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On Geotechnical Studies Relevant to the Containment of Underground Nuclear Explosions at the Nevada Test Site

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ON GEOTECHNICAL STUDIES RELEVANT TO THE CONTAINMENT OF UNDERGROUND NUCLEAR EXPLOSIONS AT THE NEVADA TEST SITE

ABSTRACT

The Department of Energy and the Department of Defense are actively pursuing a program of nuclear weapons testing by underground explosions at the Nevada Test Site (NTS). Over the past 11 years, scores of tests have been conducted and the safety record is very good. In the short run, emphasis is put on preventing the release of radioactive materials into the atmosphere. In the long run, the subsidence and collapse of the ground above the nuclear cavities also are matters of interest.

Currently, estimation of containment is based mostly on empiricism derived from extensive experience and on a combination of physical/mechanical testing and numerical modeling. When measured directly, the mechanical material properties are obtained from short-term laboratory tests on small, conventional samples. This practice does not determine the large effects of scale and time on measured stiffnesses and strengths of geological materials. Because of the limited data base of properties and in situ conditions, the input to otherwise fairly sophisticated computer programs is subject to several simplifying assumptions; some of them can have a nonconservative impact on the calculated results. As for the long-term, subsidence and collapse phenomena simply have not been studied to any significant degree.

This report examines the geomechanical aspects of procedures currently used to estimate containment of underground explosions at NTS. Based on this examination, it is concluded that state-of-the-art geological engineering practice in the areas of field testing, large scale laboratory measurements, and numerical modeling can be drawn upon to complement the current approach. Specific discussions are presented with regard to:

- The time and scale effects in the measurement of the mechanical properties of geological materials.
- Measurement of in situ stresses by hydraulic fracturing and borehole-jack fracturing.
- Measurement of in situ deformability by NX-jack tests.

- Measurement of in situ tensile strength by hydraulic fracturing and borehole-jack fracturing.
- Measurement of in situ shear strength by borehole shear tests.
- Large scale, laboratory triaxial tests.
- Large scale, laboratory direct shear tests.
- Implicit numerical modeling of subsidence and collapse processes using the output from short-term, explicit calculations of early ground response to explosions.

In cases where today's evaluations indicate marginal conditions or in cases where new test areas are contemplated for which there is no benefit of experience, it is reasonable to expect that a refined input to the calculations will provide more realistic containment estimates than does current practice.

1. INTRODUCTION

The U.S. Departments of Energy and Defense are actively pursuing a program of nuclear weapons testing by underground explosions at the Nevada Test Site (NTS). Tests take place mainly in the alluvium and tuffs of Yucca Flat, as well as in the tuffs and other volcanics of Rainier Mesa and Pahute Mesa. The most serious environmental consequence of the underground explosions is the potential atmospheric release of radioactive materials. When release takes place quickly through the geological formation, it is referred to as dynamic venting.[1] Release also may be due to failure of the stemming in the emplacement hole or tunnel and is then referred to as a leak.[1] The main factors contributing to vents and leaks are summarized in Table 1. The notion of a containment cage alluded to in Table 1 bears some further explanation. This "cage" is a region of high compressive, tangential stresses added to the in situ stresses, located at a distance of between one and two cavity radii from the working point (WP). It is created by the rebound of the material towards the cavity.[2] Based on calculations with the TENSOR code[1,2] the time required for the cage to set up goes from about 0.1 s for a 1 kt yield, to a few seconds for higher yields. As long as the cage fully surrounds the cavity with tangential stresses significantly higher than the cavity gas pressure, there is reason to expect that venting will be prevented because fractures will not propagate from the pressurized cavity.

The tangential stresses may relax with time, however, and in spite of early containment, the ground above the cavity may yield and bring about a surface collapse. Such subsidence is poorly understood and has not been modeled successfully.[2] Collapse may take from a few minutes to several years to occur. The possibility that an early collapse may allow some harmful release cannot be excluded. Thus, it appears that the behavior of the ground beyond the first few seconds after the explosion also is of some interest.

Currently, estimation of containment for nuclear events at NTS is based mostly on experience and on a combination of physical testing and numerical modeling. There is an important empirical data base which consists of the information on several hundred past events. Data include yield, depth of burial (DOB), cavity radius, density of the overburden materials, and density of the WP region. A standard suite of physical and geophysical tests usually is performed in the emplacement hole and/or adjacent exploration hole(s).

Table 1. Possible factors in atmospheric releases.

Geologic failure/dynamic venting

- Too shallow burial of the device. The tensile rarefaction wave comes back from the surface before the containment cage is established.
- Burial too close to a hard interface below the working point, WP.[a] The reflected compressive wave hampers the locking effect of the rebound.
- Burial too close to a fault. High pressure gases can move along the fault and crack overlying formations above the containment cage.

Stemming failure/leaks

- Leaks through open line-of-sight pipes or tunnels.
- Leaks caused by stemming falls or improper stemming.
- Seepage through stemming.
- Cable leaks.

[a] WP is the location of the device.

References 3 to 9 have been selected as representative of this work, during the past 10 years.

Depth of burial has been the object of particular attention, and criteria based on experience have been proposed for minimum DOB.[1] However, in areas where the geology is complex or in new test areas, calculations are performed for additional guidance. They are based on finite-difference wave-propagation programs. Predicting the mechanical response of the ground to explosions requires an input of its deformability and strength properties. Up to now, this information has been obtained during short-term tests on cores a few inches in diameter. This practice does not determine the effects of scale or strain rate on measurement of rock mass stiffness and strength.[10-14] The strength measured on standard laboratory samples, 5 to 10 cm diam, can be several times higher than the strength of larger volumes in the prototypes. The overestimation of material strength can lead to selecting a more shallow DOB than would be warranted with lower strength values, thus reducing the intended margin of safety. No attempt has been made to determine what volumes of rocks and soils must be tested as representative of the conditions in situ. Current data acquisition does not include either the long-term strength properties, which play a role in surface subsidence and collapse, or the strength properties under very high strain rates, as experienced under explosive loading.

This report is an attempt to evaluate the geomechanical aspects of procedures used to estimate the likelihood of containment for underground explosions at NTS. Chapter 2 summarizes the current geotechnical practice for containment evaluation at Lawrence Livermore National Laboratory (LLNL). Chapter 3 discusses the current practice in terms of the assumptions made and in terms of the time and scale effects involved. We then present elements of an approach to incorporate current geological engineering methods in the containment evaluation process. Chapter 4 provides the details of the suggested geotechnical studies in this approach. In cases where today's evaluations indicate marginal conditions or in cases where new test areas are contemplated for which there is no benefit of experience, it is reasonable to expect that refined procedures will provide more realistic containment estimates than does current practice. The main conclusions and recommendations from this study are summarized in Chapter 5.

2. CURRENT GEOTECHNICAL PRACTICE FOR CONTAINMENT EVALUATION

2.1. PHYSICAL PROPERTIES

For each new event, site specific geological and geotechnical data are gathered in a document called the Preliminary Site Characteristics Summary (PSCS). The PSCS is part of the containment prospectus submitted to the Containment Evaluation Panel of the Department of Energy. This information is in addition to the knowledge of physical properties already gained in the various test areas, as illustrated in Figs. 1 to 3 from Ref. 4. The PSCS information relevant to our discussion typically consists of:

- Vertical geologic cross section(s) through the working point.
- A surface effects map illustrating the subsidence effects of past events in the vicinity, the surface expressions of faults, etc.
- A history of hole drilling documenting any anomalies encountered. Typically, only Hunt sidewall scrapings are recovered from the holes after the rotary drilling has been completed. Few cores are taken.
- A summary of rock and soil physical properties at the working point, and in the overburden. The properties include bulk density, grain density, water content, and CO_2 content. Also, note is made of zones where the proportion of swelling clay is more than 20% by weight. Other properties are calculated from the first three: total porosity, water saturation, and gas porosity.
- A summary of the borehole geophysical logs, including velocity data from the dry hole acoustic log (DHAL) method and the Vibroseis method.
- A comparison of the above values with values obtained for other holes in similar material (e.g., alluvium, unsaturated tuff, etc...) at NTS.

Representative data are presented in Appendix A, Figs. A-1 to A-14. They are excerpted from two recent summaries[8,9] corresponding to one event in alluvium (TILCI - hole U4ak) and one event in unsaturated tuff (AKAVI - hole U2es). As shown in the surface effects maps, the event areas may be quite new and untested (see Fig. A-2) or they may be extensively fractured by prior events (see Fig. A-9). Regarding geotechnical properties, it is worth noting that the two methods used for sonic velocity measurements may on occasion give quite different results (see Fig. A-11). This is because the DHAL technique works over intervals of a few feet, whereas the Vibroseis method provides average values from the surface to receivers which can be several hundred

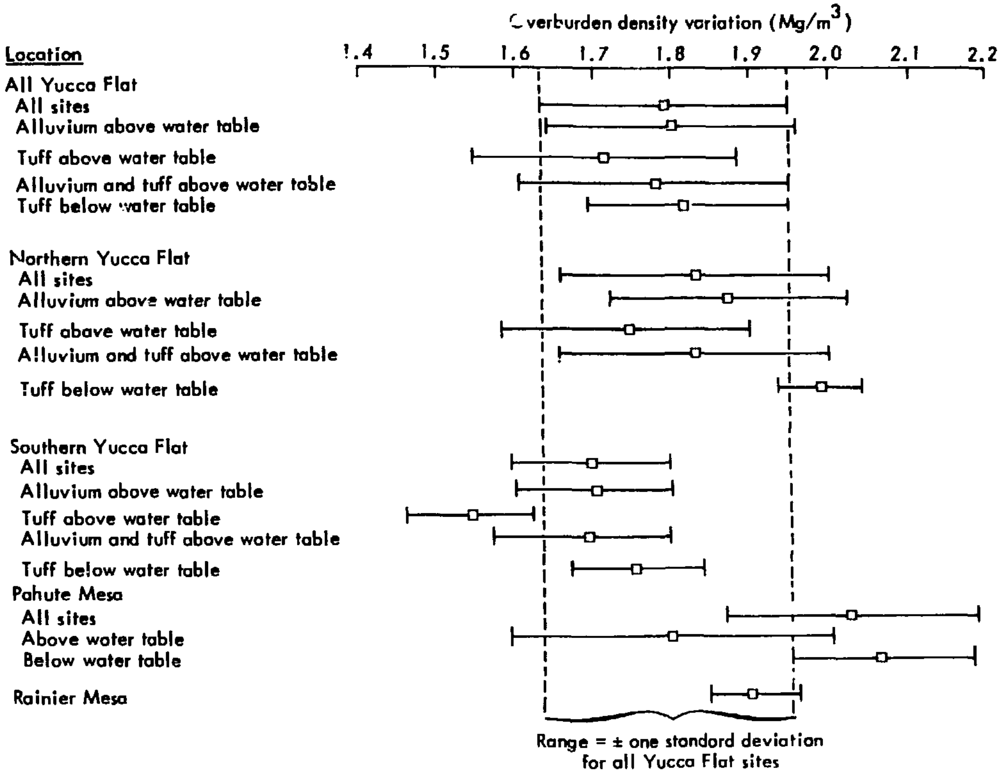


Figure 1. (After Ref. 4.)

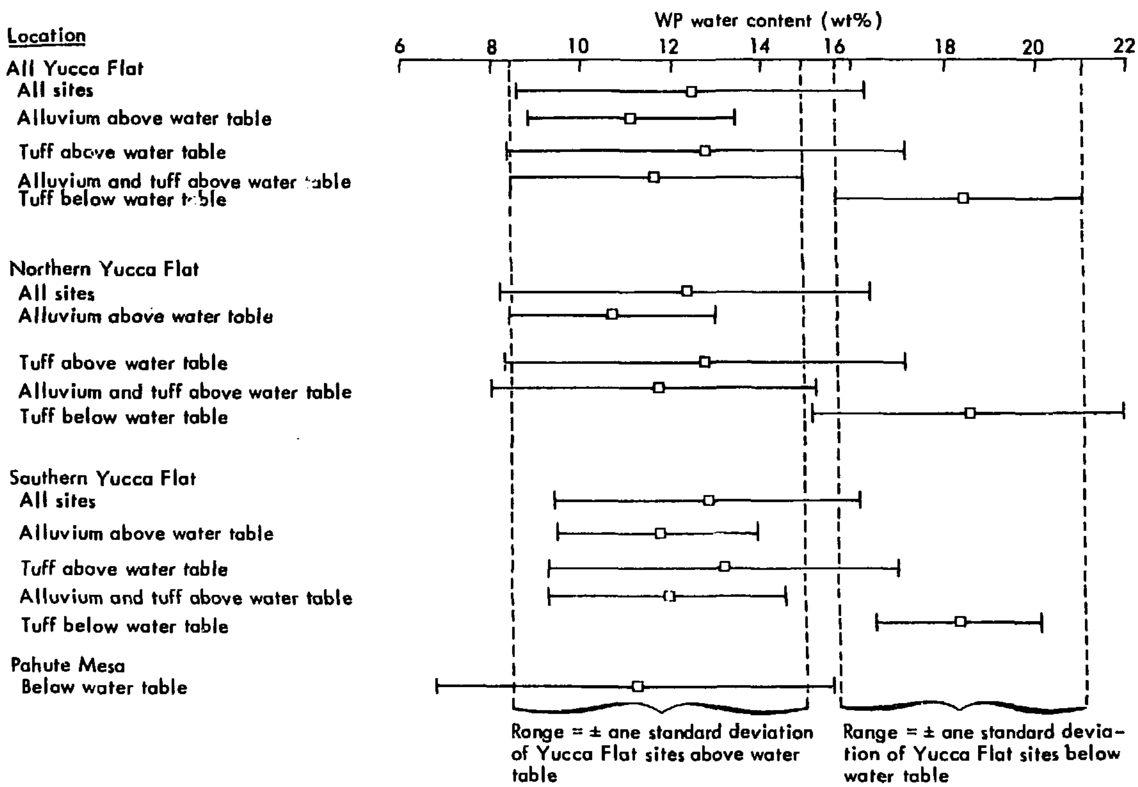
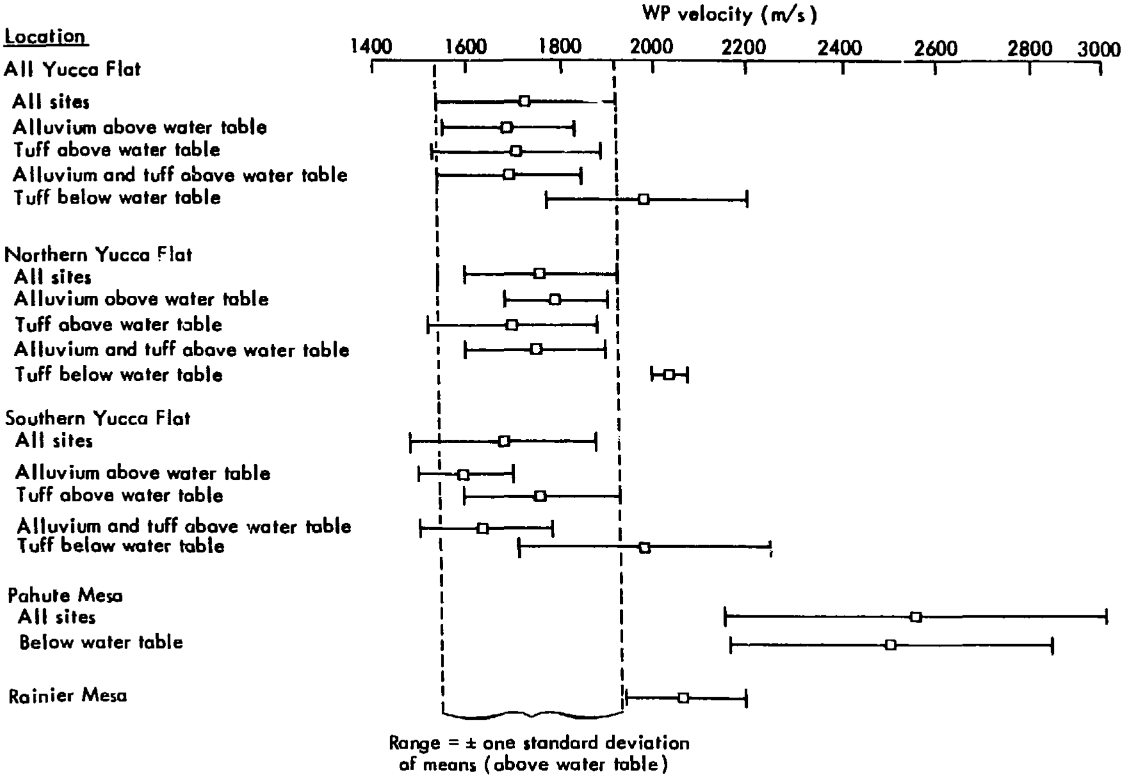


Figure 2. (After Ref. 4.)



