surveys extending up to a 480-foot depth through the alluvium and into bedrock. Based on the field tests, Sections 8.2.1 and I.2.1 of BSC (2002b) recommend average moist unit weights for alluvium of approximately 114 pcf in the upper 8 feet and 117 pcf below 8 feet. Moisture contents vary between about 5 and 7 percent. The data from the gamma-gamma surveys are the only known density measurements at lower depths of the alluvium and are generally lower in value by approximately 25 to 30%. However, Section 8.2.1 of BSC (2002b) indicates that these results may correspond to the bedrock material rather than the alluvium.

Densities from earlier soil investigations were measured by water replacement tests (McKeown 1992), laboratory tests on drive tube samples (Neal 1986), and sand cone and nuclear tests (Ho, et al. 1986). A comparison of the data obtained from these measurements to recent field tests show good agreement. Hence, a conservative moist unit weight of 114 to 117 pcf for the alluvium is recommended.

6.4.1.1.4 Shear Strength

Because undisturbed alluvial samples were not obtained in prior geotechnical investigations, correlations from several sources between the relative density and friction angle are used to estimate the strength of the alluvium. Table I-17 of BSC (2002b) presents a summary of the friction angles obtained from the various correlations used. The mean, and mean plus/minus one standard deviation values of relative density are used for the calculation. Based on the correlations between relative density and friction angle, an effective friction angle of 39 degrees, corresponding to halfway between the mean minus one standard deviation and the mean values of the relative density, is recommended for the alluvium for a pressure of 1 atmosphere.

Sections 8.2.2, I.2.2.1, I.2.2.2, I.2.2.3, I.2.2.4, and I.2.2.5 of BSC (2002b) recommend different strength envelopes to be used for different types of analyses (i.e., passive pressures, bearing capacity, and slope stability). A linear failure envelope with no cohesion (c = 0) and producing an equivalent effective friction angle, ϕ_{eff}, of 39 degrees is considered to adequately characterize the alluvial material and is recommended for design.

SPT data (discussed in Section 6.2.2.2) from borehole RF#13 and TP-NNWSI2 corroborate the conservative shear strength friction angle selected for the alluvium, revealing blow counts on the order of 100 to 300 blows/foot. This also holds true for correlations between shear wave velocity and shear strength, as the measured shear wave velocities at the site correlate to unrealistically high shear strength values.

6.4.1.1.5 Earth Pressure Coefficients

Earth pressure coefficients are calculated for at-rest, active and passive conditions to be used for analyses of lateral earth pressures (Section 7.1.5). Appendix C documents the derivation of these coefficients. The results are shown in Table 2-1.

6.4.1.1.6 Young's Modulus

Static Young's Modulus, E, for the alluvium can be calculated using the results of the elastic settlement analyses contained in Appendix C. For expected vertical loads of 3 and 5 ksf, the elastic settlements computed are 0.4 and 1.6 inches, respectively. Using a maximum alluvium thickness of 120 ft, the average strains induced in the alluvium from the 3 and 5 ksf vertical loading are determined to be 0.03% and 0.11%, respectively. Static Young's modulus can then be determined using:

$$E = \frac{\sigma}{\varepsilon}, \qquad \text{(Eq. 1)}$$

where E is the Young's modulus, ε is the axial strain, and σ is the vertical stress. E is determined to be 30 to 75 ksi.

Sections 8.2.3 and I.2.3 of BSC (2002b) recommend the following equation to be used for a strain range of 0.1 to 0.5% and for a stress range for 0 to 6 ksf:

$$E = 777.37(\varepsilon)^{-0.6505}\sigma^{0.5}\text{, where} \qquad \text{(Eq. 12 and I-66 of BSC 2002b)} \qquad \text{(Eq. 2)}$$

ε is the axial strain in percent and σ is the vertical overburden stress. Using an average overburden stress in the alluvium of 11 ksf (σ = 0.117 kcf × 60 from the alluvium weight plus 4 ksf from vertical loading) and an axial strain of 0.1% yields E to be 80 ksi. For design, use a static Young's Modulus of 30 to 75 ksi for static loading conditions (Table 2-1).

6.4.1.1.7 Resistivity

Electrical resistivity of the soil will be required for design of grounding and evaluation of corrosion potential. Field measurements will be required for the alluvium and any engineered fill that is placed. It is expected that the main source of engineered fill will be alluvium and, therefore, the resistivity of these two materials will be similar. Measurements made at eight locations on the alluvial surface prior to building the construction-support pad at the North Portal (USBR 1992, and USBR 1993) provide a typical range of values for these materials. The results indicated resistivities measuring between 60 and 540 ohm-meters.

6.4.1.2 Engineered Fill

It is anticipated that engineered fill will be obtained from alluvial soils, possibly processed to some extent. The Fran Ridge Borrow Area is an example of such material. The information presented in this section is provided for corroborative purposes only. Actual design values will be obtained after a source pit is identified.

Results of the static tests conducted on the fill obtained from the Fran Ridge Borrow Area are presented in Table 6-12 (Table 27 and Figure 214 of BSC 2002a). Results of additional static strength tests are presented in Table 28 and Figures 215 through 217 of BSC (2002a). The following sections summarize the results of the laboratory testing on disturbed samples obtained at widely spaced locations in the Fran Ridge Borrow Area.

6.4.1.2.1 General Characteristics

The borrow material is classified as a poorly graded sand to gravel (SP/GP), and, after compaction, a poorly graded sand with silt and gravel (SP-SM). Table 6-12 below presents soil properties determined for engineered fill from laboratory testing.

Table 6-12. Results from Tests Performed on Composite Sample of Fran Ridge Borrow Materials (Table 27 of BSC 2002a, DTN: MO0206EBSFRBLT.018).

Test	Results
Particle size distribution	48% (gravel) 49% (sand) 3% (fines)
Plasticity	Non-plastic
Average maximum index density Average minimum index density	112.4 pcf (passing 3-inch sieve) 94 pcf (passing 3-inch sieve)
Average specific gravity and absorption (passing 3-inch sieve)	2.39 apparent 2.24 bulk (saturated surface dry) 2.13 bulk (oven dry) 5.3% absorption
Average specific gravity and absorption (retained on No. 4 and passing 3-inch sieve)	2.45 apparent 2.24 bulk (saturated surface dry) 2.10 bulk (oven dry 6.9% absorption
Average specific gravity (passing No. 4)	2.52

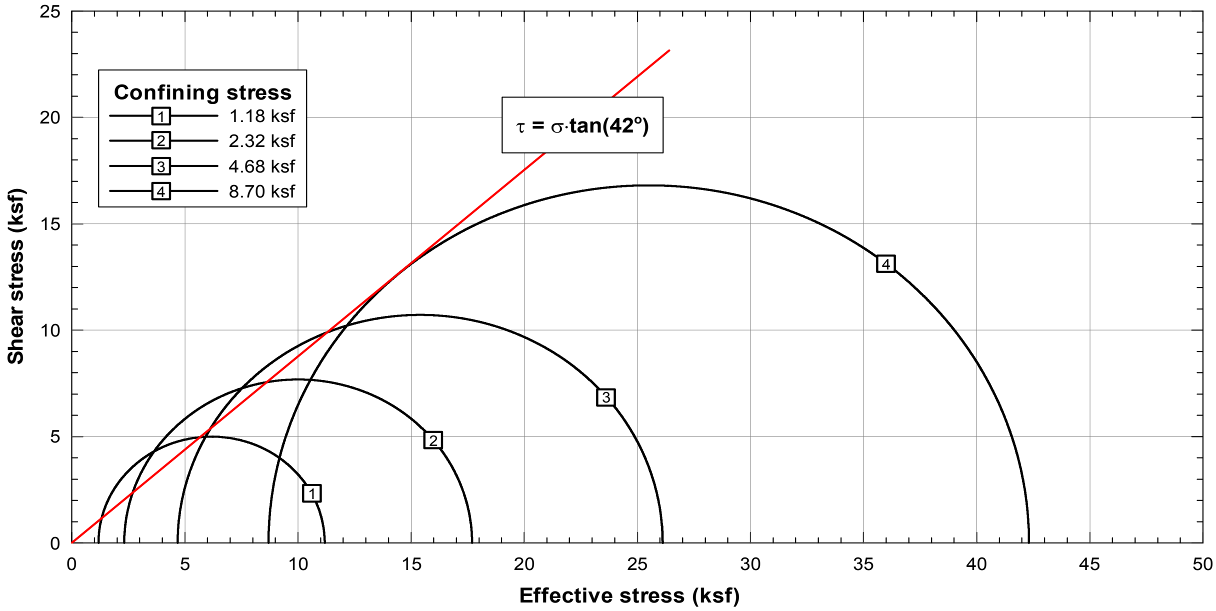
6.4.1.2.2 Total Unit Weight

The results of the compaction test on a composite sample of the Fran Ridge Borrow material indicate a maximum dry unit weight of 114.5 pcf for an optimum water content of 11 percent. Based on the results from the compaction test and on the standard practice presented in ASTM D 4718 for the correction of unit weight and water content for soils containing oversized particles, the moist unit weight for the engineered fill is computed to be 127 pcf [114.5 pcf × (1+0.11)].

6.4.1.2.3 Shear Strength

A set of four drained triaxial compression tests performed on the composite sample of the Fran Ridge material is used to obtain the shear strength of the engineered fill material. The material was compacted to an average dry density of 110 pcf and water content of 12.5%. The results of these tests are shown in Table 28 and Figures 216 and 217 of BSC (2002a). Figure 6-17 below (Figure 217 of BSC 2002a) shows the results of the triaxial tests.

Sections 8.1.2, I.1.2.3, I.1.2.4, I.1.2.5, and I.1.2.6 of BSC (2002b) recommend various strength envelopes to be used for different types of analyses (general purpose, passive pressures, bearing capacity, slope stability). However, a linear failure envelope with no cohesion (c=0) and producing an equivalent effective friction angle, ϕ_{eff}, of 42 degrees is considered to adequately characterize the engineered fill material.



DTN: MO0203EBSCTCTS.016

Figure 6-17. Strength envelopes fitted to triaxial tests on engineered fill.

6.4.1.2.4 Earth Pressures Coefficients

Earth pressure coefficients are calculated in Appendix C. Table 2-1 presents the results.

6.4.1.2.5 Young's Modulus

Typical Young's Modulus values for a dense sand and gravel material are recommended to be 14–28 ksi (Bowles 1996).

A secant Young's modulus was calculated in Sections 8.1.3 and I.1.3 of BSC (2002b) from the drained triaxial test results. Equations 6 and I-19 of BSC (2002b) are recommended for use in computing the Young's modulus:

$$E = 911.19(\sigma')^{0.4541}\text{, where} \qquad \text{(Eq. 6 and I-19 of BSC 2002b)} \qquad \text{(Eq. 3)}$$

σ' is the initial isotropic consolidation stress prior to loading. The above equation corresponds to a strain of 0.25%. For an overburden stress of 5.5 ksf, Young's Modulus is estimated to be approximately 14 ksi.

For design, use a static Young's Modulus of 14 to 28 ksi (Table 2-1).

6.4.1.3 Bedrock

6.4.1.3.1 Moist Unit Weight

Density measurements were obtained from the gamma-gamma wireline surveys and dynamic laboratory testing for the bedrock material.

Table 12 of BSC (2002a), provides a statistical summary of density measurements by lithostratigraphic unit from the borehole wireline geophysical surveys (made in boreholes RF#16, #18, #20, #21, #22, #24, and #28). Figure 101 of BSC (2002a), shows the total densities measured versus depth. This information is summarized in Table 6-13 below.

Densities of the bedrock were also measured in the dynamic laboratory tests. BSC (2002a) compares the mean values from the in situ tests and dynamic laboratory tests, as well as values obtained from previous borehole samples in the area (Table 34 of BSC 2002a). Some variability exists between the different methods of measurement. Since the number of measurements obtained from dynamic tests was too small to provide reliable numbers compared to the in situ tests, it was not considered. In accordance with Section I.3.1 of BSC (2002b), it is recommended that the lowest density value obtained for bedrock (approximately 100 pcf from the Tpki rock unit) be used for design as this provides the most conservative value for bearing capacity calculations.

Table 6-13. Mean Values of Material Density from Borehole Geophysical Surveys (adopted from Table 12 of BSC 2002a).

Unit	Mean Density (pcf)
Existing Fill	115
Alluvium, Qal	116[(1)]
Bedrock, Tmbt1	110
Bedrock, Tpki	98
Bedrock, Tpbt5	112
Bedrock, Tpcrn	117
Bedrock, Tpcpun	132
Bedrock, Tpcpul	130
Bedrock, Tpcpmn	145
Bedrock, Tpcpll	136
Bedrock, Tpcpln	132

[(1)] Assumption 4 of Section 5 in BSC (2002a) was not considered in the values.

6.4.1.3.2 Shear Strength

Since, the structures will be underlain by a significant amount of alluvium over bedrock, and noting that the shear strength of bedrock is much greater than that of alluvium, an estimation of bedrock shear strength within the depth of structural influence was conservatively ignored for the purposes of this report. This information can be derived from other project sources if needed.

6.4.2 Dynamic Soil Properties

Dynamic soil properties, including seismic wave velocity, Poisson's ratio, and strain dependent parameters of shear modulus degradation and material damping ratio were developed for use in the dynamic analyses of the structures and foundations at the surface facilities site. The available field and laboratory data was initially compiled and reported in BSC (2002a). Then additional SASW and dynamic laboratory testing were performed in 2005 through 2007 and summarized in SNL (2008). Analysis and recommendations for input ground motions for preclosure seismic design and postclosure performance assessment were initially provided in BSC (2004a). Then updated with the 2005 through 2007 data and incorporated using a different analysis approach in BSC (2008a).

6.4.2.1 Alluvium and Bedrock

Subsurface conditions vary across the site and therefore seismic analyses parameters and assumptions will vary as well, dependent on site location and structure. Therefore, the reader is directed to the tables, figures and discussions provided in BSC (2008a) for determining the applicable dynamic parameters to use for the alluvium and bedrock material in analyses underlying the various structures.

6.4.2.2 Engineered Fill

No geophysical surveys could be performed on the proposed engineered fill. The shear and compression wave velocity ranges for engineered fill are expected to be equivalent to those of alluvium at comparable depths. A test fill program, discussed in Section 7.3.10, will be performed to define pertinent properties of the fill before final design.

A Poisson's ratio of 0.3 to 0.4 is recommended based on Bowles (1996) for a dense cohesionless sand.

6.4.3 Roller Compacted Soil Cement

6.4.3.1 Recommended Properties

The following recommendations are based on the review of properties of roller compacted soil cement (RCSC) and deep soil mixes that are summarized in Section 9 of BSC (2004b). These parameters are provided as a first estimate for dynamic evaluation of roller-compacted soil-cement should RCSC be considered for use at the site:

- Percent cement: 4% to12% by weight
- Unit weight: 130 pcf to 140 pcf
- Poisson's ratio: 0.30
- Shear-wave velocity, V_s:
    - Lower bound: 2000 ft/s
    - Average: 2500 ft/s
    - Upper bound: 3000 ft/s
- Shear modulus at low strain:
    - Lower bound: 100 ksi
    - Average: 180 ksi
    - Upper bound: 270 ksi
- Strength
    - c = 400 psi (low end unconfined compressive strength for gravelly soils, ACI 230.IR-90, Table 4.1)
    - ϕ = 0 deg (ignored)

No information was found in the literature regarding shear modulus reduction curves specific to RCSC. However, the following presents a limited collection of shear modulus reduction curves for cement treated soils identified in the literature:

1. Dupas and Pecker (1979)–From cyclic triaxial tests performed on soil-cement samples. Soil was fine to medium grain sand with 5% cement by weight and compacted to 100% of the maximum density as determined by ASTM D 558. Curing time 180 days.

2. Wang (1986)–From triaxial and simple shear tests on artificially cemented sand. The material was a mixture of Monterey #0 and #20 sand with 2% cement and 74% relative density.

3. Kohata et al. (1997)–From cyclic triaxial tests performed on soil-cement samples cured for 28 days. Soil was fine-grained. Cement percentage unknown.

4. Sato et al. (1995)–From dynamic triaxial tests performed on sand-cement samples. Sand was fine to medium grained with100% less than 0.84 mm, D60 = 0.35 mm, D30 = 0.31 mm, and uniformity coefficient = 1.59. Cement percentage unknown.

5. McGinn and O'Rourke (2003)–From pressuremeter tests performed on stiff clays treated with 12% to 15% cement by weight using the deep soil mixing method.

Figure 6-18 presents the normalized shear modulus data from these sources. The EPRI (1993) curve for sand for depths of 0-20 ft is included for comparison.

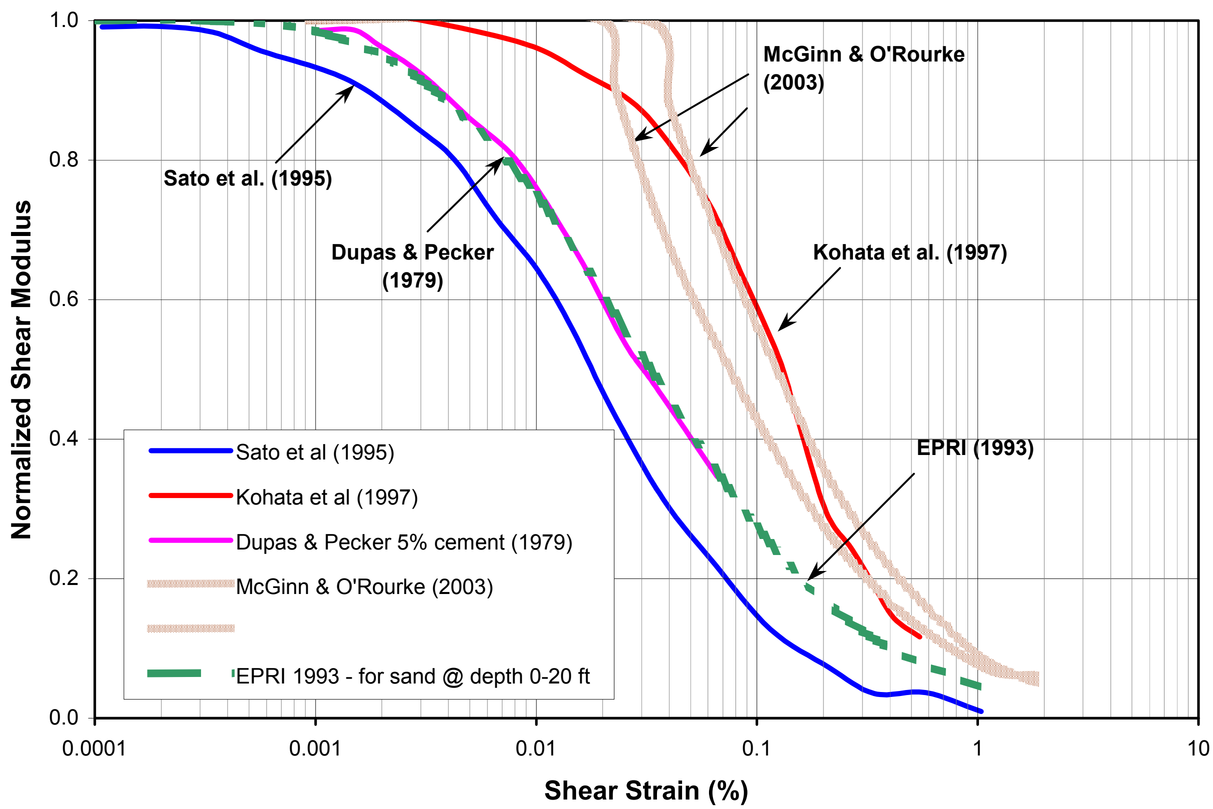


Figure 6-18. Normalized shear modulus reduction curves for cement treated soils.

Note that the curves by Kohata et al. (1997), and McGinn and O'Rourke (2003) were computed from fine-grained soils treated with cement, while all other curves were obtained from sand-cement mixtures.

No information was found in the literature regarding damping ratio reduction curves specific to RCSC. Only Dupas and Pecker (1979), Wang (1986) and Kohata et al. (1997) present damping ratio degradation data for soil cement mixes. Figure 6-19 presents the damping ratio degradation data from these sources. The EPRI (1993) curve for sand for depths of 0-20 ft is included for comparison.

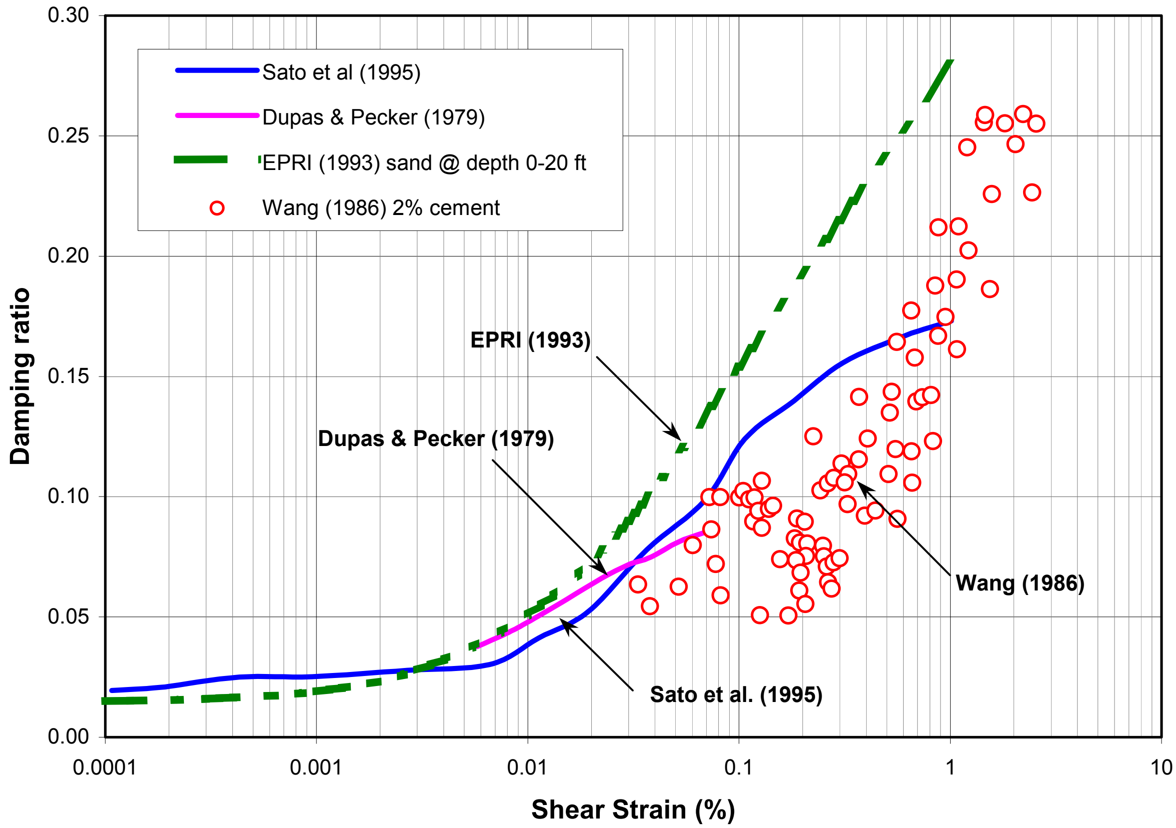


Figure 6-19. Damping ratio degradation curves for cement treated soils.

Note: the curves presented in Figure 6-19 were obtained from sand-cement mixtures.

6.4.3.2 Limitations of Use

It is not expected to use RCSC at this site. If it is considered a viable alternative to engineered fill, a comprehensive laboratory and field testing program would be conducted to determine the above listed properties prior to final design. In addition, non-linear soil-structure interaction analyses may be performed to optimize the depth and extent of soil treatment. Depending on the extent of the treatment areas, the above inputs may be used either in the free-field or in SSI models.

7 RESULTS AND CONCLUSIONS

7.1 Engineering Design Parameters

Analyses outputs obtained from the calculations prepared for this report are reasonable compared to the input parameters used. The results are considered suitable for use in the design of the surface facilities.

Figures and tables containing supporting design parameters are located at the end of this section. Table 2-2 shows key results of the analyses contained herein.

7.1.1 Material Properties

Table 2-1 summarizes the recommended static and dynamic soil properties discussed in Sections 6.4 and 7 for design.

7.1.2 Foundation Pressures

The recommended foundation pressures of the soil at the surface facilities area were determined for various conditions using conventional geotechnical bearing capacity theory. Due to the dense granular nature of the alluvium that will support the planned structures, settlement, rather than bearing capacity, will control allowable pressures. Design charts are provided for allowable foundation pressures for different footing geometries resting on alluvium. Design settlements of 1-inch, or less, and ½-inch, or less, were used in the analyses. The strength parameters used for the alluvium are discussed in Section 6.4. A minimum factor of safety of 3.0 against bearing capacity failure was used (i.e., $q_{allowable} = q_{ultimate} / 3.0$).

Figure 7-2 and Figure 7-3 show the recommended bearing pressures on square and strip footings for 2-foot and 6-foot embedment depths with widths ranging from 2 to 30 feet, for 1-inch and ½-inch design settlement, respectively. Figure 7-4 and Figure 7-5 show the variation of immediate settlement with bearing pressure for square and strip footings of 5-foot, 10-foot, and 20-foot widths, for embedment depths of 2 feet and 6 feet, respectively. Figure 7-6 shows the variation of long-term settlement with foundation width. Note that in Figure 7-6, strip and square footings provide nearly identical solutions. Details of the foundation analyses are documented in Appendix B.

For mat foundations, design should be based on allowable settlements. However, the maximum allowable bearing pressures under the mat should not exceed 50 ksf under extreme (seismic) load conditions and 10 ksf under normal load conditions (Appendix B, Section B6.7).

7.1.3 Settlement

7.1.3.1 Short-Term Settlement

Settlement of foundations is a function of the footing size, average footing load, the depth of footing embedment, and characteristics of the soil material type. Two methods were considered to estimate the settlements for the surface facilities area: (1) Burland and Burbidge (Terzaghi et al., 1996), which uses an average N_{60} blow count value (correlated from a relationship to the relative density), and (2) Schmertmann (Terzaghi et al., 1996), which uses Young's modulus (correlated from measured shear-wave velocities). Immediate settlements induced under different foundation pressures are presented in Figure 7-4 and Figure 7-5 for a variety of conditions. A detailed description of the analyses is provided in Appendix B.

7.1.3.2 Long-Term Settlement

Over time, some additional settlement will occur due to long-term, secondary settlement effects. This settlement is in addition to that estimated in Section 7.1.3.1. The long-term or secondary settlements for the surface facilities area were computed based on the method developed by Burland and Burbidge (Terzaghi et al., 1996) for a 50-yr structure design life (Section 2.2.2.7 BSC 2008b). The settlement was determined to be less than ½ inch. Long-term settlements are presented for different footing widths (square and strip footings) and different depths of foundation embedment in Figure 7-6. A detailed description of the analysis is provided in Appendix B.

7.1.3.3 Elastic Settlement

Elastic settlements were computed for a large mat foundation based on a uniform vertical stress distribution, representative average shear wave velocities (see BSC, 2008a), and Young's modulus (derived from modulus degradation curves). The dimensions of (300' × 400') were used in the analysis, which encompass the largest building dimensions. Settlements under the corner and center of the mat were determined for loads of 3, 5, and 7 ksf. A detailed description of the analysis is provided in Appendix B. Results of the analysis are shown in Table 2-2.

7.1.3.4 Differential Settlement

In accordance with Peck, et al. (1974) Chapter 14, differential settlement between adjacent footings can be ¾ of the maximum estimated value.

The following are allowable angular distortions, δ/L (allowable differential settlement over a given distance), for buildings (Fig. 5.59 of Fang 1991):

δ/L	Building type
1/500	Buildings where cracking is not permissible; Rigid circular mat or ring footing for tall and slender rigid structures

where δ = allowable differential settlement and L = spacing distance

7.1.3.5 Seismically-Induced Settlement

Seismically-induced settlement is not considered to be a significant issue due to the dry and dense nature of the soils encountered at the YMP site. In addition, cementation of the alluvium will also reduce the potential for dynamic settlement.

7.1.4 Coefficient of Subgrade Reaction and Equivalent Soil Springs

All shallow footings and mat foundations will be supported by the alluvium. For the design of large footings and mats it is typical to represent the soil with equivalent springs. The vertical coefficient of subgrade reaction for the alluvium is estimated based on Terzaghi (1955). For dry dense sand, the recommended value for a one-foot by one-foot plate, k_{s1}, is 600 to 2000 kcf (kips/cubic foot). For the dense gravelly alluvium present at the site, it is recommended that a best estimate value of 1000 kcf (580 pci) be used. For the anticipated dense engineered fill, it is recommended that a best estimate value of 600 kcf (350 pci) be used.

These values must be reduced for large loaded sizes in accordance with the following relationship:

$$k_s = k_{s1}\left(\frac{B+1}{2B}\right)^2 \qquad \text{(Eq. 4)}$$

where B is the least footing dimension and k_s is the coefficient of subgrade reaction for the footing or mat. For large footings or mats k_s will approach $\frac{k_{s1}}{4}$. Therefore, for preliminary design, it is recommended to use 155 to 520 kcf (90–300 pci) for alluvium and 75 to 250 kcf (45–145 pci) for engineered fill. It is common practice to double the static load for dynamic load cases.

Figure 9 of USN (1986) was used to estimate the horizontal coefficient of subgrade reaction by correlations with relative density. For the very dense alluvium material, the values were estimated to be 104–120 kcf (60–70 pci). For the dense engineered fill, the values were estimated to be 60–96 kcf (35–55 pci). Results are summarized in Table 2-1.

7.1.5 Lateral Earth Pressures

Currently a 55-foot below-grade wall is planned for construction of a pool for the wet-process building. Lateral earth pressures were determined to estimate the loads that will act on subgrade walls. Both static and seismic conditions for yielding and non-yielding walls were considered in the analyses, including effects from compaction-induced earth pressures and static surcharge loads. Live loads were not considered in the analyses. No factor of safety was applied to the calculated earth pressures. The calculations were performed using the measured properties of the alluvium (see Section 6.4.1). A coefficient of horizontal acceleration of 1g was used in the seismic analysis. The results from the seismic analysis may be scaled by any selected peak ground acceleration value.

A schematic summary of the results for yielding and non-yielding walls is shown in Figure 7-7 through Figure 7-15. A detailed description of the analyses is provided in Appendix C.

7.1.5.1 Lateral Earth Pressures for Temporary Shoring

For a braced and shored excavation the lateral pressures can be estimated using a uniform pressure of 17H psf, where H is the height of the wall. Details of the supporting analysis are provided in Appendix C.

7.1.5.2 Surcharge and Compaction Loads

Surcharge loading due to nearby point, line, uniform surcharge, strip, and footing loads are presented in Figure 7-8 and Figure 7-9. These relationships are based on those presented in USN (1986). In accordance with Section 4.2.11.3.5 of BSC (2007a), a minimum surcharge load of 300 psf shall be used

Compaction stresses imposed on the wall as a result of compaction are addressed in Appendix C. The calculated compaction stresses due to various compaction devices are presented Figure 7-11 through Figure 7-15. .

7.1.6 Resistance to Lateral Loads

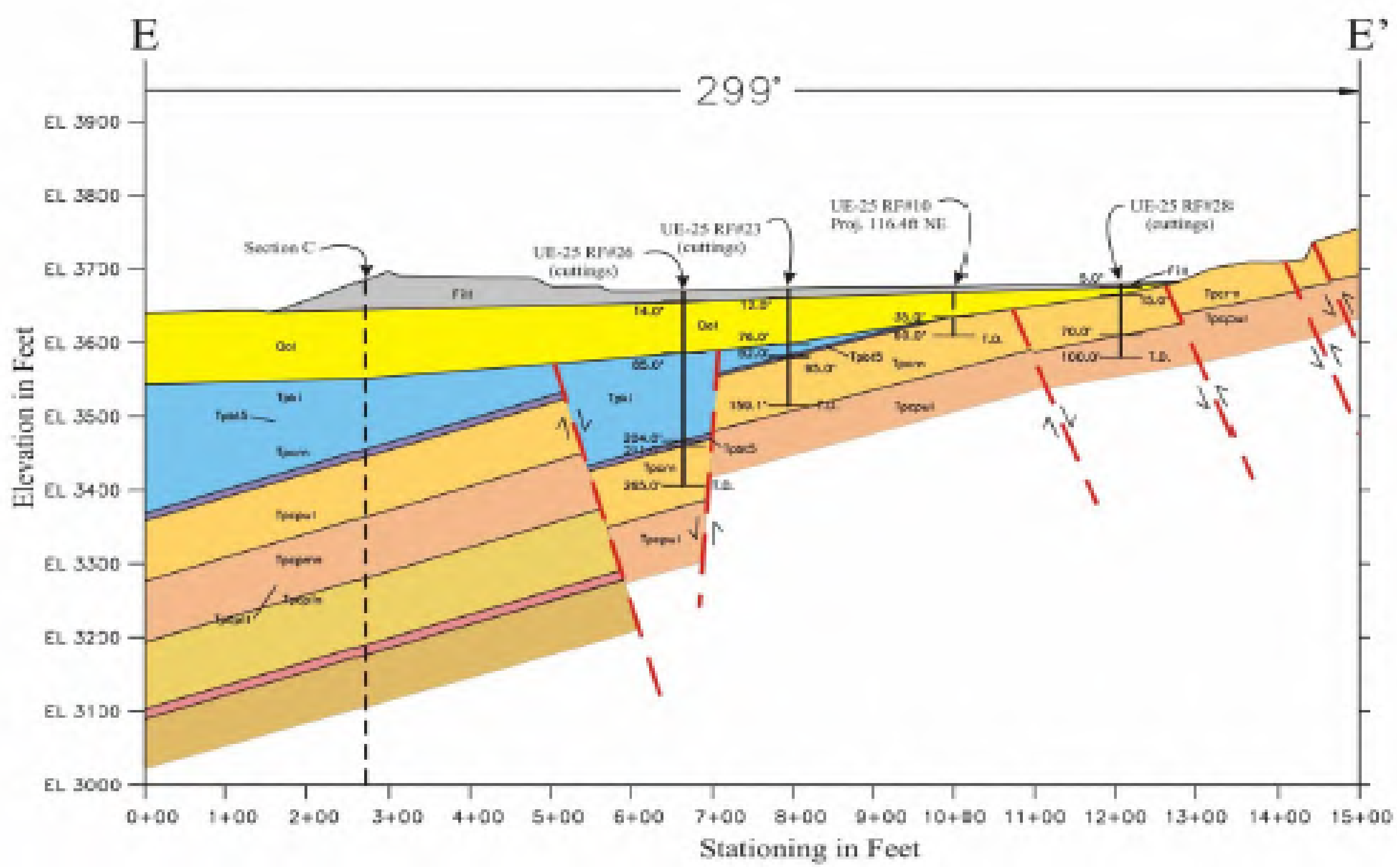
Resistance to lateral loads acting on footings, mats, and subgrade walls can be developed from passive resistance of the soil and from friction acting between the structural base and the subgrade soils.

Passive resistance can be determined assuming an equivalent fluid unit weight of 515 pcf acting on the sides of the foundations. When applying passive resistance for external footings or building walls, the effective depth of embedment should be reduced by one foot. Supporting calculations are provided in Appendix C.

The ultimate coefficients of sliding friction for mat and footing foundations underlain by alluvium and engineered fill are estimated to be 0.81 and 0.90, respectively (see Appendix C).

7.1.7 Slope Considerations

Figure 7-1 is a cross-section through the site and the lower end of Exile Hill (located on the far right hand side of the figure), which is located west of the planned surface facilities. Note that the column of geologic labels on the left-hand-side of Figure 7-1 has been shifted upward and do not label the corresponding geologic strata. The purpose for including Figure 7-1 is to illustrate the relatively gentle slope of Exile Hill directly adjacent to the surface facilities to be located east of RF#28 at the toe of the slope. Portions of Figure 7-1 are illegible, but are not pertinent to the evaluations, conclusions or recommendations of this report. As the cross-section illustrates, the surface facilities site is on relatively level ground. Exile Hill immediately west of the surface facilities site slopes at about 2.5H:1V (horizontal: vertical) in its upper portion and flattens to about 6H:1V near its base adjacent to the surface facilities site. The steeper, upper portions of Exile Hill, west of the surface facilities, are composed of rock at the surface. The alluvium and colluvium constitute the flatter lower portion. Due the flatness of the adjacent alluvial/colluvial portions and the presence of rock in the upper portions, slope stability of Exile Hill is not anticipated to be a significant concern for the surface facilities site. Additional reconnaissance of the slope as recommended in Section 7.3.1 will determine if a detailed stability analysis of Exile Hill is necessary.



Source: Figure 229, BSC 2002a

Figure 7-1. WHB Area Geologic Cross Section E-E', looking South. Excerpt from BSC (2002a), Figure 229 (see Figure 6-7 for location of cross-section)

Temporary cuts in the alluvium should be no steeper than 1.5H:1V. Permanent fill slopes should be no steeper than 2H:1V. Permanent cut and fill slopes should be provided with erosion protection by placement of at least 3 inches of coarse aggregate shouldering material.

7.1.8 Pavements

The designs of all pavement sections including the gravel construction phase pavements and any heavy transport routes should be developed using an approved pavement analysis method.

California Bearing Ratio (CBR) was not directly measured on the site materials. Pavement section designs may be based on CBR values ranging between 20 and 60 percent for the alluvium and engineered fill. As discussed in Section 7.3.4, appropriate CBR testing should be performed to finalize the value chosen for final design. These are conservative values recommended for the gravelly soils in Table 1, page 7.2-39, of USN (1986).

7.1.9 Percolation Rates

The percolation rate of soils have been measured at a number of different locations in the site vicinity, including the ESF Muck Storage Area (DTN: SNF29041993001.002, average 1.8 in/hr), at 14 locations within Midway Valley (DTN: GS960908312212.009, average 2.3 in/hr), and at 2 locations within Fortymile Wash (DTN: GS950308312213.004). Based on BSC (2002c), these sources are generally consistent with the percolation rate of about 1.8 inches per hour.

An estimate of the permeability of the alluvial soils can also be made based on the fines percentage (see Section 6.4.1.1.1) and the relationship presented in Figure 8-5 of USN (1986) for the effect of fines on permeability. Based on this figure, the permeability of the alluvium can be estimated to be between $5x10^{-5}$ fpm (0.036 in/hr) and $5x10^{-4}$ fpm (0.36 in/hr).

Alternatively, Sherard, et al (1984) determined that permeability, k, for sand and gravel filters could be calculated from $k = 0.35 \times D_{15}^2$, where D_{15} (the grain size for which 15% of material is smaller by weight) = 0.08mm (see Figure 6-14 and Figure 6-15) and k is in cm/sec. This results in a permeability of about $2.24x10^{-3}$ cm/sec = 3.2 in/hr. This estimate, and consideration that many of the tests performed in the three above-referenced DTN's of field percolation and double ring infiltrometer tests obtained higher values than the recommended 1.8 in/hr, would indicate that a reasonable range of permeability to be between about $5x10^{-5}$ fpm (0.036 in/hr) and $2x10^{-3}$ fpm (2.8 in/hr). Also, note that permeability can be highly variable in the alluvial materials which also include localized layers or zones of relatively impermeable caliche.

Care should be taken to use the proper test for the intended design function and add appropriate factors of safety. These numbers can also be refined by performing additional field percolation tests (ASTM D 5126) or laboratory constant-head permeability tests on reconstituted samples (ASTM D 2434).

7.1.10 2000 International Building Code (IBC) Soil Type

Using the averaged shear wave velocities developed in BSC (2008a) and reported in Table 2-1, a Soil Profile Type from the 2000 International Building Code (IBC) was selected to characterize the dynamic soil properties of the surface facilities area (ICC 2000, Table 1615.1.1).

Table 2-1 summarizes the soil profile type determined for the soil units at the site.