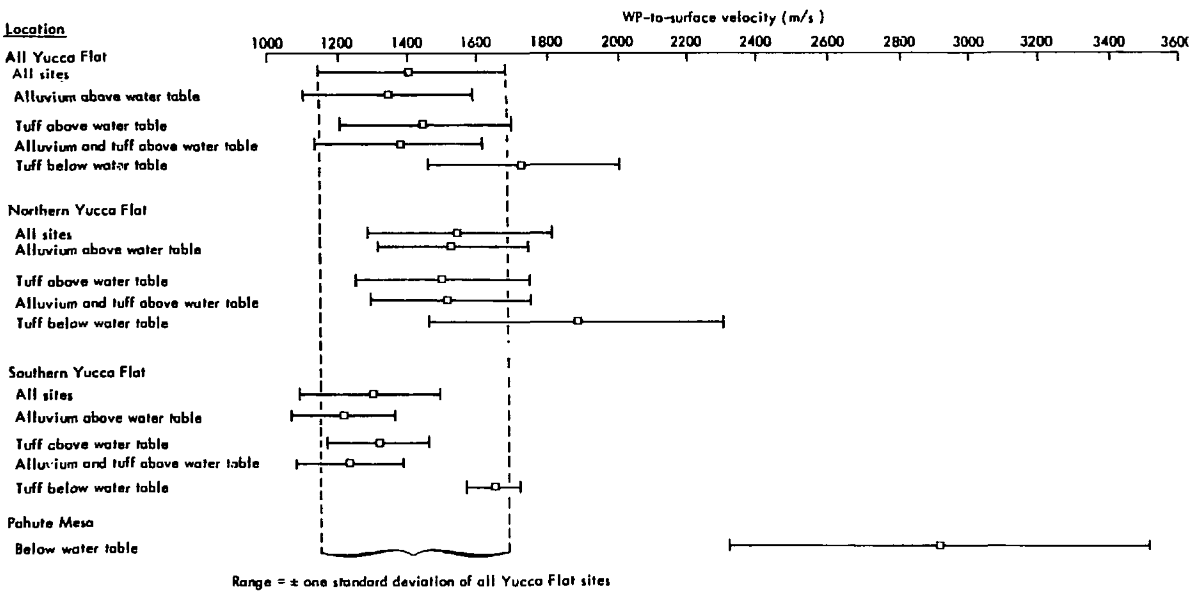


Figure 3. (After Ref. 4.)

metres away, down the hole. So, in the case of a layered geology with marked velocity contrasts, one would be tempted to use the DHAL results that are obtained over shorter intervals. On the other hand, these measurements are taken along the wall of the holes where drilling and the intrusion of the drilling fluids can disturb the in situ material and mask their true properties.

2.2. MECHANICAL PROPERTIES

As illustrated above, the typical site summaries usually do not contain data on mechanical properties of the geological materials. However, some mechanical data may be inferred from the geophysical tests. For example, the bulk modulus up to a so called "elastic limit" usually is calculated from the measured compressional wave velocity and material density, after assuming some value for the Poisson's ratio. In turn, the shear modulus can be calculated. Geological engineering experience[15-18] has shown that the modulus values calculated from various tests (e.g., static loading, dynamic cyclic loading, and seismic wave loading) vary because of the differences in stress levels and strain rates in those tests. For example, a recent series of tests on tuffs[18] gave the following results:

average E seismic/average E static = 5.4,

average E dynamic-cyclic/average E static = 1.5.

Hence, the type of test must be tailored to the level and duration of the loading in the prototype.

LLNL and its contractors also have obtained mechanical properties data for NTS rocks and soils. Pressure-volume relations and shear strength have been measured directly in triaxial compression, uniaxial compression, and Brazilian tensile tests on cores.[19-34] Whereas there is a substantial number of results for material deformability, the strength data base is quite limited. All results found in this study are reported in Appendix 3, Figs. B-1 to B-13, for the two main test media: alluvium and tuff. The tuff coverage includes samples from NTS areas 2, 8, 12, and 16, as well as from Mt. Helen, 70 miles NW of Mercury, NTS. The locations of the various areas are shown on the NTS map of Fig. 4. Data from areas 2 and 8 are very scarce. The paucity is even more acute when it comes to alluvium. All the published alluvium strength results found in this work are summarized in Figs. B-12 and B-13 and relate to areas 3 and 4. As it is, the LLNL experimental program is still far more extensive than the one at LANL.[35]

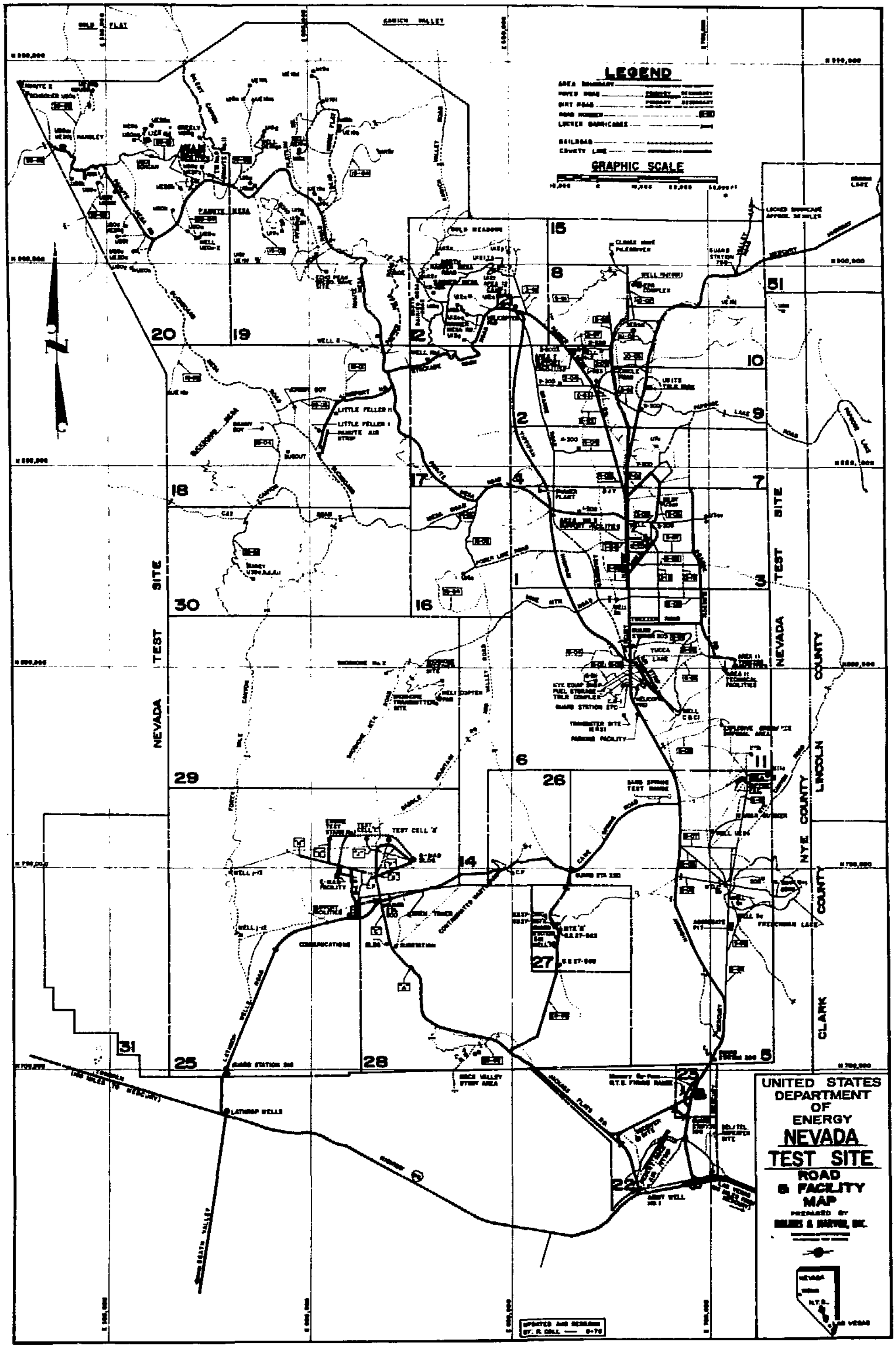


Figure 4. Map of the Nevada Test Site.

The lack of strength data is critical, considering that:

- The magnitude of the stress within the containment cage region and the distance the cage extends from the cavity surface are very sensitive to the strength of the rock surrounding the cavity.[1]
- The time required for the containment cage to form depends on the rebound time which is controlled by the shear strength.[1]
- High shear strength may be adverse to containment.[1]
- The extent of the spall zone is controlled by tensile strength.

2.3. COMPUTER-BASED CONTAINMENT CALCULATIONS

The Department of Energy, the Department of Defense, and their contractors have strived to develop models to predict the effects of nuclear and high-explosive events in geological media. References 36 to 48 have been selected as representative of work in this area over the past several years; most models are computer based. The computer codes usually operate on finite difference approximations of wave propagation equations; their material models are directed at the dynamic response of soils and rocks.[49-58] The two codes used at LLNL are the one-dimensional SOC[39] and the two-dimensional TENSOR.[47,48,57] At LANL, SOC also is available, as is the two-dimensional TOODY code.[37] Because SOC is one-dimensional, it can only model the development of the containment cage in simple geologies. At LLNL, SOC also is used in sensitivity analyses to adjust the parameters of the TENSOR models. Two-dimensional codes themselves have their drawbacks: slant planar interfaces are modeled as conical surfaces, and in situ stresses cannot be given independent values in all three principal directions. These limitations could be removed by using three-dimensional models, but the high cost and time involved in the application of such codes have precluded their use in containment calculations, so far.

Because of the short-term nonlinear dynamics involved in explosion-related ground motion phenomena, the computations typically are performed with explicit schemes. Figure 5 illustrates the steps of a single calculational cycle with TENSOR. The inelastic constitutive models for deformability and strength that are incorporated in TENSOR are depicted in Figs. 6 and 7. They accommodate loading and unloading of materials from an intact condition to a completely crushed condition and accommodate shear failure in the brittle and ductile

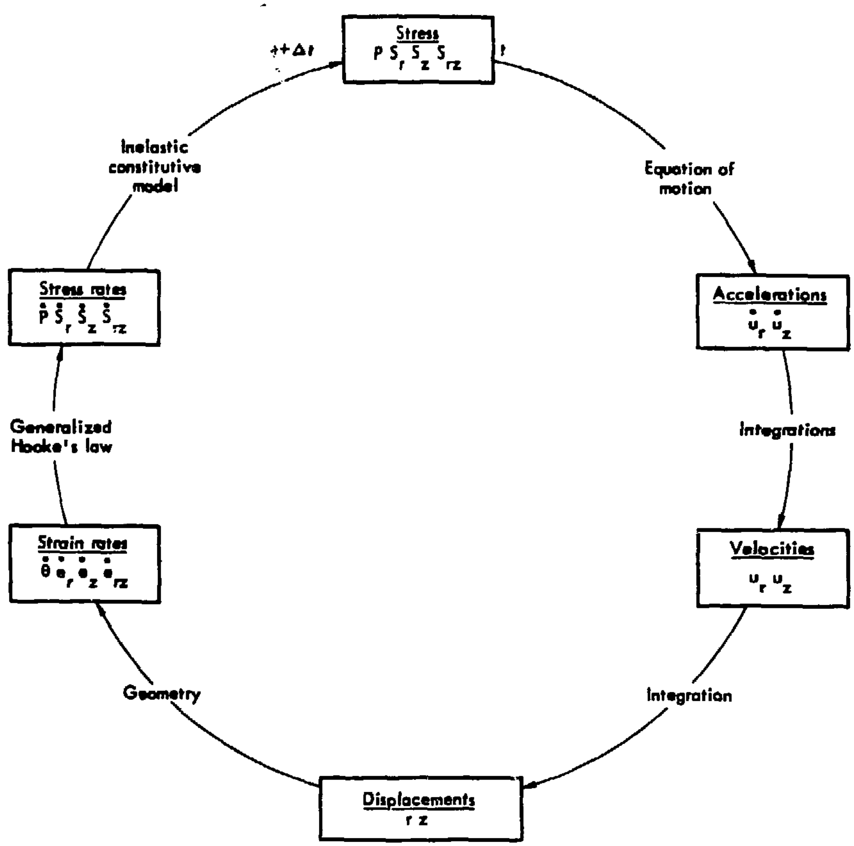


Figure 5. Calculational cycle in TENSOR.[57]

ranges. Opening and closing of tension induced fractures also can be accommodated.[58]

The phenomena of greatest interest in containment evaluation are the short-term events; i.e., the initial extent of the cavity, the extent of the region where tensile fracture occurs, and the development of the containment cage. Less critical is the long-term creep behavior of the overburden during which the cage may relax and the cavity may grow and create surface collapse. As an example of short-term calculations, the BURZET results[59] predict the existence of a spall region (see Fig. 8) and the development of a strong containment cage (see Fig. 9), in spite of the proximity of the Paleozoic surface which was actually modeled closer than it was later determined to be (see Fig. 10). In this case, there was no further evidence of potentially unsafe conditions and it was concluded that "the calculational results satisfy all the criteria for containment."[59]

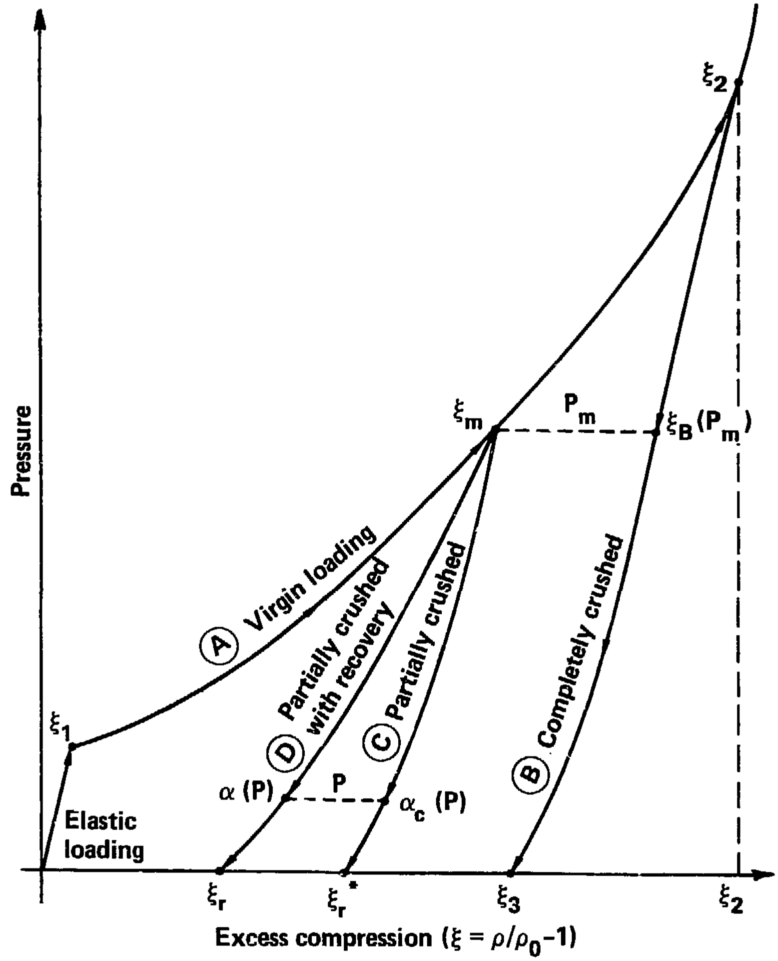


Figure 6. Pressure-volume model in TENSOR code.[57]