7.1.11 Frost Penetration

Figure 7-16 (Figure III-1 of BSC 2002b) below shows the potential frost penetration for the western United States. Based on this map, the potential for frost penetration at the YMP site is approximately 10 inches. Use 10 inches for design purposes.

7.1.12 Liquefaction Potential

As discussed in Section 6.1.4.4, groundwater is located 1270 feet below the ground surface. Therefore, there is no potential for liquefaction to occur beneath the planned structures at the site.

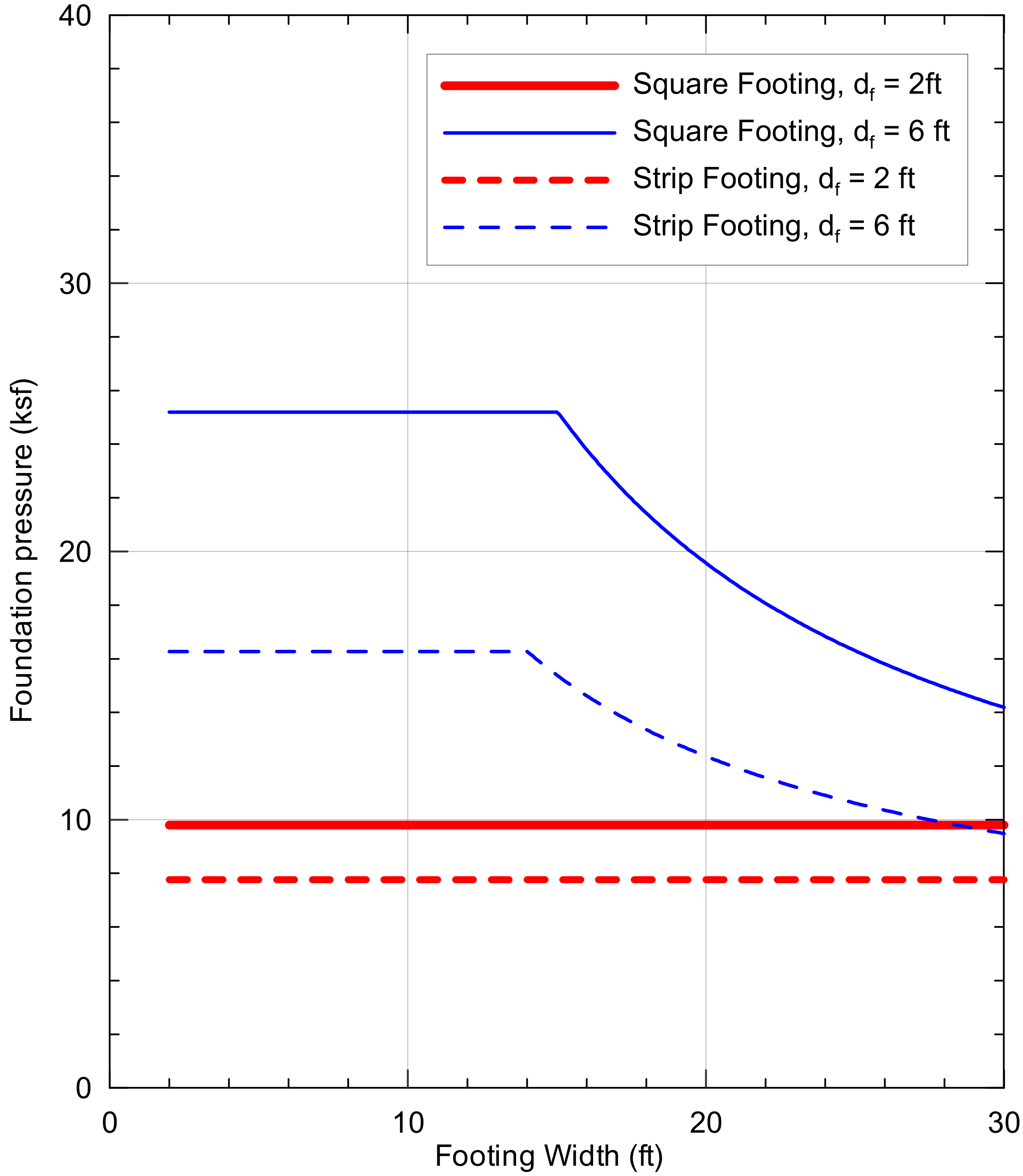


Figure 7-2. Allowable foundation pressure for square and strip footings on alluvium vs. foundation width and foundation embedment (1-inch design settlement).

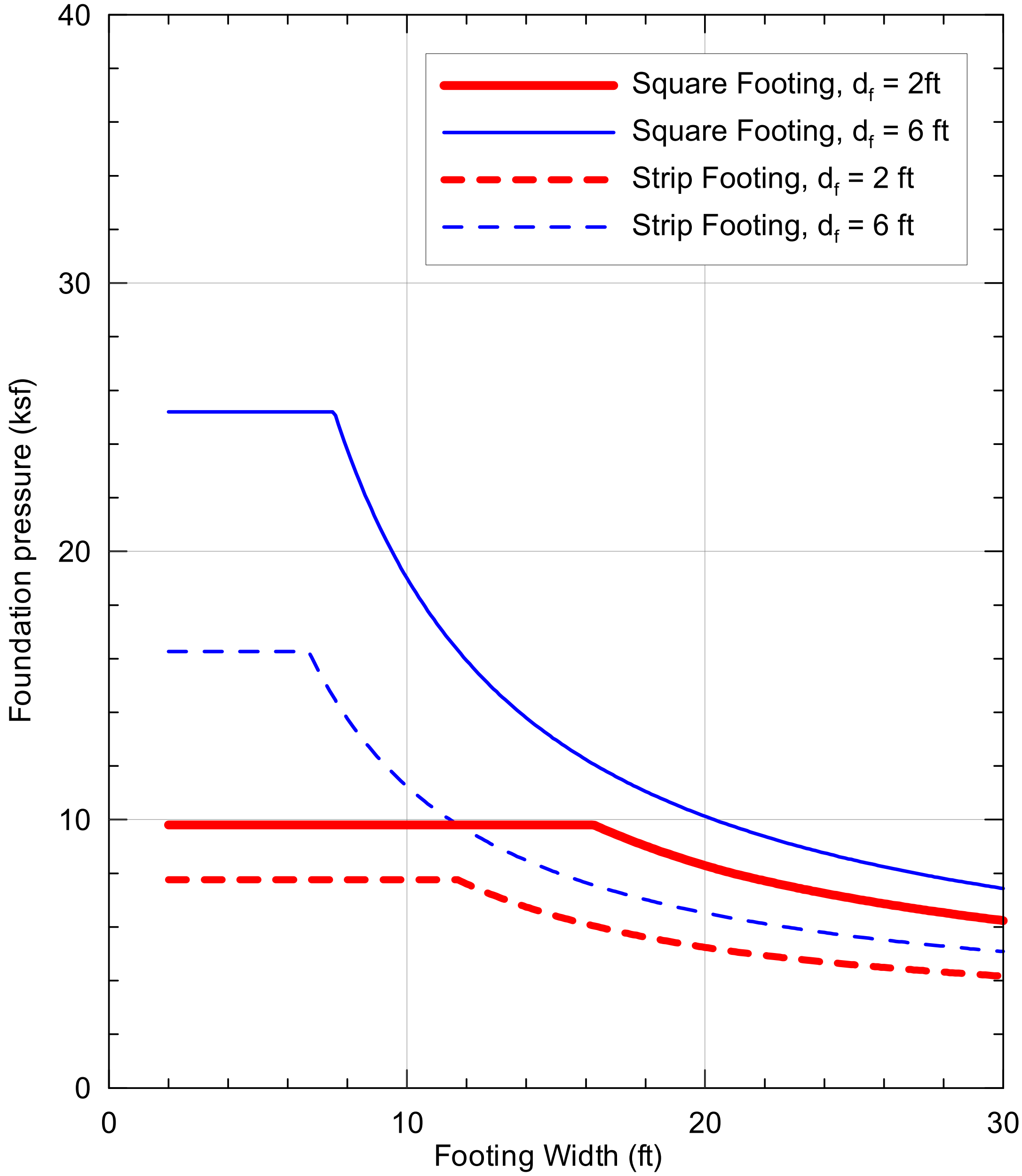


Figure 7-3. Allowable foundation pressure for square and strip footings on alluvium vs. foundation width and foundation embedment (½-inch design settlement).

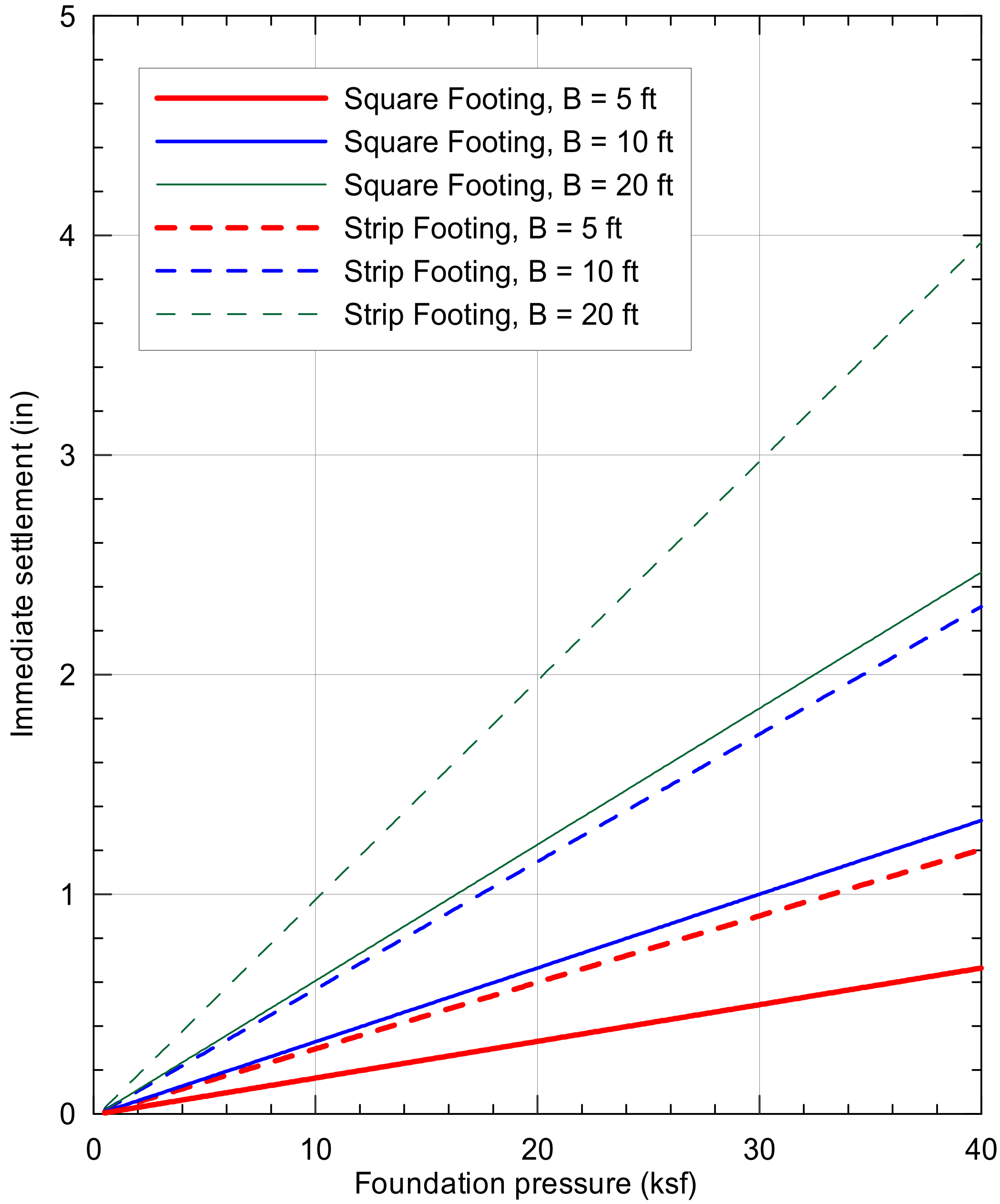


Figure 7-4. Immediate settlements for different widths of square and strip footings on alluvium vs. foundation pressure (d_f = 2 ft)

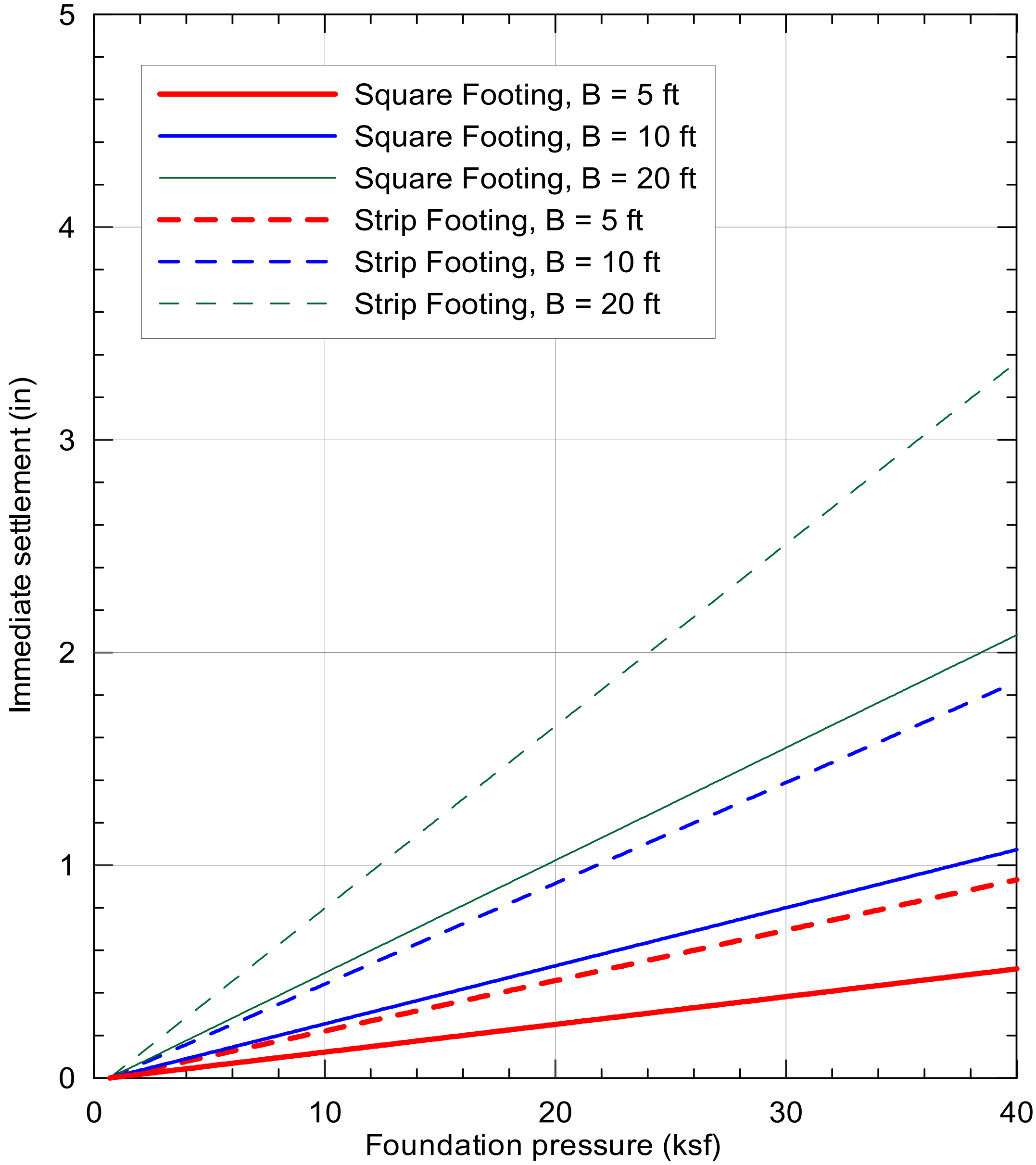


Figure 7-5. Immediate settlements for different widths of square and strip footings on alluvium vs. foundation pressure (d_f = 6 ft).

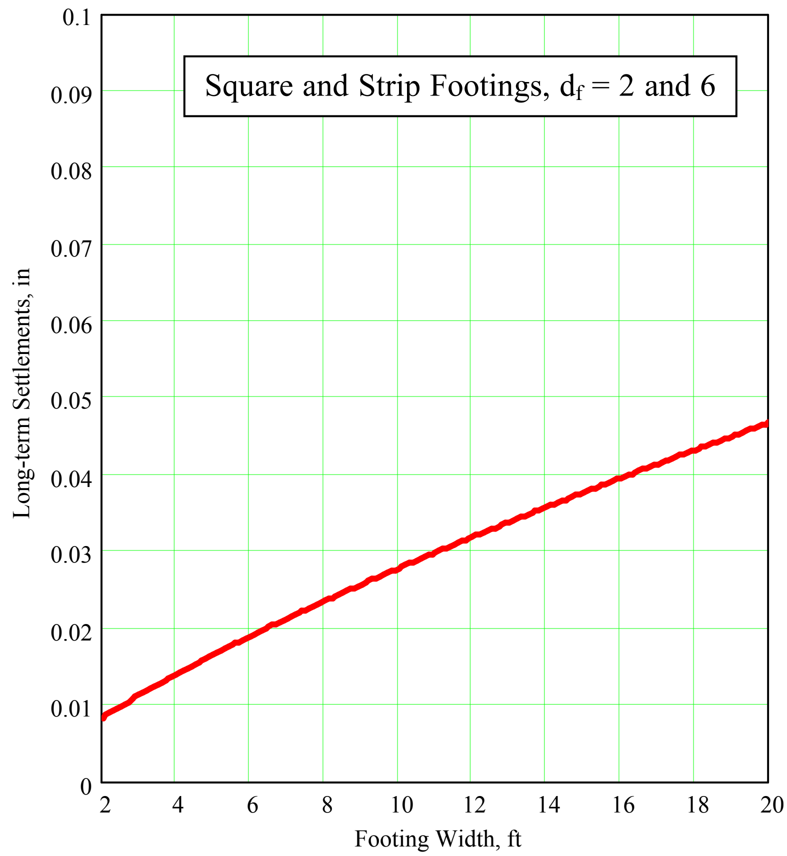
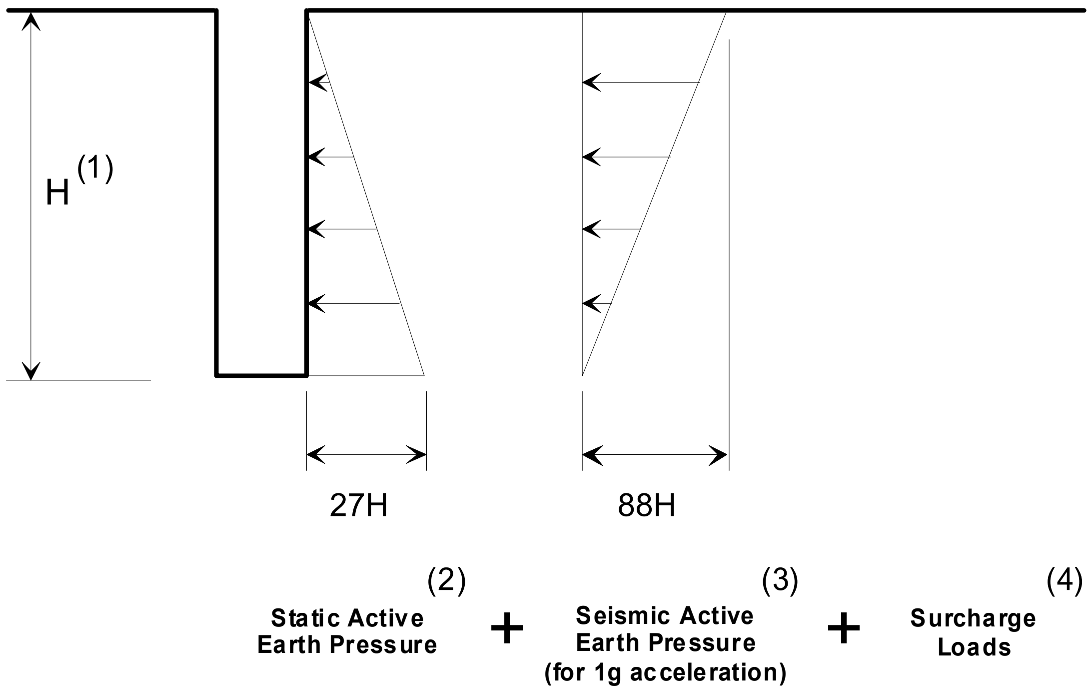


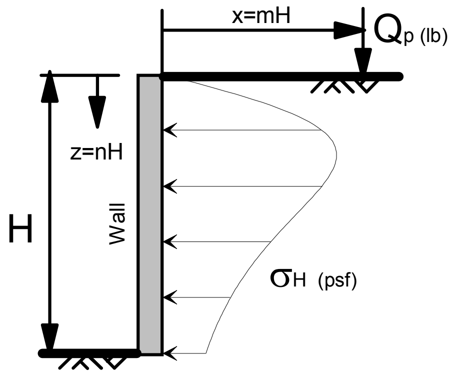
Figure 7-6. Long-term settlements for square and strip footings and different depths of foundation embedment.



Notes:

(1) Height of wall, H, is presented in feet.
(2) Static active earth pressure for alluvium: $K_A = 0.23$, $\gamma = 117$ pcf.
(3) Seismic active earth pressure for alluvium based on Seed and Whitman (1970) simplified method where $K_h = 1g$ (to be scaled by actual peak ground acceleration, PGA).
(4) Surcharge loads are shown in next figure.
(5) Pressures are presented in psf.

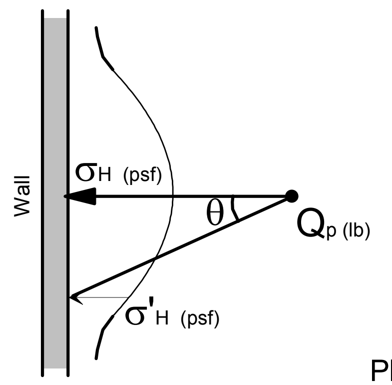
Figure 7-7. Lateral earth pressures for yielding walls



$$\sigma_H = 0.28 \frac{Q_p}{H^2} \frac{n^2}{(0.16+n^2)^3} \quad (\text{for } m \leq 0.4)$$

$$\sigma_H = 1.77 \frac{Q_p}{H^2} \frac{m^2 n^2}{(m^2+n^2)^3} \quad (\text{for } m > 0.4)$$

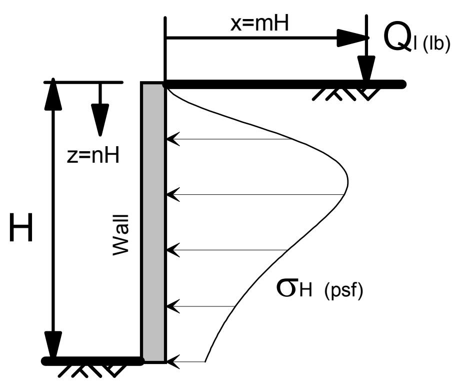
Elevation View



$$\sigma'_H = \sigma_H \cos^2(1.1\theta)$$

Plan View

Lateral Pressure due to Point Load



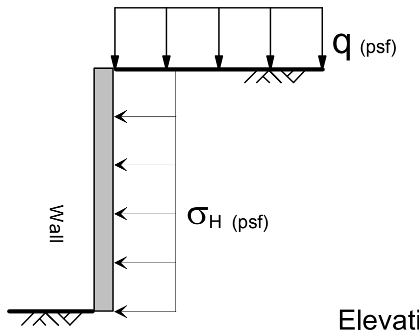
$$\sigma_H = 0.20 \frac{Q_l}{H} \frac{n}{(0.16+n^2)^2} \quad (\text{for } m \leq 0.4)$$

$$\sigma_H = 1.28 \frac{Q_l}{H} \frac{m^2 n}{(m^2+n^2)^2} \quad (\text{for } m > 0.4)$$

Elevation View

Lateral Pressure due to Line Load

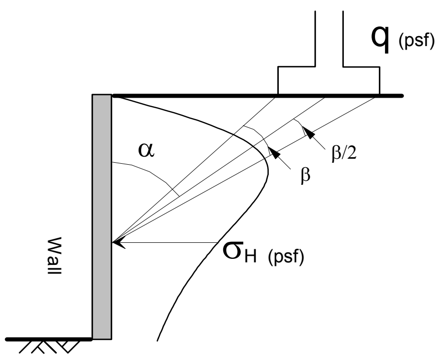
Figure 7-8. Surcharge loading for yielding walls (not drawn to scale, USN 1986)



$$\sigma_H = q\,K \text{ (psf)}$$

Elevation View

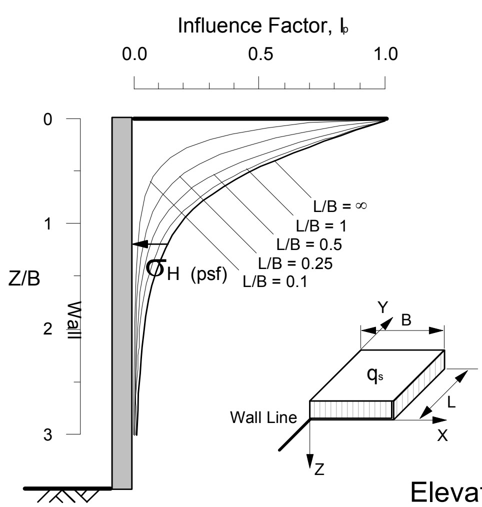
<u>Lateral Pressure due to Uniform Surcharge</u>



$$\sigma_H = \frac{2q}{\pi}\left[\beta - \sin\beta\cos2\alpha\right]$$

Elevation View

<u>Lateral Pressure due to Strip Load</u>

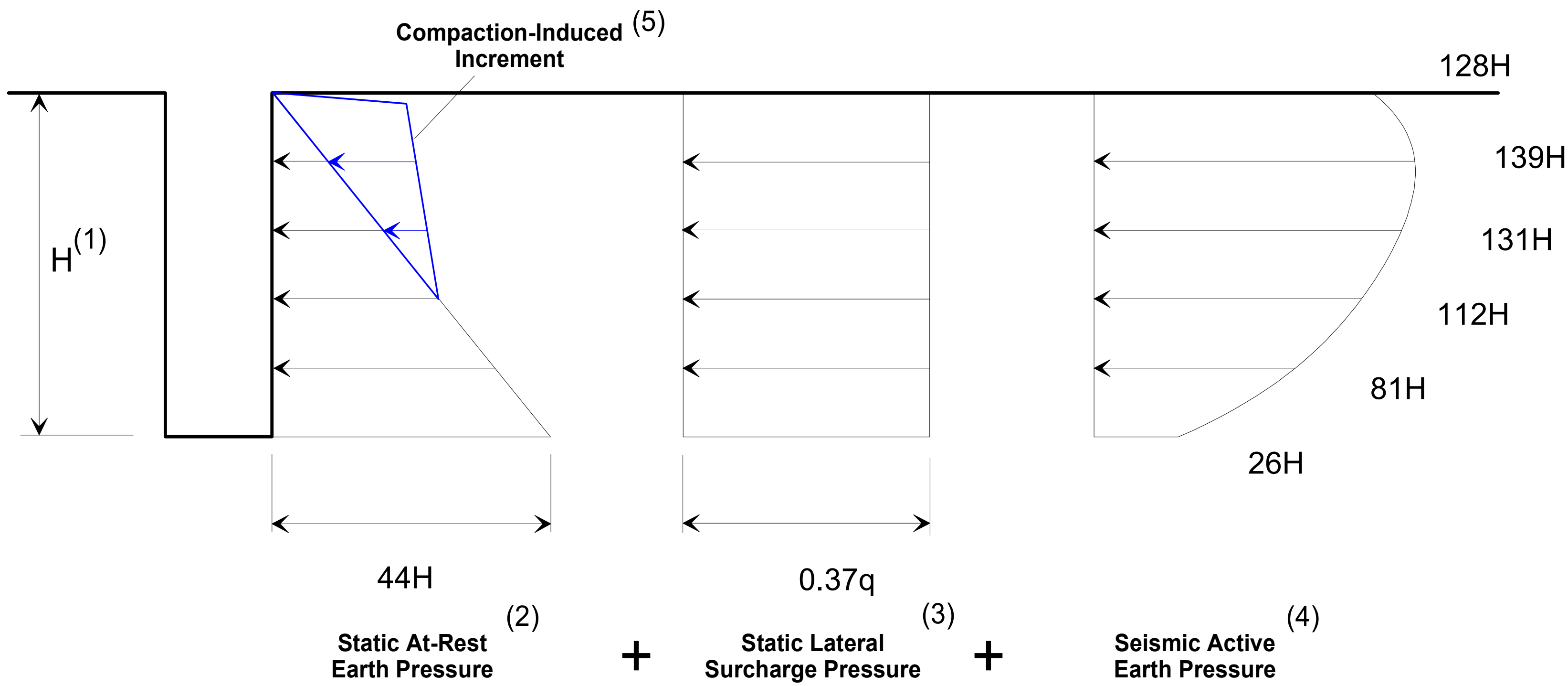


$$\sigma_H = I_p \times q_s$$

Elevation View

<u>Lateral Pressure due to Footing</u>

Figure 7-9. Surcharge loading for yielding walls, continued (not drawn to scale, USN 1986)



Notes:

(1) Height of wall, H, is presented in feet.
(2) Static at-rest earth pressures for alluvium: K_o = 0.37, γ = 117 pcf.
(3) Static lateral surcharge pressure based on K_oq, where q is surcharge to be determined.
(4) Seismic active earth pressure based on methods from ASCE 4-98 (2000), where k_h = 1g (to be scaled by actual peak ground acceleration, PGA);
Does not include dynamic contribution due to surcharge load. For dynamic contribution increase H to match surcharge pressure at surface, then redistribute stresses from Case (4) over H.
(5) Compaction-induced pressure increments for specific compaction equipment provided in the next following figures.
(6) Pressures are presented in psf.

Figure 7-10. Lateral earth pressures for non-yielding walls.

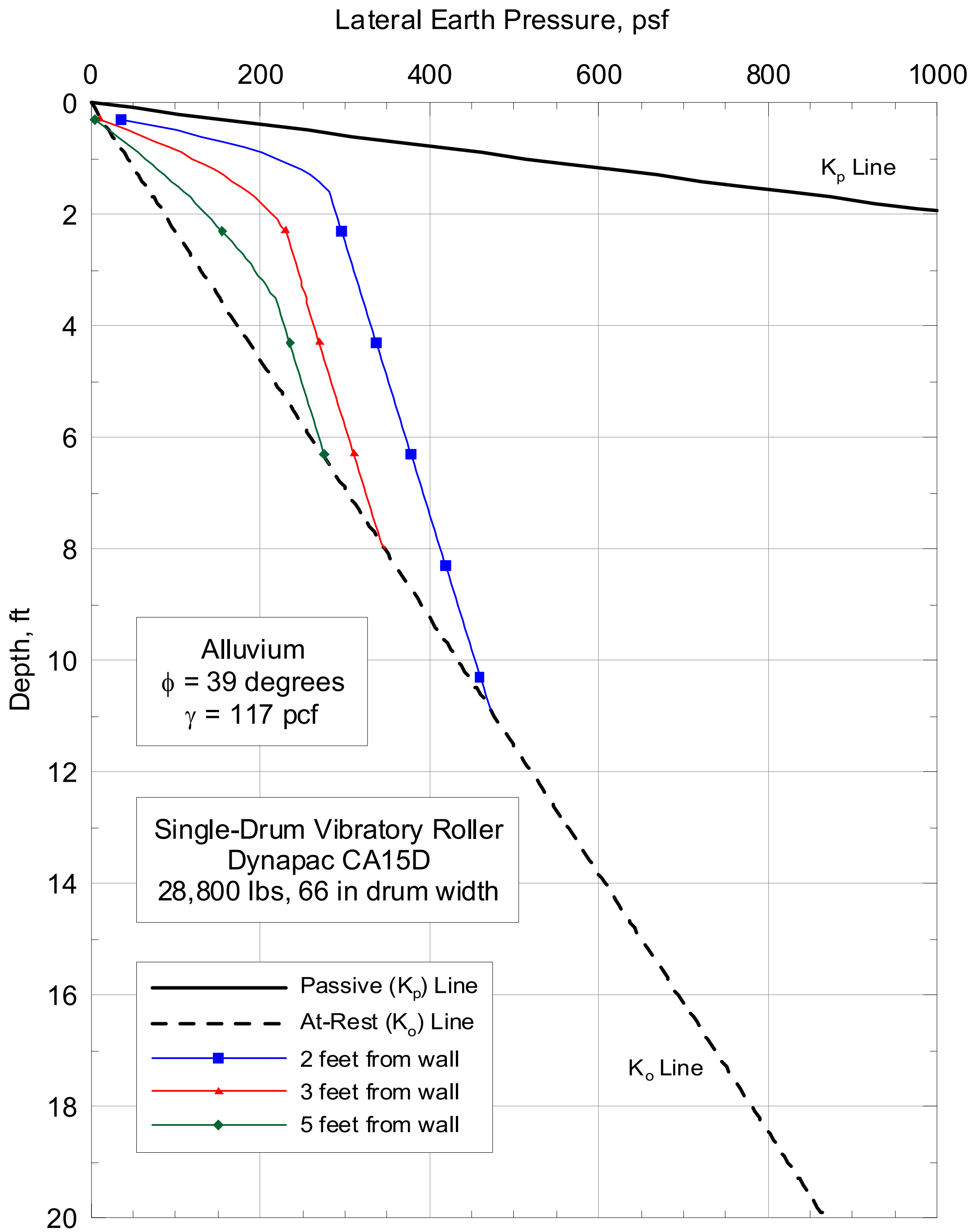


Figure 7-11. Compactor-induced pressures from roller compactor (Compactor model: Dynapac CA15D)

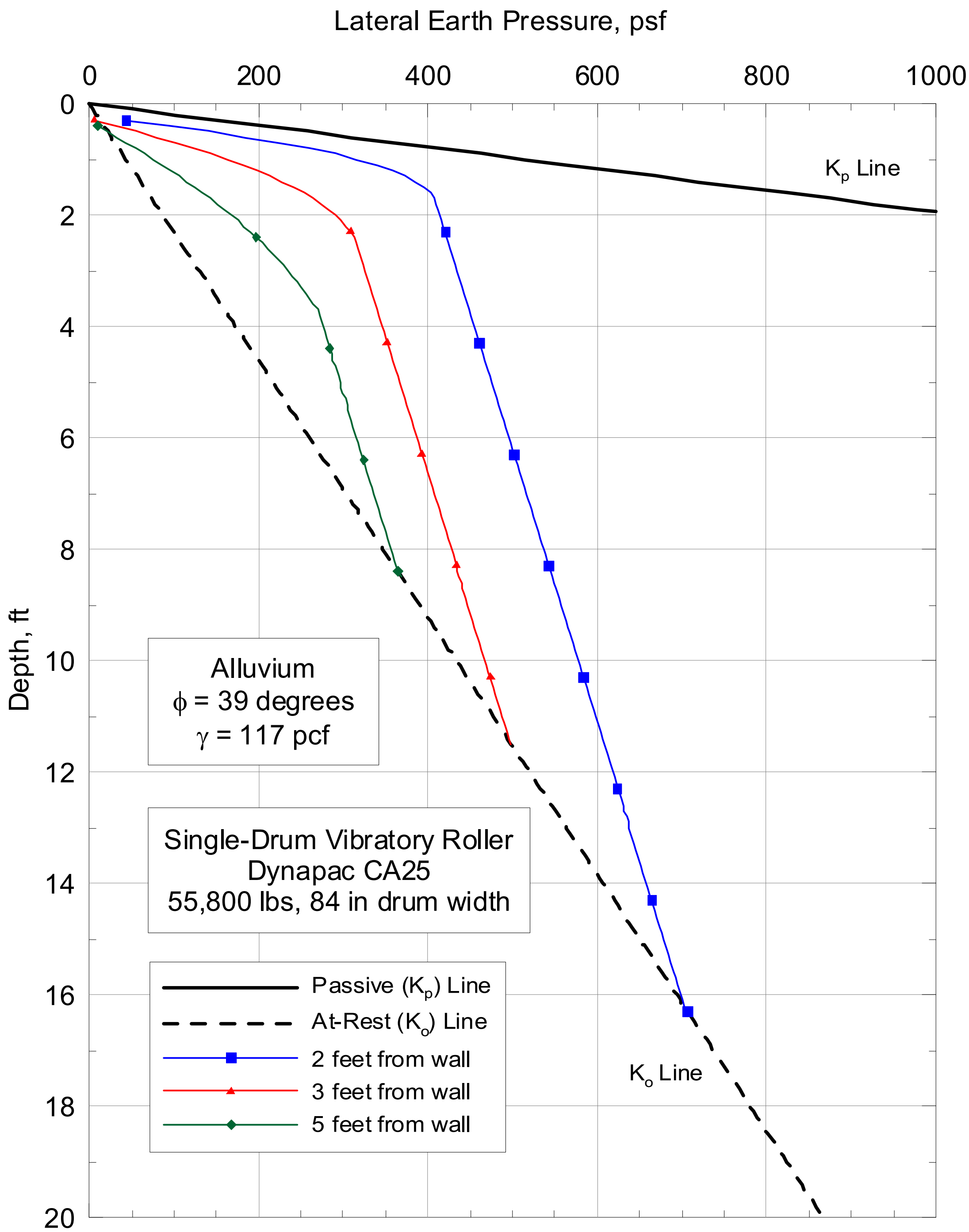


Figure 7-12. Compactor-induced pressures from roller compactor (Compactor model: Dynapac CA25)

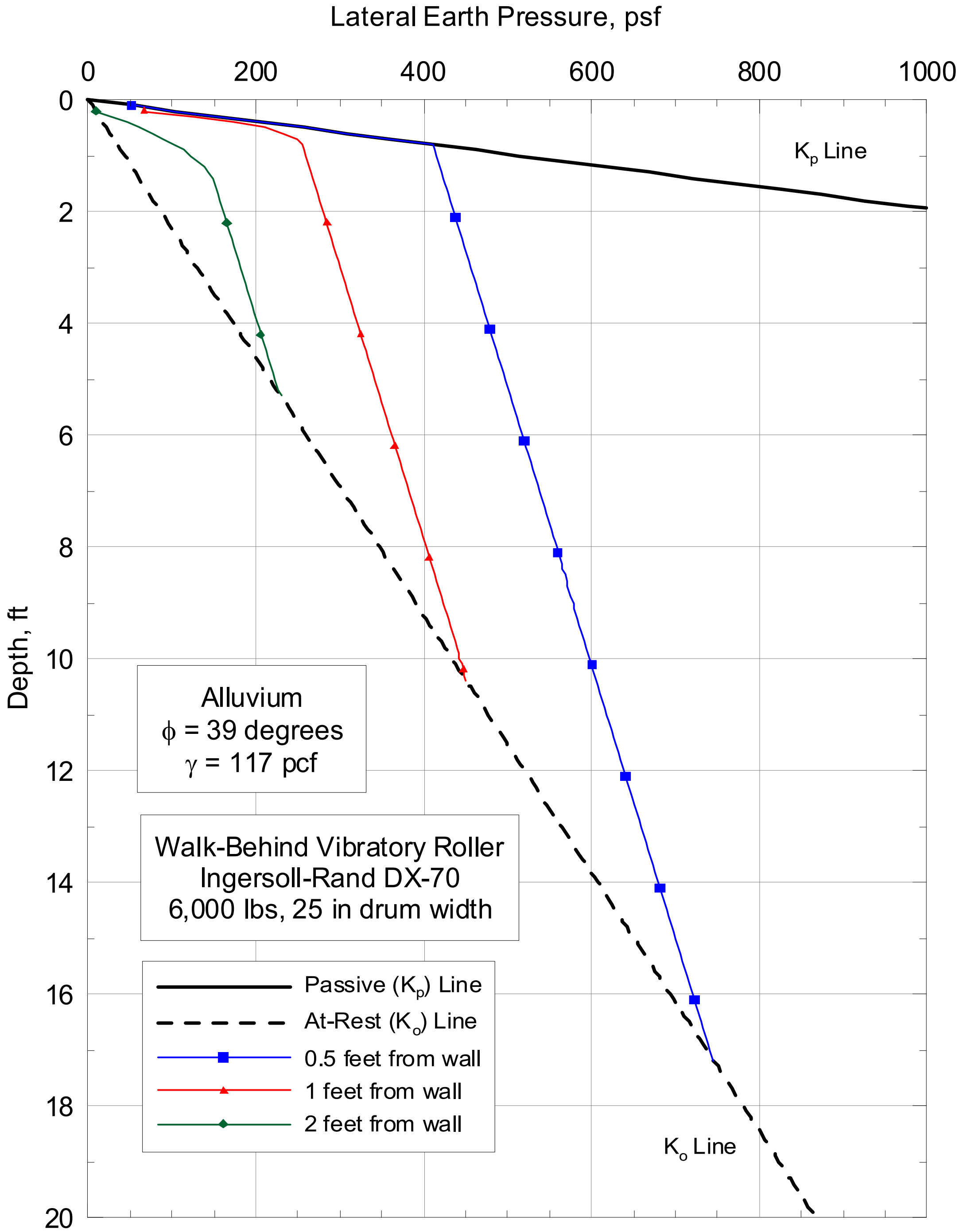


Figure 7-13. Compactor-induced pressures from roller compactor (Ingersoll-Rand DX-70).

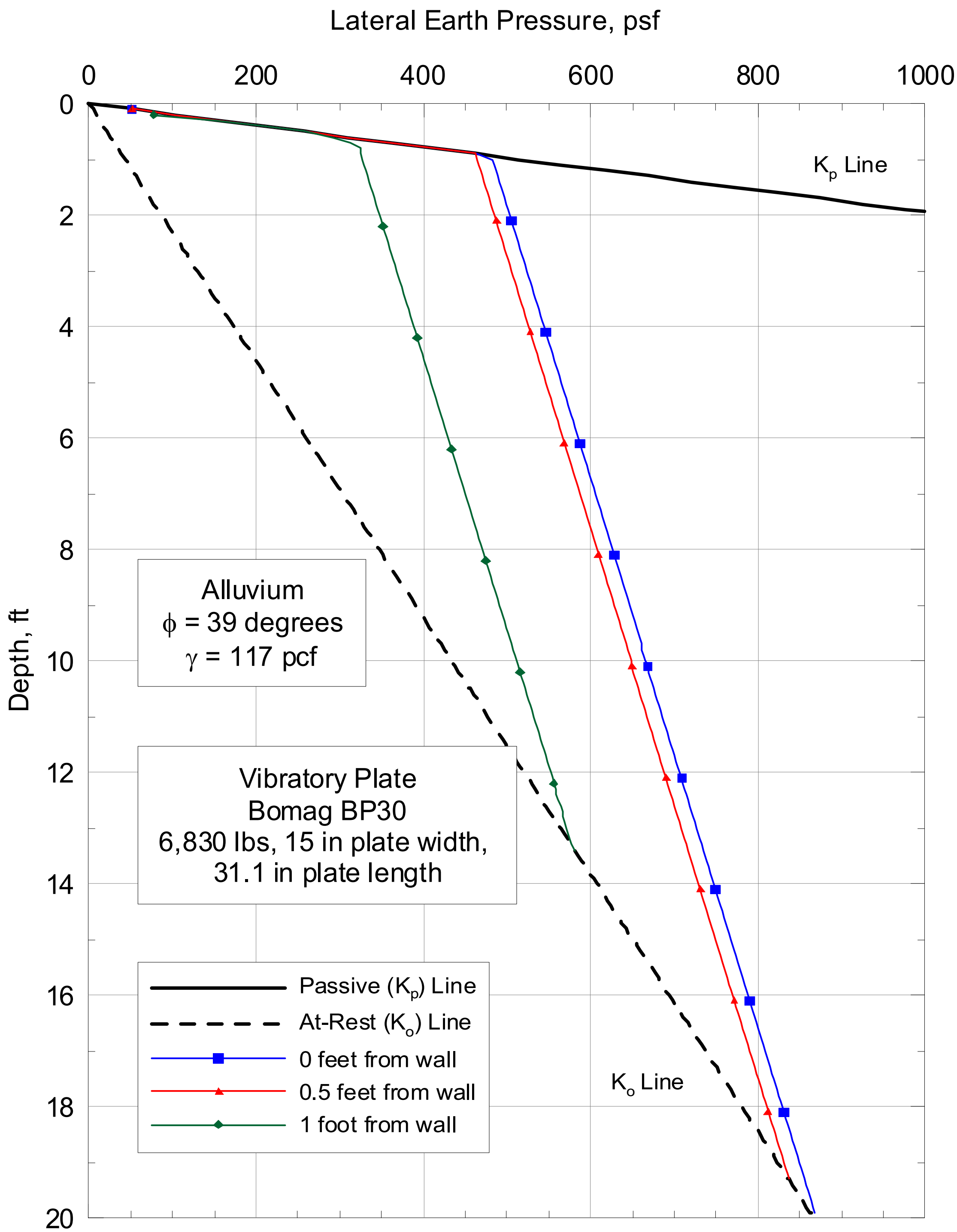


Figure 7-14. Compactor-induced pressures from plate compactor (Bomag BP30).

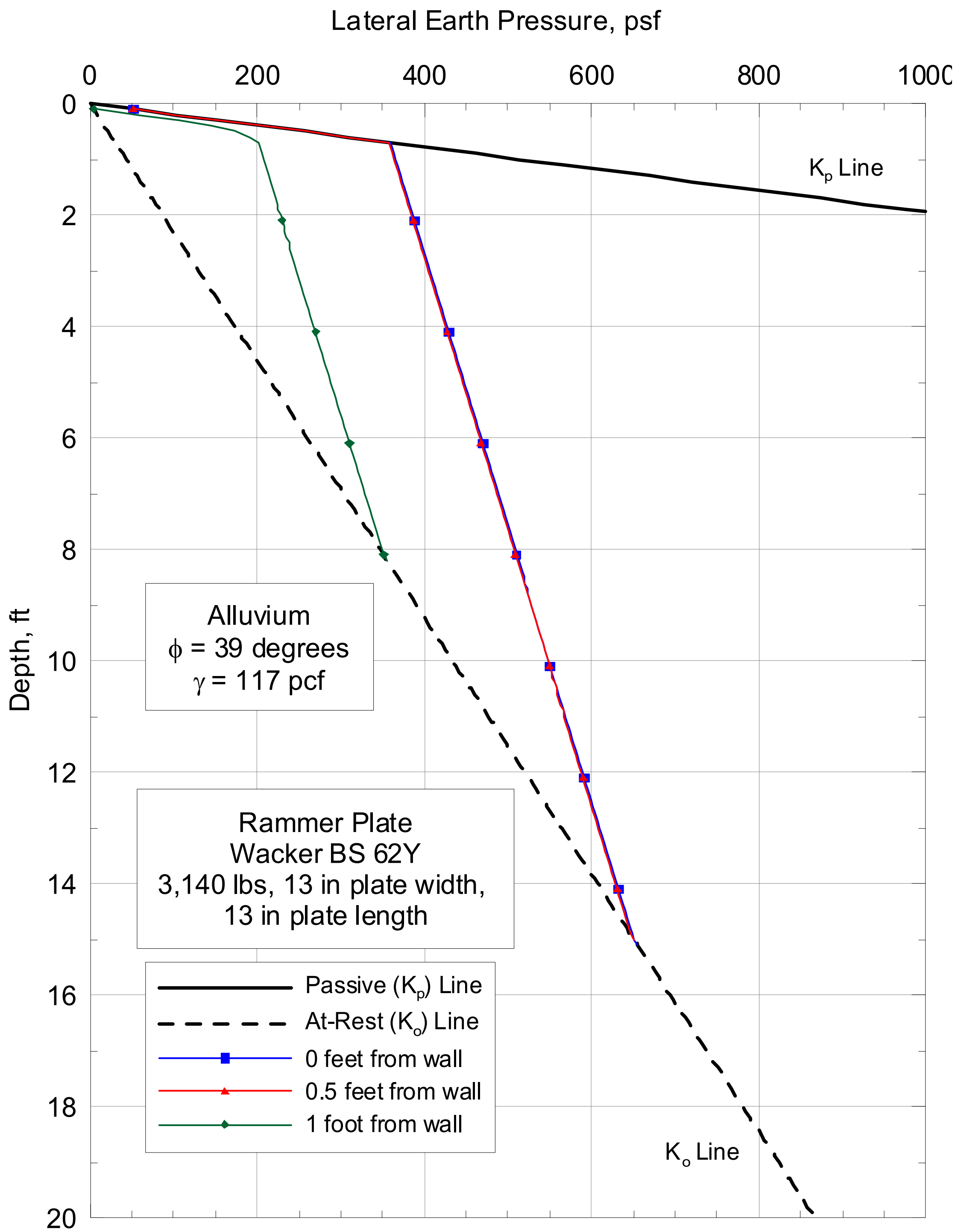


Figure 7-15. Compactor-induced pressures from plate compactor (Wacker BS 62Y).

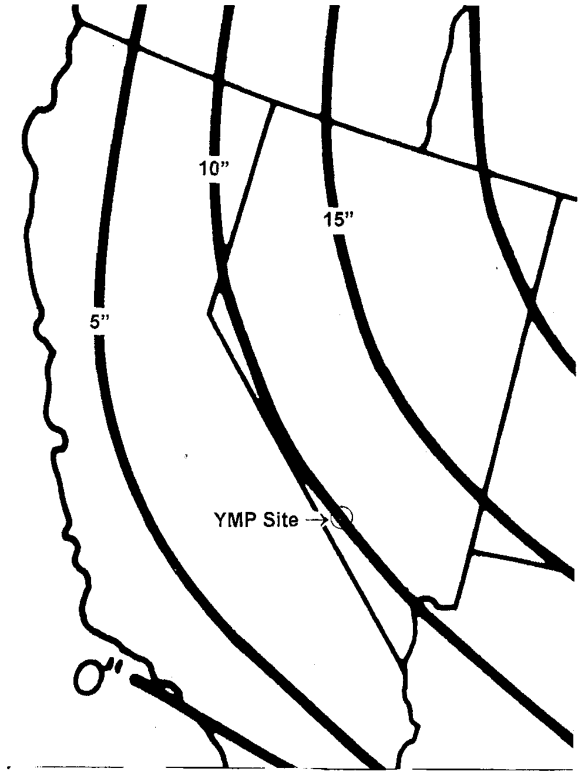


Figure 7-16. Extreme frost penetration (inches) at the North Portal Area (Figure III-1 BSC 2002b).

7.2 CONSTRUCTION CONSIDERATIONS

7.2.1 Stripping and Site Preparation

Portions of the site are currently covered with 5 to 50 feet of uncontrolled sand and gravel fill. All fill in the building areas should be removed down to the top of alluvium. Any preexisting organic materials and roots, if any, encountered at the top of the alluvium should be stripped at each of the structure sites. It is expected that no more than 6 inches of stripping of the original native surface would be needed to remove the organic materials and roots. In areas with preexisting heavy sagebrush growth, additional stripping may be required to remove the deep roots.

The excavated areas should extend outside the footing line for a distance equal to at least 1/2 the depth of the excavation up to a maximum of 5 feet outside the footing line.

The top 12 inches of the exposed subgrade surface should then be compacted to an in-place density of at least 95 percent of the maximum laboratory dry density as determined by ASTM D 1557.

The structural fill may consist of the excavated soil, or imported fill. Imported structural fill should consist of 5/8-inch minus crushed base course or 2-inch minus pit run gravel with less than 5 percent fines (minus U.S. No. 200 sieve size). All structural fill beneath or around structures should be compacted to an in-place density of at least 95 percent of the maximum laboratory dry density as determined by ASTM D 1557.

7.2.2 Foundations

All foundations should be buried a minimum of 2 feet below the ground surface. This depth exceeds the design depth of freeze of 10 inches (see Section 7.1.11), with some allowance for loss of ground around footings.

7.2.3 Excavation, Backfill and Temporary Shoring

It is recommended that all excavations be made as open excavations, with side slopes no steeper than 1.5 Horizontal to 1 Vertical (1.5H: IV). However, recognizing that some elements of certain structures may be as deep as 50 feet or more below the existing surface elevation, a combination of open cut and shoring may be necessary for those particular features. Temporary shoring to support these excavations may be designed based on the soil properties indicated in Table 2-1.

Consistent with conventional practice, actual temporary excavation slopes should be made the responsibility of the construction contractor. The construction contractor is able to observe the nature and conditions of the subsurface encountered and has the responsibility for methods, sequence, and schedule of construction. If instability is detected, slopes should be flattened or shored. All temporary excavation slopes should be accomplished in accordance with all local, state, and federal safety regulations. Excavation slopes and shoring may be designed using the soil properties shown in Table 2-1. Shoring systems, if used, should be monitored for vertical and lateral movement during construction to confirm that movements are contained within allowable limits.

The granular soils observed in the explorations can be excavated using conventional equipment such as scrapers or rubber-tired or tracked hydraulic backhoes. Excavation in most of the site soils is not expected to require any unusual equipment or procedures. Any cobbles observed in the excavations should be removed from any excavated soils that will be used as backfill. No cemented layers were identified that would require special construction equipment or techniques.

7.2.4 Excavations for Underground Utilities

Backfill above and around underground utilities should be compacted to an in-place density of at least 95 percent of the maximum laboratory dry density as determined by ASTM D 1557. Moisture content of backfill materials should be within ± 3 percent of optimum.

As an alternative to conventional trench backfilling, encasement of the conduit in controlled density fill (CDF) may be used. CDF used for pipe bedding or backfill should have a 28-day compressive strength between 50 and 200 psi (ACI 230.1R-90).

Consistent with conventional practice, actual temporary excavation slopes for utility trenches should be made the responsibility of the construction contractor. The construction contractor is able to observe the nature and conditions of the subsurface materials encountered and has the responsibility for methods, sequence, and schedule of construction. If instability is detected, slopes should be flattened or shored. All temporary excavation slopes should be accomplished in accordance with local, state, and federal safety regulations.

7.2.5 Temporary and Permanent Slopes

Temporary cut slopes should be constructed with slopes no steeper than 1.5H: IV. Fill slopes should be no steeper than 2H: 1V. This recommendation is in conformance with the Project Design Documents (BSC 2007a, Section 4.2.1.7).

Permanent cut and fill slopes should be provided with erosion protection by placement of at least 3 inches of coarse concrete aggregate.

7.2.6 Compaction

All foundation stabilization, structural fill, utility bedding, and foundation and trench backfill materials should be compacted to an in-place density of at least 95 percent of the maximum laboratory dry density as determined by ASTM D 1557. Moisture content should be controlled to be within ±3 percent of optimum.

In general, the thickness of backfill layers before compaction should not exceed 12 inches for heavy compactors and 8 inches for hand-operated mechanical compactors.

7.2.7 Suitability of On-site Materials

7.2.7.1 Structural Backfill

Based on field descriptions and laboratory testing of the alluvial materials encountered in the test pits performed at the surface facilities and the Fran Ridge borrow area (Sections 6.2.4 and 6.2.9 of BSC 2002a), these materials are suitable for use as structural backfill provided that material larger than 3 inches are removed and that a suitable means is used to test the materials for quality control purposes. Backfill placed around structures should be placed in lifts not to exceed 12 inches loose depth for heavy compactors and 8 inches for hand-operated mechanical compactors, and compacted to an in-place density of at least 95 percent of the maximum laboratory dry density as determined by ASTM D 1557.

7.2.8 Concrete Aggregates

Based on gradation tests performed on the alluvial materials encountered in the test pits performed at the surface facilities and the Fran Ridge borrow area (Section 6.5.2 of BSC 2002a), materials encountered at the site are not suitable for use as concrete aggregates without processing. The unprocessed materials contain too many large size particles. Processing these deposits to produce acceptable concrete aggregate is expected to be cost-prohibitive. However, if ballast is also processed on site, the additional processing required for concrete aggregate may become more viable.

7.2.9 Volume Coefficients

Based on density test results compiled in BSC (2002b), Section I.2.1, the mean moist unit weight of the in situ alluvium is between 114 and 117 pcf. The maximum dry unit weight for tests on Fran Ridge borrow material (also composed of alluvial soils) as reported in BSC (2002b), Section 8.1.1, was 114.5 pcf with a moisture content of 11%. Adjustments for the large particle sizes and for a moisture content one percentage point higher than the optimum resulted in a maximum estimated moist unit weight of 128 pcf (Section I.1.1 of BSC 2002b).

Therefore, assuming that the Fran Ridge material physical characteristics are similar to the in situ alluvium, the in-place relative compaction of the alluvium is estimated to be about 114/128 = 89% of its maximum value. Compaction of the excavated alluvium to 95% of its maximum dry density will result in a denser material that is smaller in volume. The difference involved with this process is therefore (114-128)/114 = -11%, or 11% percent shrinkage. Due to local variability in gravel content additional testing during construction will be necessary to determine the actual shrink or swell factors for the particular blend of materials.

7.2.10 Surface and Storm Water Drainage

Surface drainage should provide positive drainage of surface storm water away from the structures and pavement areas. It is expected that storm water disposal may utilize conventional drywells installed within the alluvium. However, cementation in the alluvium may decrease the effectiveness of drywells and additional studies and analysis would be required for this approach.

Infiltration testing is recommended for the alluvium. A factor of safety of at least 3.0 should be applied to the measured rate to accommodate plugging over time.

7.2.11 Septic System Drain Field

The septic system drain field should be designed in accordance the state and local requirements. The septic system should be designed using the average measured infiltration rate at 4 feet below the existing surface elevation in the alluvial materials. Current design standards allow septic systems to be designed based on actual infiltration rates without application of a factor of safety. Because of the expected heavy usage, provisions for reserve capacity should be included in the septic system drain field design.

7.2.12 Wet Weather Construction

Because of the granular nature of the soils at this site and the general environment of the site, wet weather construction should not be a major concern. Mitigation measures to reduce the potential impact of occasional storms would include providing positive drainage to direct storm water away from excavations and work zones. Effective maintenance of access roads and staging areas will also reduce the impact of an occasional storm.

7.2.13 Dewatering

Because of the depth of the groundwater, over 1000 feet below the ground surface elevation, dewatering is not a significant concern at this site.

7.3 ADDITIONAL INVESTIGATIONS/TESTING

The current information is considered sufficient for design of the surface facilities. Additional field (drilling or test pits) and/or laboratory testing may be needed for non-waste handling facilities or roadways, or if there are additions/alterations to the current surface facilities configuration. The following is a list of items that may be required to finalize design for the YMP facilities at the North Portal area.

7.3.1 Test Pits and Geologic Reconnaissance

Additional shallow test pits may be needed on Exile Hill to better characterize the depth to rock for embankment cuts and road design. Test pits may also be needed to determine localized undocumented fill depths in some of the building areas, primarily for bid purposes.

7.3.2 Borings

Any additional borings should be performed using mud rotary, hollow-stem auger, or air-drill techniques. Sampling would generally involve 3-inch diameter heavy-duty samplers along with 2-inch diameter SPT. Any encountered soft zones would be sampled with 3-inch diameter thin-walled Shelby tubes. Borings should extend about 15 feet into rock. Therefore, coring capability will also be needed. The cone penetrometer test (CPT) is not an option due the amount of gravel present.

7.3.3 Laboratory Testing

Laboratory testing associated with the borings would consist of gradation, Atterberg limits, direct shear, moisture and density, relative density tests, and possibly large diameter triaxial testing.

7.3.4 CBR Testing

California bearing ratio tests are needed on the alluvium and anticipated fill sources for pavement design.

7.3.5 Field Plate Load Tests

Plate load tests should be conducted on undisturbed soils in the test pits to define the elastic parameters of the alluvium and the fill source.

7.3.6 Resistivity Testing

Field electrical resistivity tests should be performed on the alluvium and fill source materials for electrical grounding design and evaluation of corrosion potential of buried metal pipes.

7.3.7 Aggregate Testing

Qualification of on-site or local aggregate will require testing for use as backfill and under pavements. Required tests include specific gravity, absorption, degradation, and soundness.

7.3.8 Ballast Testing

Additional aggregate testing suites, as described in Section 7.3.7, would be needed to evaluate tunnel muck cuttings for use as ballast when suitable samples become available.

7.3.9 Chemical Testing

Laboratory pH, chloride, sulphate tests will be needed to evaluate corrosion potential for metal pipes from alluvium and fill.

7.3.10 Test Fill Program

A test fill program should be performed to evaluate the in situ engineering properties of engineered fill, including its shear-wave and damping properties, and to determine the effect of construction equipment on the material. The test pad would also be used to establish relationships between the various density testing methods (i.e., nuclear, sand cone, and relative density).

7.3.11 Pavement and Railroad Subgrade Design

Design of temporary construction roads, operational pavements, railroad subgrade (and any special purpose roads, such as for heavy transport vehicles) may require additional field and laboratory tests.

7.3.12 RCSC Testing

If roller compacted soil cement is considered for use at the site, an additional testing program will be required to develop design parameters as discussed in Section 6.4.3.2.

APPENDIX A – Not Used

Appendix B Contents

Page Number

B1 Objective

This calculation documents the alluvium bearing capacity and short-term settlement analyses for shallow footings and mat foundations at the surface facilities area at the Yucca Mountain Project (YMP) site.

Design charts for allowable foundation pressure for square and strip footings are provided. The recommended foundation pressures consider maximum allowable bearing capacity and maximum permissible foundation settlement.

Short-term settlement evaluations under the center and corner of mat foundations are also considered in these analyses.

B2 Inputs

The following input data is required to perform the analyses:

B2.1 Foundation Geometry

Footings with widths ranging from 2 to 30 feet and foundation embedment depths of 2, 4, and 6-feet are considered in the analyses for bearing capacity and settlement analyses of shallow footings.

Footing widths

$B_0 := 2\text{ft}$ Minimum footing width

$\Delta B := 0.1\text{ft}$ Footing width increment

$B_f := 30\text{ft}$ Maximum footing width

$B_1 := B_0 + \Delta B$

$B := B_0, B_1 .. B_f$ Footing width range

Embedment depths

$d_f := 2\cdot\text{ft}, 4\text{ft} .. 6\text{ft}$ Depth of embedment range

A square 400 feet by 300 feet mat is considered in the bearing capacity and settlement analyses for mat foundations.

B2.2 Allowable Settlements

Maximum footing and mat foundation settlements of ½ and 1 inch are considered in this calculation. A 50-year lifetime for the foundations is used to estimate long-term settlements.

$\delta_{max} := 0.5in, 0.75in .. 6.00in$	Maximum settlement for calculations.
$t := 50 \cdot year$	Lifetime of structure for long-term settlement estimate (BSC 2008b, Section 2.2.2.7)

B2.3 Soil Stratigraphy and Parameters

Based on SNL (2008a, Table 6.2-2) the subsurface conditions at the site consist of 5 to 50 feet of undocumented fill underlain by alluvial material. The surface facilities will be resting directly on the alluvial material. The undocumented fill will be removed from the surface facilities area. The alluvial material thickness varies from 2 feet up to 192 feet (Table 6-1). Bedrock is found beneath the surface deposits of fill and alluvium.

The groundwater table is located at a typical depth of 1,270 feet below the present ground surface (see BSC, 2002a, Section 6.6.3).

The following material parameters for the alluvium are considered in the bearing capacity and settlement analyses:

$\gamma := 114pcf$	Moist density (see Table 2-1 of report)
$\phi_{eff} := 39deg$	Equivalent effective friction angle (see Table 2-1 of report)
$c := 0psf$	Cohesion (see see Table 2-1 of report)

The elastic settlements of shallow footings and mat foundations are evaluated with an alluvium Young's modulus profile that is obtained from the measurements of seismic shear wave velocities. Based on Figure 6.4.2-42 of BSC (2008a), lower bound, mean, and upper bound ranges are assigned visually for use in elastic settlement analysis of mats to be used in Section B6.7.

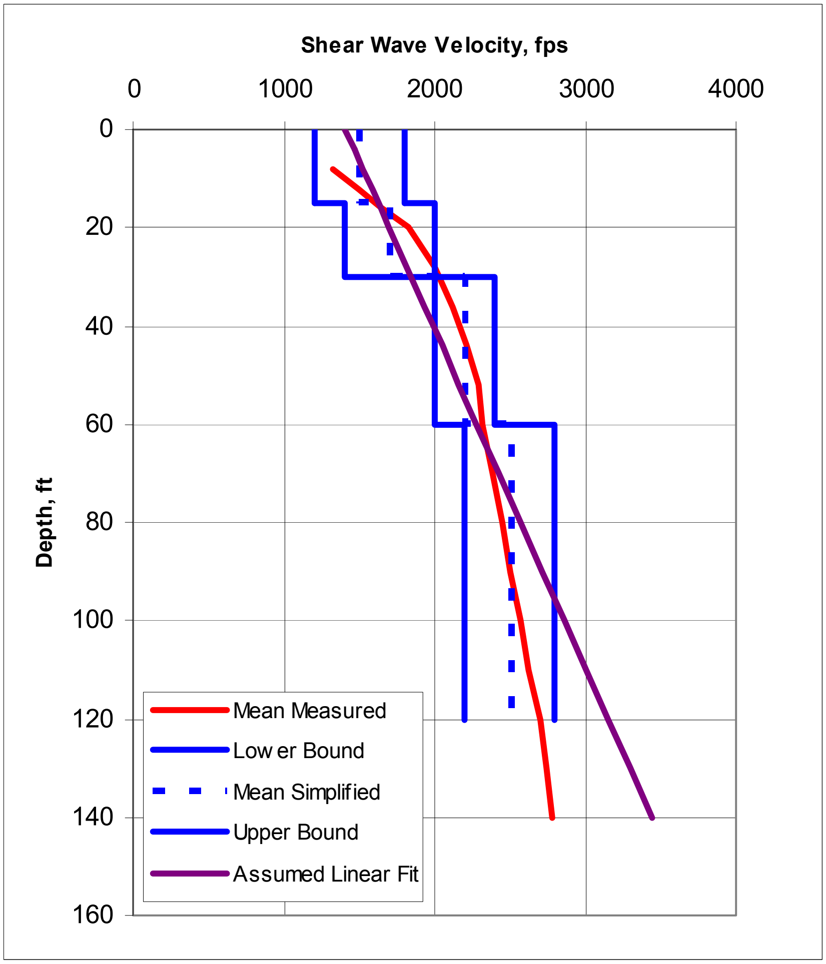


Figure B2-1. Shear wave velocity profile (based on Fig. 6.4.2-42 of BSC 2008a)

The average shear wave velocity and elastic modulus profiles are represented by the following best-fit equations:

$m_0 := 14.4 \cdot \frac{1}{s}$	Slope of equation fit
$b := 1410 \frac{ft}{s}$	Intercept of equation fit
$V(z) := m_0 \cdot z + b$	Linear fit equation for shear wave velocity vs. depth; fitted from Figure B2-1.
$\nu := 0.3$	Poisson's ratio (Figure 6.4.2-63 of BSC 2008a)

Young's modulus profile

$K := 0.1$

The fitted shear wave velocity line to obtain Young's modulus is for small strains. A reduction factor, K, of 0.1 is applied to obtain Young's modulus for large strain conditions. As demonstrated in Figure B6-19, the factor is conservative for the expected range of strains (<1%).

$G_{max}(z) := V(z)^2 \cdot \frac{\gamma}{g}$

Shear modulus (at small strains) calculated from shear wave velocity.

$E(z) := 2 \cdot K \cdot (1 + \nu) \cdot G_{max}(z)$

Young's modulus equation using linear fit shear wave velocity equation.

B2.4 Factor of Safety

A 3.0 factor of safety against bearing capacity failure of the alluvial material is implemented in the analyses to compute the allowable bearing capacity.

$FS := 3.0$

Factor of safety against bearing capacity failure

B3 Background

These analyses are the basis for recommendations and design guidelines for shallow footings and mat foundations for the surface facilities at the YMP site.

Ultimate bearing capacity values at the surface facilities area were previously presented in BSC (2002b, Section 9.2).

The current study presents shallow footings and mat foundations recommendations based on the material parameters presented in this report, Section 6.4. These recommendations are based on the field and laboratory test results reported in BSC (2002a, Section 6). These results include shear strength tests and in-situ shear wave velocity measurements in the alluvial material.

B4 Methodology

This section presents the methodology used to compute the bearing capacity and short-term settlement analyses for shallow footings and mat foundations.

B4.1 Foundation Pressures

The recommended foundation pressures for shallow footings is computed for square and strip footings and for different foundation embedment depths. These recommended pressures are limited by the following criteria:

- The recommended foundation pressure should not exceed the allowable foundation capacity that considers a factor of safety of 3.0 against the soil shear failure. This allowable value is computed using the general ultimate capacity equation reported in Bowles (1996, Table 4-1 and Table 4-5a).

- The induced footing settlements cause by the recommended foundation pressure should not exceed the maximum allowable foundation settlement. Elastic settlements are computed using the settlement analyses procedures proposed by Burland and Burbidge, and by Schmertmann et al. as reported in Terzaghi et al. (1996, Sections 50.2.5 and 50.2.6).

B4.2 Short-term Settlements for Shallow Footings

Short-term settlements of shallow foundations are computed for square and strip footings using the Burland and Burbidge, and the Schmertmann et al. methods as presented in Terzaghi et al. (1996, Sections 50.2.5 and 50.2.6). Both methods use elastic theory to evaluate immediate settlements.

The Burland and Burbidge method is based on field measurements of foundation settlements. It uses the soil average standard penetration test blow count (N_{60}) values to estimate the soil's vertical compression. The Schmertmann et al. method is based on field measurements of vertical strain beneath shallow footings. It uses the elastic soil modulus to estimate settlements.

The following discussion describes the methodology used to obtain the N_{60} values and the elastic modulus for the alluvial material to be used as input parameters in the short-term settlement estimates.

N_{60}

N_{60} results on the alluvial material are reported in only one of the exploration boreholes drilled in the WHB area. The reported values are unrealistically high and, therefore, are not used in the settlement analyses.

As an alternative to determine the N_{60} values for the alluvial material, two different procedures that correlate N_{60} values with experimental soil parameters were reviewed. The soil parameters reviewed in these correlations are as follows:

Using the correlations presented in Seed and Idriss (1970) and Seed et al. (1986), N_{60} values for the alluvial material were evaluated using the extensive seismic shear wave velocity measurements performed at the site (see BSC 2002a, Section 6). The estimated N_{60} values with these correlations were unrealistically high for the given velocity measurements and thus are not used in the settlement analyses.

N_{60} values for the alluvial material were correlated to the internal friction angle of the alluvium. The basis for the internal friction angle was from relative density measurements discussed in Section 6.4.1.1.3. The relationship proposed by Peck et al. (1974, page 310), is used to correlate N_{60} values with internal friction angle. These values were used in the short-term settlement analyses (see Section B6.2).

Young's modulus

Estimate of the soil's Young's modulus are obtained from the seismic shear wave velocity measurements performed at the site (see Figure 6.4.2-42 of BSC 2008a). The average shear wave velocity profile adopted in this calculation is presented in Section B2.3.

B4.3 Elastic Settlements for Mat Foundation

Settlements of a mat foundation on the alluvial sand were determined using elastic theory.

The stress profile under the mat was computed using a Boussinesq equation for a uniform vertical load. The incremental strain profile under the mat was computed using an iterative procedure that accounted for the degradation of Young's modulus with strain. In the iterative procedure, an initial small-strain Young's modulus was determined from the shear wave velocity profile presented in Section B2.3.

The shear modulus degradation curve for sands (Seed et al, 1986) was used to represent the Young's modulus degradation behavior of the alluvial material. For the purpose of the analysis herein, this assumption is considered conservative.

B4.4 Long-term Settlements

The Burland and Burbidge procedure was implemented to compute the long-term settlements of footings (see Terzaghi et al, 1996, Section 50.2.5). This method estimates settlements based on the soil standard penetration test blow count (N_{60}) values.

B5 Assumptions

It is conservatively assumed that bedrock is very deep and that it has no effect on the bearing capacity and settlement analyses for shallow footings and mat foundations.

Additionally, the Young's modulus, E, is assumed to degrade the same as the shear modulus, G, for sands. This yields conservative results since Poisson's ratio does not remain constant with strain. It is also assumed that there is no rock strain for the mat analysis.

No eccentric or inclined loading is considered in the analyses.

The preconsolidated characteristics of the alluvial material due to the removal of the overlaying undocumented fill is not considered in the short-term settlement analyses. This is a conservative assumption.

A 50-year lifetime for the surface structures is assumed in the long-term settlements calculations (BSC 2008b, Section 2.2.2.7).

All of these assumptions are either sufficiently conservative or represent typical standards used in the industry and do not require further verification.

B6 Calculations

Calculations were performed using Mathcad and EXCEL on a stand-alone PC. The PC is networked for printing and file storage but the programs used are loaded on the PC. These programs started and operated normally during calculation preparation.

The allowable bearing capacity results consider an adequate margin of safety against bearing capacity failure with associated tolerable footing settlement. The following schematic (Figure B6-1) for a shallow footing presents the definitions of the different symbols used in the bearing capacity and short-term settlement analyses:

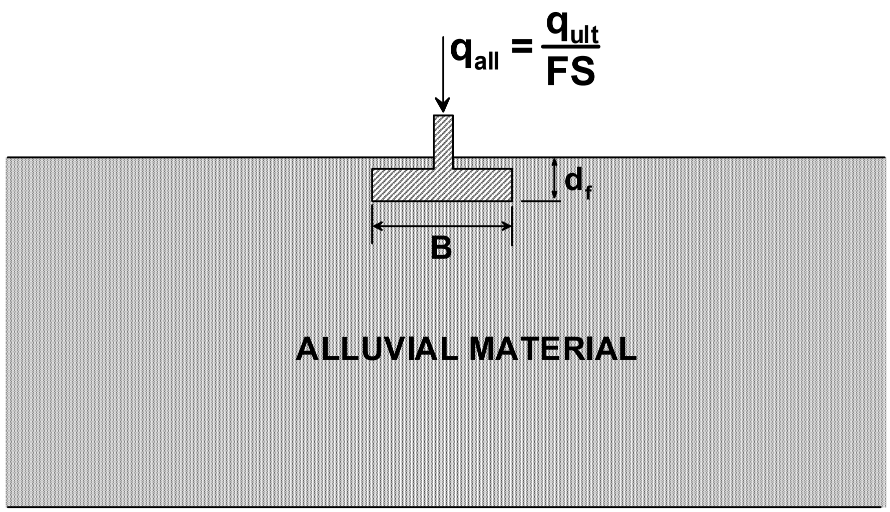


Figure B6-1. Schematic for shallow footing**.**

B6.1 Bearing Capacity for Shallow Footings

The bearing capacity of shallow footings was computed using the general ultimate capacity equation reported in Bowles (1996, Table 4-1 and Table 4-5a).

Effective overburden pressure

Check values

$q(d_f) := d_f \cdot \gamma$ $\qquad$ $q(2ft) = 228 \cdot psf$

Bearing capacity factors

Check values

$N_q(\phi) := e^{\pi \cdot \tan(\phi)} \cdot \tan\left(45deg + \frac{\phi}{2}\right)^2$ $\qquad$ $N_q(0deg) = 1$

$N_\gamma(\phi) := 2 \cdot \left(N_q(\phi) + 1\right) \cdot \tan(\phi)$ $\qquad$ $N_\gamma(0deg) = 0$

$N_c(\phi) := \begin{cases} \pi + 2 & \text{if } \phi = 0 \\ \left(N_q(\phi) - 1\right) \cdot \cot(\phi) & \text{otherwise} \end{cases}$ $\qquad$ $N_c(0deg) = 5.142$

Shape factors

Check values

Square footings

$s_{q\_square}(\phi) := 1 + \tan(\phi)$ $s_{q\_square}(0deg) = 1$

$s_{\gamma\_square} := 0.6$ $s_{\gamma\_square} = 0.6$

$s_{c\_square}(\phi) := 1 + \dfrac{N_q(\phi)}{N_c(\phi)}$ $s_{c\_square}(0deg) = 1.194$

Strip footings

$s_{q\_strip} := 1$ $s_{q\_strip} = 1$

$s_{\gamma\_strip} := 1$ $s_{\gamma\_strip} = 1$

$s_{c\_strip} := 1$ $s_{c\_strip} = 1$

Ultimate bearing capacity

Square footings

$$q_{ult\_square}(B, d_f, c, \phi, \gamma) := c \cdot N_c(\phi) \cdot s_{c\_square}(\phi) + q(d_f) \cdot N_q(\phi) \cdot s_{q\_square}(\phi) + 0.5 \cdot \gamma \cdot B \cdot N_\gamma(\phi) \cdot s_{\gamma\_square}$$

Strip footings

$$q_{ult\_strip}(B, d_f, c, \phi, \gamma) := c \cdot N_c(\phi) \cdot s_{c\_strip} + q(d_f) \cdot N_q(\phi) \cdot s_{q\_strip} + 0.5 \cdot \gamma \cdot B \cdot N_\gamma(\phi) \cdot s_{\gamma\_strip}$$

$q_{ult\_square}(10ft, 2ft, c, \phi_{eff}, \gamma) = 54638 \cdot psf$ ← Check value

$q_{ult\_strip}(10ft, 2ft, c, \phi_{eff}, \gamma) = 65339 \cdot psf$ ← Check value

Allowable bearing capacity

Square footings

$$q_{all\_square}(B, d_f, c, \phi, \gamma) := \frac{q_{ult\_square}(B, d_f, c, \phi, \gamma)}{FS}$$

Strip footings

$$q_{all\_strip}(B, d_f, c, \phi, \gamma) := \frac{q_{ult\_strip}(B, d_f, c, \phi, \gamma)}{FS}$$

$q_{all\_square}(10ft, 2ft, c, \phi_{eff}, \gamma) = 18213 \cdot psf$ ← Check value

$q_{all\_strip}(10ft, 2ft, c, \phi_{eff}, \gamma) = 21780 \cdot psf$ ← Check value

Results

Figure B6-2 presents the allowable bearing capacities for square and strip footings. Results for 2-foot and 6-foot foundation embedment depths are presented in these figures.

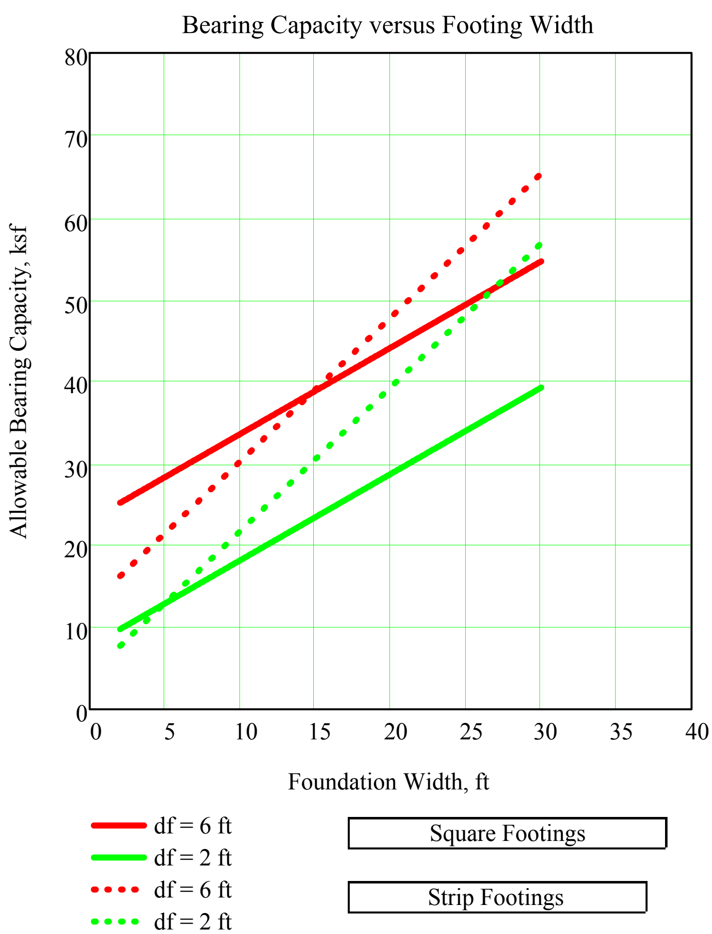


Figure B6-2. Allowable bearing pressure versus foundation width for square and strip footings

B6.2 Short-term Settlements for Shallow Footings

Short-term settlements of shallow foundations are computed for square and strip footings using the Burland and Burbidge, and Schmertmann et al. methods as presented in Terzaghi et al. (1996, Sections 50.2.5 and 50.2.6). Both methods use elastic theory to evaluate immediate settlements..