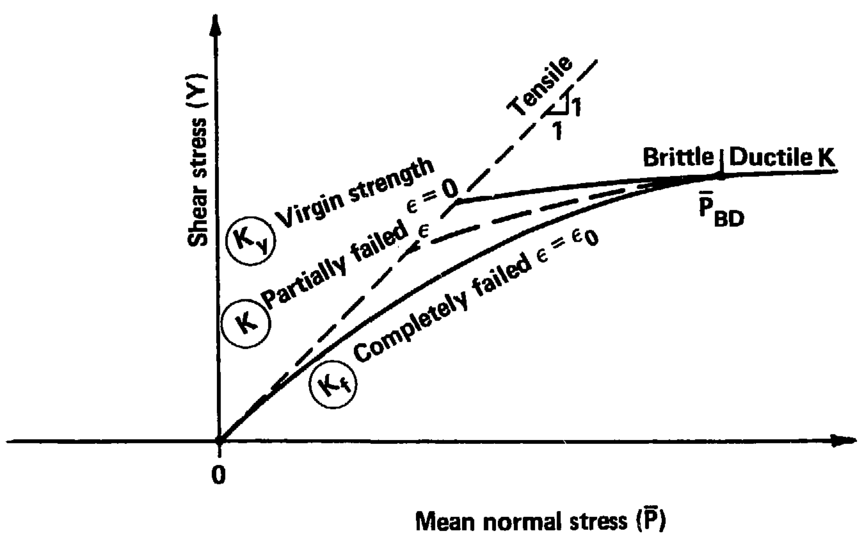


Figure 7. Shear envelopes in TENSOR code.[57]

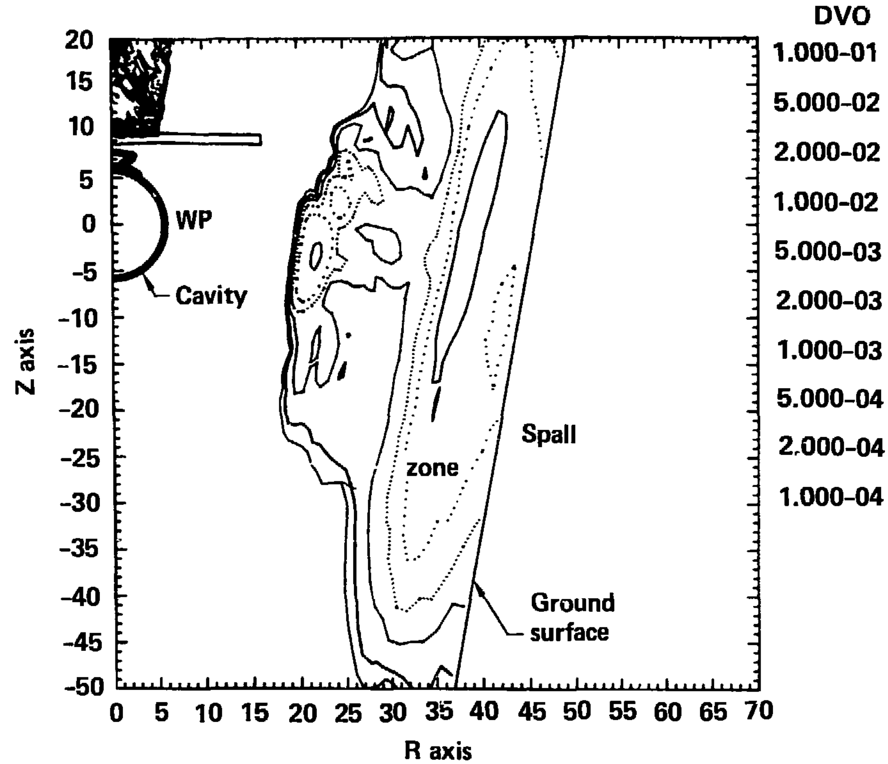


Figure 8. Region of tensile spalling in BURZET calculations.[59]

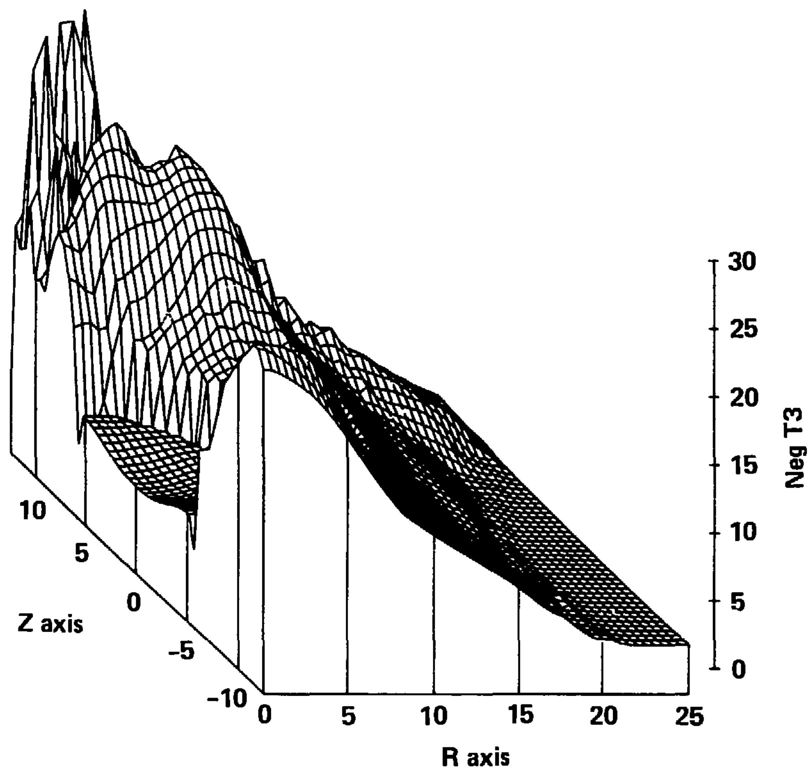


Figure 9. Containment cage calculated for BURZET.[59]

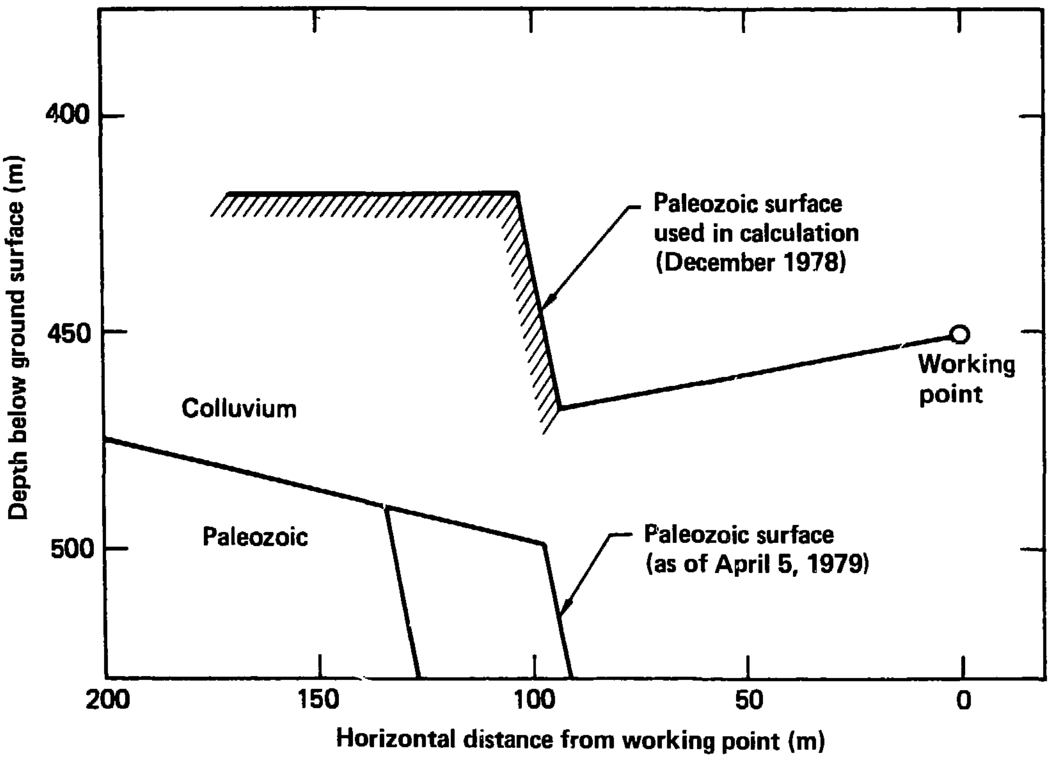


Figure 10. Cross section showing fault and scarp for BURZET model as assumed, and as it was later determined to be.[59]

3. A DISCUSSION OF CURRENT PRACTICE

3.1. COMMON ASSUMPTIONS IN THE CALCULATIONS

Several characteristics of containment calculations performed at LLNL since 1973 are summarized in Table 2. The calculation names do not imply that there was an event corresponding to each calculation because some named events eventually did not take place. Conversely, the calculations did not address all events which have taken place at NTS during that time. Many events never were calculated. The salient features revealed by the compilation of Table 2 are:

- About 40% of the calculations are two-dimensional.
- All event calculations assume hydrostatic stresses.
- When the geological materials are given some tensile strength, it is an assumed value, typically 1 to 2 bars. Occasionally the Paleozoic rocks are assigned a 5-bar tensile strength. No measurements have been made of tensile strength, in situ or in the laboratory, for alluvium or tuffs.
- The bulk modulus, K, generally is calculated from seismic compressional velocity, V_p.
- Poisson's ratio, ν, generally is assumed to be in the range of 0.15 to 0.3.

Also, within a given geologic formation, the shear strength of soils and rocks commonly is assumed to be independent of depth. The choice of a single shear strength value may be based upon an approximation of some laboratory test data. In other cases, it may be based on an empirical relation developed between a shear index and the radius of the explosion-produced cavity.[60] This index is in fact an average shear strength which, when input in TENSOR calculations, tends to result in cavity radii close to observed radii. Conversely, when cavity radii are known, the back-calculation of the event with TENSOR yields an estimate of the average shear strength for the vicinity of the cavity.

Results from such back-calculations are shown in Fig. 11 for events in NTS areas 2, 3, 4, 8, 9, 10, 11, and 12. The plots do not seem to predict a correlation between depth of burial and shear strength. In fact, the

Table 2. Some characteristics of containment calculations at LLNL for the period 1973-1981 (courtesy of J. T. Rambo, LLNL).

Calculation name	NTS area	One or two dimensions	$\sigma_{horiz} = \sigma_{vert}$	Tensile strength = 0	K from Vp	Measured ν
Asiago	2	1D	Yes	Yes	Yes	No
Azul	10	1D - 2D	Yes	Yes	Yes	No
Baneberry	8	1D - 2D	Yes	Yes	Yes	Yes[a]
Banon	2	1D	Yes	Yes	Yes	No
Burzet	4	1D - 2D	Yes	Yes	Yes	No
Caboc	2	1D - 2D	Yes	Yes	Yes	No
Camembert	19	1D	Yes	?	Yes	No
Cheshire	20	1D	Yes	Yes	Yes	No
Chevre	10	1D - 2D	Yes	Yes	Yes	No
Coulommiers	8	1D - 2D	Yes	Yes	Yes	No
Dauphin	9	1D - 2D	Yes	Yes	Yes	No
Edam	21	1D - 2D	Yes	No	Yes	No
Fallon	2	1D	Yes	Yes	Yes	No
Farallones	2	1D - 2D	Yes	Yes	Yes	No
Flax	2	1D	Yes	Yes	Yes	No
Fontina	20	1D	Yes	No	Yes	No
Handley	20	1D	Yes	Yes	Yes	No
Harzer	19	1D - 2D	Yes	No	Yes	No
Islay	2	1D	Yes	No	Yes	No
Kasseri	20	1D	Yes	No	Yes	No
Leyden	9	1D	Yes	Yes	Yes	No
Mast	19	1D	Yes	Yes	Yes	No
Molbo	20	1D - 2D	Yes	No	Yes	No
Muenster	19	1D	Yes	Yes	Yes	No
Panir	19	1D	Yes	Yes	Yes	No
Pepato	20	1D	Yes	No	Yes	No
Portmanteau	2	1D - 2D	Yes	Yes	Yes	No
Portulaca	2	1D	Yes	No	Yes	No
Scantling	4	1D	Yes	Yes	Yes	No
Scotch	19	1D	Yes	Yes	Yes	No
Serpa	19	1D	Yes	?	Yes	No
Stantan	2	1D	Yes	Yes	Yes	No
Stanyan	8	1D - 2D	Yes	Yes	Yes	No
Starwart	2	1D - 2D	Yes	?	Yes	No
Stilton	20	1D	Yes	Yes	Yes	No
Tybo	20	1D - 2D	Yes	Yes	Yes	No
Wichita	9	1D - 2D	Yes	?	Yes	No
Zaza	4	1D	Yes	Yes	Yes	No

[a] Poisson's ratio measured on laboratory samples.