Burland and Burbidge (Terzaghi et al. 1996, Section 50.2.5) Method

N_{60}

The following equation correlates N_{60} values with ϕ. This equation is the regression curve to the chart presented by Peck et al. (1974, page 310).

Note: the computed N_{60} values are bounded to a maximum value of 60 blows per foot and a minimum value of 3 blows per foot.

$$N_{60}(\phi) := \begin{cases} .0027305858 - 17.924589\cdot\frac{\phi}{deg} + 1.4246932\cdot\left(\frac{\phi}{deg}\right)^2 \ldots & \text{if } \phi \geq 28deg \\ + -.03770745\cdot\left(\frac{\phi}{deg}\right)^3 + .00035020841\cdot\left(\frac{\phi}{deg}\right)^4 & \\ 3 & \text{otherwise} \end{cases}$$

$N_{60}(\phi) := \min(60, N_{60}(\phi))$ ⟵ Bound N_{60} to a maximum value of 60 blows per foot

$N_{60}(\phi_{eff}) = 41$ ⟵ Check value

Effective preconstruction pressure at the footing base

	Check value
$\sigma_{vo}(d_f) := d_f\cdot\gamma$	$\sigma_{vo}(1ft) = 114\cdot psf$

Zone of footing influence

The following equation corresponds to Equation 50.6 presented by Terzaghi et al. (1996, page 395).

	Check value
$Z_I(B) := \left(\frac{B}{m}\right)^{0.75} m$	$Z_I(10ft) = 2.307\,m$

Average coefficient of vertical compression

The following equation corresponds to Equation 50.7 presented by Terzaghi et al. (1996, page 395).

Check value

$$m_v(\phi) := \frac{1.7}{N_{60}(\phi)^{1.4}} MPa^{-1}$$

$$m_v(\phi_{eff}) = 0.0093 \cdot MPa^{-1}$$

Foundation length-to-width ratio

The following values are derived from Equation 50.14 presented by Terzaghi et al. (1996, page 397).

Square Footings

$$S_{c\_sq} := 1$$

Strip Footings

$$S_{c\_st} := 1.56$$

Immediate settlement equation for square and strip

The following equations correspond to Equations 50.11a and 50.11b presented by Terzaghi et al. (1996, page 396). Equation 50.11a is applicable for foundation pressures greater than the effective preconsolidation pressure. Equation 50.11b is applicable for foundation pressures less than the effective preconsolidation pressure.

Square footings

$$S_{c1\_sq}(B, d_f, c, \phi, \gamma) := \begin{cases} Z_I(B) \cdot m_v(\phi) \cdot \left[q_{all\_square}(B, d_f, c, \phi, \gamma) \ldots \atop + \left(\frac{2}{3} \cdot \sigma_{vo}(d_f) \right) \cdot (-1) \right] \cdot S_{c\_sq} & \text{if } q_{all\_square}(B, d_f, c, \phi, \gamma) > \sigma_{vo}(d_f) \\ \frac{1}{3} \cdot Z_I(B) \cdot m_v(\phi) \cdot q_{all\_square}(B, d_f, c, \phi, \gamma) \cdot S_{c\_sq} & \text{otherwise} \end{cases}$$

Strip footings

$$S_{c1\_st}(B, d_f, c, \phi, \gamma) := \begin{cases} Z_I(B) \cdot m_v(\phi) \cdot \left[q_{all\_strip}(B, d_f, c, \phi, \gamma) \ldots \atop + -\left(\frac{2}{3} \cdot \sigma_{vo}(d_f) \right) \cdot (-1) \right] \cdot S_{c\_st} & \text{if } q_{all\_strip}(B, d_f, c, \phi, \gamma) > \sigma_{vo}(d_f) \\ \frac{1}{3} \cdot Z_I(B) \cdot m_v(\phi) \cdot q_{all\_strip}(B, d_f, c, \phi, \gamma) \cdot S_{c\_st} & \text{otherwise} \end{cases}$$

$S_{c1\_sq}(5ft, 0ft, c, \phi_{eff}, \gamma) = 0.126 \cdot in$ ⟵ Check value

$S_{c1\_st}(5ft, 0ft, c, \phi_{eff}, \gamma) = 0.328 \cdot in$ ⟵ Check value

Schmertmann (Terzaghi et al. 1996, Section 50.2.6) Method

Embedment correction factor (regression equation)

This equation is the regression curve to the chart presented in Figure 50.10 by Terzaghi et al. (1996, Section 50.2.6).

$$C_1(B, d_f) := \frac{1.0561309 + 0.66610907 \cdot \left(\frac{d_f}{B}\right)}{1 + 1.2514064 \cdot \left(\frac{d_f}{B}\right) - 0.0024535149 \cdot \left(\frac{d_f}{B}\right)^2}$$

$C_1(B, d_f) := \min(1, C_1(B, d_f))$ ⟵ Bound C_1 to a maximum value of 1

Strain influence equations for square and strip footings

These equations correspond to the curves presented in Figure 50.9 presented by Terzaghi et al. (1996, Section 50.2.6) for square (L/B = 1) and strip (L/B > 10) footings. L is the footing length.

Square footings

$$I_{z\_sq}(z, B, d_f) := \begin{cases} \frac{4}{5 \cdot B} \cdot (z - d_f) + \frac{1}{5} & \text{if } z \le \left(d_f + \frac{B}{2}\right) \\ \frac{-2}{5 \cdot B} \cdot (z - d_f) + \frac{4}{5} & \text{otherwise} \end{cases}$$

Strip footings

$$I_{z\_st}(z, B, d_f) := \begin{cases} \frac{4}{5 \cdot B} \cdot (z - d_f) + \frac{1}{5} & \text{if } z \le \left(d_f + \frac{B}{2}\right) \\ \frac{-6}{35 \cdot B} \cdot (z - d_f) + \frac{24}{35} & \text{otherwise} \end{cases}$$

Immediate settlement equations

These equations represent the continuous form of Equations 50.23a and 50.23b presented by Terzaghi et al. (1996, Section 50.2.6).

Square footings

$$S_{c2a\_sq}(B, d_f) := \int_{d_f}^{d_f+2B} \frac{I_{z\_sq}(z, B, d_f)}{E(z)}\, dz$$

$$S_{c2\_sq}(B, d_f, c, \phi, \gamma) := C_1(B, d_f)\cdot\left(q_{all\_square}(B, d_f, c, \phi, \gamma) - \sigma_{vo}(d_f)\right)\cdot S_{c2a\_sq}(B, d_f)$$

Strip footings

$$S_{c2a\_st}(B, d_f) := \int_{d_f}^{d_f+4B} \frac{I_{z\_st}(z, B, d_f)}{E(z)}\, dz$$

$$S_{c2\_st}(B, d_f, c, \phi, \gamma) := C_1(B, d_f)\cdot\left(q_{all\_strip}(B, d_f, c, \phi, \gamma) - \sigma_{vo}(d_f)\right)\cdot S_{c2a\_st}(B, d_f)$$

$S_{c2\_sq}(5ft, 0ft, c, \phi_{eff}, \gamma) = 0.104\cdot in$ ⟵ Check value

$S_{c2\_st}(5ft, 0ft, c, \phi_{eff}, \gamma) = 0.313\cdot in$ ⟵ Check value

Results

Figures B6-3 and B6-4 present settlement estimates versus allowable bearing capacities for square and strip footings, respectively. Settlements are evaluated with the Burland and Burbidge, and Schmertmann Methods. Results for 2-foot and 6-foot foundation embedment depths are presented in these figures.

For plotting purposes let:

$$q_{all\_sq\_6ft}(B) := q_{all\_square}(B, 6ft, c, \phi_{eff}, \gamma)$$

$$q_{all\_sq\_2ft}(B) := q_{all\_square}(B, 2ft, c, \phi_{eff}, \gamma)$$

$$q_{all\_st\_6ft}(B) := q_{all\_strip}(B, 6ft, c, \phi_{eff}, \gamma)$$

$$q_{all\_st\_2ft}(B) := q_{all\_strip}(B, 2ft, c, \phi_{eff}, \gamma)$$

in Figures B6-3 and B6-4 below.

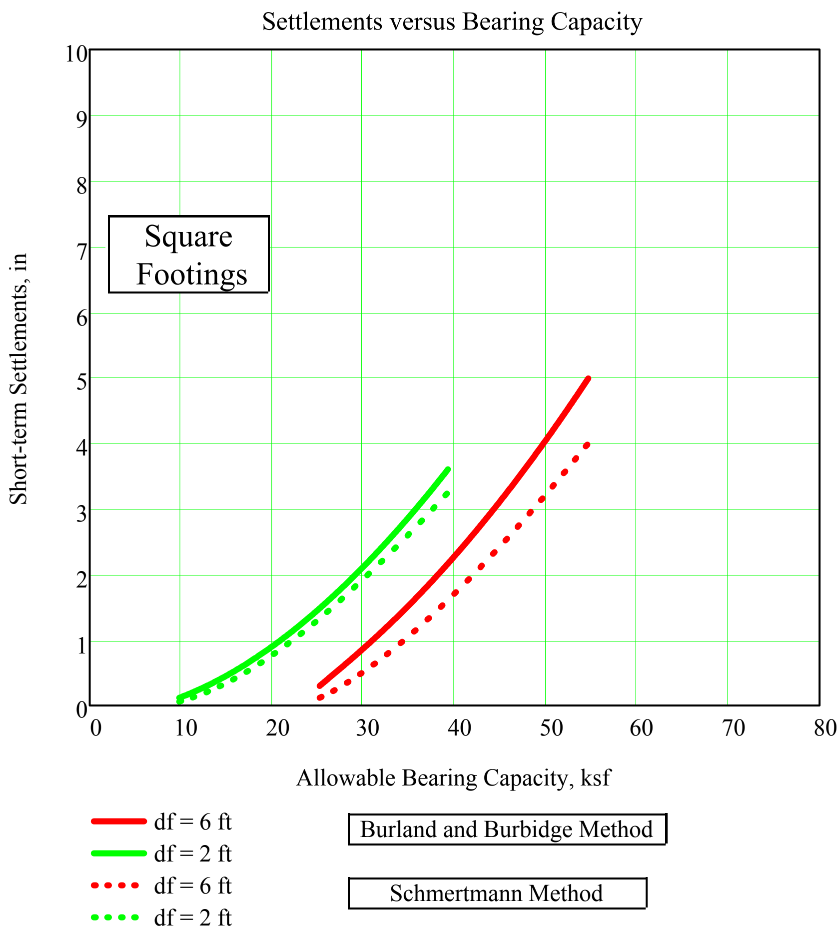


Figure B6-3. Short-term settlement estimates versus allowable bearing capacities for square footings

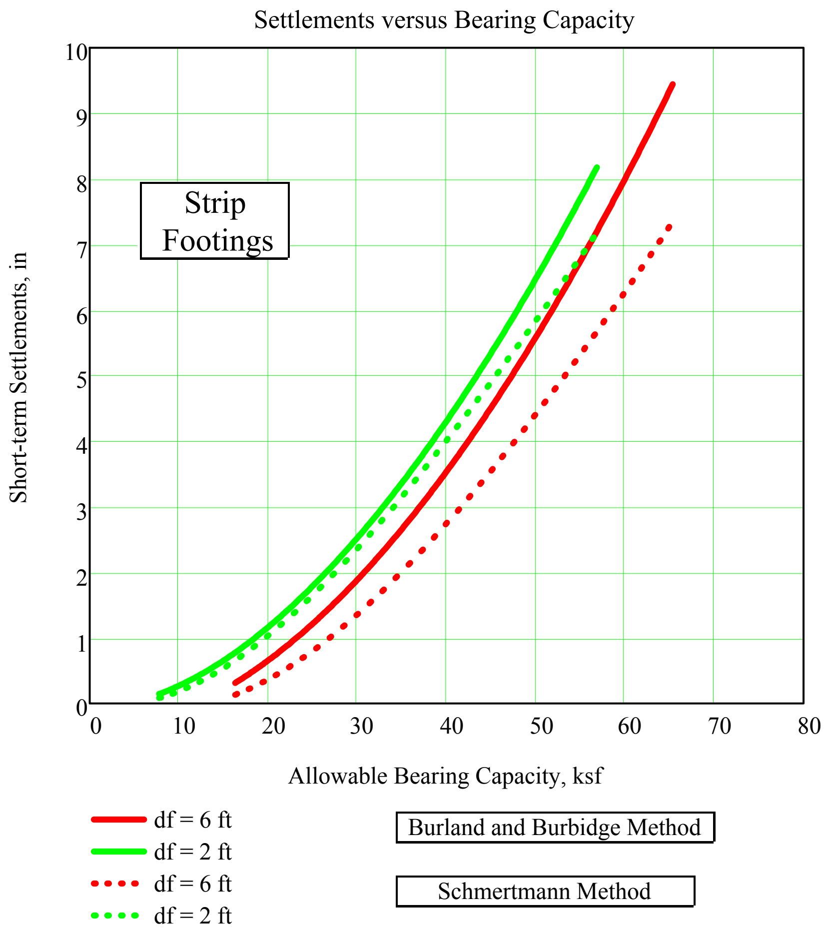


Figure B6-4. Short-term settlement estimates versus allowable bearing capacities for strip footings

B6.3 Foundation Pressure Considering a Maximum Allowable Short-term Settlement ($S_c = \delta_{max}$)

The allowable foundation pressure is constrained to a pressure that produces a footing maximum allowable short-term settlement, δ_{max}. This capacity is computed using the methods proposed by Burland and Burbidge, and Schmertmann et al. as reported in Terzaghi et al. (1996, Sections 50.2.5 and 50.2.6).

Burland and Burbidge (Terzaghi et al. 1996, Section 50.2.5) Method

The following equations correspond to Equations 50.11a and 50.11b presented by Terzaghi et al. (1996), page 396.

Square footings

$$q_{\delta max\_c1\_sq}(B, d_f, c, \phi, \gamma, \delta_{max}) := \begin{cases} \dfrac{\delta_{max}}{Z_I(B) \cdot m_v(\phi) \cdot S_{c\_sq}} + \dfrac{2}{3} \cdot \sigma_{vo}(d_f) & \text{if } q_{all\_square}(B, d_f, c, \phi, \gamma) > \sigma_{vo}(d_f) \\ \dfrac{3 \cdot \delta_{max}}{Z_I(B) \cdot m_v(\phi) \cdot S_{c\_sq}} & \text{otherwise} \end{cases}$$

Strip footings

$$q_{\delta max\_c1\_st}(B, d_f, c, \phi, \gamma, \delta_{max}) := \begin{cases} \dfrac{\delta_{max}}{Z_I(B) \cdot m_v(\phi) \cdot S_{c\_st}} + \dfrac{2}{3} \cdot \sigma_{vo}(d_f) & \text{if } q_{all\_strip}(B, d_f, c, \phi, \gamma) > \sigma_{vo}(d_f) \\ \dfrac{3 \cdot \delta_{max}}{Z_I(B) \cdot m_v(\phi) \cdot S_{c\_st}} & \text{otherwise} \end{cases}$$

$q_{\delta max\_c1\_sq}(5ft, 0ft, c, \phi_{eff}, \gamma, 0.5in) = 20.825 \cdot ksf$ ⟵ Check value

$q_{\delta max\_c1\_st}(5ft, 0ft, c, \phi_{eff}, \gamma, 0.5in) = 13.35 \cdot ksf$ ⟵ Check value

Schmertmann (Terzaghi et al. 1996, Section 50.2.6) Method

These equations represent the continuous form of Equations 50.23a and 50.23b presented by Terzaghi et al. (1996, Section 50.2.6).

Square footings

$$q_{\delta max\_c2\_sq}(B, d_f, \delta_{max}) := \frac{\delta_{max}}{C_1(B, d_f) \cdot (S_{c2a\_sq}(B, d_f))} + \sigma_{vo}(d_f)$$

Strip footings

$$q_{\delta max\_c2\_st}(B, d_f, \delta_{max}) := \frac{\delta_{max}}{C_1(B, d_f) \cdot (S_{c2a\_st}(B, d_f))} + \sigma_{vo}(d_f)$$

$q_{\delta max\_c2\_sq}(5ft, 0ft, 0.5in) = 25.391 \cdot ksf$ ⟵ Check value

$q_{\delta max\_c2\_st}(5ft, 0ft, 0.5in) = 14.017 \cdot ksf$ ⟵ Check value

Results

Figures B6-5 and B6-6 present the maximum foundation pressure versus foundation width for square and strip footings, respectively. Settlements are evaluated with the Burland and Burbidge, and Schmertmann methods. Results for 2-foot and 6-foot foundation embedment depths are presented in these figures.

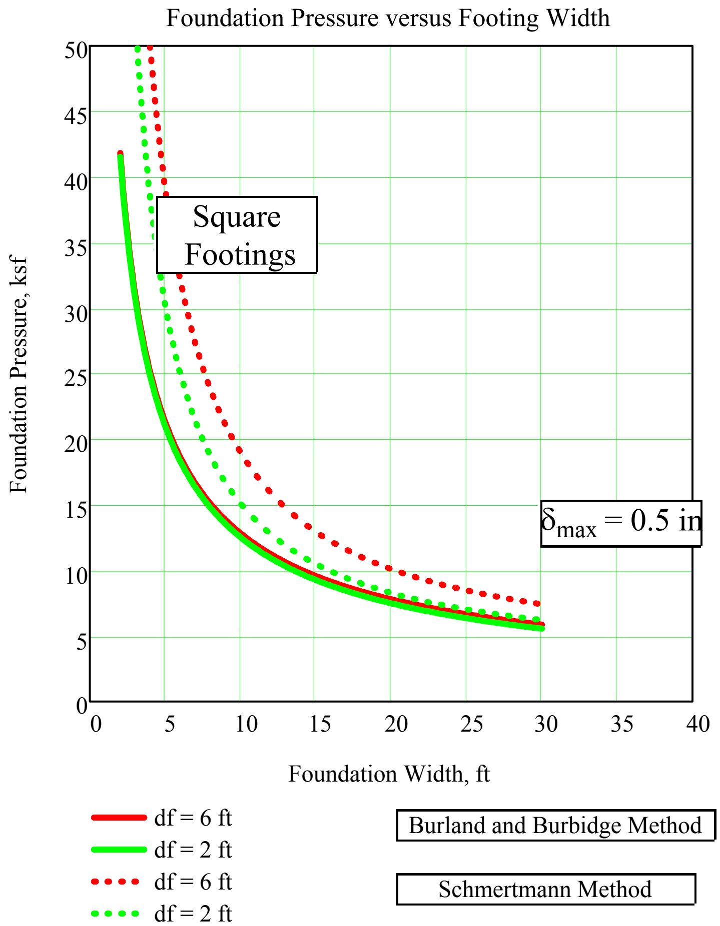


Figure B6-5. Foundation pressure versus foundation width for square footings considering a maximum allowable foundation settlement of 0.5 in

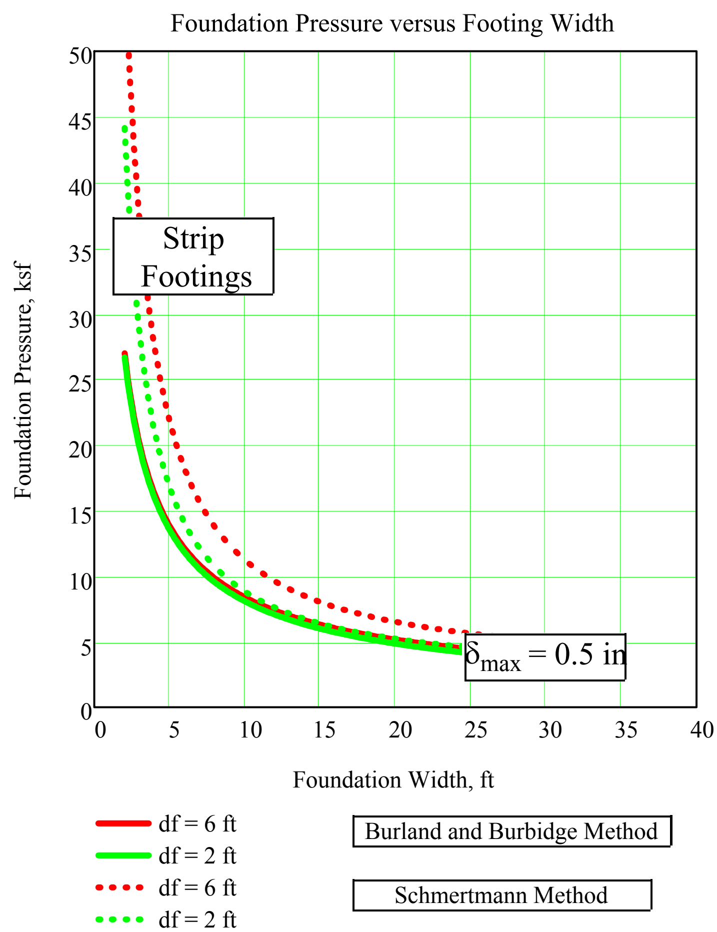


Figure B6-6. Foundation pressure versus foundation width for strip footings considering a maximum allowable foundation settlement of 0.5 in

B6.4 Design Foundation Pressure

The design foundation pressure is computed as the minimum of the allowable bearing capacity or the foundation pressure as determined above from Sections B6.1 and B6.3.

The maximum foundation pressure for design is further limited by a cutoff value. This value corresponds to the minimum pressure of the values determined in Sections B6.1 and B6.3 for a 2-foot wide footing.

Burland and Burbidge (Terzaghi et al. 1996) method

Square footings

$$q_{fp\_c1\_sq}(B, d_f, c, \phi, \gamma, \delta_{max}) := \left| \begin{array}{l} q_{all\_square}(B, d_f, c, \phi, \gamma) \text{ if } S_{c1\_sq}(B, d_f, c, \phi, \gamma) \le \delta_{max} \\ q_{\delta max\_c1\_sq}(B, d_f, c, \phi, \gamma, \delta_{max}) \text{ otherwise} \end{array} \right.$$

$$q_{fp\_c1\_sq0}(d_f, c, \phi, \gamma, \delta_{max}) := q_{fp\_c1\_sq}(B_0, d_f, c, \phi, \gamma, \delta_{max}) \longleftarrow \text{Cutoff value}$$

$$q_{fp\_c1\_sq}(B, d_f, c, \phi, \gamma, \delta_{max}) := \left| \begin{array}{l} q_{fp\_c1\_sq0}(d_f, c, \phi, \gamma, \delta_{max}) \text{ if } q_{fp\_c1\_sq}(B, d_f, c, \phi, \gamma, \delta_{max}) > q_{fp\_c1\_sq0}(d_f, c, \phi, \gamma, \delta_{max}) \\ q_{fp\_c1\_sq}(B, d_f, c, \phi, \gamma, \delta_{max}) \text{ otherwise} \end{array} \right.$$

Strip footings

$$q_{fp\_c1\_st}(B, d_f, c, \phi, \gamma, \delta_{max}) := \left| \begin{array}{l} q_{all\_strip}(B, d_f, c, \phi, \gamma) \text{ if } S_{c1\_st}(B, d_f, c, \phi, \gamma) \le \delta_{max} \\ q_{\delta max\_c1\_st}(B, d_f, c, \phi, \gamma, \delta_{max}) \text{ otherwise} \end{array} \right.$$

$$q_{fp\_c1\_st0}(d_f, c, \phi, \gamma, \delta_{max}) := q_{fp\_c1\_st}(B_0, d_f, c, \phi, \gamma, \delta_{max}) \longleftarrow \text{Cutoff value}$$

$$q_{fp\_c1\_st}(B, d_f, c, \phi, \gamma, \delta_{max}) := \left| \begin{array}{l} q_{fp\_c1\_st0}(d_f, c, \phi, \gamma, \delta_{max}) \text{ if } q_{fp\_c1\_st}(B, d_f, c, \phi, \gamma, \delta_{max}) > q_{fp\_c1\_st0}(d_f, c, \phi, \gamma, \delta_{max}) \\ q_{fp\_c1\_st}(B, d_f, c, \phi, \gamma, \delta_{max}) \text{ otherwise} \end{array} \right.$$

$$q_{fp\_c1\_sq}(20ft, 0ft, c, \phi_{eff}, \gamma, 0.5in) = 2.103 \cdot ksf \longleftarrow \text{Check value}$$

$$q_{fp\_c1\_st}(20ft, 0ft, c, \phi_{eff}, \gamma, 0.5in) = 3.505 \cdot ksf \longleftarrow \text{Check value}$$

Schmertmann (Terzaghi et al. 1996) method

Square footings

$$q_{fp\_c2\_sq}(B, d_f, c, \phi, \gamma, \delta_{max}) := \left| \begin{array}{l} q_{all\_square}(B, d_f, c, \phi, \gamma) \text{ if } S_{c2\_sq}(B, d_f, c, \phi, \gamma) \le \delta_{max} \\ q_{\delta max\_c2\_sq}(B, d_f, \delta_{max}) \text{ otherwise} \end{array} \right.$$

$$q_{fp\_c2\_sq0}(d_f, c, \phi, \gamma, \delta_{max}) := q_{fp\_c2\_sq}(B_0, d_f, c, \phi, \gamma, \delta_{max})$$ ⟵ Cutoff value

$$q_{fp\_c2\_sq}(B, d_f, c, \phi, \gamma, \delta_{max}) := \begin{cases} q_{fp\_c2\_sq0}(d_f, c, \phi, \gamma, \delta_{max}) & \text{if } q_{fp\_c2\_sq}(B, d_f, c, \phi, \gamma, \delta_{max}) > q_{fp\_c2\_sq0}(d_f, c, \phi, \gamma, \delta_{max}) \\ q_{fp\_c2\_sq}(B, d_f, c, \phi, \gamma, \delta_{max}) & \text{otherwise} \end{cases}$$

Strip footings

$$q_{fp\_c2\_st}(B, d_f, c, \phi, \gamma, \delta_{max}) := \begin{cases} q_{all\_strip}(B, d_f, c, \phi, \gamma) & \text{if } S_{c2\_st}(B, d_f, c, \phi, \gamma) \le \delta_{max} \\ q_{\delta max\_c2\_st}(B, d_f, \delta_{max}) & \text{otherwise} \end{cases}$$

$$q_{fp\_c2\_st0}(d_f, c, \phi, \gamma, \delta_{max}) := q_{fp\_c2\_st}(B_0, d_f, c, \phi, \gamma, \delta_{max})$$ ⟵ Cutoff value

$$q_{fp\_c2\_st}(B, d_f, c, \phi, \gamma, \delta_{max}) := \begin{cases} q_{fp\_c2\_st0}(d_f, c, \phi, \gamma, \delta_{max}) & \text{if } q_{fp\_c2\_st}(B, d_f, c, \phi, \gamma, \delta_{max}) > q_{fp\_c2\_st0}(d_f, c, \phi, \gamma, \delta_{max}) \\ q_{fp\_c2\_st}(B, d_f, c, \phi, \gamma, \delta_{max}) & \text{otherwise} \end{cases}$$

$$q_{fp\_c2\_sq}(20ft, 0ft, c, \phi_{eff}, \gamma, 0.5in) = 2.103 \cdot ksf$$ ⟵ Check value

$$q_{fp\_c2\_st}(20ft, 0ft, c, \phi_{eff}, \gamma, 0.5in) = 3.505 \cdot ksf$$ ⟵ Check value

Results

Figures B6-7 and B6-8 present the design foundation pressure versus foundation width for square and strip footings, respectively. Settlements are evaluated with the Burland and Burbidge, and Schmertmann methods. Results for 2-foot and 6-foot foundation embedment depths are presented in these figures.

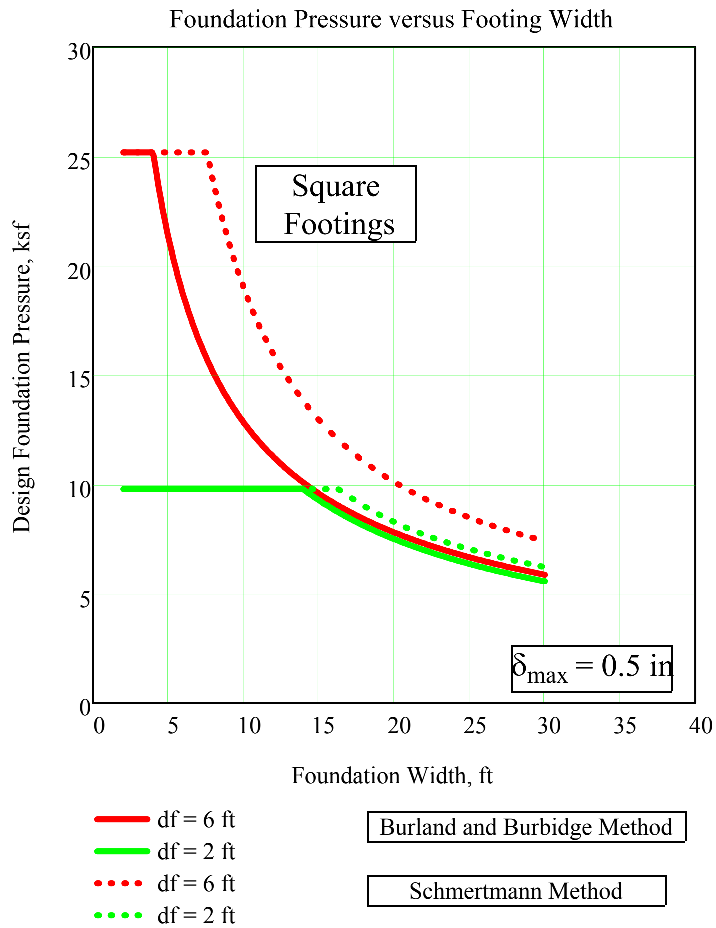


Figure B6-7. Design foundation pressure versus foundation width for square footings considering a maximum allowable foundation settlement of 0.5 in

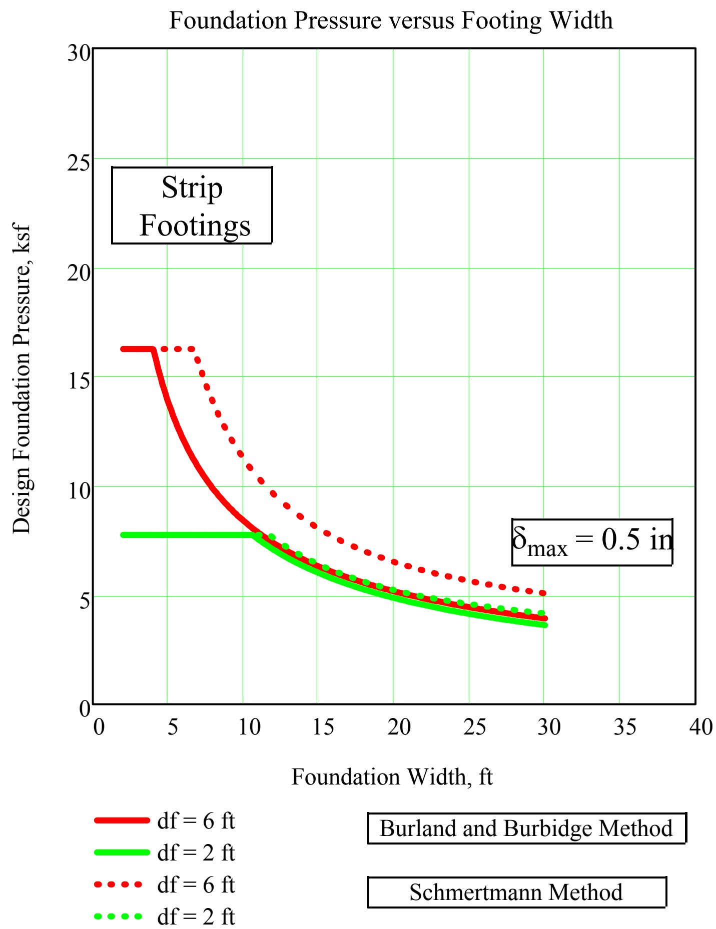


Figure B6-8. Design foundation pressure versus foundation width for strip footings considering a maximum allowable foundation settlement of 0.5 in

B6.5 Settlements for Different Foundation Pressures

The short-term settlements for different foundation pressures are computed using the procedures by Burland and Burbidge, and Schmertmann et al. as reported in Terzaghi et al. (1996, Sections 50.2.5 and 50.2.6).

The following bearing pressure range is considered in the analyses:

$q_{bp} := 0.5ksf, 0.6ksf .. 40ksf$ Bearing pressure range

Burland and Burbidge (Terzaghi et. al 1996) method

The following equations correspond to Equations 50.11a and 50.11b presented by Terzaghi et al. (1996, Section 50.2.5).

Square Footings

$$S_{bp\_c1\_sq}(B, d_f, \phi, q_{bp}) := \begin{cases} Z_I(B) \cdot m_v(\phi) \cdot \left(q_{bp} - \frac{2}{3} \cdot \sigma_{vo}(d_f)\right) \cdot S_{c\_sq} & \text{if } q_{bp} > \sigma_{vo}(d_f) \\ \frac{1}{3} \cdot Z_I(B) \cdot m_v(\phi) \cdot q_{bp} \cdot S_{c\_sq} & \text{otherwise} \end{cases}$$

Strip Footings

$$S_{bp\_c1\_st}(B, d_f, \phi, q_{bp}) := \begin{cases} Z_I(B) \cdot m_v(\phi) \cdot \left(q_{bp} - \frac{2}{3} \cdot \sigma_{vo}(d_f)\right) \cdot S_{c\_st} & \text{if } q_{bp} > \sigma_{vo}(d_f) \\ \frac{1}{3} \cdot Z_I(B) \cdot m_v(\phi) \cdot q_{bp} \cdot S_{c\_st} & \text{otherwise} \end{cases}$$

Check values

$S_{bp\_c1\_sq}(5ft, 0ft, \phi_{eff}, q_{bp}) =$ $q_{bp} =$

$S_{bp\_c1\_sq}$ (in)	q_{bp} (ksf)
0.012	0.5
0.014	0.6
...	...

$S_{bp\_c1\_st}(5ft, 0ft, \phi_{eff}, q_{bp}) =$ $q_{bp} =$

$S_{bp\_c1\_st}$ (in)	q_{bp} (ksf)
0.019	0.5
0.022	0.6
...	...

Schmertmann (Terzaghi et. al 1996) method

These equations represent the continuous form of Equations 50.23a and 50.23b presented by Terzaghi et al. (1996, Section 50.2.6).

Square Footings

$$S_{bp\_c2a\_sq}(B, d_f) := \int_{d_f}^{d_f+2B} \frac{I_{z\_sq}(z, B, d_f)}{E(z)}\, dz$$

$$S_{bp\_c2\_sq}(B, d_f, q_{bp}) := C_1(B, d_f)\cdot(q_{bp} - \sigma_{vo}(d_f))\cdot S_{bp\_c2a\_sq}(B, d_f)$$

Strip Footings

$$S_{bp\_c2a\_st}(B, d_f) := \int_{d_f}^{d_f+4B} \frac{I_{z\_st}(z, B, d_f)}{E(z)}\, dz$$

$$S_{bp\_c2\_st}(B, d_f, q_{bp}) := C_1(B, d_f)\cdot(q_{bp} - \sigma_{vo}(d_f))\cdot S_{bp\_c2a\_st}(B, d_f)$$

Check values

$S_{bp\_c2\_sq}(5ft, 0ft, q_{bp}) =$ $\cdot in$

0.01
0.012
...

$q_{bp} =$ $\cdot ksf$

0.5
0.6
...

$S_{bp\_c2\_st}(5ft, 0ft, q_{bp}) =$ $\cdot in$

0.018
0.021
...

$q_{bp} =$ $\cdot ksf$

0.5
0.6
...

Results

Figures B6-9 through B6-12 present the estimated settlements versus foundation pressure for square and strip footings. Settlements are evaluated with the Burland and

Burbidge, and Schmertmann methods. Figures B6-9 and B6-10 present the results for square and strip footings with 6-foot foundation embedment depth, respectively. Figures B6-11 and B6-12 present the results for square and strip footings with 2-foot foundation embedment depth, respectively.

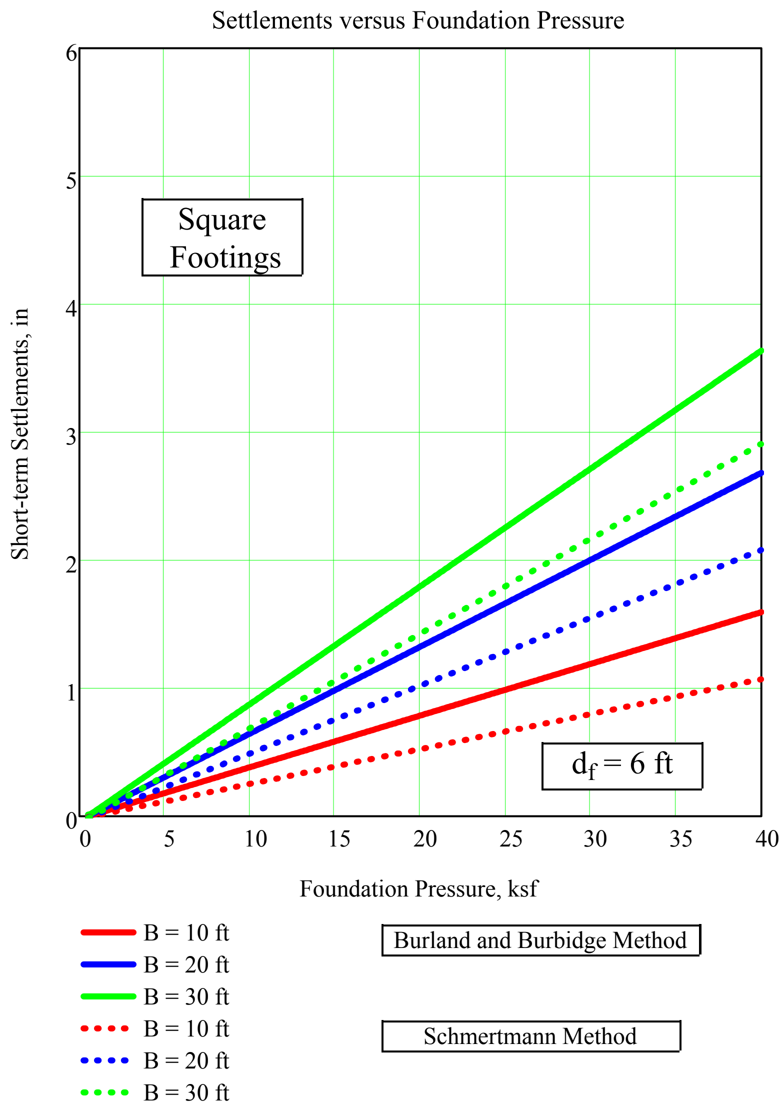


Figure B6-9. Short-term settlements versus foundation pressure for square footings

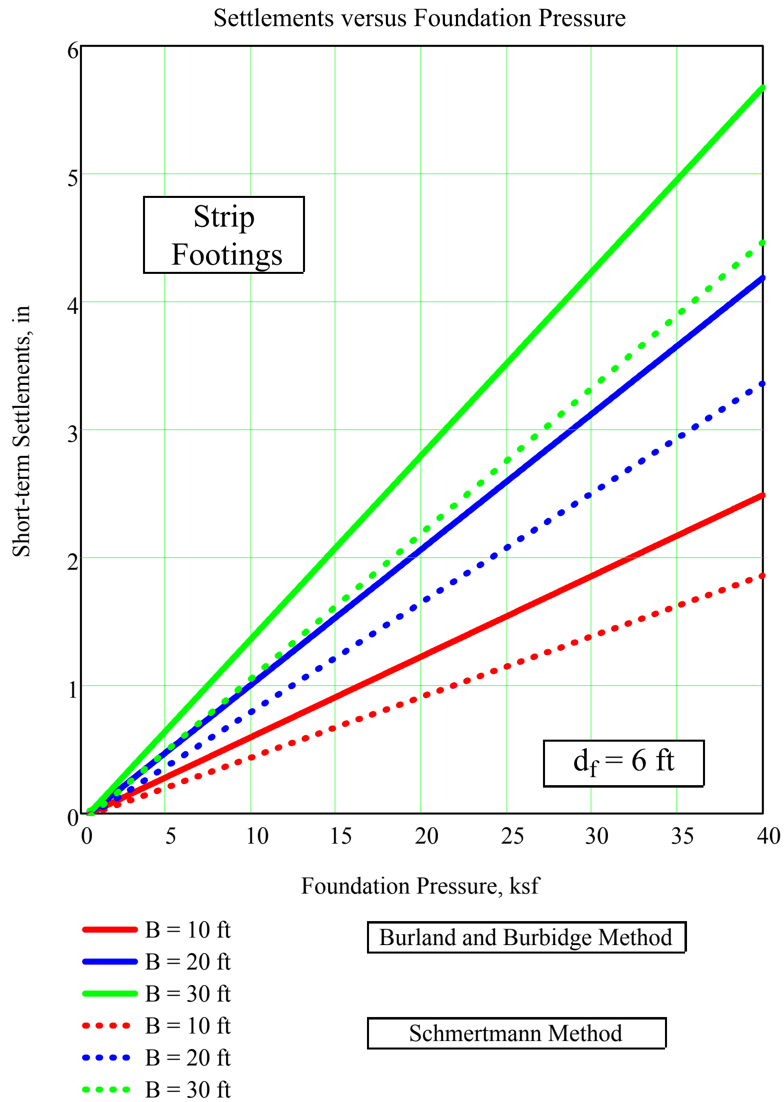


Figure B6-10. Short-term settlements versus foundation pressure for strip footings

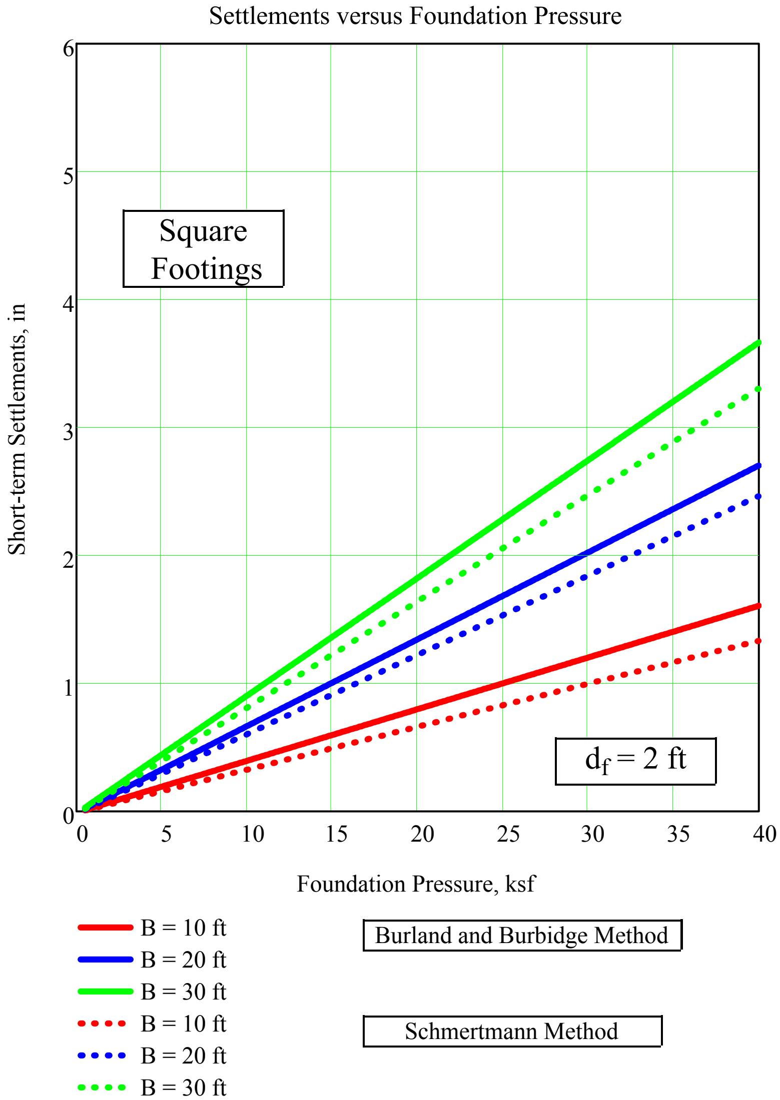


Figure B6-11. Short-term settlements versus foundation pressure for square footings

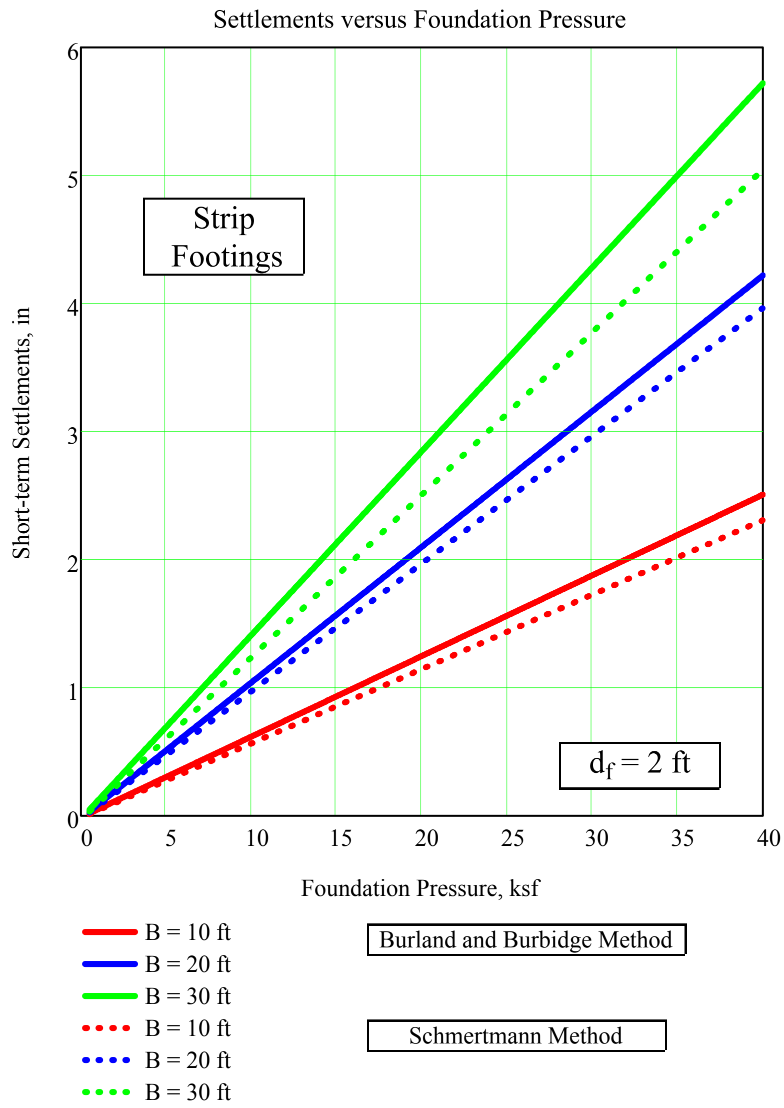


Figure B6-12. Short-term settlements versus foundation pressure for strip footings

A comparison of the above methods show similar results for the design pressure.

Results from the Schmertmann method are adopted since more data from the project (shear wave velocity) is available for this method. The Burland and Burbidge method uses an N_{60} value, which was derived from relative density measurements. The design pressure calculated by the Schmertmann method is limited for larger footing sizes for conservatism.

Figures B6-13 through B6-16 present our recommendations to the project for allowable foundation pressures and immediate settlements.

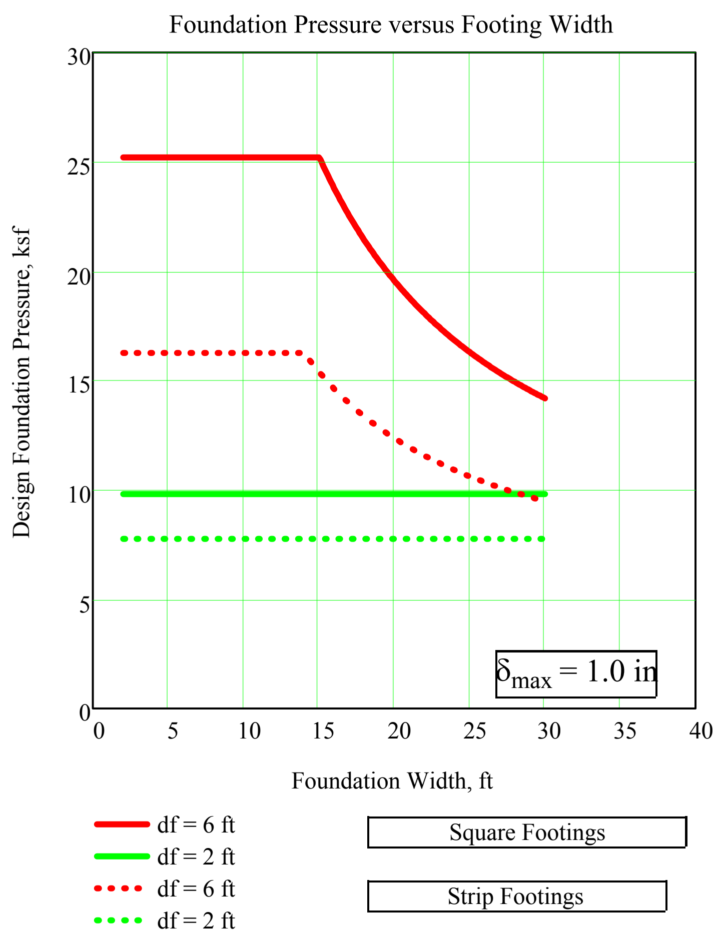


Figure B6-13. Design foundation pressure versus foundation width for square and strip footings considering a maximum allowable foundation settlement of 1.0 in

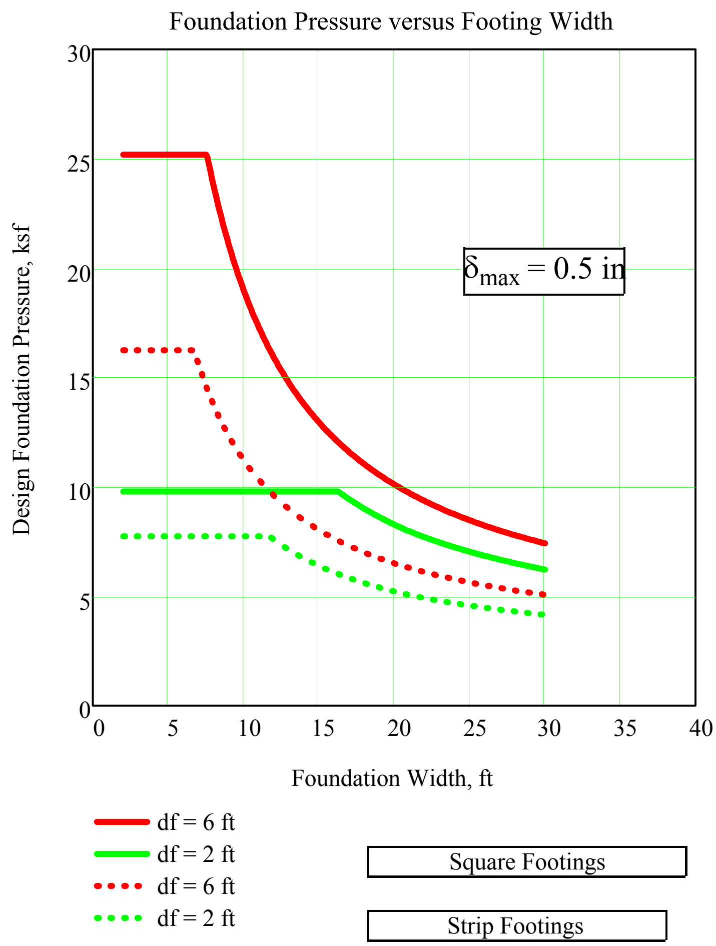


Figure B6-14. Design foundation pressure versus foundation width for square and strip footings considering a maximum allowable foundation settlement of 0.5 in

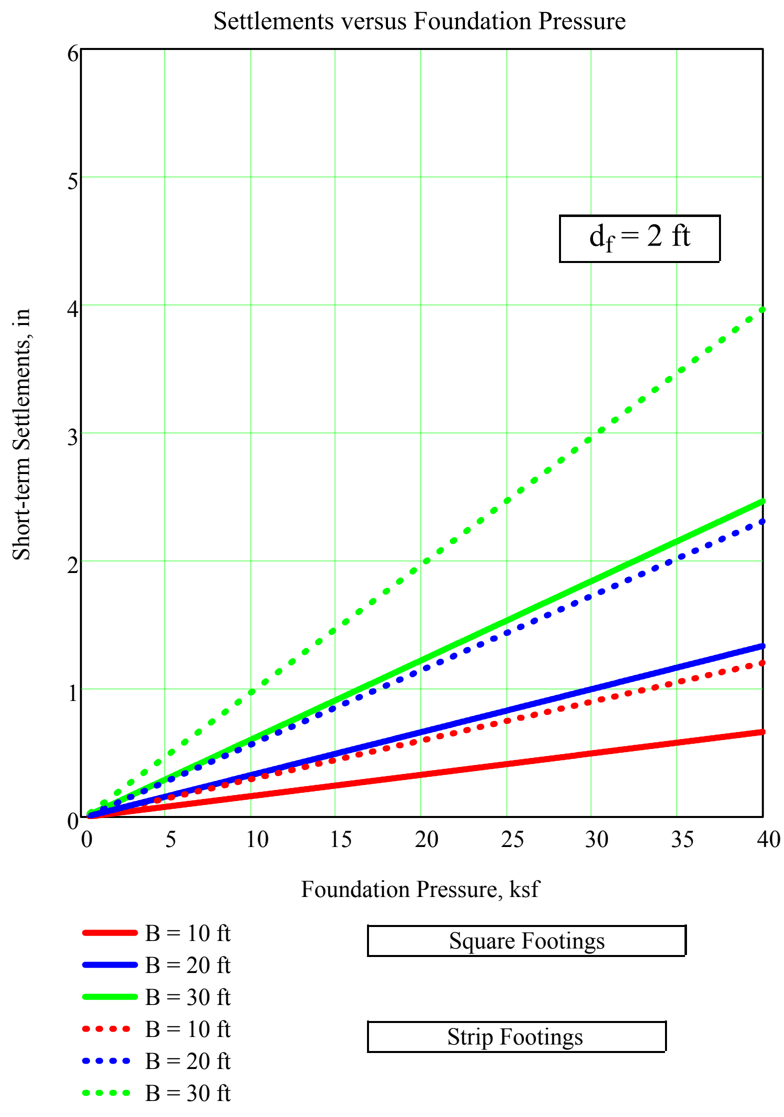


Figure B6-15. Short-term settlements versus foundation pressure for strip footings

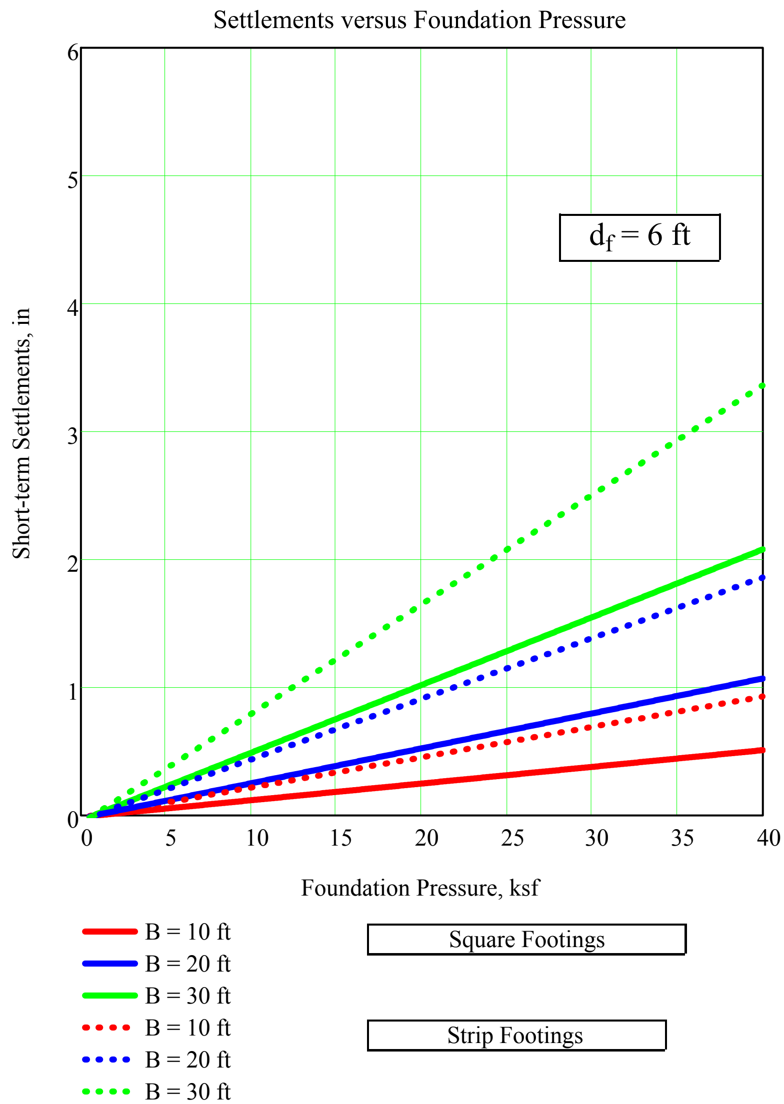


Figure B6-16. Short-term settlements versus foundation pressure for strip footings

B6.6 Long-term Settlements

The Burland and Burbidge procedure was implemented to compute the footings long-term settlements (see Terzaghi et al, 1996, Section 50.2.5). This method estimates settlements based on the soil standard penetration test blow count (N_{60}) values.

Compression strain

Square footings

$$\varepsilon_{c\_sq}(B, d_f, c, \phi, \gamma) := \begin{cases} \dfrac{1.4}{N_{60}(\phi)^{1.4}} & \text{if } q_{all\_square}(B, d_f, c, \phi, \gamma) > \sigma_{vo}(d_f) \\ \dfrac{1}{3} \cdot \dfrac{1.4}{N_{60}(\phi)^{1.4}} & \text{if } q_{all\_square}(B, d_f, c, \phi, \gamma) \le \sigma_{vo}(d_f) \end{cases}$$

Strip footings

$$\varepsilon_{c\_st}(B, d_f, c, \phi, \gamma) := \begin{cases} \dfrac{1.4}{N_{60}(\phi)^{1.4}} & \text{if } q_{all\_strip}(B, d_f, c, \phi, \gamma) > \sigma_{vo}(d_f) \\ \dfrac{1}{3} \cdot \dfrac{1.4}{N_{60}(\phi)^{1.4}} & \text{if } q_{all\_strip}(B, d_f, c, \phi, \gamma) \le \sigma_{vo}(d_f) \end{cases}$$

$$\varepsilon_{c\_sq}(5\text{ft}, 0\text{ft}, c, \phi_{eff}, \gamma) = 7.647 \times 10^{-3} \longleftarrow$$ Check value

$$\varepsilon_{c\_st}(5\text{ft}, 0\text{ft}, c, \phi_{eff}, \gamma) = 7.647 \times 10^{-3} \longleftarrow$$ Check value

Secondary compression strain index

Square footings

$$\varepsilon_{\alpha\_sq}(B, d_f, c, \phi, \gamma) := 0.02 \cdot \varepsilon_{c\_sq}(B, d_f, c, \phi, \gamma)$$

Strip footings

$$\varepsilon_{\alpha\_st}(B, d_f, c, \phi, \gamma) := 0.02 \cdot \varepsilon_{c\_st}(B, d_f, c, \phi, \gamma)$$

$$\varepsilon_{\alpha\_sq}(5ft, 0ft, c, \phi_{eff}, \gamma) = 1.529 \times 10^{-4}$$ ← Check value

$$\varepsilon_{\alpha\_st}(5ft, 0ft, c, \phi_{eff}, \gamma) = 1.529 \times 10^{-4}$$ ← Check value

Long-term settlement equation

Square footings

$$S_{c3\_sq}(B, d_f, c, \phi, \gamma) := \varepsilon_{\alpha\_sq}(B, d_f, c, \phi, \gamma) \cdot Z_I(B) \cdot \log\left(\frac{\frac{t}{year}\,day}{1 \cdot day}\right)$$

Strip footings

$$S_{c3\_st}(B, d_f, c, \phi, \gamma) := \varepsilon_{\alpha\_st}(B, d_f, c, \phi, \gamma) \cdot Z_I(B) \cdot \log\left(\frac{\frac{t}{year}\,day}{1 \cdot day}\right)$$

$$S_{c3\_sq}(5ft, 0ft, c, \phi_{eff}, \gamma) = 0.014 \cdot in$$ ← Check value

$$S_{c3\_st}(5ft, 0ft, c, \phi_{eff}, \gamma) = 0.014 \cdot in$$ ← Check value

Results

Figures B6-17 presents the estimated long-term settlements versus foundation pressure for square and strip footings and embedment depth considered herein. Settlement are evaluated with the Burland and Burbidge method.

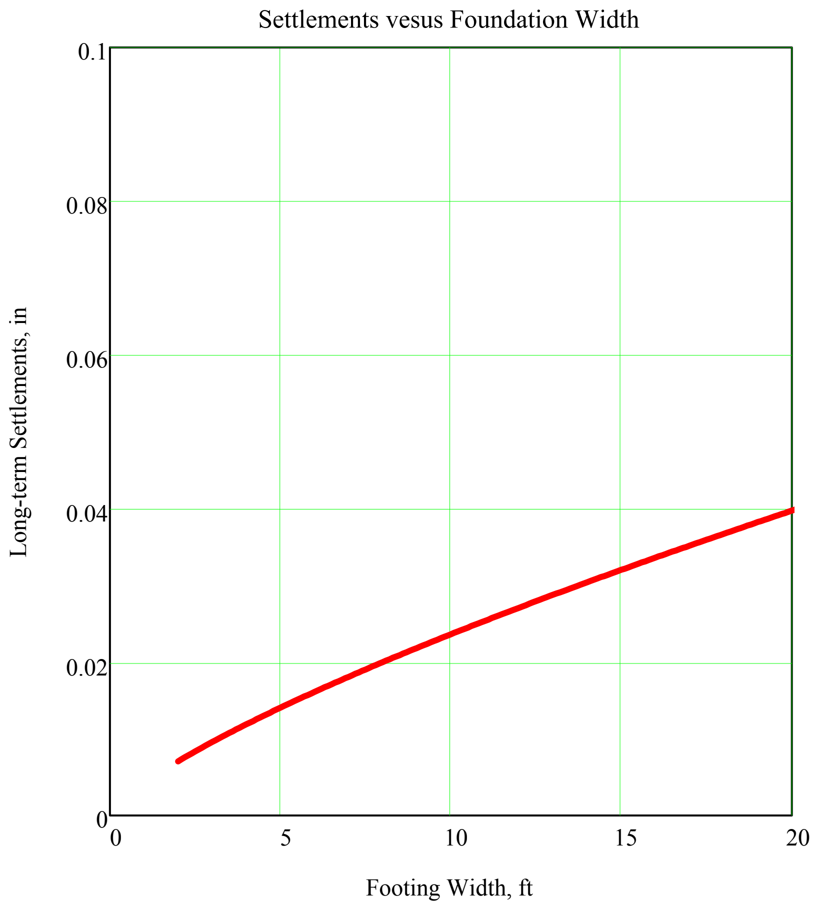


Figure B6-17. Long-term settlements versus footing width for square and strip footings and embedment depth considered herein (i.e., 2 ft and 6 ft).

Units:

$kPa \equiv 1000 \cdot Pa$ $\quad psf \equiv \frac{lbf}{ft^2}$ $\quad pcf \equiv \frac{lbf}{ft^3}$ $\quad year \equiv 365 \cdot day$

$MPa \equiv 10^3 \cdot kPa$ $\quad ksf \equiv \frac{1000lbf}{ft^2}$ $\quad tsf \equiv \frac{2000lbf}{ft^2}$ $\quad fps \equiv \frac{ft}{s}$

B6.7 Elastic Settlement for Mat Foundation

Elastic settlements are computed based on a uniform vertical stress distribution, representative average shear wave velocities, and modulus degradation curves for sands. The settlements are determined for uniform vertical loads of 3, 5, and 7 ksf. The following are performed for the computation:

- **Alluvium Thickness** – Divide the alluvium layer (120 feet thick, a reasonable average for the major building sites) into 1 ft sublayers (h1, h2…h_i), where i = sublayer number. Since the mat thickness used in the analysis is assumed to be 3 feet, subtract 3 feet from the top portion of the alluvium.

- **Vertical Stress Distribution, σ_z** – Compute the vertical stress distribution below the mat (corner and center) for the entire alluvium layer. For a uniform load on a rectangular mat (beneath the mat corner), use the following equation from pp. 54 of Poulos and Davis (1991):

$$\sigma_z = \frac{q}{2\pi}\left[\tan^{-1}\frac{\ell b}{zR_3} + \frac{\ell bz}{R_3}\left(\frac{1}{R_1^{\,2}} + \frac{1}{R_2^{\,2}}\right)\right], \text{ where} \qquad \text{(B1)}$$

$z = depth$

$\ell = \dfrac{L}{2}$ (for distribution at center of foundation)

$\ell = L$ (for distribution at corner of foundation)

$b = \dfrac{B}{2}$ (for distribution at center of foundation)

$b = B$ (for distribution at corner of foundation)

$R_1 = \left(\ell^2 + z^2\right)^{1/2}$

$R_2 = \left(b^2 + z^2\right)^{1/2}$

$R_3 = \left(\ell^2 + b^2 + z^2\right)^{1/2}$

Multiply Eq. (B1) by 4 for the stress distribution at the center of foundation. Figure B6-18 below shows the stress distributions for the 3 uniform vertical loads.

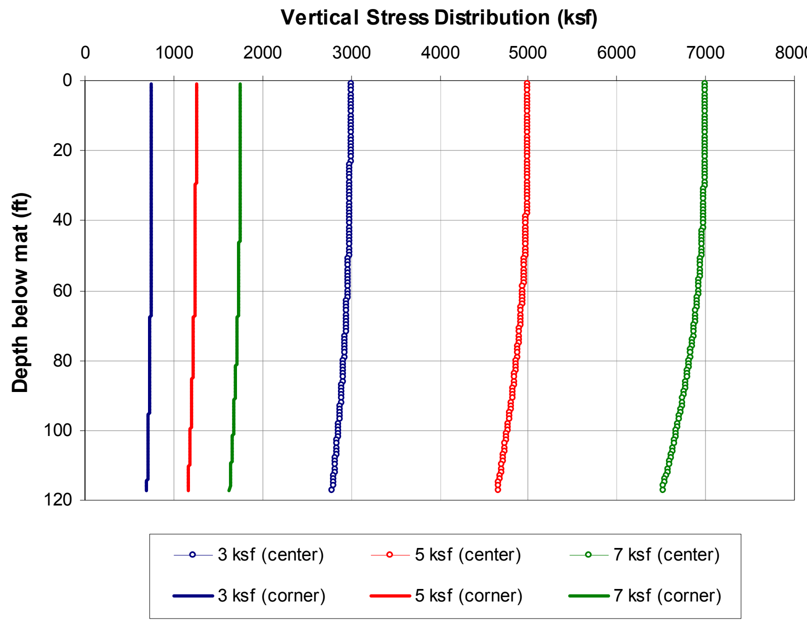


Figure B6-18. Vertical stress distribution versus depth for vertical loads of 3, 5, and 7 ksf.

- **Modulus Degradation Curves** – Select appropriate modulus degradation curves (G/G_{max} versus shear strain, γ_t) to be used to determine the strains induced in the alluvium layer during vertical loading. Dynamic testing was performed on one reconstituted alluvium sample in (DTN:MO0203DHRSSWHB.001). The modulus degradation curve obtained from the testing lies between the mean and upper bound curves from Seed and Idriss (1970) for sands as shown below:

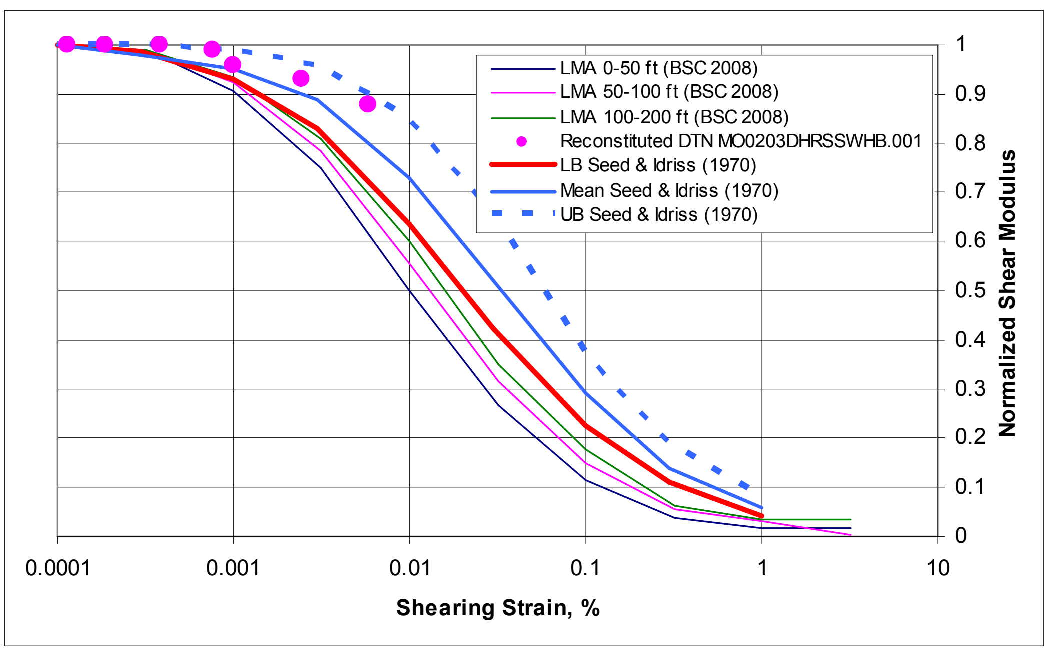


Figure B6-19. Modulus degradation curves for sandy material.

The recommended project design curves are those that begin with LMA in Figure B6-19 above, and were derived from BSC 2008a. The lower bound curve from Seed and Idriss (1970) is used in the analyses.

- **Shear Wave Velocity, Vs –** Select representative shear wave velocity values for the alluvium layer to be used for the analyses. Table B6-1 (data determined from Figure B2-1) summarizes the lower bound (mean minus one standard deviation) and mean V_s values used at different depths in the alluvium for the analysis:

Table B6-1. Average shear wave velocity values (shown in Figure B2-1).

Depth from ground surface (ft)	Lower bound (ft/s)	Mean (ft/s)
0-15	1,200	1,500
15-30	1,400	1,700
30-60	2,000	2,200
60-120	2,200	2,500

- **Young's Modulus of Elasticity, E and axial strain, ε_a –** Use the vertical stress distribution (3, 5, and 7 ksf), the modulus degradation curves (mean and lower bound), and shear wave velocity averages (mean and lower bound) to determine Young's Modulus and the amount of axial strain induced in the alluvium layer.

  The modulus degradation curves are modified to show elastic modulus versus axial strain. It is assumed that the shear modulus degradation relationship, G/G_{max} is analogous to the elastic modulus degradation, E/E_{max}. This is a conservative assumption since it is known that the elastic modulus degrades less than the shear modulus. Calculate dynamic G_{max} from the shear wave velocity values using:

$$G_{\max} = \frac{V_s^{\,2}\gamma}{g} \qquad \text{(B2)}$$

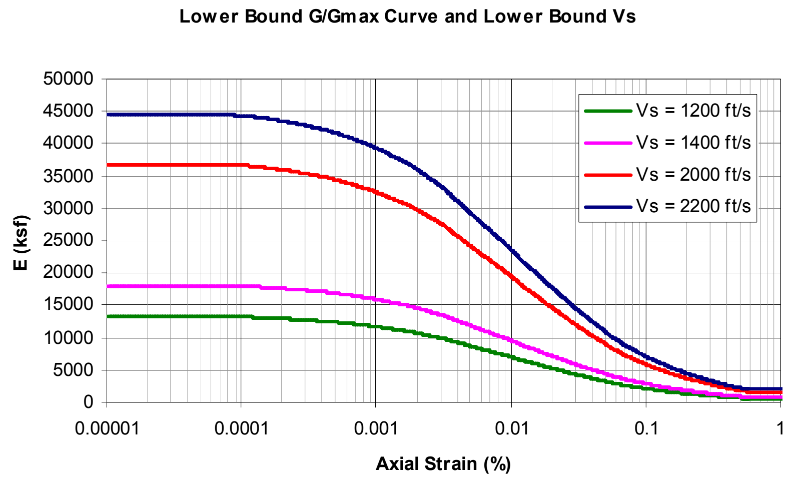
  where γ = 114 pcf (unit weight of alluvium). Using this, the degradation curves can be modified to show G versus γ_t for each velocity average. E can then be determined by:

$$E = 2G(1+\upsilon) \qquad \text{(B3)}$$

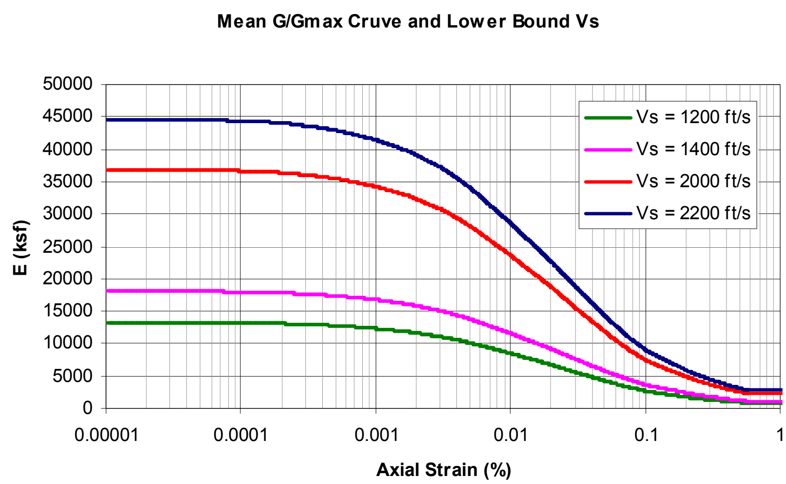
  where υ = 0.3 (Poisson's ratio of alluvium). The shear strain, γ_t, can be expressed as axial strain, ε_a, by the following relationship (Equation 11 of Vucetic and Dobry 1986):

$$\varepsilon_a = \frac{\gamma_t}{1.73} \qquad \text{(B4)}$$

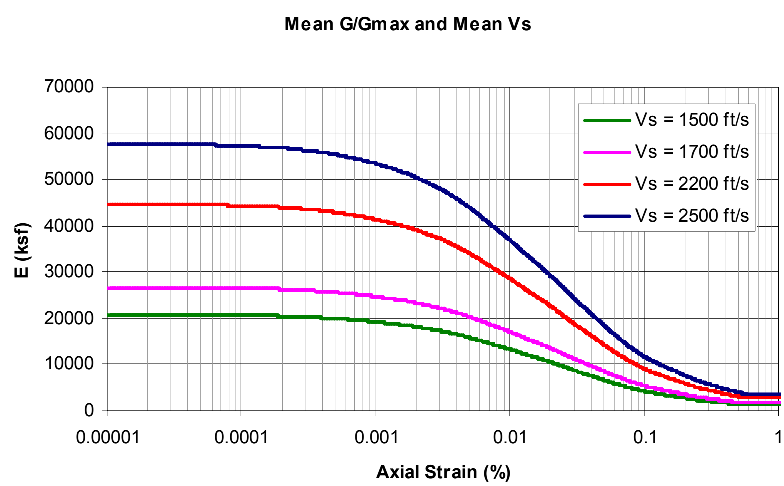
  Using (B3) and (B4), the degradation curves can be modified to show E versus ε_a. The following curves for combinations of mean and lower bound values of modulus degradation curves and shear wave velocity value are generated:



(a)



(b)



(c)

Figure B6-20 (a)-(c). Young's Modulus versus axial strain.

Using the appropriate curve, an initial axial strain can be used to determine the corresponding E. The new axial strain can then be computed using:

$$\varepsilon_a = \frac{\sigma_z}{E} \qquad \text{(B5)}$$

where σ_z is computed in (B1) for 3, 5, and 7 ksf vertical loading. The new strain can then be used with the curves to determine a new E. This iterative process using (B5) is continued until the axial strain converges, which represents the amount of strain induced in the alluvium due to the vertical loading.

- **Settlement** – Compute the total settlement of the alluvium from the final axial strains by summing the settlements in each alluvium layer using:

$$Settlement = \sum_{i=1}^{120} \varepsilon_i h_i \qquad \text{(B6)}$$

The calculations are performed for each vertical load case (3, 5, and 7 ksf) for the following bound conditions of modulus degradation and shear wave velocity (Table B6-2):

Table B6-2. Shear wave velocity and modulus degradation curve bound conditions used in analysis.

Shear wave velocity	Modulus degradation
Lower	Lower
Lower	Mean
Mean	Mean

Table B6-3 shows a sample EXCEL spreadsheet calculation (center of the mat foundation under 5 ksf loading using mean values of the shear wave velocity and modulus degradation curve for sands).

The results of the analyses (center and corner of the mat for different shear wave velocity and modulus degradation bound conditions and for various loadings) are shown in Table B6-4. A summary of the expected elastic settlements is shown in Section B7 of this calculation. Because of the conservatism in assuming that Young's modulus, E, degrades the same as the shear modulus for sands, the calculated settlements may be unrealistically high. Hence, for the summary table in Section B7, the settlements computed using the lower bounds of the shear wave velocity and modulus degradation are not used.

- **Bearing Pressure -** For mat foundations, design should be based on settlement considerations. To prevent soil bearing failure, based on conservative extrapolations from Figure B6-2 and from Figure 2 of BSC 2002b, the maximum allowable bearing pressures under mat foundations should not exceed 50 ksf under extreme (seismic) load conditions and 10 ksf under normal load conditions (Figure B7-2).

Table B6-3. Example EXCEL spreadsheet to calculate elastic settlement.