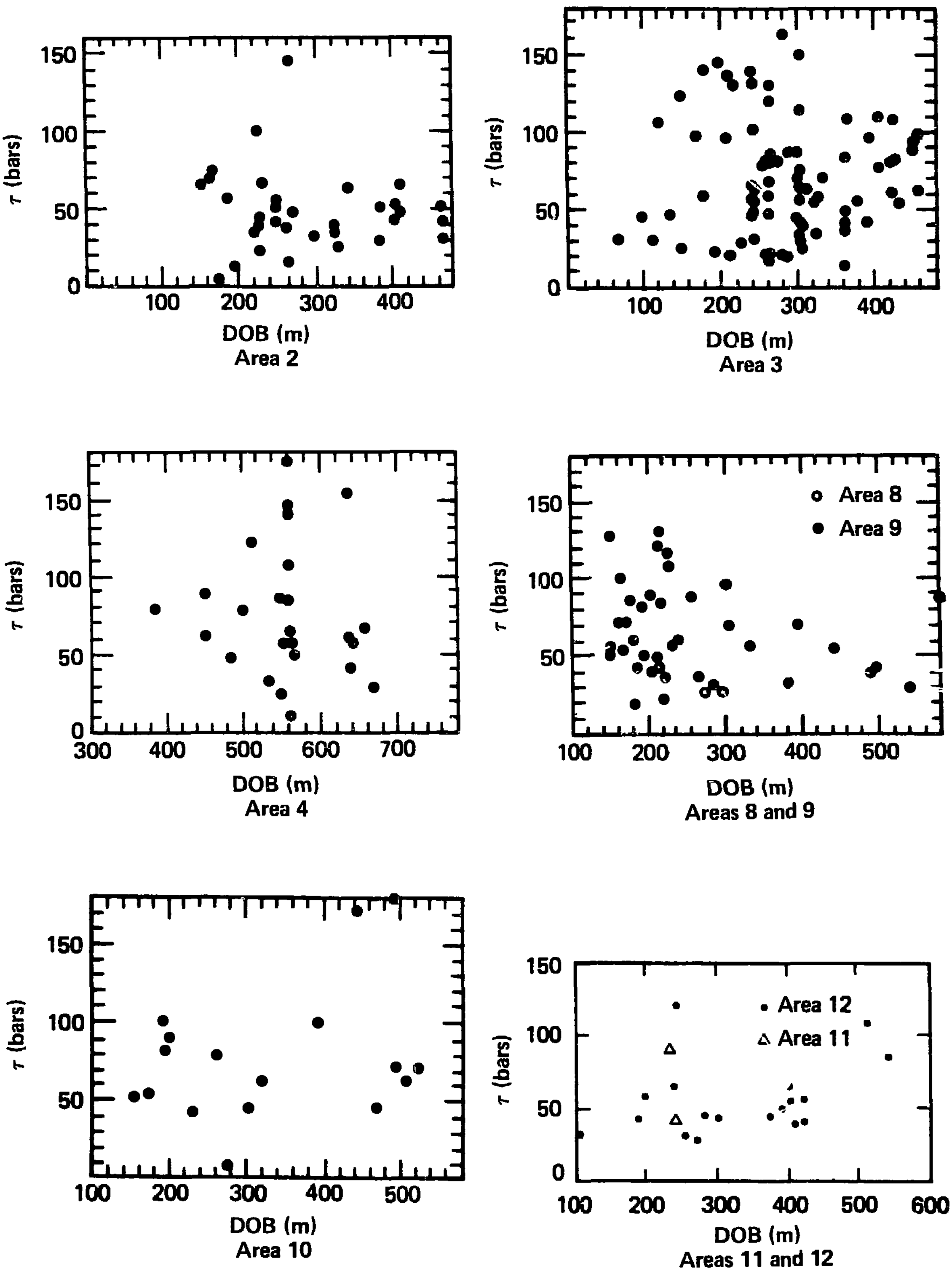


Figure 11. Average material shear strength back-calculated with TENSOR from known cavity radii, for various areas at the NTS.[60]

materials in these areas are frictional materials which have been consolidated; i.e., there are no excess pore pressures prior to loading. Under dynamic loading, there is no time for drainage; the properties to be used are those obtained in consolidated-undrained (C-U) tests.[50] In such tests, the shear strength is independent of pressure during the test but depends upon the preconsolidation stress. On the contrary, in drained tests the shear strength is pressure dependent. This is very clearly shown, particularly below a 1 kbar mean pressure, in the figures of Appendix B. The assumption of constant shear strength, regardless of overburden (preconsolidation) stress and test pressure, corresponds to an unconsolidated-undrained (U-U) condition, which prevails if no excess pore-pressure dissipation is permitted either before or during loading. This is not the case in the containment problem at hand. In spite of the common assumption of constant shear strength, TENSOR does offer the option of including pressure-dependent strength. On the other hand, TENSOR does not offer yet the capability to model pore-pressure dissipation, but work has started to couple a fluid flow model to the continuum mechanics computations.

Yet another procedure which is used in preliminary calculations, such as with SOC, consists in letting a pressure-dependent shear envelope be generated by the code from stored average properties of NTS materials.[52] In this case, the user need only specify the bulk density, grain density of the solid matrix, weight fraction of water, and bulk modulus. The author of this procedure clearly points out that it is not intended to replace a rock property measurement program.[52]

3.2. THE EFFECT OF TIME

The calculations usually are carried out from a few tenths of seconds to a few seconds, depending upon the yield, until the velocity field is close to zero and a pattern of stresses has emerged which reveals the presence or absence of a satisfactory containment cage. The long-term behavior of the overburden is not investigated to estimate future collapse. The SOC and TENSOR codes were not intended to handle creep and progressive failure. Long-term strength and creep properties are not being obtained on the geological materials. However, with respect to this last point, it should be noted that some existing triaxial test data could be reanalyzed to give indication of long-term strength level. According to published theory,[61] the volumetric behavior of brittle materials in triaxial compression can permit a recognition of the stress levels at which stable and unstable fracture propagations will occur. The threshold between the two types of fracture propagation is thought to be the ultimate strength for long-term loading. On volumetric plots, this level is marked by a reversal of the sign of the increments of volumetric strain. The only published volumetric strain plot found in this work is that for the Diamond Mine tuff of area 16,[24] shown in Fig. 12. Undoubtedly, more such plots could be generated from past triaxial tests, where longitudinal and radial strains were recorded. On Fig. 12, the stress level of point A would be the long-term strength of the tuff on the scale of a few cubic inches; it appears to be about 75% of the ultimate short-term strength. A corollary idea proposed for the long-term predictions is that the total strain at failure under a stress exceeding level A will be that which can be read from a complete stress-strain curve in triaxial compression.[62,63] Turning to the one-dimensional representation of Fig. 13, this means that if a stress level σ_B is maintained on the material, it is predicted that failure will take place when the total accumulated strain is ε_c. The path BC is supposed to be traveled in a steady-state creep mode. The steady-state creep rate for the material can be determined in appropriate creep tests so that the total time for failure at strain ε_c would be predictable.

At the other end of the strain-rate spectrum, the very short-term, dynamic properties of the geologic materials are not being obtained either. There is substantial evidence that both the stiffness and strength values increase when the strain rate increases.[11,12] For example, results obtained on tuff are summarized in Fig. 14 and Table 3. Unlike in the case of long-term strength, one may argue that ignoring the dynamic effects provides conservative results. However, using static strength values which can be too low by a factor of 2 or more may lead to a deeper burial of the device, at unnecessary additional cost.

3.3. THE EFFECT OF SCALE

The concepts and test procedures discussed above apply only as long as the volumes of materials which are tested are representative of the in situ material conditions. Today, there is not a clear indication of what the

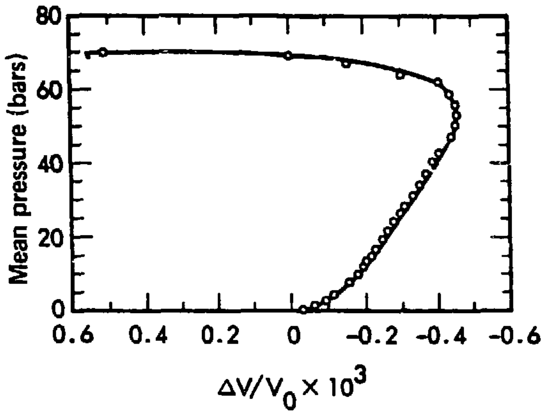


Figure 12. Pressure-volumetric strain relation for Diamond Mine tuff in uniaxial compression.[24]

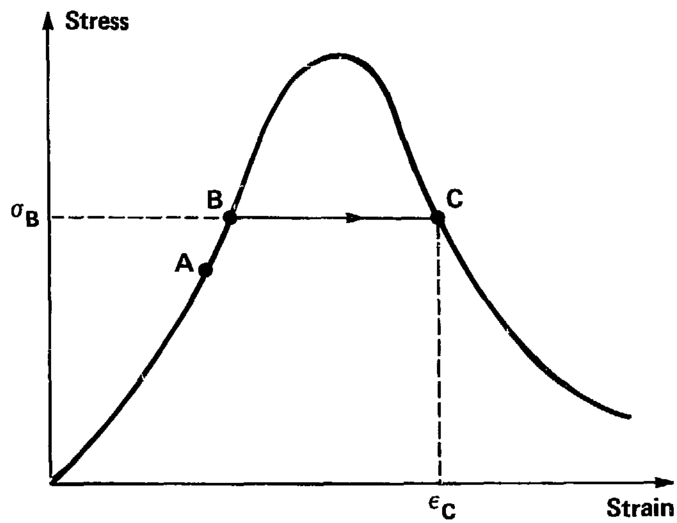


Figure 13. Concept for long-term strength of rocks.[62]

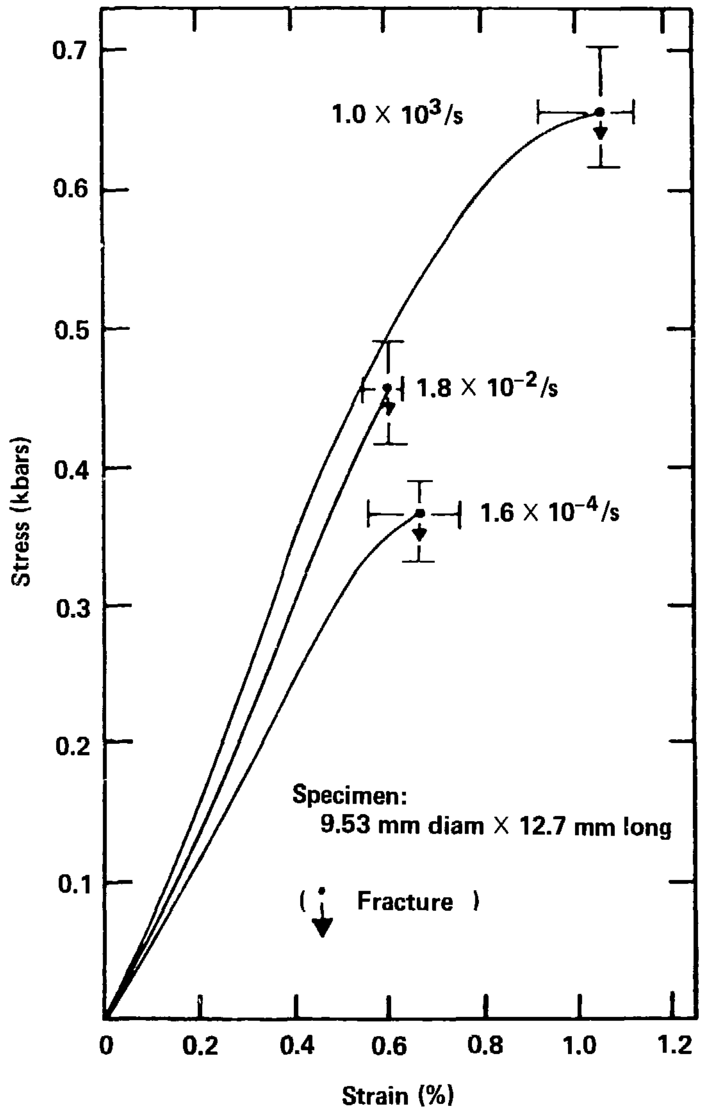


Figure 14. Stress-strain behavior of dry volcanic tuff for various strain rates at room temperature and atmospheric pressure.[12]

Table 3. Results of unconfined compression tests on NTS tuff under various loading rates.[11]

Loading rate (psi/s)	Ultimate compressive strength (psi)	Modulus of elasticity (10^6 psi)	Poisson's ratio	No. of specimens
50	1,640	0.54	0.19	3
1.68×10^5	1,850	0.33	0.42	1
3.11×10^5	2,490	0.41	0.49	1
8.46×10^5	4,230	0.91	0.36	1

minimum representative volumes are for NTS materials. Undoubtedly, this knowledge should be developed with a high priority to overcome the scale effects which plague the application of laboratory test results to field situations.

It is expected that, because of the smaller particle sizes, the minimum volume for alluvium should be significantly smaller than the one for jointed rocks. Some useful guidance can be found in large laboratory triaxial tests performed on rockfill materials by the Geotechnical Engineering Division of U.C. Berkeley, at the Richmond Field Station,[64,65] and by others.[66-68] Table 4 summarizes strength results for several rockfills at confinement of 350 psi (2.4 MPa); ϕ is the drained friction angle, ϵ_{1f} is the axial strain at failure, ϵ_{vf} is the volumetric strain at failure, and σ_{3f} critical is the minimum value of confinement to restrain volumetric dilation before failure. Figure 15 shows the variation of the friction angle with confining pressure. The effect of scale can be studied in Figs. 16 and 17 for the dredger tailings from Oroville Dam, California, and the crushed basalt of a San Francisco rockfill. In these tests, the specimen diameter was kept at six times the maximum particle size. The figures show that for intermediate confining pressures (140 and 650 psi) the sample size has no significant effect on the test results. From this, it seems reasonable to infer that the ratio of 6 for specimen diameter over maximum particle size is adequate; however, there is no evidence as to whether a smaller ratio also would be

Table 4. Strength and deformation properties of selected large rockfill specimens.[65]

Symbol	Dam or Place	Material	Gradation	Particle shape	$\phi(°)$	$\epsilon_1{}^f$ (%)	$\epsilon_v{}^f$ (%)	$\sigma_3{}^f$ critical PS
△	Oroville	Dredger tailings	Well graded 6" to fines	Rounded	40	6.5	1.5	120
▽	Pinzandaran	Sand and gravel (dry)	Well graded 8" to fines	Rounded	39	8	4.7	60
⊕	San Francisco	Basalt	Well graded 8" to 1/4"	Angular	39	15	6	60
□	San Francisco	Basalt	Well graded 3" to 1/4"	Angular	38			
◆	San Francisco	Basalt	Poorly graded 6" to fines	Angular	37	20	6.5	40
●	Malpaso	Conglomerate	Well graded 8" to fines	Angular	37	13	4.5	20
○	El Infiernillo	Silicified conglomerate (dry)	Poorly graded 8" to fines	Angular	36.5	14	5.5	30
■	Pyramid	Argillite	Poorly graded 6" to fines	Angular	36.5	20	5.5	25
○	El Infiernillo	Diorite (dry)	Poorly graded 8" to fines	Angular	35	15	10	25
▼	El Granero	Shale[a]	Well graded 8" to 1/4"	Angular	35	>14	>10	10
▲	El Granero	Shale[a]	Poorly graded 8" to 1/4"	Angular	33	>14	>10	5
⊞	Mica	Granitic Gneiss	Well graded 8" to fines	Angular	32	>14	6	20
Not shown	Mica	Granitic Gneiss	Well graded 8" to 1 1/3"	Angular	25	>14	6	20

[a] Test not continued to failure.

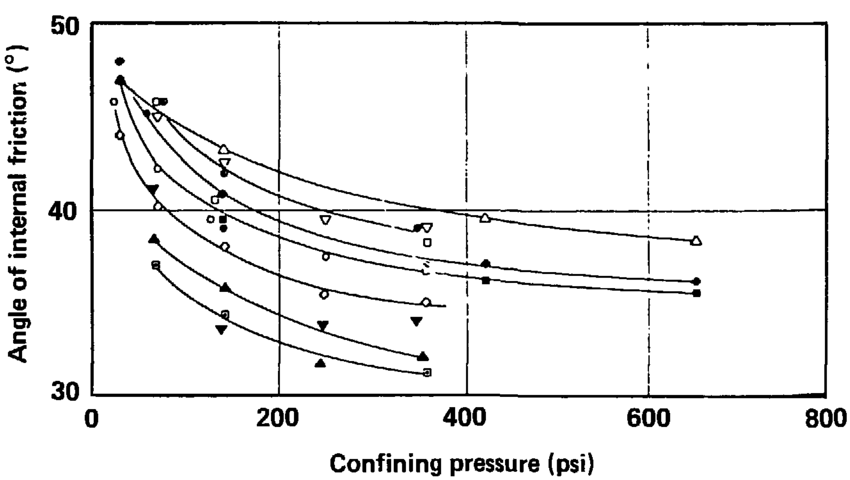


Figure 15. Angle of friction vs confining pressure for large rockfill specimens.[65]

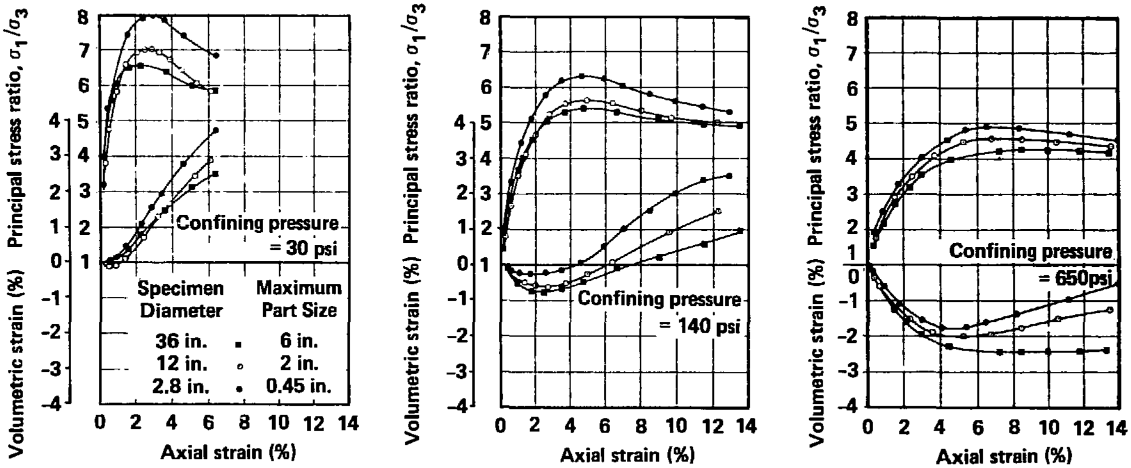


Figure 16. Strength and deformability properties of Oroville Dam rockfill material for various sample sizes in triaxial compression.[65]

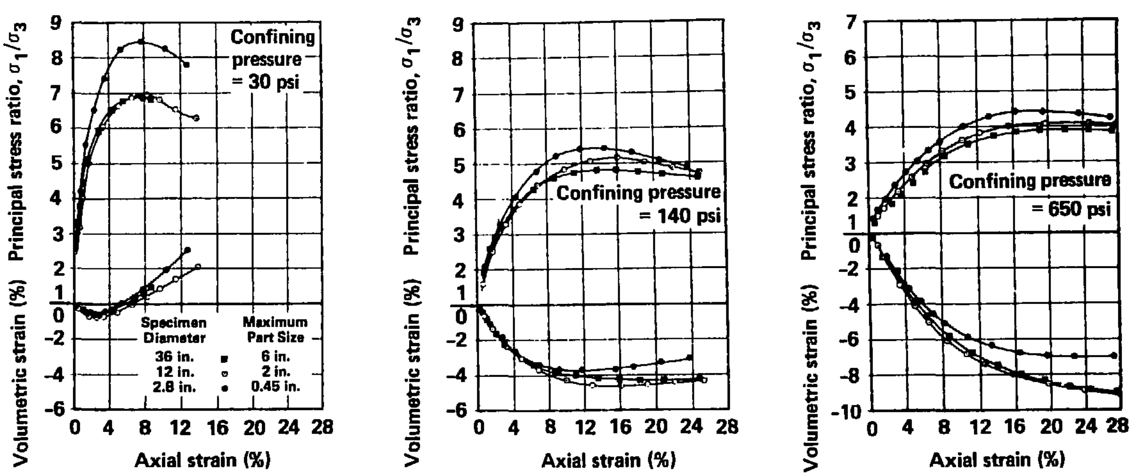


Figure 17. Strength and deformability properties of crushed basalt for various sample sizes in triaxial compression.[65]

acceptable. An alternate method to triaxial testing, for determining friction, is the use of a large size direct shear machine such as used by the Rock Mechanics Laboratory at U.C. Berkeley for granular materials and weak rocks.[69] The large triaxial cell and the large shear box of U.C. Berkeley are shown on Figs. 18 and 19 respectively. Consideration could be given to making use of this equipment on an ad hoc basis in cooperation with U.C. Berkeley.

As for the rock materials with joints and fractures, the effect of scale on mechanical and fluid flow properties is well recognized.[10,13,14,70-76] Significant information exists regarding the ratio of field-measured rock mass deformability, E_F to laboratory-measured rock material modulus, E_ℓ. Based on surveys by the author and one of his colleagues,[13,76] involving 37 large rock projects worldwide, the mean of 103 E_F/E_ℓ ratios was about 0.40. The numbers obtained on small cores were on the average 2.5 times higher than deformability values measured in place. Data are much scarcer regarding scale effects on the compressive or shear strength of large rock volumes. Figure 20 shows a typical variation of compressive strength versus size of sample for a hard rock.[70] The striking features are the precipitous decrease in strength, when the size exceeds that of conventional laboratory samples (15 to 20 cm diameter), and the large scatter of results at the laboratory scale, when compared to the field scale. Today, one can only speculate as to what the corresponding strength/size relations for tuff and other volcanics may look like. Regarding the shear strength of joints, it has been suggested that the minimum test size should be that of a natural block bounded by the existing joints and fractures.[14]

The in situ tensile strength of rocks also is poorly documented, although a measurement procedure based on hydraulic fracturing tests in boreholes has been suggested.[63] Recently, approximate strength criteria have been published for intact and jointed rock masses.[77] The values summarized in Table 5 are qualified by their authors as being somewhat tentative, since they are based upon so little field information. Column 4 refers to rhyolite, which is one of the test media at NTS, such as for the MOLBO event. Line 3 of column 5 could apply to the Climax granite of the past HARD HAT and PILEDRIVER events, but there is no direct reference to tuffs. They may fit in the lower half of column 4, depending upon their degree of fracturing.

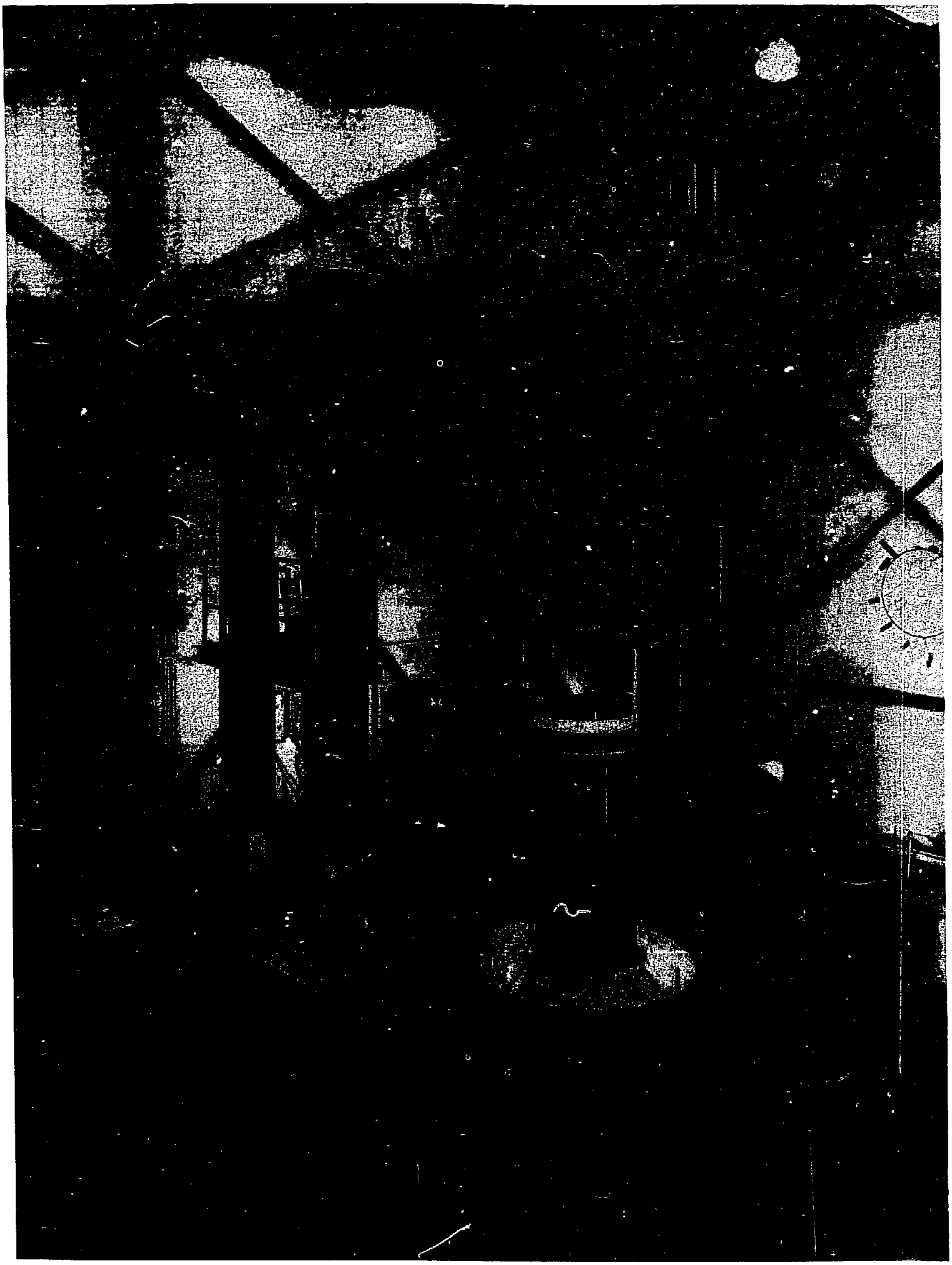


Figure 18. Large scale triaxial system at U.C. Richmond Field Station (LBL photograph).

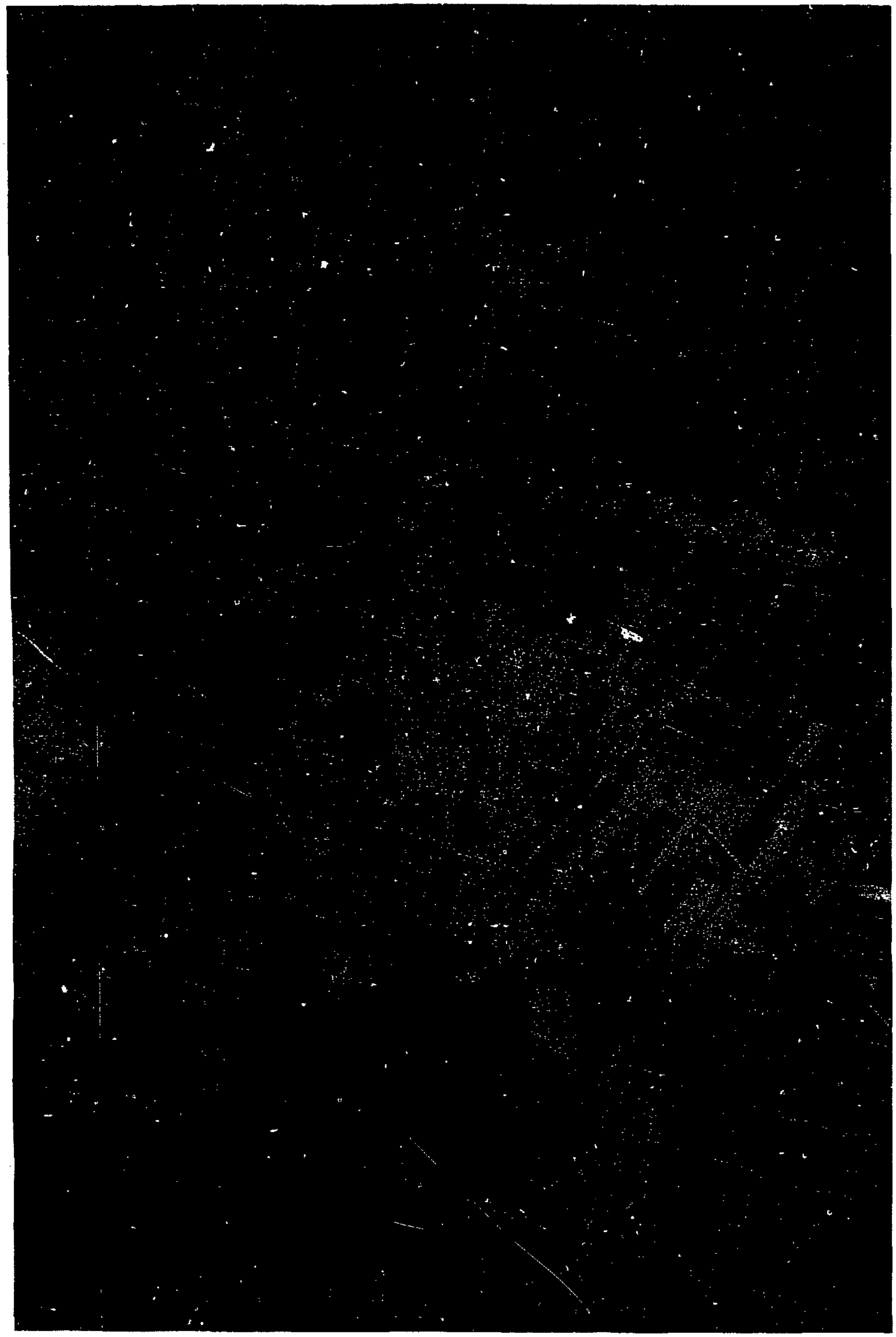


Figure 19. Large scale direct shear machine of U.C. Berkeley's Rock Mechanics Laboratory. Maximum sample area is 30 × 45 cm.[69]

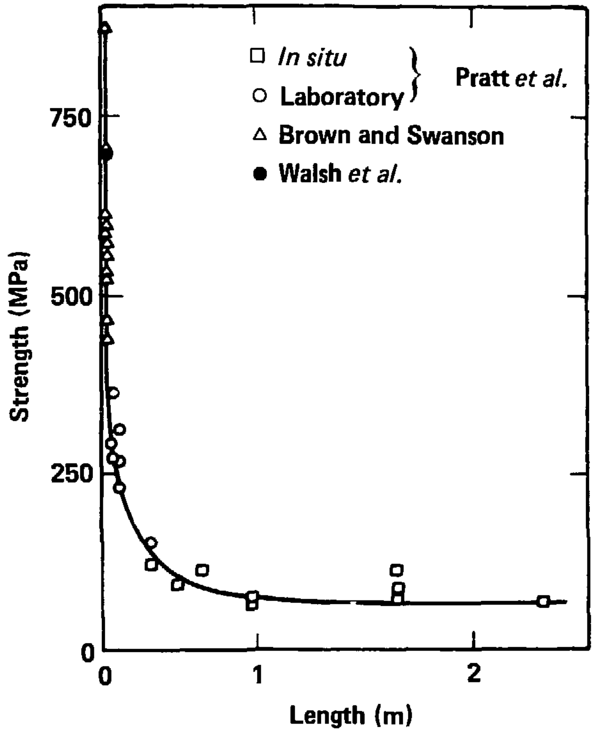


Figure 20. Scale effect on unconfined compressive strength of diorite.[70]

3.4. AN OUTLINE OF SUGGESTED COMPLEMENTARY STUDIES

Based on the above discussion, several suggestions for new and modified procedures are grouped under the headings of field testing, laboratory testing, and numerical modeling. Table 6 outlines the types of activities which are involved, and the purpose of each activity. An attempt also is made to establish priority needs regarding these activities. The following comments are in order:

- The recommendations entail some redundancy in the testing. In particular, it is proposed that short-term deformability and strength properties be measured both in the field and in the laboratory. This is related to the effect of scale on the one hand and to the imperfections of both laboratory and field tests on the other. The exact volume of rock involved in a borehole test is not precisely defined, as it would be in a laboratory test.[13] On the

Table 5. Approximate strength criteria for intact rock and jointed rock masses.[72]