MEAN VELOCITY PROFILE
MEAN SEED AND IDRISS (1970) CURVE

B =	300	ft	B/2 =	150	ft
L =	400	ft	L/2 =	200	ft
q =	**5000**	**psf**	γ =	114	pcf
			υ =	0.3	

SETTLEMENT TOTAL	**0.54**	**in**
STRAIN LEVEL	**0.15**	**%**

Depth from bottom of mat (ft)	Δz (ft)	Stress distribution for Uniform Loading on Rectangular Area R1	R2	R3	σz (psf)	Initial strains from Vs Vs (ft/s)	Gmax (ksf)	Emax (ksf)	$\gamma_{initial}$ (%)	$\varepsilon_{a\_initial}$ (%)	Iterative process E static (ksf)	ε_{a_final} (%)	Sett. (in)
0		200	150	250									
1	1	200	150	250	5000	1500	7966	20711	0.024	0.014	3326	0.15	0.018
2	1	200	150	250	5000	1500	7966	20711	0.024	0.014	3326	0.15	0.018
3	1	200	150	250	5000	1500	7966	20711	0.024	0.014	3326	0.15	0.018
4	1	200	150	250	5000	1500	7966	20711	0.024	0.014	3326	0.15	0.018
5	1	200	150	250	5000	1500	7966	20711	0.024	0.014	3326	0.15	0.018
6	1	200	150	250	5000	1500	7966	20711	0.024	0.014	3326	0.15	0.018
7	1	200	150	250	5000	1500	7966	20711	0.024	0.014	3326	0.15	0.018
8	1	200	150	250	5000	1500	7966	20711	0.024	0.014	3326	0.15	0.018
9	1	200	150	250	4999	1500	7966	20711	0.024	0.014	3326	0.15	0.018
10	1	200	150	250	4999	1500	7966	20711	0.024	0.014	3326	0.15	0.018
11	1	200	150	250	4999	1500	7966	20711	0.024	0.014	3326	0.15	0.018
12	1	200	150	250	4999	1500	7966	20711	0.024	0.014	3326	0.15	0.018
13	1	200	151	250	4998	1700	10232	26602	0.019	0.011	6381	0.08	0.009
14	1	200	151	250	4998	1700	10232	26602	0.019	0.011	6381	0.08	0.009
15	1	201	151	250	4997	1700	10232	26602	0.019	0.011	6381	0.08	0.009
16	1	201	151	251	4997	1700	10232	26602	0.019	0.011	6381	0.08	0.009
17	1	201	151	251	4996	1700	10232	26602	0.019	0.011	6381	0.08	0.009
18	1	201	151	251	4995	1700	10232	26602	0.019	0.011	6381	0.08	0.009
19	1	201	151	251	4995	1700	10232	26602	0.019	0.011	6381	0.08	0.009
20	1	201	151	251	4994	1700	10232	26602	0.019	0.011	6381	0.08	0.009
21	1	201	151	251	4993	1700	10232	26602	0.019	0.011	6381	0.08	0.009
22	1	201	152	251	4992	1700	10232	26602	0.019	0.011	6381	0.08	0.009
23	1	201	152	251	4990	1700	10232	26602	0.019	0.011	6381	0.08	0.009
24	1	201	152	251	4989	1700	10232	26602	0.019	0.011	6394	0.08	0.009
25	1	202	152	251	4988	1700	10232	26602	0.019	0.011	6394	0.08	0.009
26	1	202	152	251	4986	1700	10232	26602	0.019	0.011	6394	0.08	0.009
27	1	202	152	251	4985	1700	10232	26602	0.019	0.011	6394	0.08	0.009
28	1	202	153	252	4983	2200	17135	44552	0.011	0.006	21057	0.02	0.003
29	1	202	153	252	4981	2200	17135	44552	0.011	0.006	21057	0.02	0.003
30	1	202	153	252	4979	2200	17135	44552	0.011	0.006	21057	0.02	0.003
31	1	202	153	252	4977	2200	17135	44552	0.011	0.006	21102	0.02	0.003
32	1	203	153	252	4975	2200	17135	44552	0.011	0.006	21102	0.02	0.003
33	1	203	154	252	4973	2200	17135	44552	0.011	0.006	21102	0.02	0.003
34	1	203	154	252	4970	2200	17135	44552	0.011	0.006	21102	0.02	0.003
35	1	203	154	252	4968	2200	17135	44552	0.011	0.006	21102	0.02	0.003
36	1	203	154	253	4965	2200	17135	44552	0.011	0.006	21102	0.02	0.003
37	1	203	154	253	4962	2200	17135	44552	0.011	0.006	21191	0.02	0.003
38	1	204	155	253	4959	2200	17135	44552	0.011	0.006	21191	0.02	0.003
39	1	204	155	253	4956	2200	17135	44552	0.011	0.006	21236	0.02	0.003
40	1	204	155	253	4952	2200	17135	44552	0.011	0.006	21236	0.02	0.003
41	1	204	156	253	4949	2200	17135	44552	0.011	0.006	21236	0.02	0.003
42	1	204	156	254	4945	2200	17135	44552	0.011	0.006	21236	0.02	0.003
43	1	205	156	254	4942	2200	17135	44552	0.011	0.006	21269	0.02	0.003
44	1	205	156	254	4938	2200	17135	44552	0.011	0.006	21269	0.02	0.003
45	1	205	157	254	4934	2200	17135	44552	0.011	0.006	21269	0.02	0.003

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46	1	205	157	254	4929	2200	17135	44552	0.011	0.006	21269	0.02	0.003
47	1	205	157	254	4925	2200	17135	44552	0.011	0.006	21269	0.02	0.003
48	1	206	157	255	4920	2200	17135	44552	0.011	0.006	21314	0.02	0.003
49	1	206	158	255	4916	2200	17135	44552	0.011	0.006	21314	0.02	0.003
50	1	206	158	255	4911	2200	17135	44552	0.011	0.006	21314	0.02	0.003
51	1	206	158	255	4906	2200	17135	44552	0.011	0.006	21381	0.02	0.003
52	1	207	159	255	4901	2200	17135	44552	0.011	0.006	21381	0.02	0.003
53	1	207	159	256	4895	2200	17135	44552	0.011	0.006	21425	0.02	0.003
54	1	207	159	256	4890	2200	17135	44552	0.011	0.006	21425	0.02	0.003
55	1	207	160	256	4884	2200	17135	44552	0.011	0.006	21425	0.02	0.003
56	1	208	160	256	4878	2200	17135	44552	0.011	0.006	21470	0.02	0.003
57	1	208	160	256	4872	2200	17135	44552	0.011	0.006	21470	0.02	0.003
58	1	208	161	257	4866	2500	22127	57531	0.008	0.005	32481	0.01	0.002
59	1	209	161	257	4860	2500	22127	57531	0.008	0.005	32481	0.01	0.002
60	1	209	162	257	4853	2500	22127	57531	0.008	0.005	32481	0.01	0.002
61	1	209	162	257	4847	2500	22127	57531	0.008	0.005	32539	0.01	0.002
62	1	209	162	258	4840	2500	22127	57531	0.008	0.005	32539	0.01	0.002
63	1	210	163	258	4833	2500	22127	57531	0.008	0.005	32582	0.01	0.002
64	1	210	163	258	4826	2500	22127	57531	0.008	0.005	32582	0.01	0.002
65	1	210	163	258	4818	2500	22127	57531	0.008	0.005	32582	0.01	0.002
66	1	211	164	259	4811	2500	22127	57531	0.008	0.005	32640	0.01	0.002
67	1	211	164	259	4803	2500	22127	57531	0.008	0.005	32640	0.01	0.002
68	1	211	165	259	4795	2500	22127	57531	0.008	0.005	32683	0.01	0.002
69	1	212	165	259	4788	2500	22127	57531	0.008	0.005	32683	0.01	0.002
70	1	212	166	260	4779	2500	22127	57531	0.008	0.005	32741	0.01	0.002
71	1	212	166	260	4771	2500	22127	57531	0.008	0.005	32741	0.01	0.002
72	1	213	166	260	4763	2500	22127	57531	0.008	0.005	32799	0.01	0.002
73	1	213	167	260	4754	2500	22127	57531	0.008	0.005	32799	0.01	0.002
74	1	213	167	261	4746	2500	22127	57531	0.008	0.005	32842	0.01	0.002
75	1	214	168	261	4737	2500	22127	57531	0.008	0.005	32842	0.01	0.002
76	1	214	168	261	4728	2500	22127	57531	0.008	0.005	32871	0.01	0.002
77	1	214	169	262	4719	2500	22127	57531	0.008	0.005	32871	0.01	0.002
78	1	215	169	262	4709	2500	22127	57531	0.008	0.005	32871	0.01	0.002
79	1	215	170	262	4700	2500	22127	57531	0.008	0.005	32957	0.01	0.002
80	1	215	170	262	4690	2500	22127	57531	0.008	0.005	33015	0.01	0.002
81	1	216	170	263	4681	2500	22127	57531	0.008	0.005	33015	0.01	0.002
82	1	216	171	263	4671	2500	22127	57531	0.008	0.005	33087	0.01	0.002
83	1	217	171	263	4661	2500	22127	57531	0.008	0.005	33130	0.01	0.002
84	1	217	172	264	4651	2500	22127	57531	0.008	0.005	33130	0.01	0.002
85	1	217	172	264	4640	2500	22127	57531	0.008	0.005	33173	0.01	0.002
86	1	218	173	264	4630	2500	22127	57531	0.008	0.005	33173	0.01	0.002
87	1	218	173	265	4620	2500	22127	57531	0.008	0.005	33332	0.01	0.002
88	1	219	174	265	4609	2500	22127	57531	0.008	0.005	33332	0.01	0.002
89	1	219	174	265	4598	2500	22127	57531	0.008	0.005	33433	0.01	0.002
90	1	219	175	266	4587	2500	22127	57531	0.008	0.005	33476	0.01	0.002
91	1	220	175	266	4576	2500	22127	57531	0.008	0.005	33476	0.01	0.002
92	1	220	176	266	4565	2500	22127	57531	0.008	0.005	33534	0.01	0.002
93	1	221	176	267	4554	2500	22127	57531	0.008	0.005	33591	0.01	0.002
94	1	221	177	267	4543	2500	22127	57531	0.008	0.005	33649	0.01	0.002
95	1	221	178	267	4531	2500	22127	57531	0.008	0.005	33649	0.01	0.002
96	1	222	178	268	4520	2500	22127	57531	0.008	0.005	33707	0.01	0.002
97	1	222	179	268	4508	2500	22127	57531	0.008	0.005	33721	0.01	0.002
98	1	223	179	269	4496	2500	22127	57531	0.008	0.005	33721	0.01	0.002
99	1	223	180	269	4484	2500	22127	57531	0.008	0.005	33779	0.01	0.002
100	1	224	180	269	4472	2500	22127	57531	0.008	0.004	33836	0.01	0.002
101	1	224	181	270	4460	2500	22127	57531	0.008	0.004	33836	0.01	0.002
102	1	225	181	270	4448	2500	22127	57531	0.008	0.004	33894	0.01	0.002
103	1	225	182	270	4436	2500	22127	57531	0.008	0.004	33937	0.01	0.002
104	1	225	183	271	4423	2500	22127	57531	0.008	0.004	33966	0.01	0.002
105	1	226	183	271	4411	2500	22127	57531	0.008	0.004	33966	0.01	0.002
106	1	226	184	272	4398	2500	22127	57531	0.008	0.004	33995	0.01	0.002
107	1	227	184	272	4386	2500	22127	57531	0.008	0.004	34053	0.01	0.002
108	1	227	185	272	4373	2500	22127	57531	0.008	0.004	34053	0.01	0.002
109	1	228	185	273	4360	2500	22127	57531	0.008	0.004	34096	0.01	0.002
110	1	228	186	273	4347	2500	22127	57531	0.008	0.004	34139	0.01	0.002
111	1	229	187	274	4334	2500	22127	57531	0.008	0.004	34226	0.01	0.002
112	1	229	187	274	4321	2500	22127	57531	0.008	0.004	34327	0.01	0.002
113	1	230	188	274	4308	2500	22127	57531	0.007	0.004	34356	0.01	0.002
114	1	230	188	275	4295	2500	22127	57531	0.007	0.004	34471	0.01	0.001
115	1	231	189	275	4282	2500	22127	57531	0.007	0.004	34485	0.01	0.001
116	1	231	190	276	4269	2500	22127	57531	0.007	0.004	34485	0.01	0.001
117	1	232	190	276	4255	2500	22127	57531	0.007	0.004	34529	0.01	0.001

APPENDIX B – BEARING CAPACITY AND SETTLEMENT

Table B6-4. Results of elastic settlement analyses.

Load (ksf)	V_s Bound	G / G_{max} Bound	Depth (ft)	V_s (ft/s)	Settlement Under Center of Mat					Settlement Under Corner of Mat				
					Sett (in)	Emax (ksf)	$\varepsilon_{a\_initial}$ (%)	E final (ksf)	ε_{a_final} (%)	Sett (in)	Emax (ksf)	$\varepsilon_{a\_initial}$ (%)	E final (ksf)	ε_{a_final} (%)
3	Lower	Lower	0 - 12	1200	**0.8**	13255	0.01	1028	0.29	**0.1**	13255	0.00	6776	0.01
			13 - 27	1400		18042	0.01	2733	0.11		18042	0.00	10806	0.01
			28 - 57	2000		36820	0.00	14533	0.02		36820	0.00	28364	0.00
			58 - 117	2200		44552	0.00	20633	0.01		44552	0.00	35784	0.00
	Lower	Mean	0 - 12	1200	**0.4**	13255	0.01	2352	0.13	**0.0**	13255	0.00	8929	0.01
			13 - 27	1400		18042	0.01	5535	0.05		18042	0.00	13459	0.01
			28 - 57	2000		36820	0.00	21176	0.01		36820	0.00	31670	0.00
			58 - 117	2200		44552	0.00	28313	0.01		44552	0.00	39328	0.00
	Mean	Mean	0 - 12	1500	**0.2**	20711	0.01	7693	0.04	**0.0**	20711	0.00	16038	0.00
			13 - 27	1700		26602	0.01	12520	0.02		26602	0.00	21773	0.00
			28 - 57	2200		44552	0.00	28045	0.01		44552	0.00	39305	0.00
			58 - 117	2500		57531	0.00	40151	0.01		57531	0.00	52057	0.00
5	Lower	Lower	0 - 12	1200	**2.8**	13255	0.02	587	0.85	**0.1**	13255	0.01	4494	0.03
			13 - 27	1400		18042	0.02	798	0.63		18042	0.00	8093	0.02
			28 - 57	2000		36820	0.01	7823	0.06		36820	0.00	23879	0.01
			58 - 117	2200		44552	0.01	12850	0.04		44552	0.00	31194	0.00
	Lower	Mean	0 - 12	1200	**1.6**	13255	0.02	818	0.61	**0.1**	13255	0.01	7082	0.02
			13 - 27	1400		18042	0.02	2208	0.23		18042	0.00	11217	0.01
			28 - 57	2000		36820	0.01	14626	0.03		36820	0.00	28954	0.00
			58 - 117	2200		44552	0.01	21503	0.02		44552	0.00	36521	0.00
	Mean	Mean	0 - 12	1500	**0.5**	20711	0.01	3326	0.15	**0.1**	20711	0.00	13629	0.01
			13 - 27	1700		26602	0.01	6381	0.08		26602	0.00	19079	0.01
			28 - 57	2200		44552	0.01	21057	0.02		44552	0.00	36509	0.00
			58 - 117	2500		57531	0.00	32481	0.01		57531	0.00	49051	0.00
7	Lower	Lower	0 - 12	1200	**4.4**	13255	0.03	587	1.19	**0.2**	13255	0.01	2879	0.06
			13 - 27	1400		18042	0.02	798	0.88		18042	0.01	6013	0.03
			28 - 57	2000		36820	0.01	4199	0.17		36820	0.00	20760	0.01
			58 - 117	2200		44552	0.01	7666	0.09		44552	0.00	27242	0.01
	Lower	Mean	0 - 12	1200	**2.9**	13255	0.03	818	0.86	**0.1**	13255	0.01	5421	0.03
			13 - 27	1400		18042	0.02	1114	0.63		18042	0.01	9476	0.02
			28 - 57	2000		36820	0.01	8721	0.08		36820	0.00	26333	0.01
			58 - 117	2200		44552	0.01	15476	0.04		44552	0.00	33784	0.01
	Mean	Mean	0 - 12	1500	**1.3**	20711	0.02	1587	0.44	**0.1**	20711	0.00	11693	0.01
			13 - 27	1700		26602	0.02	3717	0.19		26602	0.00	16906	0.01
			28 - 57	2200		44552	0.01	14951	0.05		44552	0.00	33739	0.01
			58 - 117	2500		57531	0.01	26038	0.03		57531	0.00	46352	0.00

Notes:

1. Assume 120 ft thick Alluvium layer
2. Assume that upper 3 ft of Alluvium will be removed for mat thickness (120 - 3 = 117 total feet of alluvium)
3. Assume Poisson's ratio of Alluvium = 0.3
4. Assume unit weight of soil = 114 pcf
5. Mat dimensions, B = 300 ft, L = 400 ft
6. G/Gmax for Seed and Idriss (1970) curve assumed constant for strains > 1%
7. Iterate strains until difference < 0.001%

B7 Results and Conclusion

The following figures and table summarize the results of the bearing capacity and settlement analyses contained herein:

Figure	Descpription
B7-1	Allowable foundation pressure for square and strip footings on alluvium vs. foundation width and foundation embedment (1-inch design settlement).
B7-2	Allowable foundation pressure for square and strip footings on alluvium vs. foundation width and foundation embedment (½-inch design settlement).
B7-3	Immediate settlements for different widths of square and strip footings on alluvium vs. foundation pressure ($d_f = 2$ ft).
B7-4	Immediate settlements for different widths of square and strip footings on alluvium vs. foundation pressure ($d_f = 6$ ft).
B7-5	Long-term settlements for square and strip footings with different depths of foundation embedment.
B7-6	Elastic settlement of mat foundation (3 ksf vertical load).
B7-7	Elastic settlement of mat foundation (5 ksf vertical load).
B7-8	Elastic settlement of mat foundation (7 ksf vertical load).

Table	Description
B7-1	Results of elastic settlement of 400’ × 300’ mat foundation analyses

Figures B7-1 through B7-4 pertain to bearing capacity and immediate settlement calculations for shallow square and strip footings. For these figures, the results from the Schmertmann method are used since more data (shear wave velocity) is available for this method. The Burland and Burbidge (Terzaghi et al. 1996) method, which is based on blow counts, was not considered reliable. Few blow counts were recorded at the site and due to the high gravel content of the alluvium, are not representative of the more compressible matrix material.

Figure B7-5 presents the long-term settlements evaluation for square and strip footings using the Burland and Burbidge (Terzaghi et al. 1996) method.

Figures B7-6 to B7-8 show the variation with depth of percent of total settlement and percent strain for elastic settlements in the center and corner of a mat foundation. A summary of the predicted total and maximum differential elastic settlements (center and corner of mat foundation) is shown in Table B6-1. The predicted settlements are considered to be very conservative due to the assumption that Young's modulus degrades the same as the shear modulus for sands (alluvium). In actuality, the predicted settlements should be less. Additionally, for the elastic settlement analyses, the stiffness of the mat foundation is not considered to redistribute the loads.

The results of the analyses contained herein appear reasonable for the design of foundations for the expected loading at the Yucca Mountain Project.

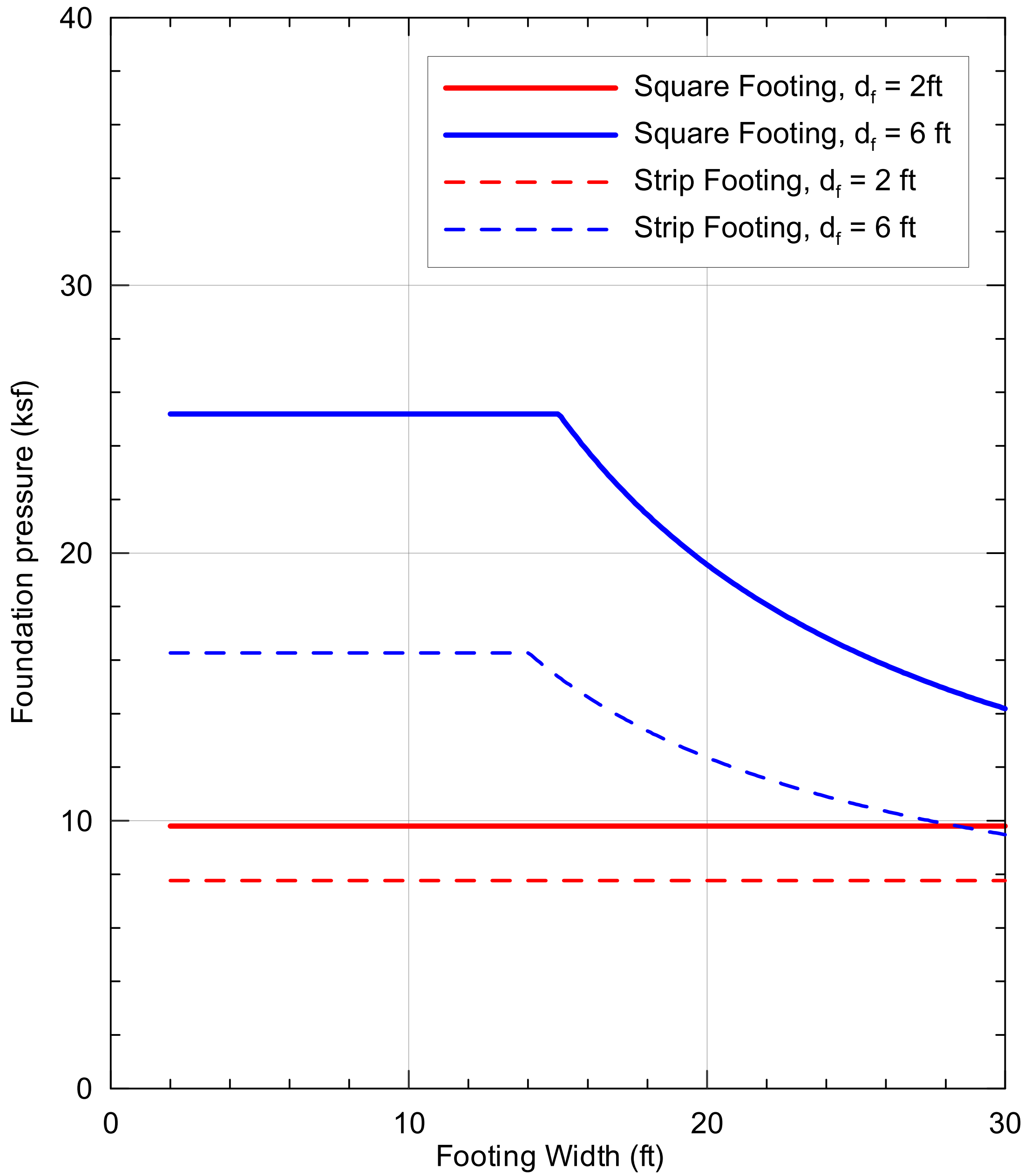


Figure B7-1. Allowable foundation pressure for square and strip footings on alluvium vs. foundation width and foundation embedment (1-inch design settlement).

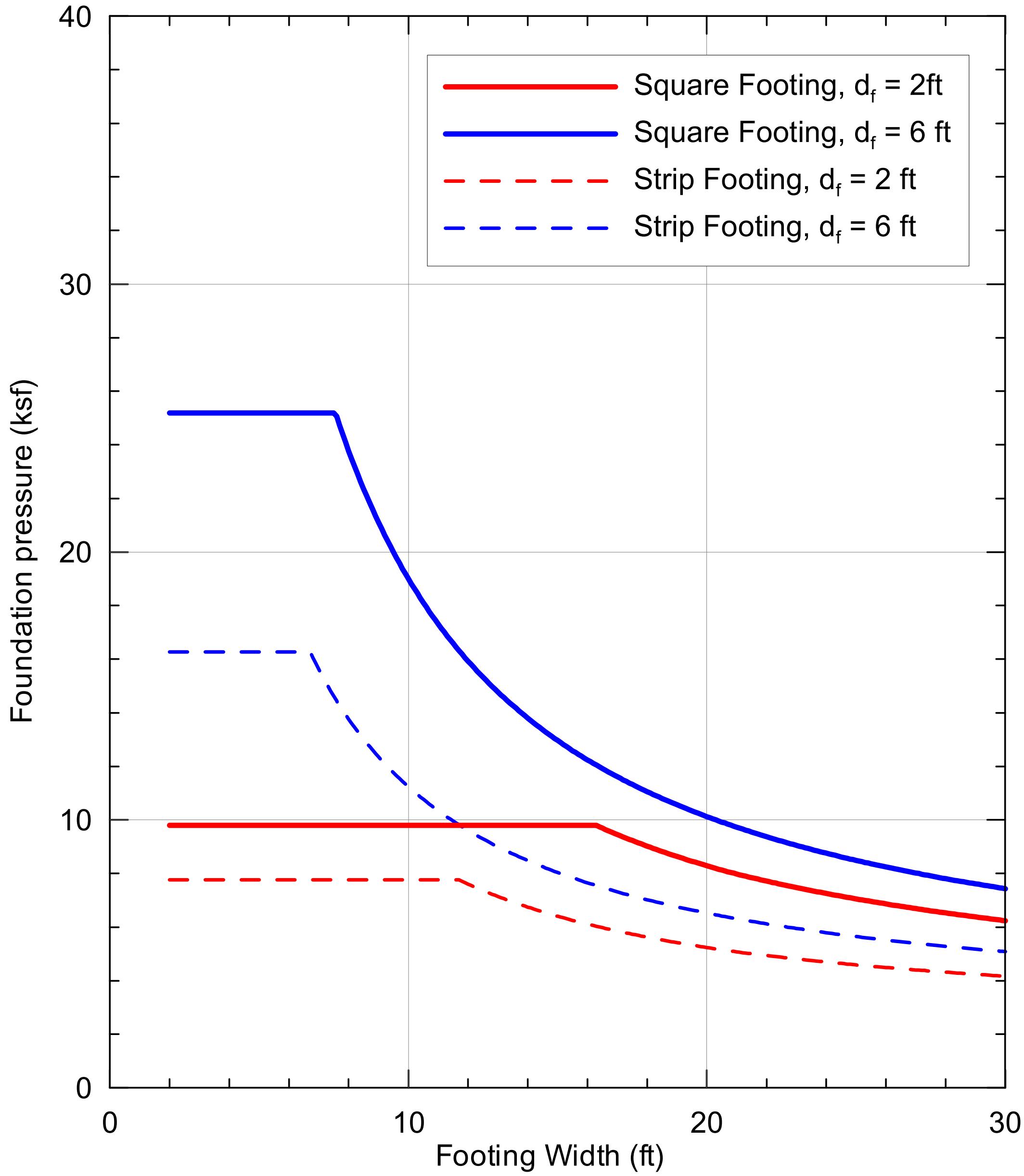


Figure B7-2. Allowable foundation pressure for square and strip footings on alluvium vs. foundation width and foundation embedment (½-inch design settlement).

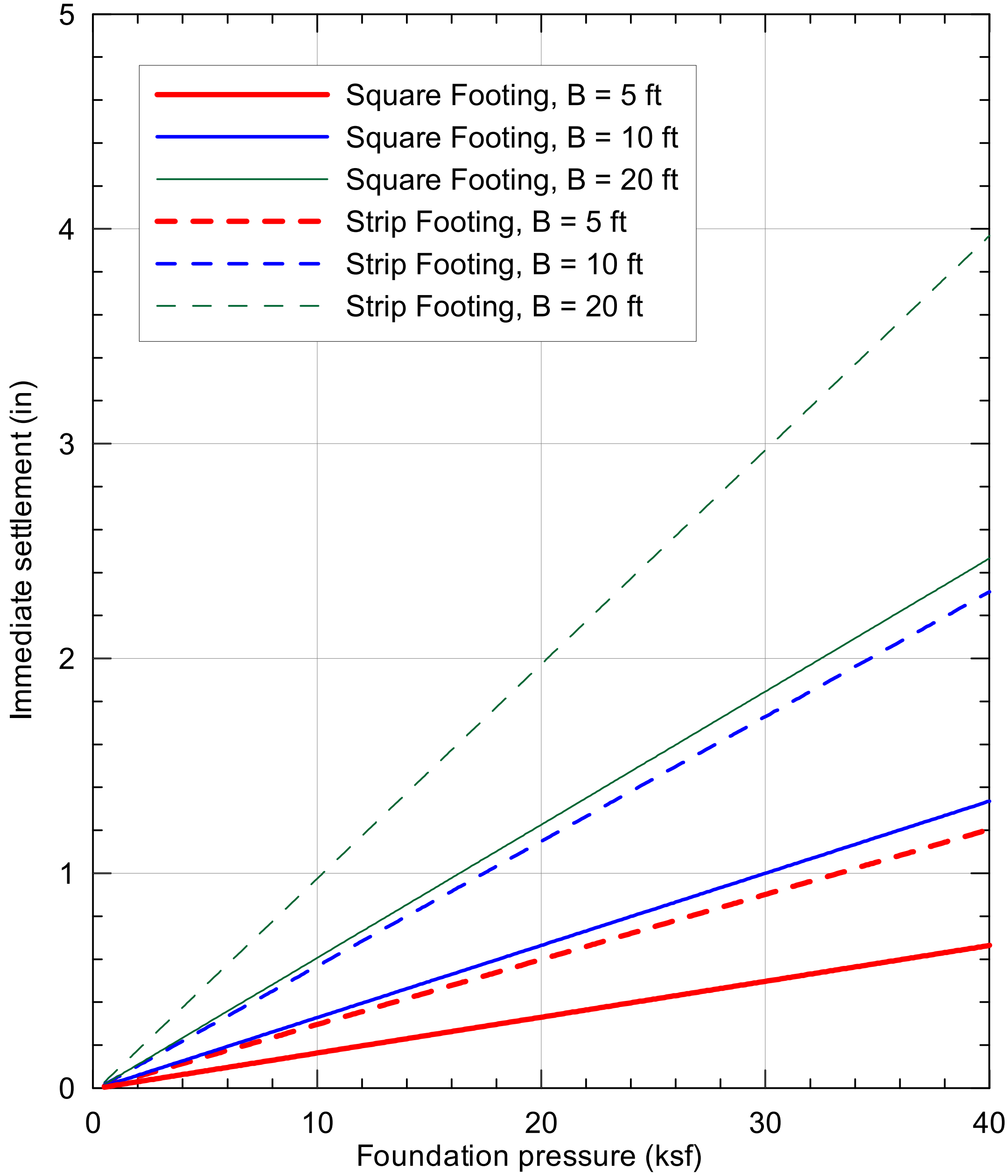


Figure B7-3. Immediate settlements for different widths of square and strip footings on alluvium vs. foundation pressure (d_f = 2 ft)

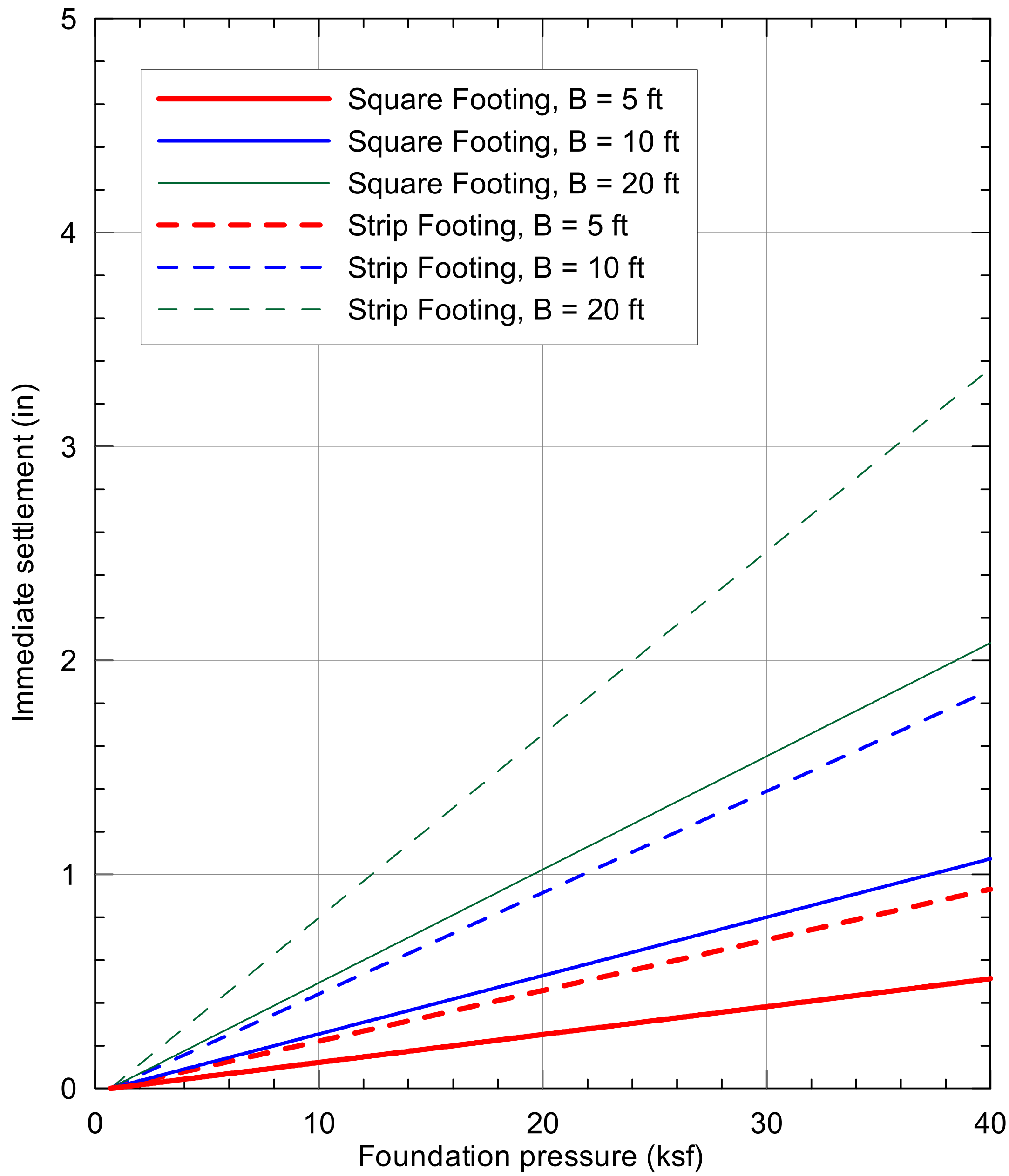


Figure B7-4. Immediate settlements for different widths of square and strip footings on alluvium vs. foundation pressure (d_f = 6 ft).

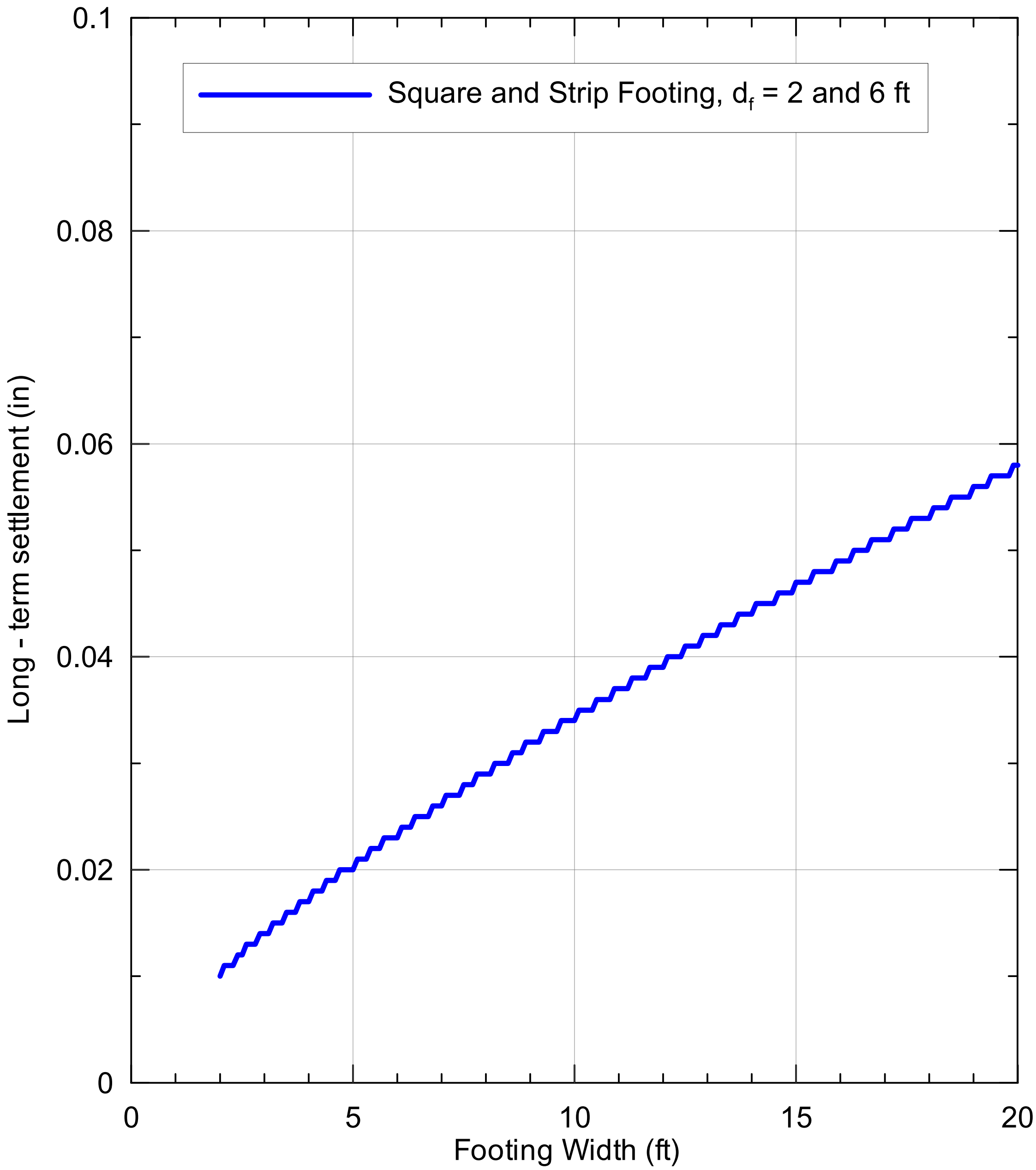


Figure B7-5. Long-term settlements for square and strip footings and different depths of foundation embedment.