APPROXIMATE RELATIONSHIP BETWEEN ROCK MASS QUALITY AND CONSTANTS					
Empirical failure criterion $\sigma_1 = \sigma_3 + \sqrt{m\sigma_c\sigma_3 + s\sigma_c^2}$ σ_1 = major principal stress σ_3 = minor principal stress σ_c = uniaxial compressive strength of intact rock, and m, s = empirical constants.	CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE *dolomite, limestone and marble*	LITHIFIED ARGILLACEOUS ROCKS *mudstone, siltstone, shale and slate (normal to cleavage)*	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE *sandstone and quartzite*	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *andesite, dolerite, diabase and rhyolite*	COARSE GRAINED POLYMINERALLIC IGNEOUS AND METAMORPHIC CRYSTALLINE ROCKS *amphibolite, gabbro, gneiss, granite, norite and quartz-diorite*
INTACT ROCK SAMPLES *Laboratory size specimens free from joints* CSIR rating 100 NGI rating 500	m = 7.0 s = 1.0	m = 10.0 s = 1.0	m = 15.0 s = 1.0	m = 17.0 s = 1.0	m = 25.0 s = 1.0
VERY GOOD QUALITY ROCK MASS *Tightly interlocking undisturbed rock with unweathered joints at 1 to 3m.* CSIR rating 85 NGI rating 100	m = 3.5 s = 0.1	m = 5.0 s = 0.1	m = 7.5 s = 0.1	m = 8.5 s = 0.1	m = 12.5 s = 0.1
GOOD QUALITY ROCK MASS *Fresh to slightly weathered rock, slightly disturbed with joints at 1 to 3m.* CSIR rating 65 NGI rating 10	m = 0.7 s = 0.004	m = 1.0 s = 0.004	m = 1.5 s = 0.004	m = 1.7 s = 0.004	m = 2.5 s = 0.004
FAIR QUALITY ROCK MASS *Several sets of moderately weathered joints spaced at 0.3 to 1m.* CSIR rating 44 NGI rating 1	m = 0.14 s = 0.0001	m = 0.20 s = 0.0001	m = 0.30 s = 0.0001	m = 0.34 s = 0.0001	m = 0.50 s = 0.0001
POOR QUALITY ROCK MASS *Numerous weathered joints at 30 to 500mm with some gouge. Clean compacted waste rock.* CSIR rating 23 NGI rating 0.1	m = 0.04 s = 0.00001	m = 0.05 s = 0.00001	m = 0.08 s = 0.00001	m = 0.09 s = 0.00001	m = 0.13 s = 0.00001
VERY POOR QUALITY ROCK MASS *Numerous heavily weathered joints spaced < 50mm with gouge. Waste rock with fines.* CSIR rating 3 NGI rating 0.01	m = 0.007 s = 0	m = 0.010 s = 0	m = 0.015 s = 0	m = 0.017 s = 0	m = 0.025 s = 0

Table 6. Suggested geotechnical studies complementary to current containment practice.

Type of information	Type of activity	Suggested priority
In Situ Testing[a]		
In situ stresses	Hydrofracturing (P)[b] Borehole jack fracturing (S)	1
In situ deformability	In situ velocity and stress measurements (P)[c] Borehole jack fracturing (S) Petite sismique (S) Rock mass classification (S)	1
In situ tensile strength	Modified hydrofrac (P) Borehole jack fracturing (S)	2
In situ shear strength	Borehole shear	1
Laboratory Tests		
Very-short-term (dynamic) deformability and strength	Large-scale triaxial	2[d]
Short-term deformability	Large-scale triaxial	1
Short-term shear strength	Large-scale triaxial (P) Large direct shear (S)	1 1
Long-term deformability	Large-scale triaxial creep	2
Long-term strength	Short-term triaxial plus creep tests	1
Numerical Modeling		
Short-term effects (cavity growth, spall region, containment cage)	Explicit calculations using refined input of in situ stresses and material properties	1
Long-term effects (cavity collapse, subsidence, and surface collapse)	Implicit calculations with secondary creep using input from short-term calculations	2

[a] All short-term tests.

[b] P: primary method; S: secondary method.

[c] Such instrumentation recently was fielded with the TILCI event. Results of the measurements are being analyzed.[78]

[d] Although this is very desirable information, we do not know of equipment now suitable for such testing.

other hand, the material in place certainly is less disturbed than when extracted and transported to a laboratory.

- The typical stresses involved in the proposed field and laboratory tests range from 1 bar to 1 kbar (0.1 to 100 MPa). This is the very range where the greatest need exists for refinement of material properties in containment calculations.[78]
- The recommendations offered in the section on physical properties do not affect the current practice for measuring these properties.
- The suggested priority for the various activities involve an attempt to weigh various potentially conflicting factors such as the desirability of having the information, the practicality of obtaining the information, and the possibility of using this information when it is generated. For example, it is desirable to know whether the principal stresses in the horizontal direction are unequal, because the more unequal they are the higher the horizontal shear stress. It is practical but not trivial to obtain a measurement of such stresses by hydraulic fracturing, for example. However, this information cannot be used directly in a two-dimensional calculation because it requires the horizontal principal stresses to be equal. Then, one may choose to adopt the smaller principal stress as the most conservative value.

Table 6 shows that first priority is given to the short-term effects and to the large scale determination of deformability and shear strength in the field or, when possible in the laboratory. Of less urgency are the long-term effects and the determination of tensile strength. This latter parameter, however, can be obtained as a by-product of hydrofracturing stress measurements, which rank as a very desirable task.

Definitions and details for the new in situ tests, laboratory tests, and computer models are given in the following chapter.

4. DETAILS OF THE SUGGESTED GEOTECHNICAL STUDIES

4.1. IN SITU TESTING

4.1.1. In Situ Stresses

The recommended method to obtain in situ stresses at depths up to several hundred metres in the alluvium and tuffs is the hydraulic fracturing technique.[63,79,80] Hydrofracturing results obtained in the tuffs of N, E, and T tunnels at NTS, did correlate well with results obtained by the overcoring method.[81-83] However, overcoring at great depth still is an experimental technique,[84] whereas deep hydrofracturing is a proven procedure. In the Mesas, it has been shown that one of the principal stresses is near vertical.[81] Such an assumption is reasonable as well for the weakly consolidated materials of Yucca Flat. Thus, the two remaining principal stresses are in horizontal directions, and hydrofracturing in vertical holes is a suitable approach. These horizontal stresses are not necessarily equal to each other. This was confirmed by recent hydrofracturing in soils,[85] as well as in the tuffs of Area 12, where major and minor horizontal stresses were measured as 88 and 35 MPa, respectively.[81] The symbols used in this discussion are illustrated in Figs. 21 and 22 for a dry medium:

σ_{hM} is the maximum horizontal stress.

σ_{hm} is the minimum horizontal stress.

σ_t is the tensile strength of the formation.

p_{c1} is the initial breakdown pressure (hydraulic pressure when the first pressurization overcomes the tensile strength and the in situ stress concentrations).

p_s is the shut-in pressure or steady pressure achieved when pumping continues beyond p_{c1}.

p_{c2} is the new peak pressure obtained when repressurizing a hole after letting the hole pressure fall below p_s. p_{c2} does not enter the equations in conventional hydraulic fracturing to measure stresses. When there is no pore pressure, two equations allow σ_{hM} and σ_{hm} to be calculated if the in situ tensile strength is known:

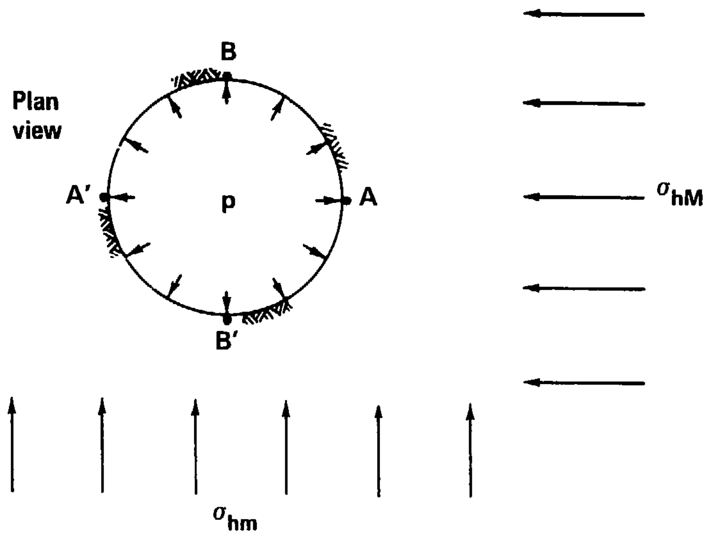


Figure 21. Internal pressure and in situ stresses in hydraulic fracturing.[63]

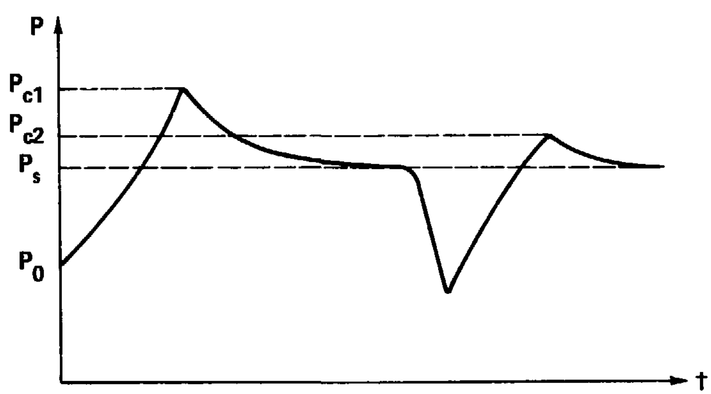


Figure 22. Pressure-time curve in hydraulic fracturing.[63]

$$3\sigma_{hm} - \sigma_{hM} = p_{c1} - \sigma_t \quad , \tag{1}$$

$$\sigma_{hm} = p_s \quad . \tag{2}$$

When there is a pore pressure, p_o, in the formation, p_o is subtracted from the total stresses in Eqs. (1) and (2). The above equations also assume that the hydrofracturing fluid does not percolate in the geological medium. A previous attempt to measure stresses in the NTS alluvium by hydrofracturing did not provide reliable values.[86] Improvements in the technique, such as the use of highly viscous gels and of loss additives should enhance the prospect of obtaining better results today.[85] Standard equipment available in the geotechnical community permits hydrofracturing in NX (7.5 cm diam) boreholes.

An alternate method is to use the modified NX borehole-jack method which is described elsewhere.[87,88] The technique is still somewhat experimental but should work well in nonfractured formations. A recent application at NTS in the Climax granite was only partially successful because of the existing fractures in the rock.[89] Such fractures would affect the quality of hydrofrac measurements as well.

4.1.2. In Situ Deformability

The preferred method consists of direct measurements with the NX borehole jack, which is particularly well suited for soft rock formations.[90,91] The jack can apply wall pressure up to 70 MPa. A large number of measurements can be made at different orientations in a single hole. The volume of rock involved in the test is estimated to be about 4.6 ft^3, or 0.13 m^3.[13] This is the same volume as that of a 44-cm-diam cylinder with a length/diameter ratio of 2. Based on an earlier discussion, this means that the jack certainly is adequate when the fracture spacing or the maximum particle size are less than about 3 in. (7.5 cm). In fact, experience in rocks[16,90,92] has shown the jack to be applicable in cases when the spacing was up to 10 in. (25 cm).

Additional estimates of rock mass deformability can be obtained with indirect methods such as the empirical correlations based on the petite sismique method and rock mass classifications.[89] A recent application of these methods in the Climax granite at NTS demonstrates the value of obtaining redundant estimates of rock mass properties.[92]

4.1.3. In Situ Tensile Strength

There is no proven method to determine the tensile strength of geologic materials in situ. However, two methods have been proposed which, in theory can provide the required information, i.e., an extended hydraulic fracturing procedure and the use of borehole jack fracturing.

If, in hydrofracturing, the borehole pressure is dropped after reaching the shut-in pressure, p_s, and raised again, the hydraulic fracture will close and reopen.[63] Let the new peak pressure be p_{c2} (see Fig. 21). It is smaller than p_{c1}, because the tensile strength of the material is now zero. Thus, replacing σ_t by 0 and p_{c1} by p_{c2}, Eq. (1) becomes

$$3\sigma_{hm} - \sigma_{hM} = p_{c2} \quad . \tag{3}$$

Subtracting Eq. (1) from Eq. (3) gives:

$$\sigma_t = p_{c1} - p_{c2} \quad . \tag{4}$$

Turning around, one realizes that σ_{hm} and σ_{hM} can now be calculated without assuming σ_t. The extended hydrofracturing procedure is self-contained and provides σ_{hm}, σ_{hM}, and σ_t. A caveat is in order at this point. The hydraulic fracturing experiment does not yield the above results if the fracture is not vertical.[63] Assuming that the tensile strengths for propagation of horizontal and vertical fractures are the same, the vertical fracture could form only at a depth below which the vertical stress is[63]:

$$\sigma_v \geq 3\sigma_{hm} - \sigma_{hM} \quad . \tag{5}$$

Let K be the ratio of the average horizontal stress to the vertical stress,

$$K = (\sigma_{hm} + \sigma_{hM})/2\sigma_v \quad , \tag{6}$$

and z be the depth in metres. A recent survey of numerous published in situ stress values[93] has led to the conclusion that most K values are within the limits

$$0.3 + 100/z \leq K \leq 0.5 + 1500/z \quad . \tag{7}$$

Table 7.

Horizontal stress ratio σ_{hm}/σ_{hM}	Minimum depth (m) for vertical hydrofrac, assuming	
	$z = 100/(K - 0.3)$	$z = 1500/(K - 0.5)$
0.33	0	0
0.40	31	500
0.50	83	1,500
0.60	143	3,000
0.667	188	4,505
0.70	211	5,495
0.80	292	10,490
0.90	386	25,424
1.00	500	∞

Equations (5) and (7) combine to give the range of the minimum depth to obtain a vertical hydraulic fracture corresponding to different values of N (see Table 7).[63]

The preceding discussion highlights the requirement for determining the orientation of the hydraulic fracture. This can be done either by using an impression packer, or by visual inspection of the borehole with a camera.

An independent estimate of in situ tensile strength can be obtained by borehole-jack fracturing.[94] The borehole-jack loading in a medium subjected to a biaxial stress field is illustrated in Fig. 23. In Fig. 24 the distribution of induced tangential stress, σ_θ, shows a maximum at an angle, β, which is half the angular width of the jack plates. With the conventional NX borehole jack in which $\beta = 45^o$, the tensile strength estimate was obtained as[94]:

$$\sigma_t = \sigma_{hM} + \sigma_{hm} - 2(\sigma_{hM} - \sigma_{hM}) \cos 2(45 - \alpha) + 2P_i/r \quad , \qquad (8)$$

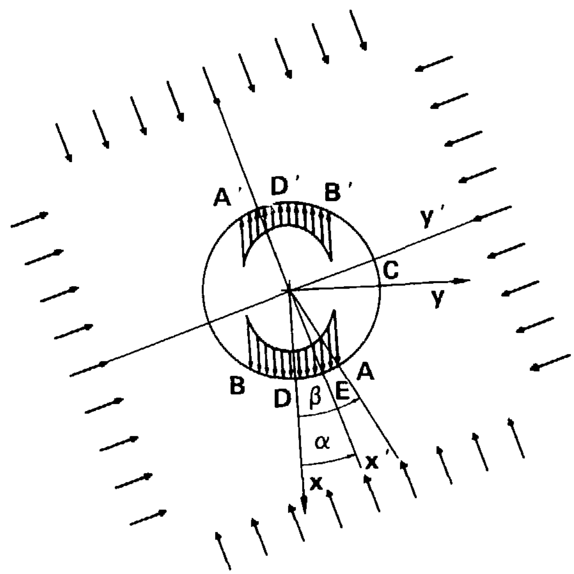


Figure 23. Borehole-jack loading in a biaxial stress field.[83]

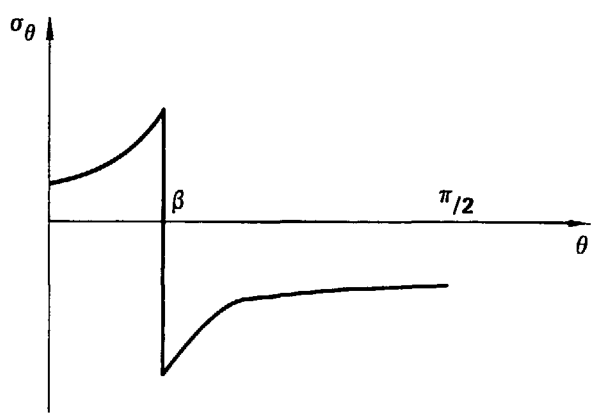


Figure 24. Tangential stress on borehole wall under jack loading.[83]

where

- α is the angle between the direction of borehole loading and σ_{hM},
- P_i is the borehole jack pressure at initiation of tensile fracture around the hole, and
- r is the borehole radius.

An estimate of P_i can be obtained from a softening in the load-displacement curve of the jack record; σ_{hM}, σ_{hm} and α can be gained in an independent hydrofracturing test. Thus, an additional estimate of σ_t can be acquired to be compared with the value derived from the extended hydrofracturing procedure.

4.1.4. In Situ Shear Strength

The only instrument available today to measure rock shear strength at depth is the Rock Borehole Shear Tester (RBST), which has been developed for use in NX holes.[95,96] The principle is to expand a borehole jack with specially serrated loading plates and then to pull the shoes in contact with the rock, parallel to the axis of the borehole, under a constant normal force. The shear surface is in the material adjacent to the borehole wall. The normal stress range is 0.1 to 80 MPa, and the shear stress range is 0 to 50 MPa. From the measured pull force, a rock shear strength is derived which is expressed in terms of cohesion and friction angle. The instrument has been used in softer rocks (coal, marlstone, mudstone, trona, . . .) to the apparent satisfaction of the users.[96] A possible limitation of the method is that the material adjacent to the borehole wall would have been so disturbed in the drilling process that it would not be representative any longer.

4.1.5. Suppliers and Costs

The above tests can be done either by subcontracting or by having LLNL purchase and field the equipment, which would allow modifications and enhancement as desired. It is quite remarkable that all but one of the procedures can be performed in NX-size holes with off-the-shelf equipment. The exception is the petite sismique (shear-wave propagation) for which current down-the-hole equipment probably can be adapted to NX size at moderate cost. The cost of drilling a 1000-ft-deep hole with a diameter of 3 to 12 in.

in alluvium currently is estimated at about $200,000.[97] This compares to about $600,000 for a 1000-ft deep, 96-in.-diameter emplacement hole.[97] The cost of NX holes currently is not included in the budget for routine tests.

Hydraulic fracturing in NX holes is routinely performed by Dr. B. C. Haimson of the University of Wisconsin at Madison. Costs incurred involve only time and expenses. No charge is made for equipment rental. The cost of LLNL acquiring this type of equipment is not known at present. Another man with considerable experience in hydrofracturing is Dr. J. C. Roegiers who is presently with Dowell Company of Tulsa, Oklahoma, while on leave of absence from the University of Toronto, Canada.

The conventional NX borehole jack can be purchased from Slope Indicator Company in Seattle (SINCO). A system to operate down to a few hundred metres would cost between $10,000 and $15,000. The equipment can also be rented from SINCO on a weekly or monthly basis, as was done for recent NTS work in the Climax granite.[92]

The modified borehole jack for borehole fracturing is available on a service basis from Dr. R. V. de la Cruz, also of the University of Wisconsin at Madison. Again, only personnel time and expenses are involved. Since this instrument is a prototype, there is no firm price on LLNL acquisition of such a system. Because of the still experimental nature of the procedure it would be advisable to consider purchase only after the method has been proven in the materials of interest.

The Rock Borehole Shear Tester is manufactured by Handy Geotechnical Instruments of Ames, Iowa. The service is also available for rent from Geotest in Chicago. Current depth limitation is about 100 m. The cost of a system operating to a depth of about 100 m, in connection with a wireline, would be about $10,000. Rental costs can be obtained on request.

4.2. Laboratory Testing

The discussions of Chapter 3 clearly pointed to the need for resolving time effects and scale effects on geological material properties. Laboratory testing seems attractive for two reasons. In terms of time, it would be most impractical to attempt performing the field tests previously described for extended periods of time; e.g., the days and, possibly, weeks required for investigation of creep. In terms of scale, the field tests in borehole do have a set size, whereas the sample size can be varied in the laboratory.

It is not necessary to dwell on the well known triaxial test and the direct shear test. However, it is worth noting that large scale triaxial equipment is accessible within the University of California's own laboratories at Berkeley and Richmond. The large scale triaxial testing machine shown on Fig. 18 is currently on loan to the Earth Sciences Division of the Lawrence Berkeley Laboratory (LBL) by the Civil Engineering Department at U.C. Berkeley. The normal frame capacity is 4 Mlb (17.8 MN); the system is servocontrolled and is equipped with an HP 9845 computer. Accessibility to this test equipment should increase now that the LBL/Stripa program, under the direction of Dr. P. A. Witherspoon, is being concluded. The large scale shear machine can be made available by Dr. R. E. Goodman of U.C. Berkeley on an ad hoc basis. Specific costs can be negotiated at a later date.

Even though the maximum confining pressure in the LBL machine is limited to about 1000 psi (7 MPa), the independent testing of various sizes of samples in the U.C. labs would indicate whether the test volumes involved in the field borehole measurements are representative. There is little doubt that the maximum volume that the Richmond triaxial system can accommodate would exceed the minimum representative volume for the great majority of the NTS alluvium. Testing of volumes up to 0.1 m^3 was also proposed recently at LLNL,[98] but this capability does not exist yet. As for the hard rocks, large granite cores have been tested on the recent LBL/Stripa program (see Fig. 25). The technology used to sample and transport these cores probably can be transferred to NTS materials such as welded tuff and rhyolite.

4.3. Numerical Modeling

For the analysis of short-term events (cavity growth, spall region, containment cage) a wave-propagation explicit finite difference code such as TENSOR provides a framework for computations. It contains algorithms which should accommodate the refined input of geology, in situ stresses, and mechanical properties of rocks and soils. The credibility of containment calculations can only benefit from a systematic effort to develop this refined input.

For the long-term aspects (cavity collapse, subsidence, and surface collapse) the numerical models should accommodate steady-state creep and failure mechanisms. Recent developments for calculations of subsidence over

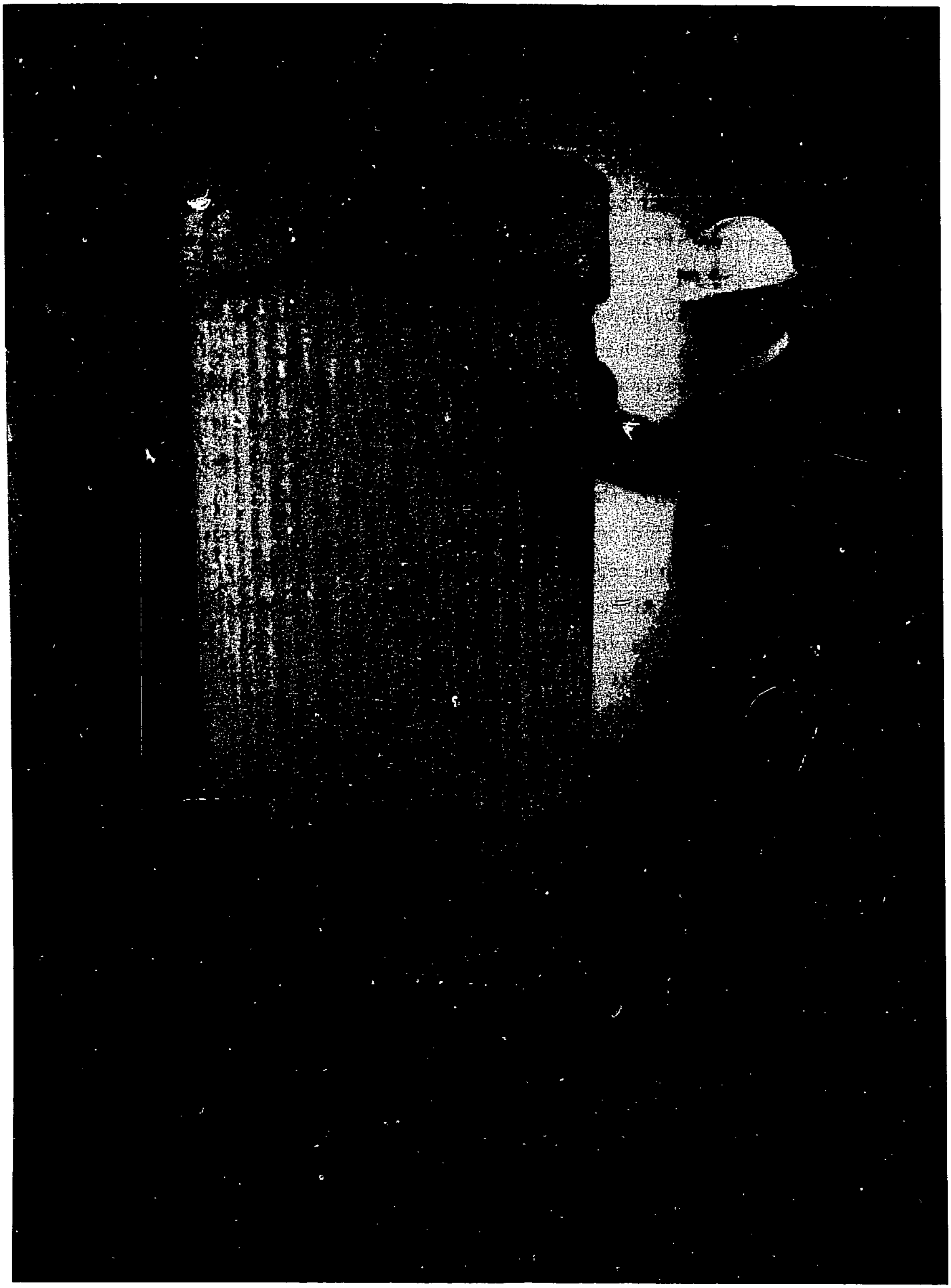


Figure 25. Large scale granite core for uniaxial test (LBL photograph).

underground coal gasification areas [99-101] and recent extensive surveys of the geotechnical computing field [102,103] indicate that implicit procedures are likely to be more efficient and versatile in performing the long-term calculations. It is suggested that development work be pursued on containment studies to couple the results of explicit short-term analysis to the input of implicit long-term calculations. TENSOR does have an implicit option but does not model transient or steady-state creep yet. Such creep models could be incorporated in the code. An alternative is to use a code already available at LLNL, such as the SANGRE finite element program developed at LANL,[104] and couple it to TENSOR.

5. SUMMARY

Currently, estimation of containment of underground explosions at NTS is based mostly on empiricism derived from extensive experience and on a combination of physical/mechanical testing and numerical modeling. When measured directly, the mechanical material properties are obtained from short-term laboratory tests on small conventional samples. This practice does not determine the large effects of scale and time on measured stiffnesses and strength of geologic materials. Because of the limited data base of properties and in situ conditions, the input to otherwise fairly sophisticated computer programs is subject to several simplifying assumptions; some of them can have a nonconservative impact on the calculated results. As for the long-term, subsidence and collapse phenomena simply have not been studied to any significant degree.

This report has examined the geomechanical aspects of procedures currently used to estimate containment of underground explosions at NTS. Based on this examination, it was concluded that state-of-the-art geological engineering practice in the areas of field testing, large scale laboratory measurements, and numerical modeling can be drawn upon to complement the current approach. Specific discussions were made with regard to:

- The time and scale effects in the measurement of mechanical properties of geological materials.
- Measurement of in situ stresses by hydraulic fracturing and borehole jack fracturing.
- Measurement of in situ deformability by NX-jack tests.
- Measurement of in situ tensile strength by hydraulic fracturing and borehole jack fracturing.
- Measurement of in situ shear strength by borehole shear tests.
- Large-scale laboratory triaxial tests.
- Large-scale laboratory direct shear tests.
- Implicit numerical modeling of subsidence and collapse processes, using the output from short-term explicit calculations of early ground response to explosions.

In cases where today's evaluations indicate marginal conditions or in cases where new test areas are contemplated for which there is no benefit of experience, it is reasonable to expect that a refined input to the calculations will provide more realistic containment estimates than current practice does.

6. ACKNOWLEDGMENTS

This research was supported by the Containment Program funded by the Department of Energy at the Lawrence Livermore National Laboratory.

The author gratefully acknowledges the contributions of the following colleagues: A. E. Abey, D. E. Burton, M. S. Costantino, H. C. Heard, N. W. Howard, H. L. McKague, W. McKinnis, F. A. Morrison, Jr., C. W. Olsen, R. P. Swift, L. Thigpen, R. K. Thorpe, O. R. Walton, and particularly J. T. Rambo and R. W. Terhune. Special thanks also are due to Mrs. L. Burrow for her fine typing of the manuscript.

LR/KT

APPENDIX A

SUMMARY OF THE GEOLOGICAL, GEOTECHNICAL, AND GEOPHYSICAL INFORMATION PRESENTED TO THE CEP FOR EVENTS TILCI[8] and AKAVI[9]

TILCI: Figures A-1 to A-7; alluvium.

AKAVI: Figures A-8 to A-14; unsaturated tuff.