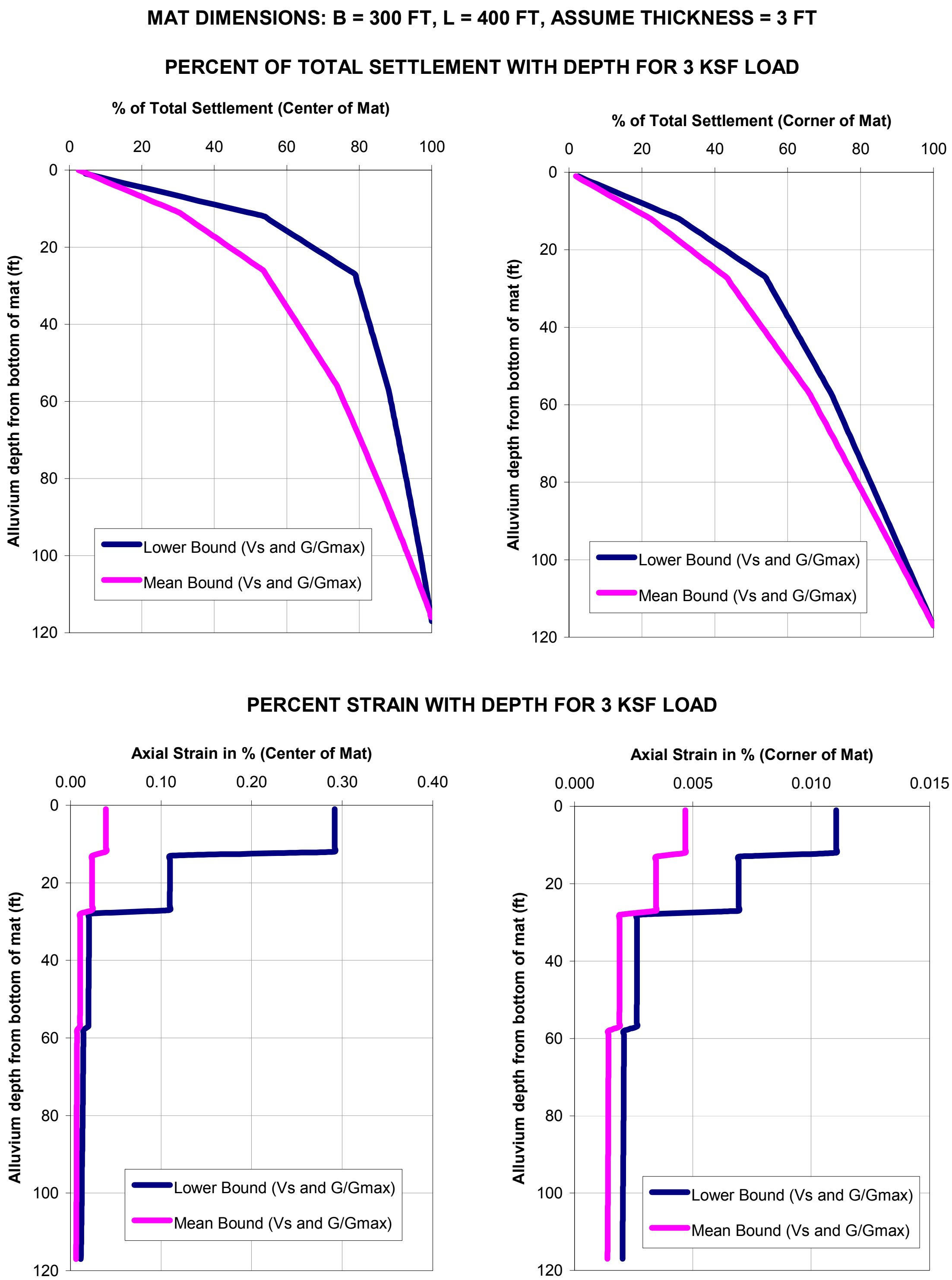
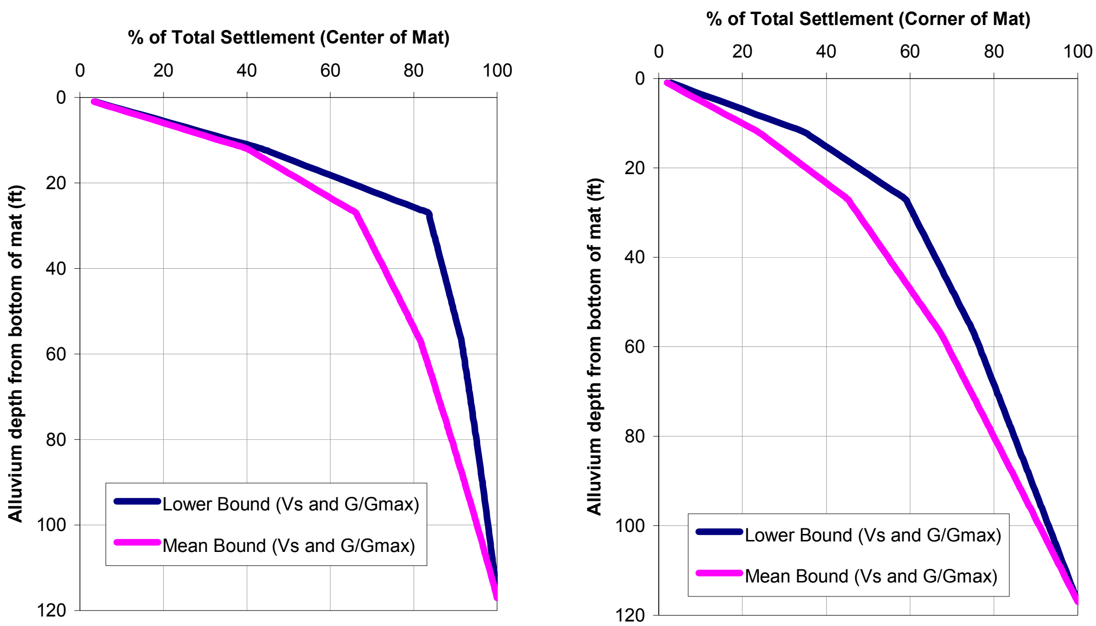


Figure B7-6. Elastic settlement of mat foundation (3 ksf vertical load).

APPENDIX B – BEARING CAPACITY AND SETTLEMENT

MAT DIMENSIONS: B = 300 FT, L = 400 FT, ASSUME THICKNESS = 3 FT

PERCENT OF TOTAL SETTLEMENT WITH DEPTH FOR 5 KSF LOAD



PERCENT STRAIN WITH DEPTH FOR 5 KSF LOAD

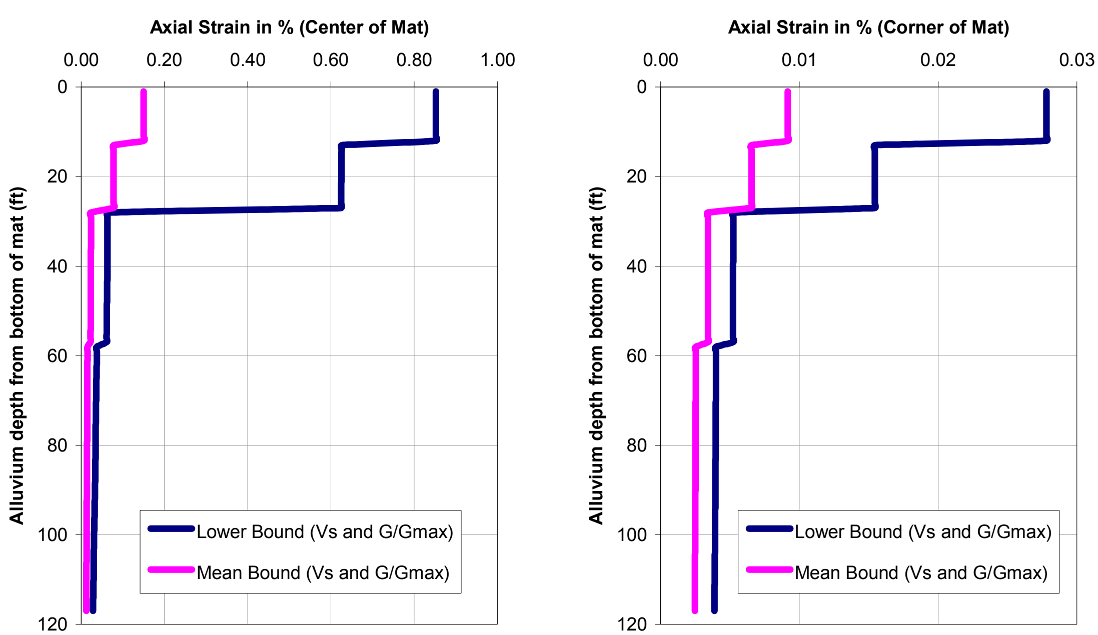


Figure B7-7. Elastic settlement of mat foundation (5 ksf vertical load).

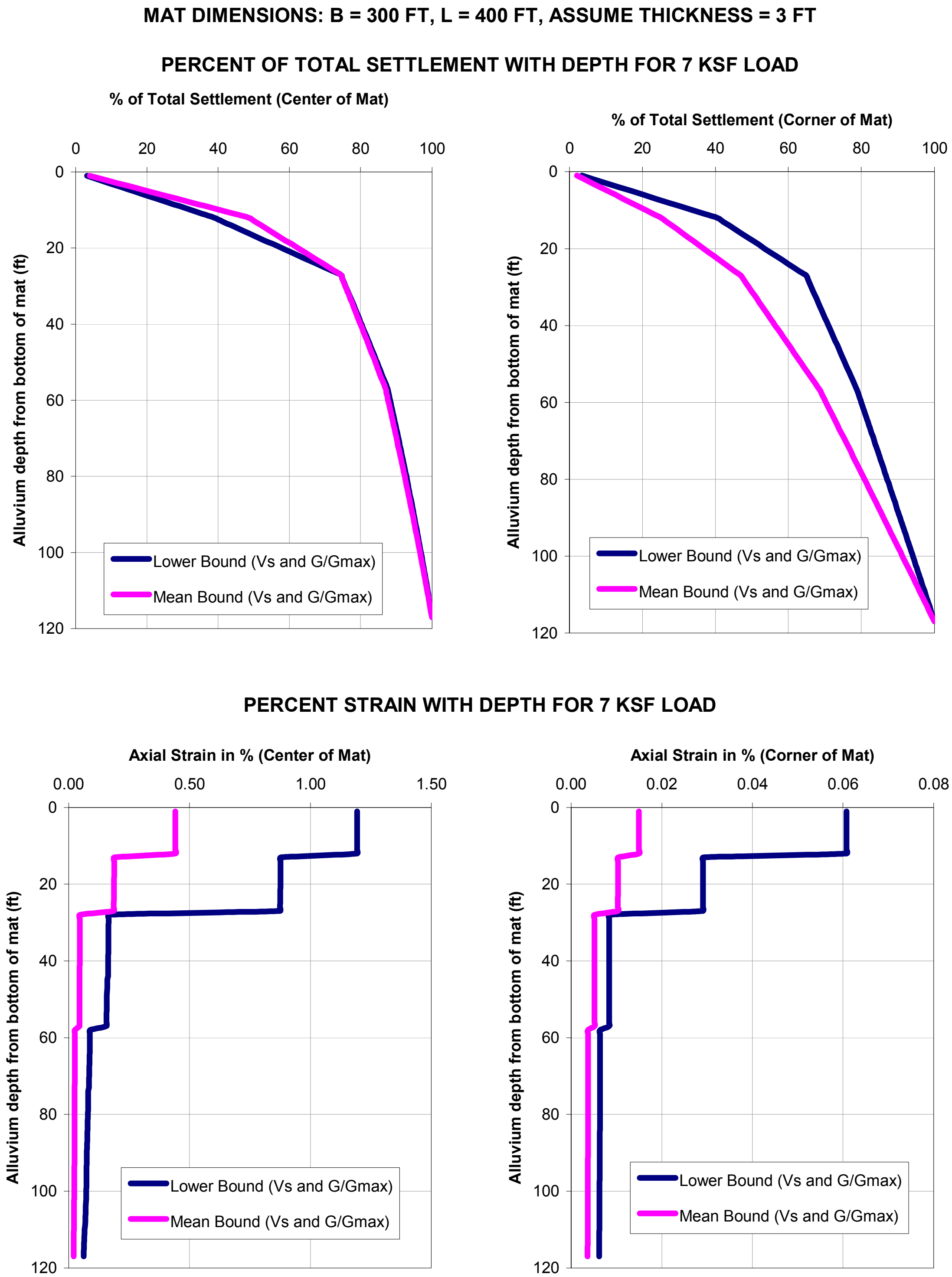


Figure B7-8. Elastic settlement of mat foundation (7 ksf vertical load).

Table B7-1. Results of elastic settlement of 400’ × 300’ mat foundation analyses.

Load (ksf)	Total Settlement		Maximum Differential
	Center of Mat	Corner of Mat	Corner to Center of Mat (340ft)
3	0.2 – 0.4 in	Negligible	0.4 in
5	0.5 – 1.6 in	~ 0.1 in	1.5 in
7	1.3 – 3 in	~ 0.1 in	3 in

Appendix C Contents

Page Number

C1 Objective

The purpose of this analysis is to estimate the potential lateral pressures acting at the Waste Handling Building (WHB) at Yucca Mountain for yielding and non-yielding walls under static and dynamic conditions. Lateral pressures due to roller and plate compactors and surcharge loads, and lateral pressures acting on temporary shoring are also considered.

Calculations for the resistance to lateral loads resulting from passive resistance or base friction are also performed.

C2 Inputs

Table C2-1 below lists the parameters used in the analysis contained herein (as determined in Section 6.4.1 of this study). Although engineered backfill may be used locally at the site (and is stronger than alluvium), the properties of the alluvium are used in the calculations of this analysis for conservatism. A horizontal seismic coefficient, k_h, and Poisson's ratio, υ, are necessary to determine the dynamic lateral earth pressures and, hence, are also listed below. A coefficient of horizontal acceleration, k_h, of 1 g is used in the analysis so that it may be scaled for any selected peak ground acceleration, PGA.

Table C2-1. Parameter Inputs

Parameter	Alluvium	Engineered Fill
Friction angle, ϕ	39 deg	42 deg
Unit weight, γ	117 pcf	127 pcf
Horizontal seismic coefficient, k_h	1.0*	-
Poisson's ratio, υ	0.3	-

*to be scaled for any selected PGA

Several input parameters are needed in order to estimate the lateral earth pressures created from compaction equipment acting on the soil. Table C2-2 below lists the input parameters used in the analysis herein. If a plate compactor is considered, the width and length of the particular equipment is needed for the analysis and thus is also shown in the table below.

Table C2-2. Compaction Equipment Inputs (Duncan et al. 1991).

Compactor		Static & Dynamic Force (lbf)	Roller Width (in)	Plate Width (in)	Plate Length (in)	Compactor Distance from Wall (ft)
Name	Type					
Dynapac CA15D	Single-drum vibratory roller	28,800	66	-	-	2, 3, 5
Dynapac CA25	Single-drum vibratory roller	55,800	84	-	-	2, 3, 5
Ingersoll-Rand DX-70	Walk-behind vibratory roller	6,000	25	-	-	0.5, 1, 2
Bomag BP30	Vibratory plate	6,830	-	15	31.1	0, 0.5, 1
Wacker BS62Y	Rammer plate	3,140	-	13	13	0, 0.5, 1

C3 Background

The surface of the WHB area is currently covered by generally 5 to 50 feet of existing fill. One isolated location recorded 28 ft of fill, but there is some doubt as to its validity (see Section 5, Assumption 10 of BSC 2002b). The fill is underlain by approximately 2 to 192 feet of alluvium (Table 6-1). It is understood that all of the existing fill is to be removed, and the WHB facility will lie directly on the alluvium.

At this time, a 55-foot below-grade pool is planned to be constructed within the wet-process building. Upon completion of its walls, backfill will be placed against it. Hence, stresses induced by compaction equipment must be considered in calculating the earth pressures acting on this wall.

Due to continuous changes in design, other walls (yielding and non-yielding) may potentially be constructed at the YMP site.

C4 Methodology

The following sections outline the methods used and provide the theory and references that are adopted for the analysis.

C4.1 Static lateral earth pressures

The static analysis is based on the Rankine theory (Fang 1991) for determining earth pressures acting on a wall. Lateral at-rest (for non-yielding walls) and active (for yielding walls) earth pressure forces for a vertical wall with horizontal backfill are determined.

C4.2 Dynamic lateral earth pressures (yielding walls)

The seismic analysis for yielding walls is based on simplified methods to determine the dynamic active earth pressure force. The simplified method developed by Seed and Whitman (1970) is used in this analysis to determine the seismic active pressure force increment.

C4.3 Dynamic lateral earth pressures (non-yielding walls)

Procedures outlined in Section 3.5 of ASCE 4-98 are followed to determine the seismic stress increment acting on non-yielding walls. The analysis does not include the dynamic contribution due to surcharge loads.

C4.4 Surcharge pressures

Static lateral surcharge pressures for non-yielding walls are determined based on elastic solutions. Equations used for various surcharge loads for yielding walls (USN 1986) are shown in Figure C7-2 and Figure C7-3 of this appendix. Live loads are not considered in the analysis.

C4.5 Compaction-induced pressures

Procedures outlined in Duncan and Seed (1986), Duncan et al. (1991), and USN (1986) are followed to determine the additional lateral earth pressures that will develop due to various types of compaction equipment. A comparison with the method outlined in USN (1986) is also performed.

C4.6 Temporary shoring pressure

The pressure acting on temporary shoring during excavation of the alluvium is estimated using Figure 12.22e of Fang (1991) for soldier piles.

C4.7 Resistance to lateral loads

Sliding friction is estimated based on Table 1 of USN (1986) and per recommendations in BSC (2002a). The Rankine theory is used to estimate passive resistance.

C5 Assumptions

The following assumptions are made in the analysis:

- Walls are to be vertical with a horizontal backfill.
- Groundwater is deep enough that it will not affect the lateral earth pressures.
- Bedrock is deep enough that it will not affect the lateral earth pressures.
- Wall friction is conservatively assumed to be zero.

All of these assumptions are either sufficiently conservative or represent typical standards used in the industry and do not require further verification.

C6 Calculations

All calculations are conducted using the computer program Mathcad. The Mathcad worksheets containing the calculations are all located in Section C8 of this calculation. The following sections outline the procedures performed.

C6.1 Static lateral earth pressures

The following equations are used to determine the static earth pressure coefficients, K, for various conditions:

$$K_o = 1 - \sin\phi \qquad \text{At-rest} \qquad \text{(C1)}$$

$$K_A = \tan\left(45 - \frac{\phi}{2}\right)^2 \qquad \text{Active} \qquad \text{(C2)}$$

$$K_P = \tan\left(45 + \frac{\phi}{2}\right)^2 \qquad \text{Passive} \qquad \text{(C3)}$$

The distributed pressure and resultant force of each condition are calculated using the following equations:

$$p = K\gamma H \qquad \text{Distributed pressure} \qquad \text{(C4)}$$

$$P = K\gamma\frac{H^2}{2} \qquad \text{Resultant force} \qquad \text{(C5)}$$

where,

H = height of wall

Pressure distribution diagrams are shown in Section C7 of this calculation. Table C6-1 below shows the computed static earth pressure coefficients using the properties of the alluvium. Equivalent fluid weights, $K\gamma$, is multiplied by the wall height, H, to determine lateral earth pressures.

Table C6-1. Earth Pressure Coefficients.

Condition	Earth Pressure Coefficient, K	
	Alluvium	Engineered Fill
At-Rest, K_o	0.37	0.33
Active, K_A	0.23	0.20
Passive, K_P	4.4	5.0

C6.2 Dynamic Lateral Pressures

Using the simplified method developed by Seed and Whitman (1970), the seismic active earth pressure increment coefficient for a yielding wall is calculated using the following equation:

$$\Delta K_{AE} = \frac{3}{4} k_h \qquad \text{(C6)}$$

As stated in Section C2, a coefficient of horizontal acceleration, k_h, of 1 g is used in the analysis so that it may be scaled to any given PGA. The distributed pressure and resultant force increment are calculated using the following equations:

$$\Delta p_{ae} = \Delta K_{AE} \gamma H \qquad \text{Distributed pressures} \qquad \text{(C7)}$$

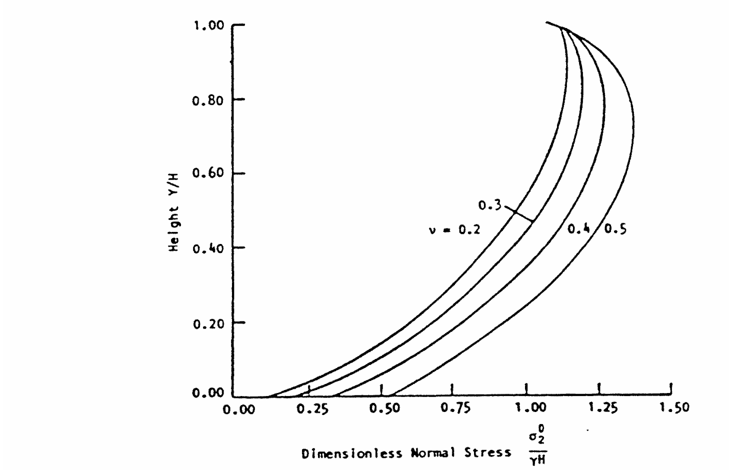
$$\Delta P_{AE} = \Delta K_{AE} \gamma \frac{H^2}{2} \qquad \text{Resultant force} \qquad \text{(C8)}$$

Seed and Whitman (1970) suggest that the component of the resultant force may be taken to act at approximately 0.6H above the wall base. The sum of the initial static active earth pressure force (equation C5), and the dynamic active earth pressure force increment (equation C8) produces the dynamic lateral pressure for a yielding wall:

$$P_{AE} = \Delta P_{AE} + P_A \qquad \text{(C9)}$$

For non-yielding walls, procedures outlined in Section 3.5 of ASCE 4-98 are followed to determine the incremental stresses developed due to seismic loading. A conservative estimate of the dynamic soil pressures may be obtained from Figure 3500-1 of ASCE 4-98 shown as Figure C6-1 below.

APPENDIX C – LATERAL EARTH PRESSURES AND RESISTANCE TO LATERAL LOADS



Explanation

H	=	embedment height
Y	=	distance from base of retaining structure
γ	=	soil unit weight
υ	=	Poisson's ratio
σ_2^0	=	lateral dynamic soil pressure against the retaining structure for 1.0g horizontal earthquake acceleration

Figure C6-1. Variation of normal dynamic soil pressures for the elastic solution.

Assuming H = 50ft, γ = 117 pcf, and υ = 0.3 (Section 2) for the alluvium, the seismic pressure is determined using Figure C6-1. The pressure can then be scaled to any given coefficient of horizontal acceleration. For the above parameters, Figure C6-2 shows a plot of the seismic pressure coefficient scaled to 1g acceleration versus the unit height of a non-yielding wall. Note that the analysis does not include the dynamic contribution due to surcharge loads.

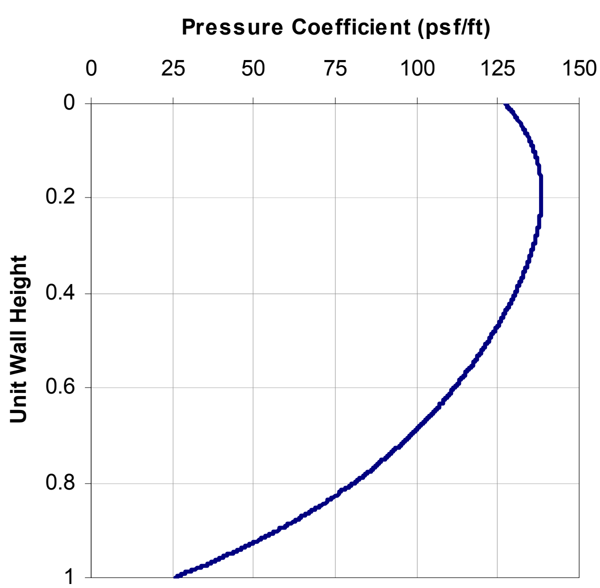


Figure C6-2. Seismic pressure coefficient scaled to 1g versus unit height for non-yielding walls (per ASCE 4-98).

C6.3 Surcharge pressures

Static surcharge pressures for non-yielding walls may be calculated as K_o x q where K_o is the static earth pressure coefficient at rest (0.37), and q is the surcharge load to be applied. The pressure distribution diagrams are shown in Section 7 of this calculation. For yielding walls, refer to the schematic recommendations provided in Section C7 of this calculation. The analysis does not include live loads.

C6.4 Compaction-Induced Pressures

The procedures outlined in Duncan and Seed (1986) are followed to determine the incremental horizontal stresses due to compaction. The equation for the incremental horizontal pressure due to a point load ($\Delta\sigma_h$) presented in Poulos and Davis (1991) is used and modified to taken into account either a roller or plate compactor, the compactor distance from the wall, roller width, plate area, and friction angle of the soil.

Duncan et al. (1991) is used to select various compaction equipment (summarized in Table C2-2). Results are shown as lateral earth pressures due to compaction versus depth. For the analysis, the lateral pressure increment due to compaction is determined and limited to not exceed the passive earth pressures. The pressure increment linearly increases from the depth where it intersects the passive pressure line or where it the pressure is locally at a maximum value near the surface, whichever is larger, until it converges with the at-rest soil pressure line.

The calculations and equations used are provided in Section C8 of this calculation. To avoid redundancy, only one sample calculation for a roller compactor is shown. A check of the results against recommendations from USN (1986) is also included. Figure C6-3 shown below from the USN (1986) manual is used for the check.

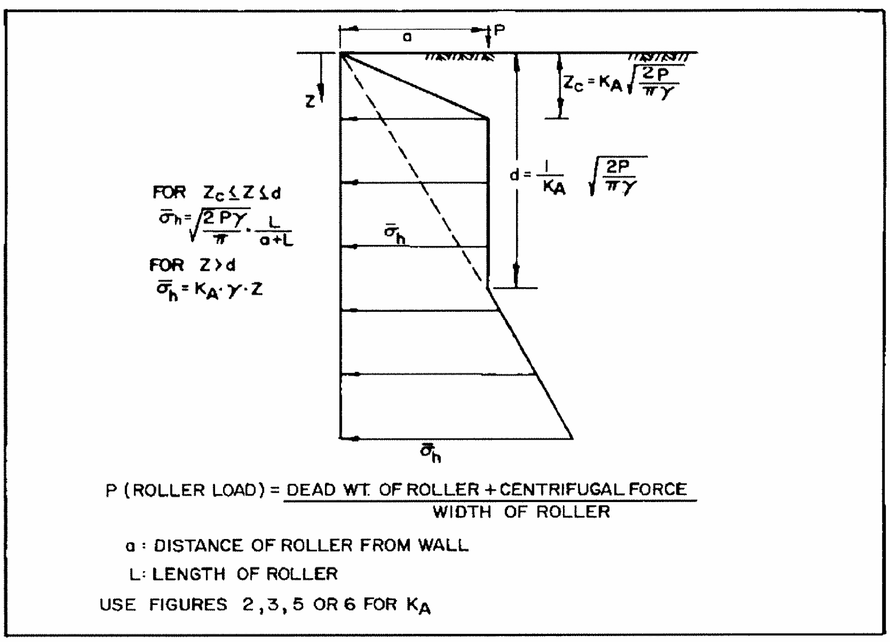


Figure C6-3. Design pressure envelope for non-yielding walls with compaction effects (Figure 13 from USN 1986).

Results of the analysis are shown in Section C7 of this calculation.

C6.5 Temporary Shoring Pressure

The pressure of the alluvium acting on temporary shoring provided by soldier piles are estimated using Figure 12.22e in Fang (1991) for sands. The pressure is considered to be uniform acting on the full height of the shoring wall and is expressed by:

$$p_{ts} = 0.65 K_A \gamma H = 17.5H \qquad \text{(C10)}$$

C6.6 Resistance to Lateral Loads

Resistance to lateral loads can be developed from passive pressure against the vertical face of the sub-grade walls and footings, and from the friction against the base.

C6.6.1 Passive Pressures

The coefficient for resistance developed from passive pressures was calculated in Section C6.1. The distributed passive pressure is calculated to be:

$$p_p = K_P \gamma H = 515H \qquad \text{(C11)}$$

C6.6.2 Interface Friction Coefficient

The interface resistance between the soil and structures placed in it is a function of the soil and the structure. Typically, the interface friction coefficient, f, is estimated to be equal to tan ϕ, where ϕ is the internal friction angle of the soil. Other adjustments, based on the structural material type and a factor of safety, FS, are also included in the final design value.

The recommended interface friction coefficient between alluvium and concrete is derived from consideration of the soil internal friction angle determined in Section 6.4.1 of this study and recommended typical values of interface friction angles published in the literature as described below.

- Internal friction angle, ϕ (see Section C2) = 39 deg
- Estimated base friction, f = tan 39 deg = 0.81

Bowles 1996 recommends f = tan (ϕ). This corresponds to f = 0.81.

USN 1986 (pp. 7.2-121) recommends for cofferdam allowable design to use f = 0.5 on smooth rock, or tan(ϕ) otherwise. The worst case is 0.5 (for steel acting against soil). USN 1986 (pp. 7.2-63) recommends ultimate interface friction coefficients between mass concrete and the following soils:

- Clean gravel, gravel sand mixtures and coarse sand 0.55 to 0.6
- Clean fine to med. sand, silty med. to coarse sand, silty or clayey gravel 0.45 to 0.55
- Clean fine sand, silty or clayey fine to med. sand 0.35 to 0.45

The alluvium materials at the site consist of coarse sand and gravel. Hence, the average ultimate interface friction coefficient between mass concrete and the alluvial material is estimated to be about 0.55.

It is recommended to use **0.81** as the ultimate friction coefficient for the alluvium. The ultimate interface resistance for engineered fill is calculated in the same fashion as the alluvium, except that the internal friction angle is 42 degrees. The ultimate interface friction coefficient for the engineered fill is determined to be 0.90. For engineered fill, a reduced value corresponding to a factor of safety of at least 1.5, should be used when determining the overall resistance against sliding for a structural element.

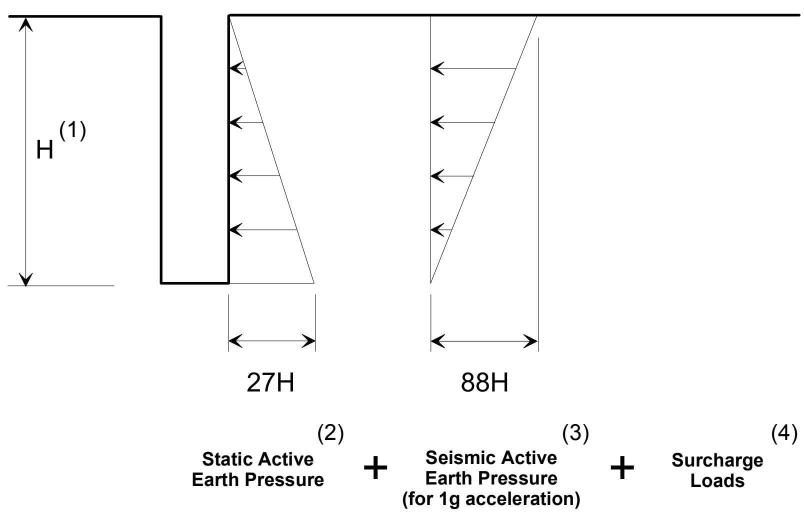
C7 Results / Conclusions

C7.1 Lateral Earth Pressures on Yielding Walls:

The combined lateral earth pressures acting on a yielding wall are as follows:

Static Active + Seismic Active Increment + Static Surcharge

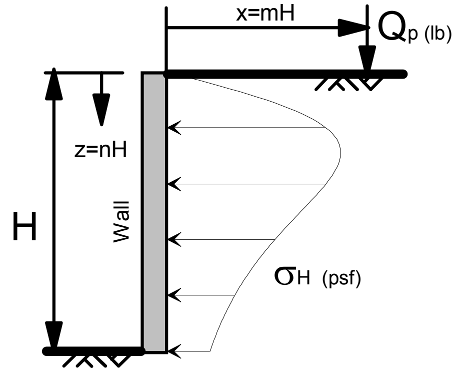
Figure C7-1, Figure C7-2, and Figure C7-3 show the pressure distribution sketch.



Notes:

(1) Height of wall, H, is presented in feet.
(2) Static active earth pressure for alluvium: $K_A = 0.23$, $\gamma = 117$ pcf.
(3) Seismic active earth pressure for alluvium based on Seed and Whitman (1970) simplified method where $K_h = 1g$ (to be scaled by actual peak ground acceleration, PGA).
(4) Surcharge loads are shown in next figure.
(5) Pressures are presented in psf.

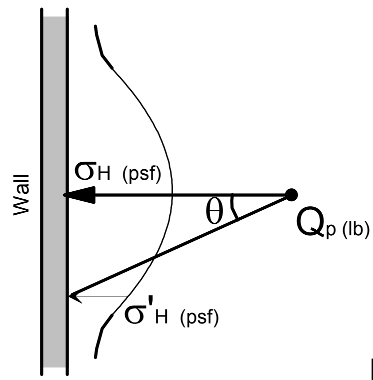
Figure C7-1. Pressure Distribution Sketch for Yielding Walls (not to scale)



$$\sigma_H = 0.28\,\frac{Q_p}{H^2}\,\frac{n^2}{(0.16+n^2)^3} \quad (\text{for } m \le 0.4)$$

$$\sigma_H = 1.77\,\frac{Q_p}{H^2}\,\frac{m^2n^2}{(m^2+n^2)^3} \quad (\text{for } m > 0.4)$$

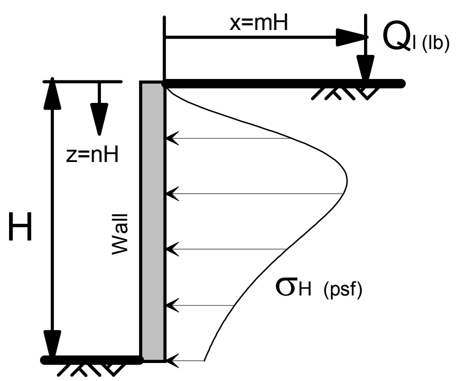
Elevation View



$$\sigma'_H = \sigma_H \cos^2(1.1\theta)$$

Plan View

Lateral Pressure due to Point Load



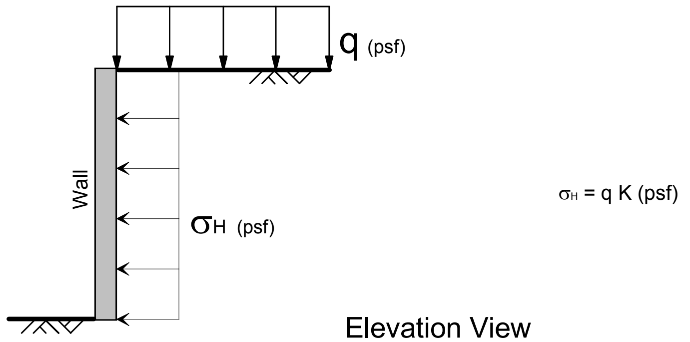
$$\sigma_H = 0.20\,\frac{Q_l}{H}\,\frac{n}{(0.16+n^2)^2} \quad (\text{for } m \le 0.4)$$

$$\sigma_H = 1.28\,\frac{Q_l}{H}\,\frac{m^2n}{(m^2+n^2)^2} \quad (\text{for } m > 0.4)$$

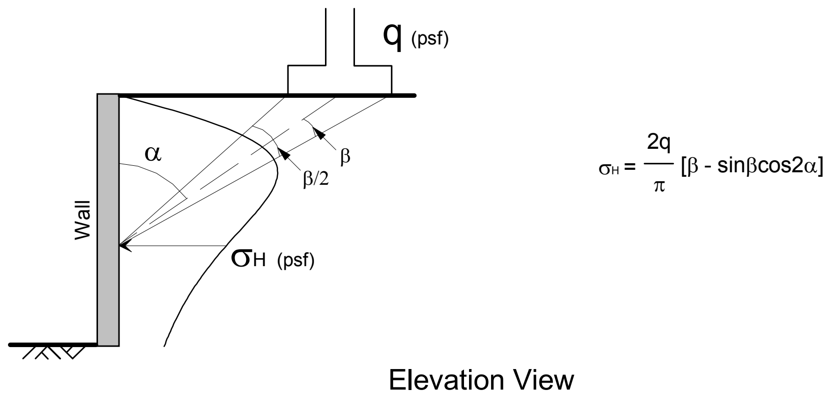
Elevation View

Lateral Pressure due to Line Load

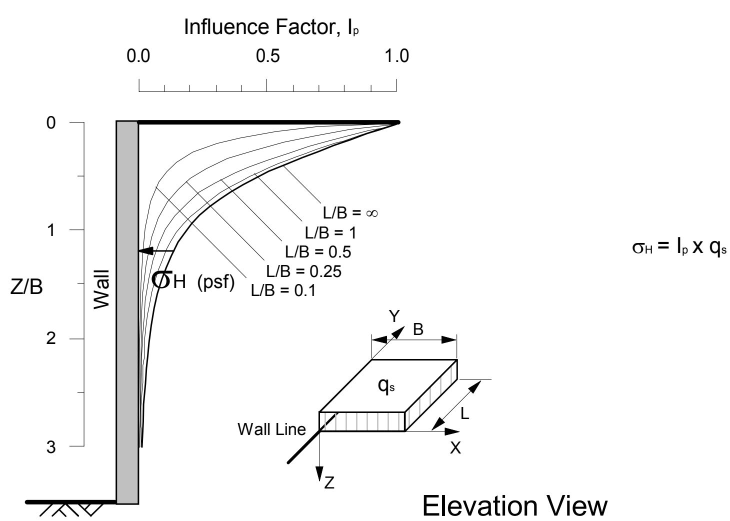
Figure C7-2. Surcharge loads for yielding walls (taken from USN 1986).



Lateral Pressure due to Uniform Surcharge



Lateral Pressure due to Strip Load



Lateral Pressure due to Footing

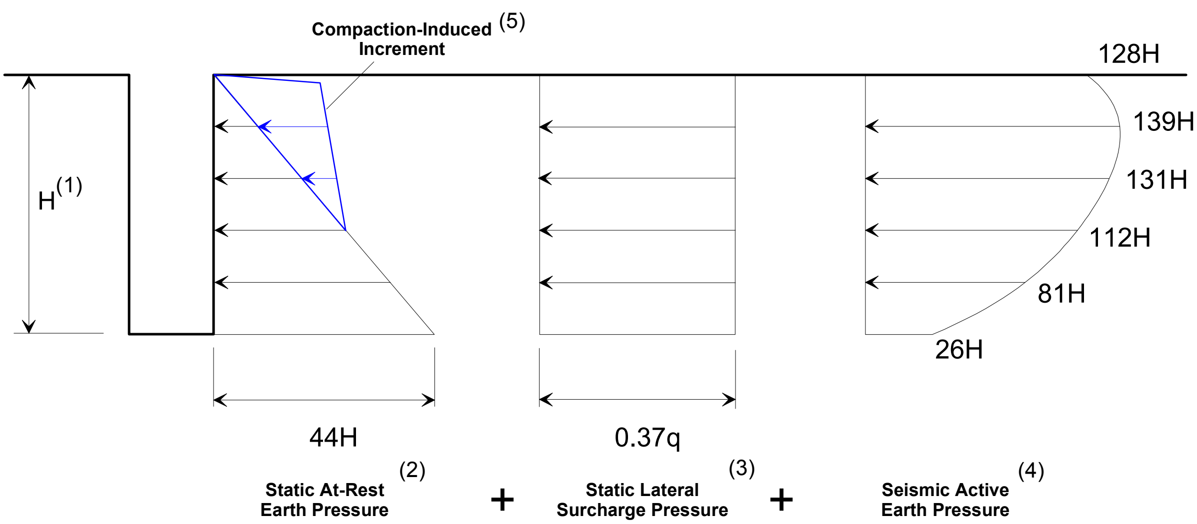
Figure C7-3. Surcharge loads for yielding walls continued (taken from USN 1986)

C7.2 Lateral Earth Pressures on Non-Yielding Walls

The combined lateral earth pressures acting on a non-yielding wall are as follows:

Static At-Rest + Compaction-Induced Increment + Static Surcharge + Seismic Active

The pressure distribution sketch for non-yielding walls is shown in Figure C7-4 through Figure C7-9.



Notes:

1. (1) Height of wall, H, is presented in feet.
2. (2) Static at-rest earth pressures for alluvium: $K_o = 0.37$, $\gamma = 117$ pcf.
3. (3) Static lateral surcharge pressure based on $K_o q$, where q is surcharge to be determined.
4. (4) Seismic active earth pressure based on methods from ASCE 4-98, where $k_h = 1g$ (to be scaled by actual peak ground acceleration, PGA); does not include the dynamic contribution due to surcharge loads.
5. (5) Compaction-induced pressure increments for specific compaction equipment provided in the next following figures.
6. (6) Pressures are presented in psf.

Figure C7-4. Pressure Distribution Sketch for Non-Yielding Walls (not to scale)

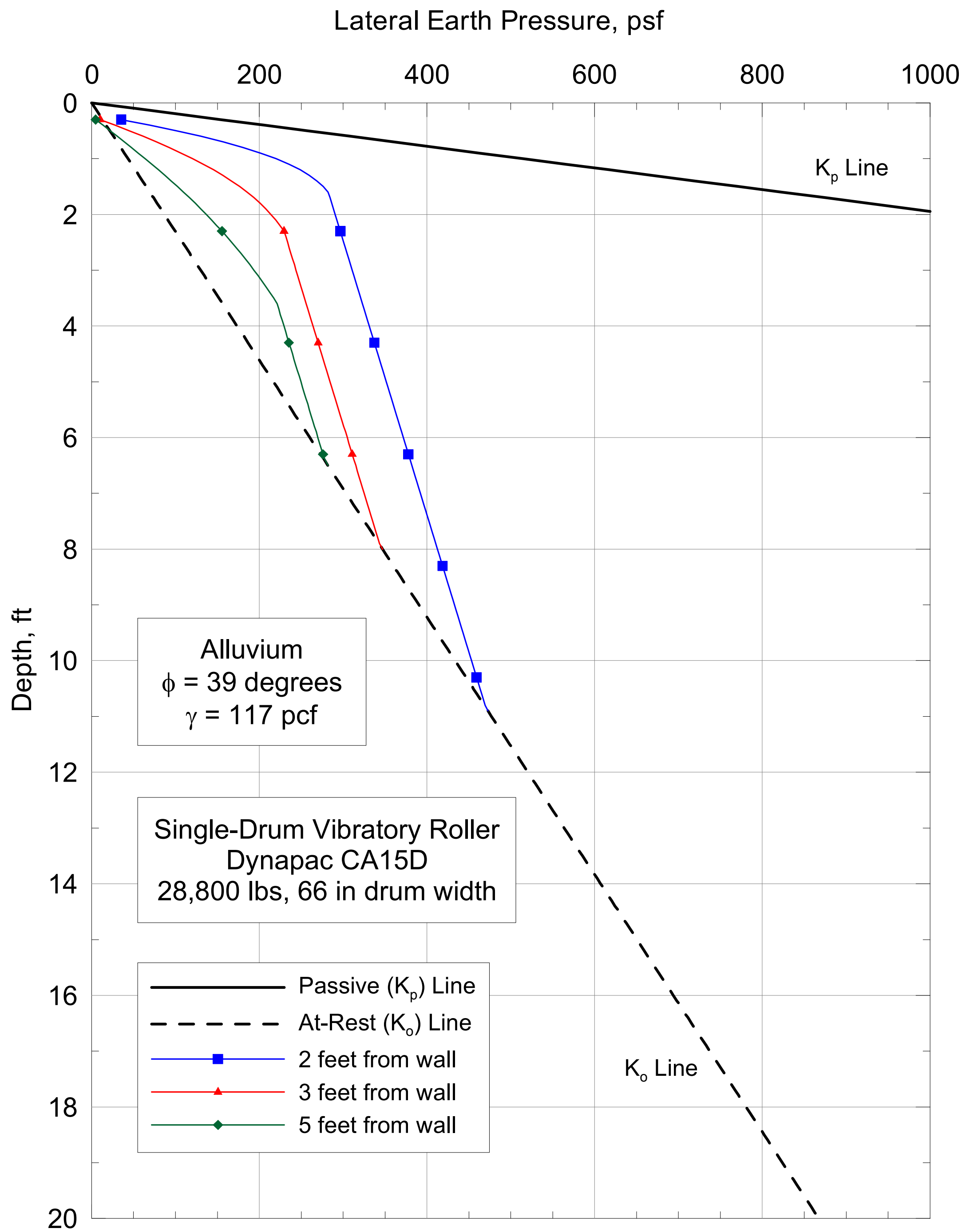


Figure C7-5. Compactor-induced pressures from roller compactor (Dynapac CA15D).

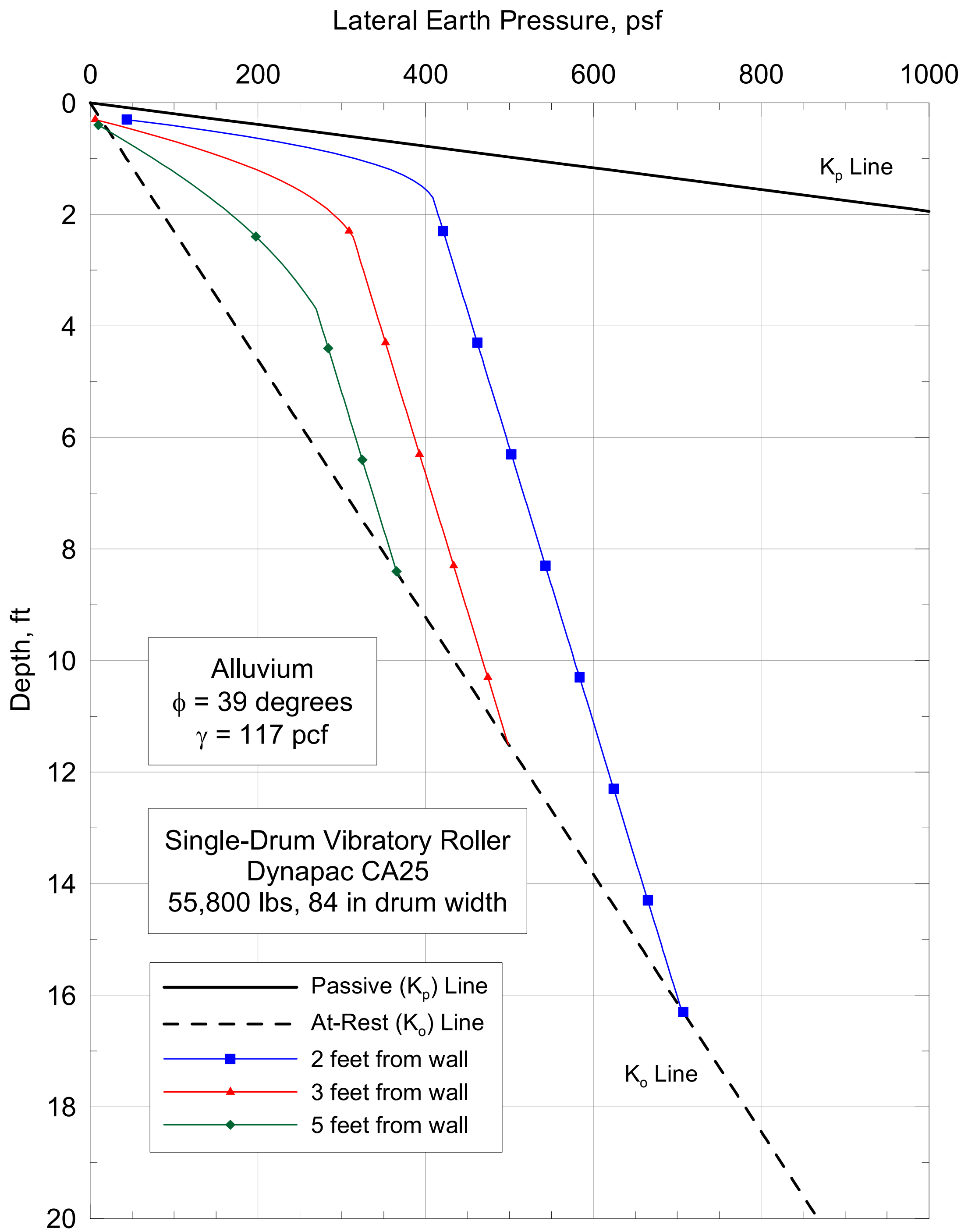


Figure C7-6. Compactor-induced pressures from roller compactor (Dynapac CA25).

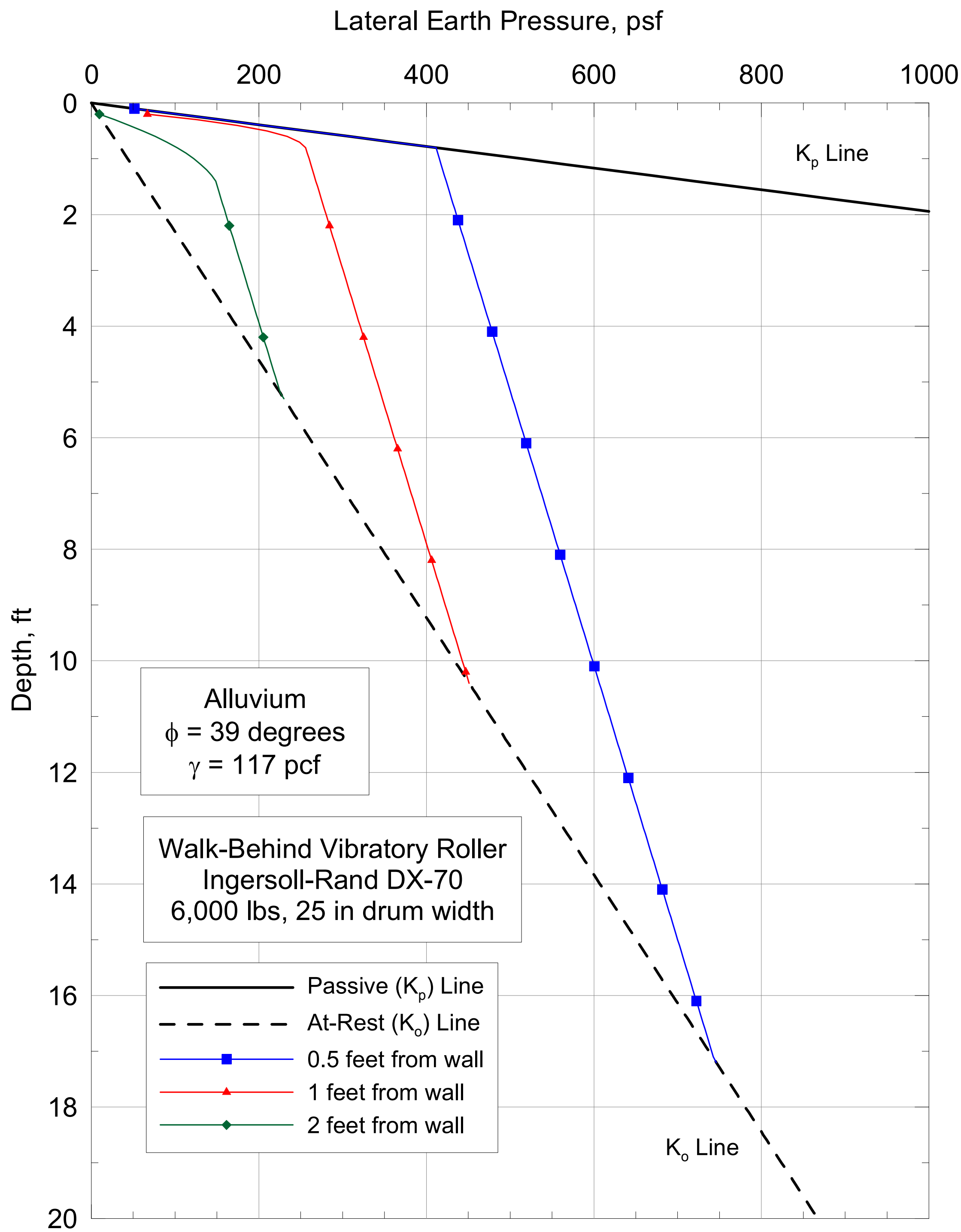


Figure C7-7. Compactor-induced pressures from roller compactor (Ingersoll-Rand DX-70).

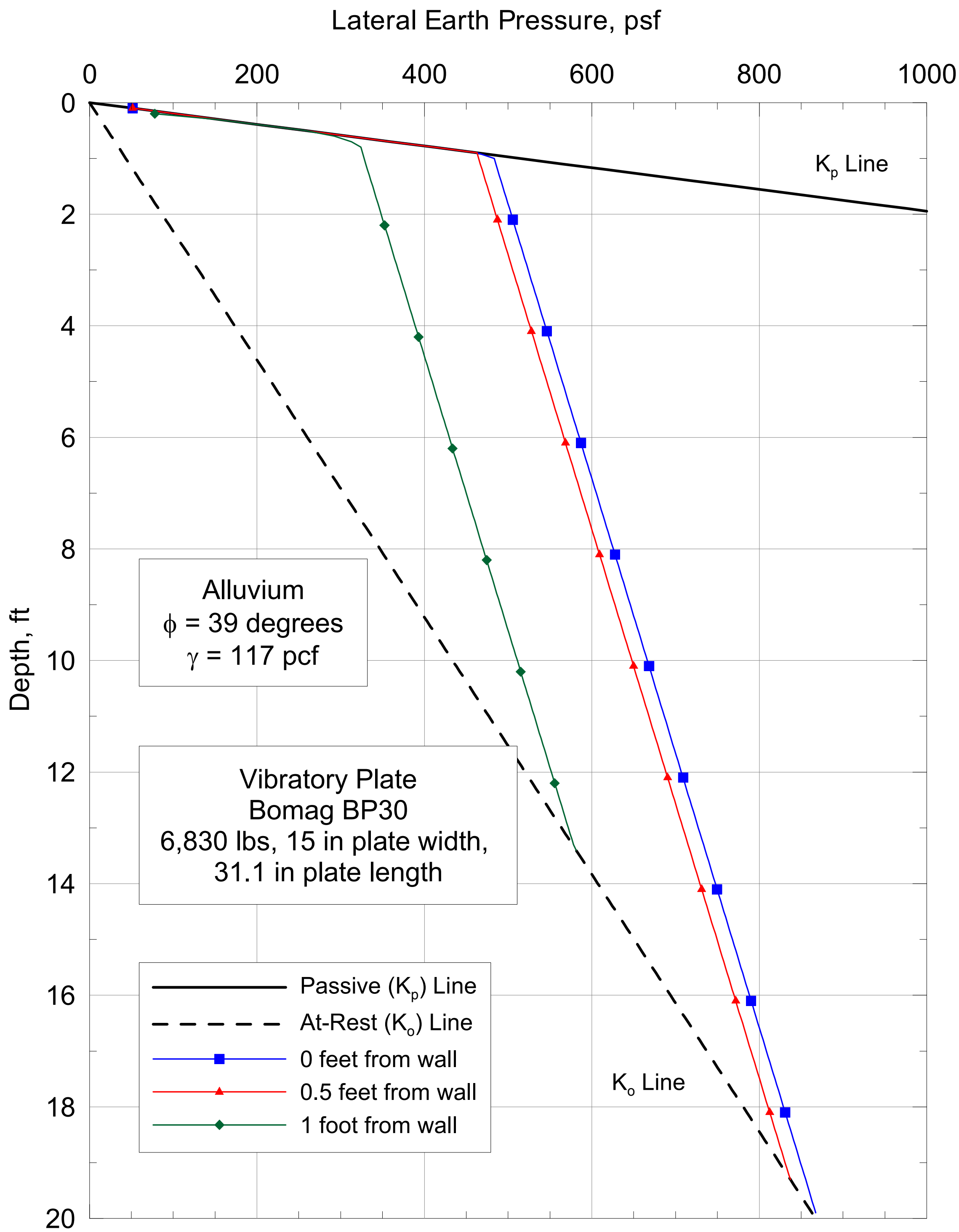


Figure C7-8. Compactor-induced pressures from plate compactor (Bomag BP30).

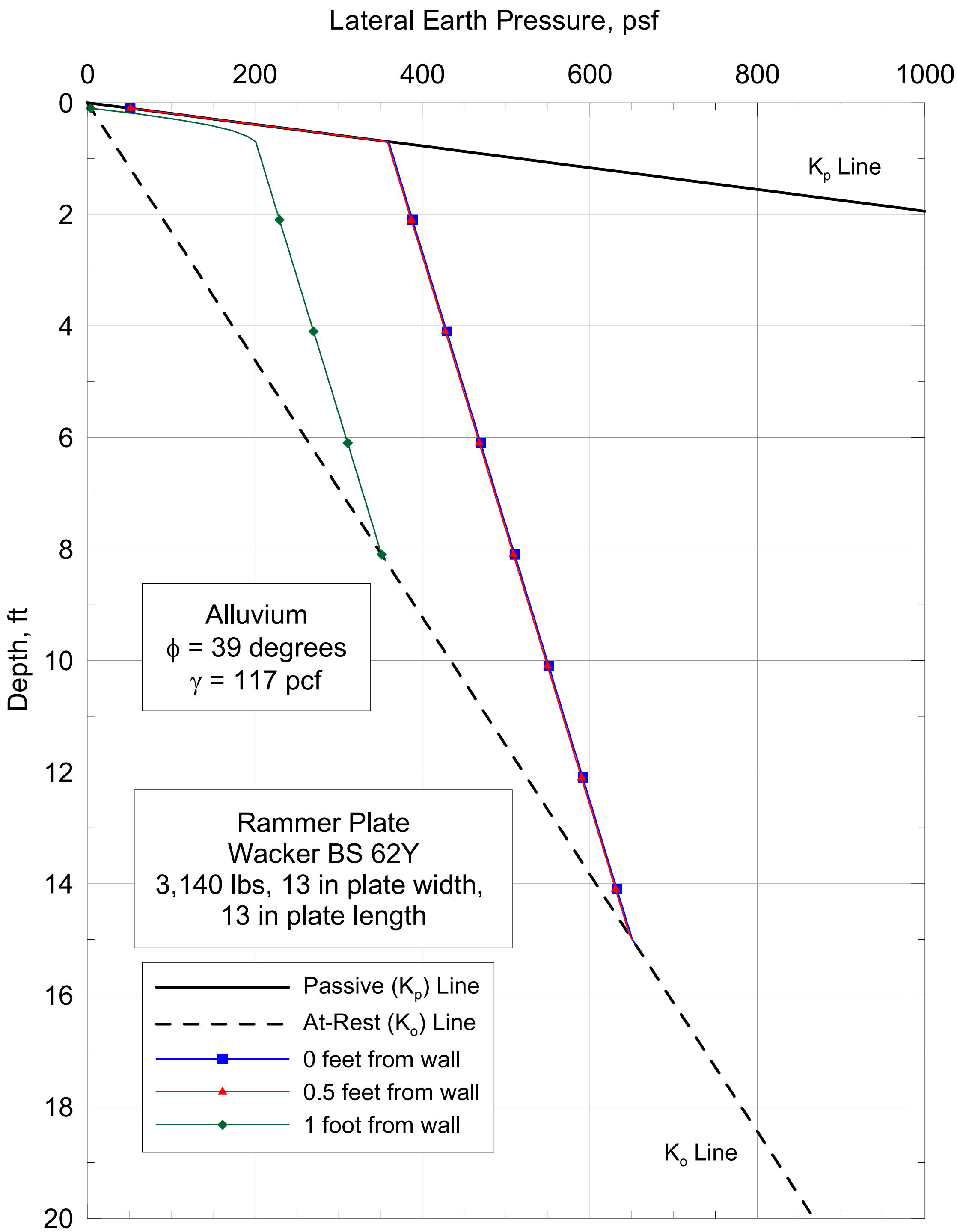


Figure C7-9. Compactor-induced pressures from plate compactor (Wacker BS 62Y).

C7.3 Temporary Shoring Pressure

The pressure of the alluvium acting on temporary shoring provided by soldier piles is estimated to be 17.5H.

C7.4 Resistance to Lateral Loads

The coefficient for resistance developed from passive pressures was calculated in Section C6.1. The passive pressure against the vertical face of the sub-grade walls and footings is calculated to be 515H.

The average interface friction coefficient between mass concrete and the alluvium or potential engineered fill is estimated to be **0.90**, where $\tan \phi = 0.90$. An appropriate factor of safety should be applied to this value.

C8 MathCad Worksheets

MathCad Calculations

Alluvium parameters

$\phi_{alluv} := 39deg$ Friction angle

$\gamma_{alluv} := 117pcf$ Unit weight

$\nu := 0.3$ Poisson's ratio

Static Lateral Pressures

For Non-Yielding Walls:

- At Rest Pressures (based on alluvium properties)

$K_o := 1 - \sin(\phi_{alluv})$

$K_o = 0.37$ Static At-Rest Earth Pressure Coefficient

$p_r(H) := K_o \cdot \gamma_{alluv} \cdot H$ Distributed Static At-Rest Earth Pressure

$$p_r(H) = 43.37 \cdot H \cdot \frac{psf}{ft}$$

$$P_R(H) := K_o \cdot \gamma_{alluv} \cdot \frac{H^2}{2}$$ Resultant Static At-Rest Earth Force

For Yielding Walls:

- Active Pressures (based on alluvium properties)

$$K_A := \tan\left(45deg - \frac{\phi_{alluv}}{2}\right)^2$$

$K_A = 0.23$ Static Active Earth Pressure Coefficient

$p_a(H) := K_A \cdot \gamma_{alluv} \cdot H$ Distributed Static Active Earth Pressure

$$p_a(H) = 26.618 \cdot H \cdot \frac{psf}{ft}$$

$$P_A(H) := K_A \cdot \gamma_{alluv} \cdot \frac{H^2}{2}$$ Resultant Static Active Earth Force

Dynamic Lateral Pressures (yielding walls)

- Active Pressures

$k_h := 1$

$\Delta K_{AE} := \frac{3}{4} \cdot k_h$ — Seismic Active Earth Pressure Increment Coefficient

$\Delta K_{AE} = 0.75$

$\Delta p_{ae}(H) := \Delta K_{AE} \cdot \gamma_{alluv} \cdot H$ — Distributed Seismic Active Earth Pressure Increment

$\Delta p_{ae}(H) = 87.75 \cdot H \cdot \frac{psf}{ft}$

$\Delta P_{AE}(H) := \Delta K_{AE} \cdot \gamma_{alluv} \cdot \frac{H^2}{2}$ — Resultant Seismic Active Earth Pressure Force Increment. It is suggested that the component may be taken to act at approximately 0.6H per Seed and Whitman (1970).

$P_{AE}(H) := \Delta P_{AE}(H) + P_A(H)$ — Sum of initial static active earth pressure force and dynamic active earth pressure force increment

Dynamic Lateral Pressures (non-yielding walls)

Acceleration [g], to be multiplied by k_h

$H := 20ft$ — Wall height

$d := 0ft, 0.1ft .. H$

Coefficients for ASCE 4-98 seismic stresses:

$$M := \begin{pmatrix} 1.0829167 & 0.070869084 & -3.1836133 & 3.5952709 & -2.0641442 \\ 1.0888187 & 1.1176702 & -4.0053697 & 4.333532 & -2.3203657 \\ 1.0968336 & 1.7075112 & -5.3728278 & 5.6727378 & -2.7717642 \\ 1.0788775 & 2.2549514 & -5.719958 & 5.1033643 & -2.1980003 \end{pmatrix}$$

$$y(eqtn, x) := M_{eqtn,0} + M_{eqtn,1} \cdot x + M_{eqtn,2} \cdot x^2 + M_{eqtn,3} \cdot x^3 + M_{eqtn,4} \cdot x^4$$

$$eqtn_0 := \begin{cases} \mathrm{trunc}\left(\frac{\nu}{0.1}\right) - 2 & \text{if } \nu \ge 0.2 \\ 0.2 & \text{otherwise} \end{cases}$$

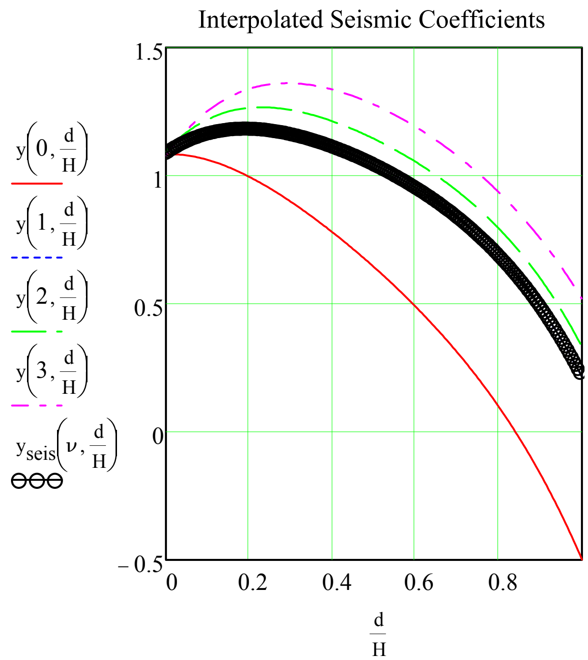
$$eqtn_1 := eqtn_0 + 1$$

APPENDIX C - LATERAL EARTH PRESSURES AND RESISTANCE TO LATERAL LOADS

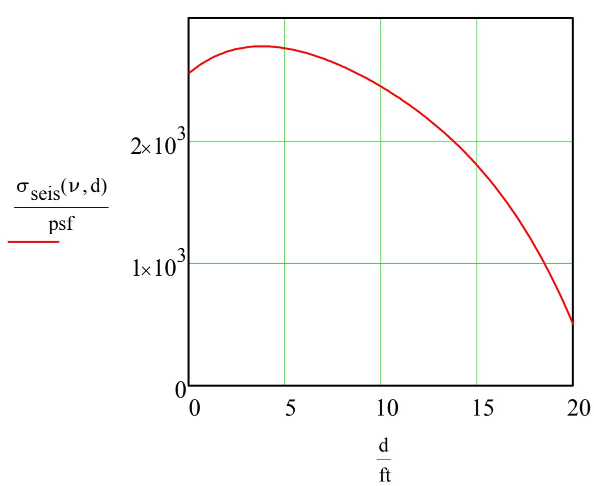
$eqtn_0 = 1$ $\qquad$ $eqtn_1 = 2$

$$y_{seis}(\nu, d) := \frac{\nu - \left(eqtn_0 + 2\right) \cdot 0.1}{.1} \cdot \left(y\left(eqtn_1, d\right) - y\left(eqtn_0, d\right)\right) + y\left(eqtn_0, d\right)$$

The interpolated seismic coefficients per ASCE 4-98 are shown below:



The seismic pressure increment are calculated from $\sigma_{seis}(\nu, x) := y_{seis}\left(\nu, \frac{x}{H}\right) \cdot \gamma_{alluv} \cdot H \cdot a$



$$\sigma_{seis}(\nu, 0H) = 127.392 \cdot H \cdot \frac{psf}{ft}$$

$$\sigma_{seis}(\nu, .2H) = 138.422 \cdot H \cdot \frac{psf}{ft}$$

$$\sigma_{seis}(\nu, .4H) = 130.218 \cdot H \cdot \frac{psf}{ft}$$

$$\sigma_{seis}(\nu, .6H) = 111.479 \cdot H \cdot \frac{psf}{ft}$$

$$\sigma_{seis}(\nu, .8H) = 80.48 \cdot H \cdot \frac{psf}{ft}$$

$$\sigma_{seis}(\nu, H) = 25.071 \cdot H \cdot \frac{psf}{ft}$$

Compaction-Induced Lateral Earth Pressures (Duncan and Seed 1986 and Duncan et al. 1991 procedure)

Methodology:

1. Solve Bousinesq stress due to load
2. Reduce Item 1 using factor, F, and add to Ko stress
3. Limit Item 2 so as to not exceed Kp stress
4. Find depth to peak stress
5. Smooth relationiship below peak Bousinesq stress

Input:

Example using Dynapac CA15D

$P := 28800\text{lbf}$	Static + dynamic force of compactor
$CHD := 0.01\text{ft}$	Closest distance from compactor edge to wall
$\phi := 39\text{deg}$	Internal friction angle of alluvium
$\gamma := 117\text{pcf}$	Unit weight of alluvium
Type := "roller"	Type of analysis (plate or roller), **use lower case**
$\text{width} := 66\text{in}$	Compactor width
$\text{length} := 31.1\text{in}$	Compactor length
$\nu := \dfrac{4 - 3\sin(\phi)}{8 - 4\sin(\phi)}$	Poisson's ratio per Duncan et. al (1991)

Calculations :

Roller Calcs: $R(x,y,z) := \sqrt{x^2 + y^2 + z^2}$

$$\Delta\sigma_h(x,y,z) := \frac{P}{2\cdot\pi}\cdot\left[\frac{3\cdot\left(x^2+y^2\right)\cdot z}{R(x,y,z)^5} - \frac{1-2\cdot\nu}{R(x,y,z)^2 + z\cdot R(x,y,z)}\right]$$

Equation 2.2b from pp. 16 of Poulos and Davis (1991)

$\nu = 0.385$

Bousinesq stress due to compaction:

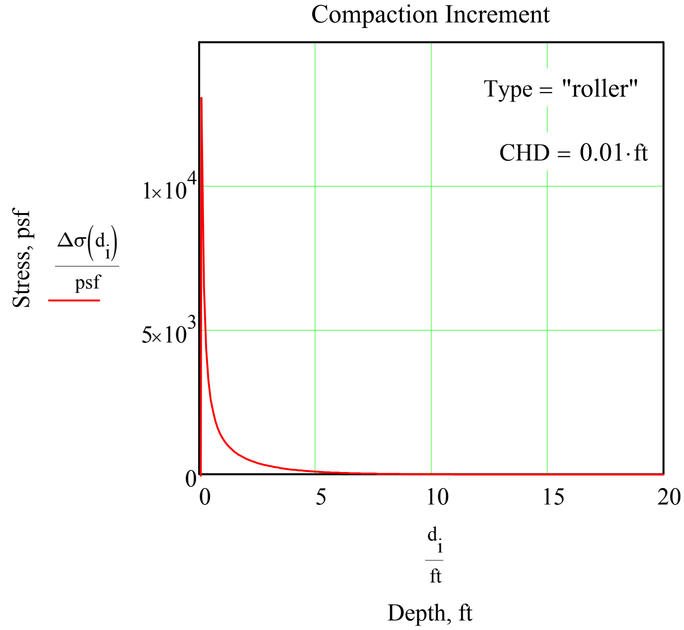
$$\Delta\sigma(d) := \begin{cases} \dfrac{2}{\text{width}}\left(\displaystyle\int_{CHD}^{CHD+\text{width}} \Delta\sigma_h(x, 0\text{ft}, d)\, dx\right) & \text{if Type} = \text{"roller"} \\ \dfrac{2}{\text{width}\cdot\text{length}}\left(\displaystyle\int_{CHD}^{CHD+\text{width}}\int_{\frac{-\text{length}}{2}}^{\frac{\text{length}}{2}} \Delta\sigma_h(x,y,d)\, dy\, dx\right) & \text{otherwise}\end{cases}$$

Double stress increment per Duncan et al. (1991)

$i := 0 .. \frac{H}{0.1ft}$

$d_i := i \cdot 0.1ft$

The unmodified stresses due only to compaction are shown below:



This stress increment is modified per Duncan et al. (1986):

$\alpha := 0.7794423 - 0.51338219 \cdot e^{-19.574578 \cdot \sin(\phi)^{4.9554863}}$ $\alpha = 0.708$

$F := \frac{5^{\alpha}}{4} - 0.25$ $F = 0.531$

$K_p := \tan\left(45deg + \frac{\phi}{2}\right)^2$ passive pressure $K_p = 4.395$

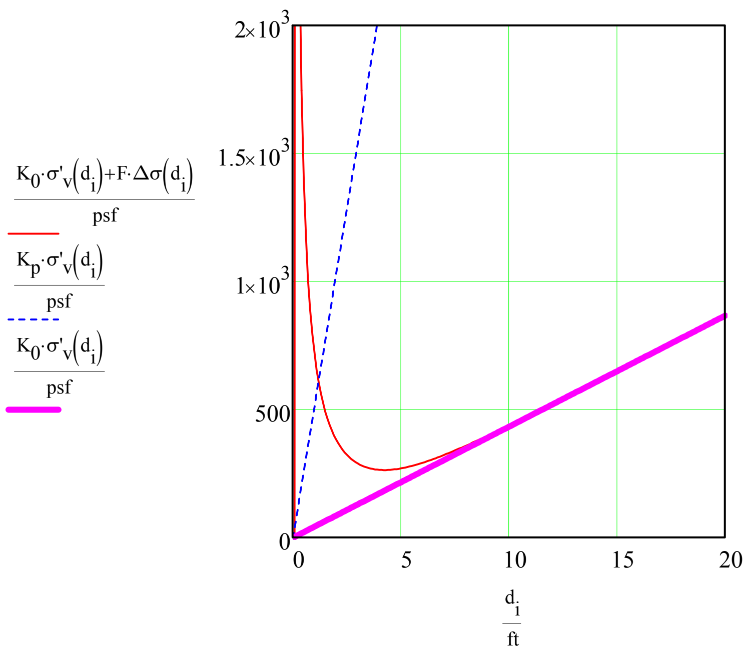
$K_0 := 1 - \sin(\phi)$ at-rest pressure $K_0 = 0.371$

$K_2 := K_0 \cdot (1 - F)$ $K_2 = 0.174$

$\sigma'_v(d) := \gamma \cdot d$

Limit stress in upper portion of wall to passive pressure

$$\sigma'_h(d) := \begin{cases} K_0 \cdot \sigma'_v(d) + F \cdot \Delta\sigma(d) & \text{if } \left(K_0 \cdot \sigma'_v(d) + F \cdot \Delta\sigma(d)\right) \le K_p \cdot \sigma'_v(d) \\ K_p \cdot \sigma'_v(d) & \text{otherwise} \end{cases}$$



$F = 0.531$

$K_0 = 0.371$

$K_p = 4.395$

$K_2 = 0.174$

Find critical depth where stress, $\sigma'_h(d)$, is a maximum off the Ko-line

$k_i := \sigma'_h(d_i) - K_0 \cdot \sigma'_v(d_i)$ k_max is the maximum stress increment off the Ko-line

$depth := .2ft$

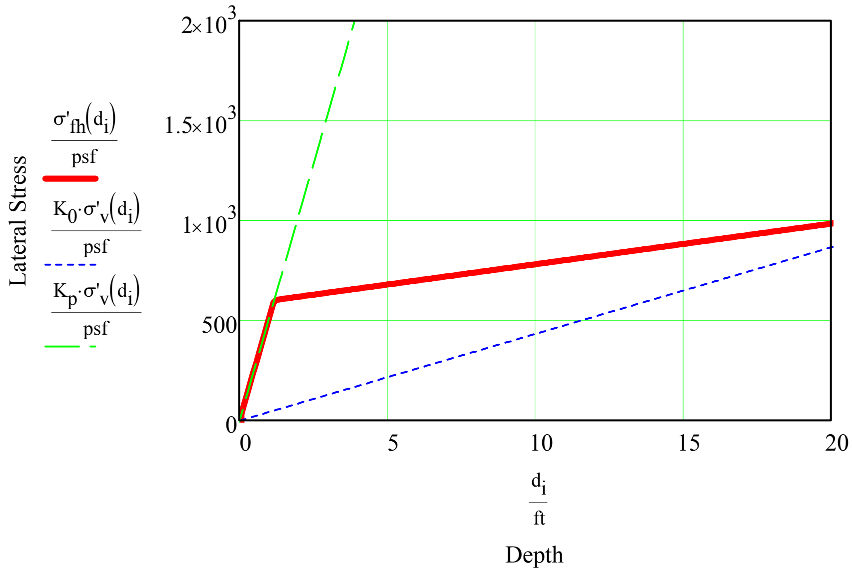
$d_c := root\left(max(k) - \sigma'_h(depth) + K_0 \cdot \sigma'_v(depth), depth\right)$

$d_c = 1.174 \cdot ft$ Critical depth

The total of static and compaction stresses for the wall are determined as follows (note: stress must not go below Ko line):

$$\sigma'_{fh}(d) := \begin{cases} \sigma'_h(d) & \text{if } d \le d_c \\ \text{otherwise} & \\ \quad \begin{cases} \sigma'_h(d_c) + K_2 \cdot \sigma'_v(d - d_c) & \text{if } \left(\sigma'_h(d_c) + K_2 \cdot \sigma'_v(d - d_c)\right) \ge K_0 \cdot \sigma'_v(d) \\ K_0 \cdot \sigma'_v(d) & \text{otherwise} \end{cases} & \end{cases}$$

The combined static and compaction stresses are shown below:



$\sigma'_{fh}(d_i) =$

$-2.029\cdot10^4$	·psf
51.427	
102.855	
154.282	
205.709	
257.136	
308.564	
359.991	
411.418	
462.846	
514.273	
565.7	
604.084	
606.117	
608.149	
...	

Check results against NavFac DM7.02 (USN 1986)
Using equations from Figure 13:

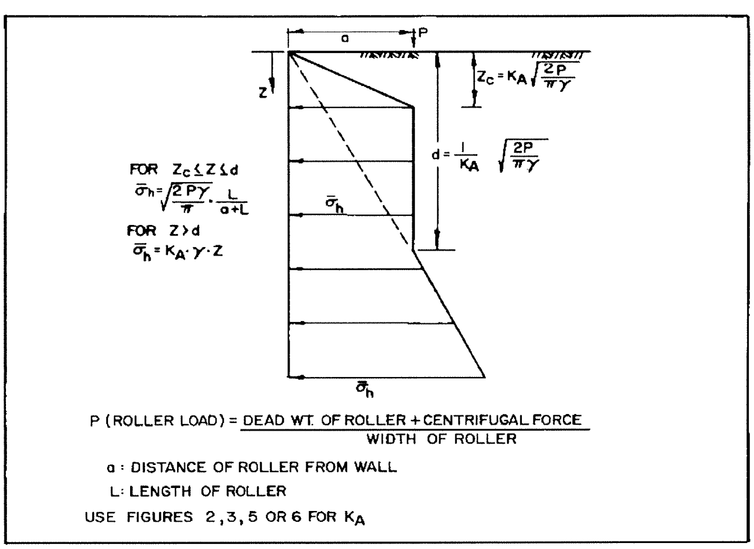


FIGURE 13
Horizontal Pressure on Walls from Compaction Effort

Redefine some variables to correspond to NavFac:

$K_a := \frac{1}{K_p}$ $\qquad P := \frac{P}{width}$ $\qquad z_i := d_i$ $\qquad a := 0ft$

$K_a = 0.228$ $\qquad P = 5.236 \times 10^3 \cdot \frac{lbf}{ft}$

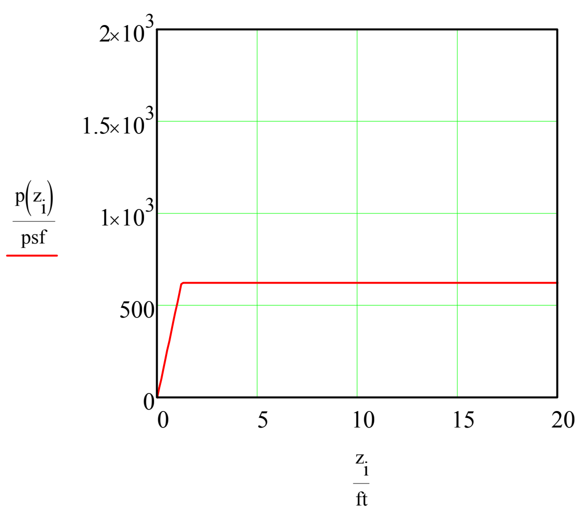
$z_{cr} := K_a \sqrt{\frac{2 \cdot P}{\pi \cdot \gamma}}$ Critical depth

$z_{cr} = 1.214 \cdot ft$

$d := \frac{1}{K_a} \sqrt{\frac{2 \cdot P}{\pi \cdot \gamma}}$ Depth where compaction effects merge with pressure line

$d = 23.462 \cdot ft$

$$p(z) := \begin{cases} \left(\sqrt{\frac{2 \cdot P \cdot \gamma}{\pi}} \cdot \frac{length}{a + length}\right) \cdot \left(\frac{z}{z_{cr}}\right) & \text{if } z < z_{cr} \\ \text{otherwise} & \begin{cases} \sqrt{\frac{2 \cdot P \cdot \gamma}{\pi}} \cdot \frac{length}{a + length} & \text{if } z_{cr} \le z \le d \\ K_a \cdot \gamma \cdot z & \text{if } z > d \end{cases} \end{cases}$$



Solution assumes that the compactor is used at the wall
(distance from wall = 0')

This matches relatively well with the solution obtained from the Duncan et al. (1986) and (1991) solution.

$psf \equiv \frac{lbf}{ft^2}$ $\qquad pcf \equiv \frac{lbf}{ft^3}$