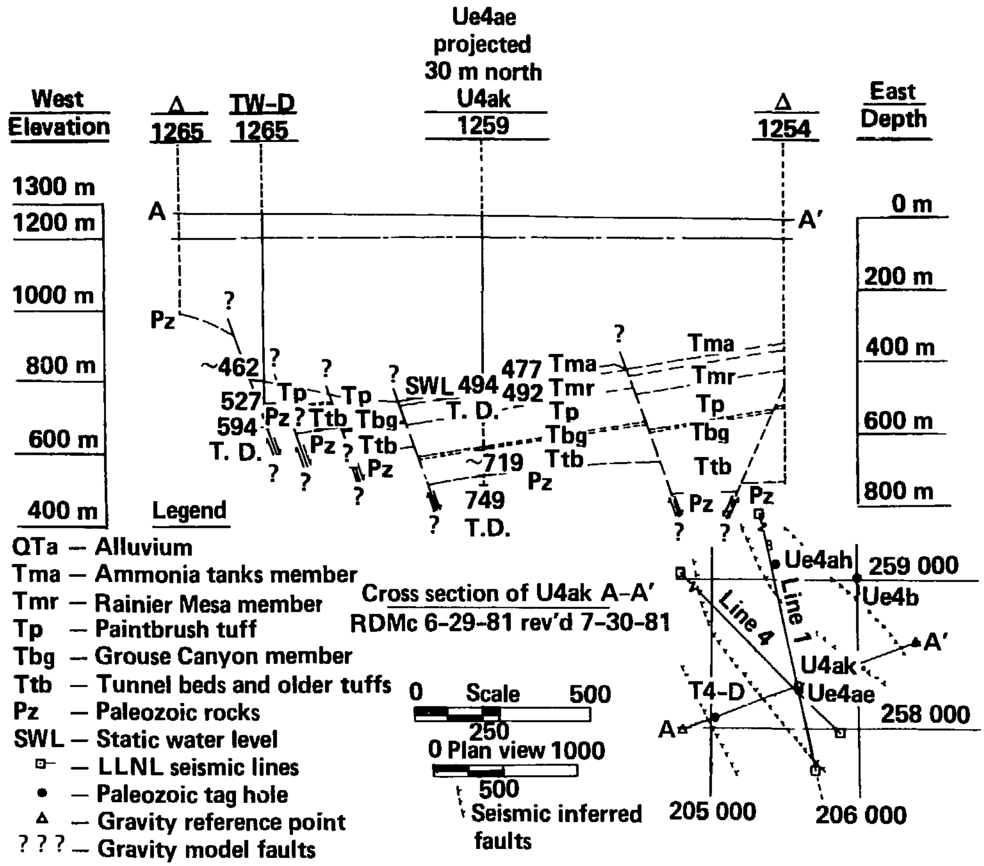
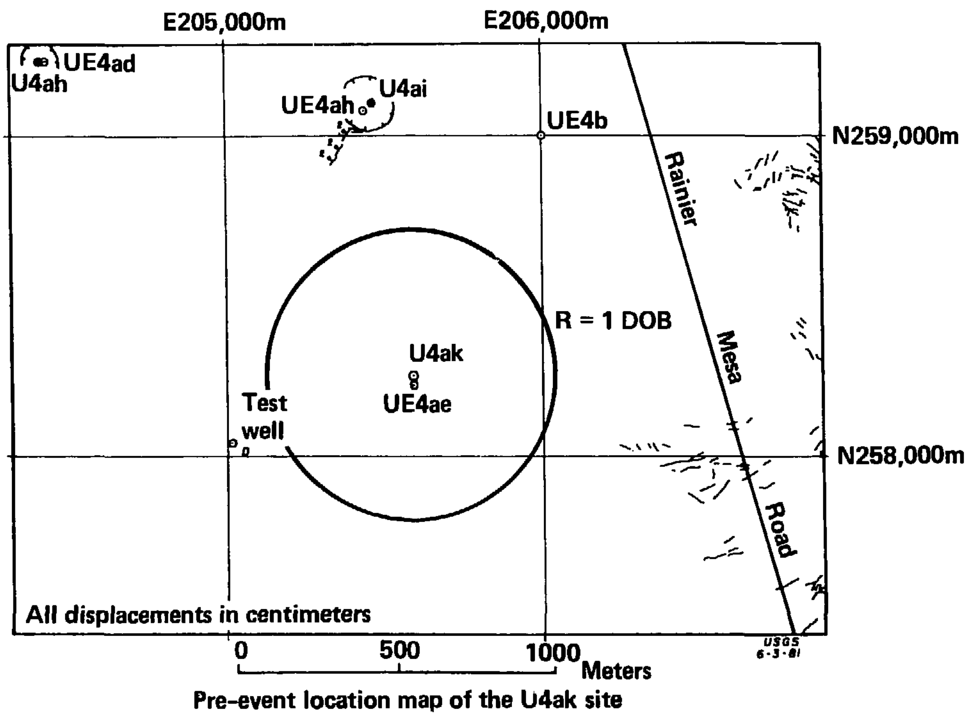


Figure A-1.



Pre-event location map of the U4ak site

Figure A-2.

DRILLING HISTORY

Ue4ae was spudded 4-24-74 as part of the LLNL Area 4 exploration program. The hole, drilled with conventional air-foam was completed on 5-20-74 at a depth of 749 m in Paleozoic rocks. Logs were run 5-20- to 5-23-74, the hole was sidewall sampled on 6 m intervals 5-23 to 5-26-74, then plugged back to 698 m (above the Paleozoic surface). The hole was muddeo up; 3-D, dipmeter and electric logs were run. In 1981, additional siaewall samples on 3 m intervals were taken on 5-10, and logs on 5-6 and 5-7-81.

U4ah was spudded on 4-13-81, 30 m north of Ue4ae. It was completed using dual string reverse air and water circulation to a depth of 497 m by 5-6-81. No difficulties were encountered in drilling. Total cuttings samples were collected, and logs were run from May to July 1981.

MEDIUM CHARACTERISTICS

Medium characteristics calculated for a 15.2 m radius averaging interval centered at 445 m in tuffaceous alluvium at U4ak are derived from Ue4ae and summarized in Table I. Between the two holes, magnetometer correlations indicate 2^{0} dip at the averaging interval, centered at 444 m in Ue4ae which corresponds to 445 m in U4ak. The sources of information and assumptions made in this analysis are given in Table II. The previous experience for selecteo sites in alluvium in southern Area 2 and western Area 4 is shown in Table III, and graphically in the histograms Figs. 1 and 2.

All of the properties measured and calculated at U4ak are within the range of previous experience for these areas. Slightly high W.P. water content and porosity are similar to those measured at the nearby U4ai (BURZET) site, whose W.P. was in tuffaceous alluvium at a similar depth (450 m). However, Burzet's averaging interval went into the Ammonia Tanks unit, which has lower density and lower velocity and of course, the averaging interval properties reflect this.

Figure A-3.

UE4AE GENERATED 07/09/81

DATA SUMMARY FOR WP = 444.0 M = 445 m U4ak

VP MEDIUM IS AL

AVERAGING RADIUS = 15.20 M (15.07 UP, 15.32 DOWN)
AVERAGING INTERVAL = 428.9 TO 459.3 M

PARAMETER	MEAN VALUE	UNITS	NO. POINTS	STD DEV	ESTIMATED ERROR	DATA-RANGE	DEPTH-RANGE	AVERAGING METHOD	LOG-TYPE	RUN NO.
BULK DENSITY	1.83	MG/M3	151	0.19	0.09	1.31 TO 2.40	428.9 TO 458.8	INT	DENSITY BC	1
GRAIN DENSITY	2.54	MG/M3	10	0.04	0.01	2.46 TO 2.61	431.0 TO 457.2	NUM	SAMPLE HNT	1,2
WATER CONTENT	16.1	WT%	10	2.2	0.7	13.1 TO 18.7	431.0 TO 457.2	NUM	SAMPLE HNT	1,2
POROSITY	39.6	VOL%	10	*.	3.1	*. TO *.	428.9 TO 459.3	CALC	CALCULATED	
SATURATION	74.3	VOL%	10	*.	9.7	*. TO *.	428.9 TO 459.3	CALC	CALCULATED	
GAS POROSITY	10.1	VOL%	10	*.	4.6	*. TO *.	428.9 TO 459.3	CALC	CALCULATED	
CO2 CONTENT	1.6	WT%	11	1.9	0.6	0.1 TO 6.1	431.0 TO 457.2	NUM	SAMPLE HNT	1C,2
DUAL SR VELOCITY	2141.	M/S	10	314.	*.	1703. TO 2605.	429.8 TO 457.2	INT	DUAL SR	1
SEISMIC VELOCITY	1890.	M/S	2	*.	293.	*. TO *.	411.5 TO 472.4	INT	VIBROSEIS	1

OVERBURDEN PARAMETERS (WP-TO-SURFACE AVERAGES)

BULK DENSITY	=	1.89	MG/M3	+ - 0.09
DUAL SR VELOCITY	=	1668.	M/S	+ - 0.
SEISMIC VELOCITY	=	1600.	M/S	+ - 80.
GAS POROSITY	=	14.8	VOL%	

Zones of swelling clay
greater than 20%
@ 381 m, U4ak cuttings sample

BULK DENSITY IS WATER-CORRECTED

Figure A-4.

PHYSICAL PROPERTIES DISTRIBUTION FOR SELECTED W.P. IN UNSATURATED ALLUVIUM, AREA 4 AND SOUTHERN AREA 2

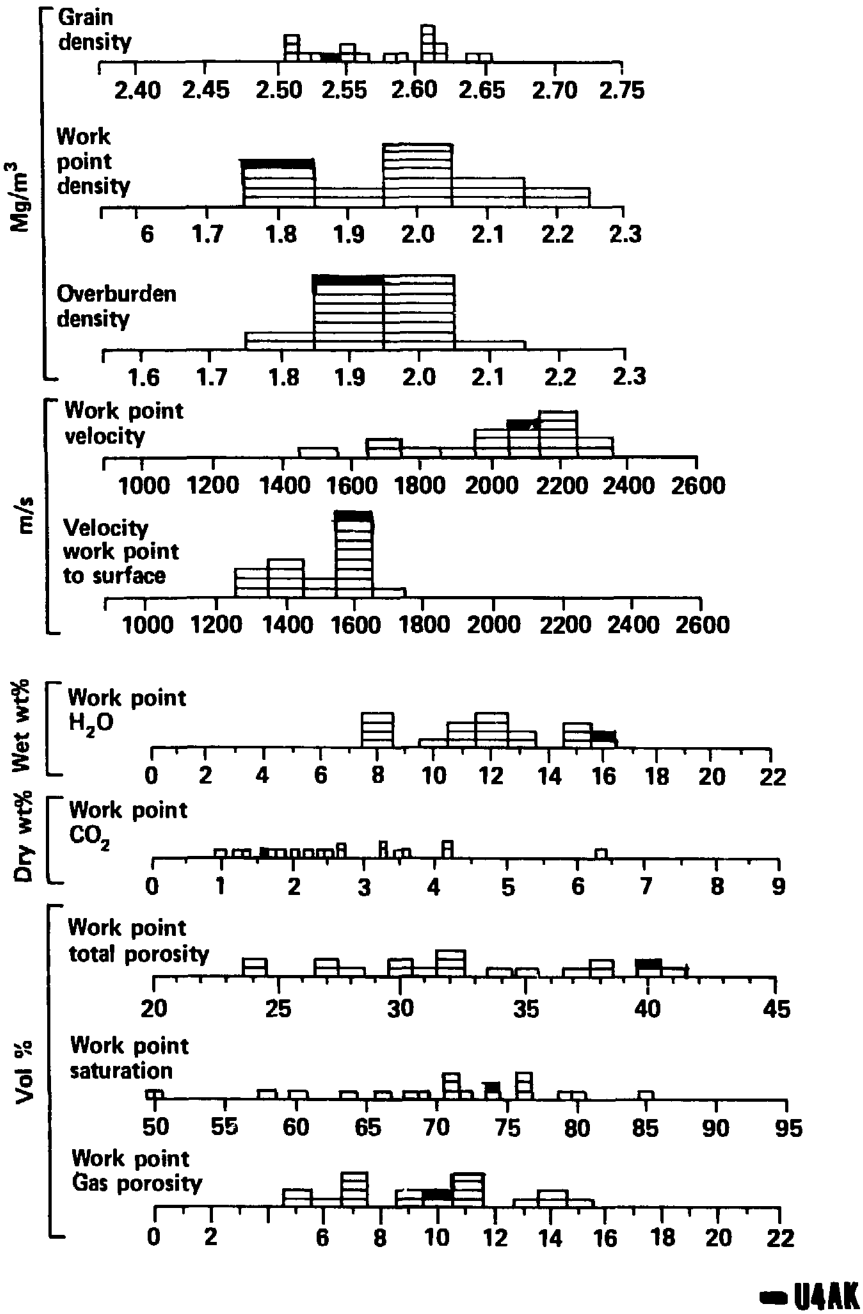


Figure A-5.

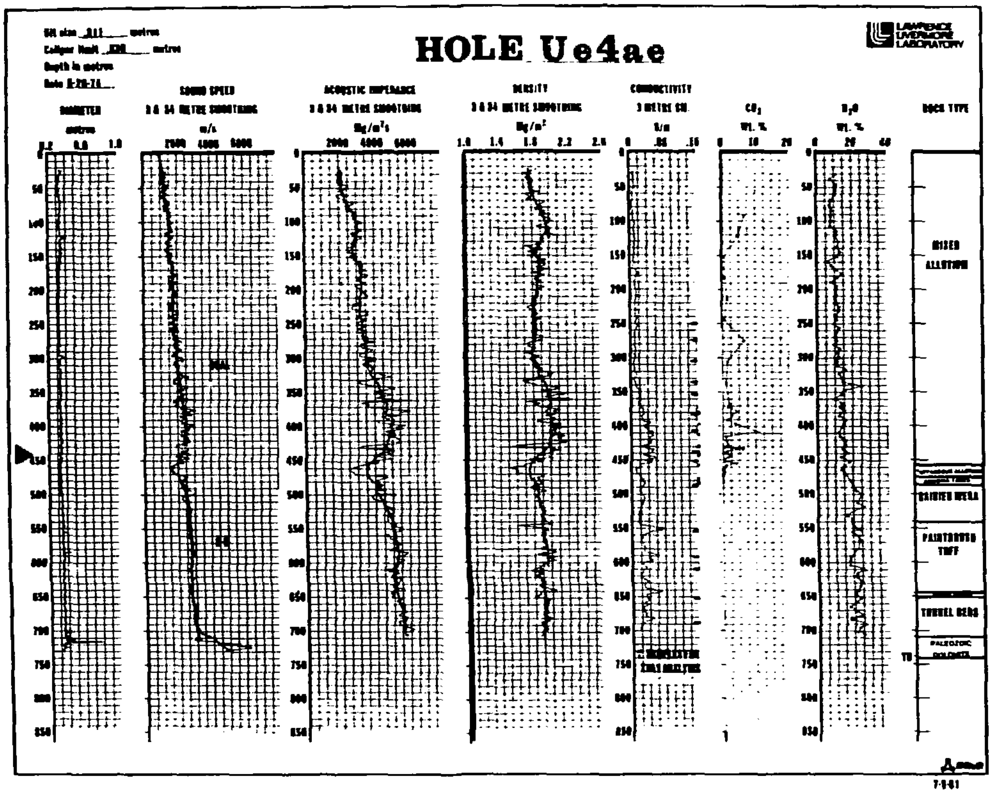


Figure A-6.

SOUTHWESTERN AREA 2 AND WESTERN AREA 4
ALLUVIUM CHARACTERISTICS SUMMARY

Hole	W.P.	Density, Mg/m^3 Overburden	W.P.	Grain	Velocity m/s W.P. to Sur	W.P.	Wt % H_2O	ϕ	Vol % Sat	Gasϕ	Wt % CO_2
U2dh-2	198	1.80	2.10	2.62	1280	1740	8	27	58	11	2.7
U2dh-3	259	1.90	2.10	2.61	1370	2260	10	27	76	7	6.3
U2di	331	2.00	2.00	2.52	1646	2073	13	30	85	5	2.2
U2dk	323	2.00	2.00	2.59	1585	2195	11	35	60	14	2.5
U2dl	331	2.00	1.90	2.53	1645	2195	12	34	66	11	3.5
U2dm	326	2.00	2.00	2.56	1615	1961	12	32	76	7	3.3
U2dn	204	1.90	2.20	2.62	1280	1525	8	24	71	7	2.0
U2do	326	1.90	2.00	2.58	1585	2350	12	32	72	9	3.3
U2dp	296	2.00	2.10	2.61	1494	2134	11	28	80	5	4.2
U2du	183	1.80	2.00	2.61*	1340	2070	8	30	50	15	1.2
U2dv	466	2.00	2.00	2.65*	1615	1890	11	32	71	9	4.2
U2dw	374	2.00	2.00	2.61*	1554	2225	12	31	79	7	3.6
U2dw	272	2.00	2.20	2.64*	1402	1951	8	24	76	6	2.4
U2dz	536	2.08	1.88*	2.55*	1747	2204	15	37	74	10	1.8
U4aa	263	1.90	1.80	2.51*	1500	2175	15	38	71	11	1.7
U4ab	263	1.90	1.80	2.51*	1425	1975	13	38	64	14	1.0
U4af	208	1.87	1.80*	2.55*	1388	1752	15	40	68	13	2.7
U4ai	450	1.92	1.77*	2.51*	1589	1725	16	41	69	11	1.3
U4ak	445	1.89	1.83*	2.54*	1600	2141	16	40	74	10	1.6

*water corrected

NWH 7-8-81

Figure A-7.

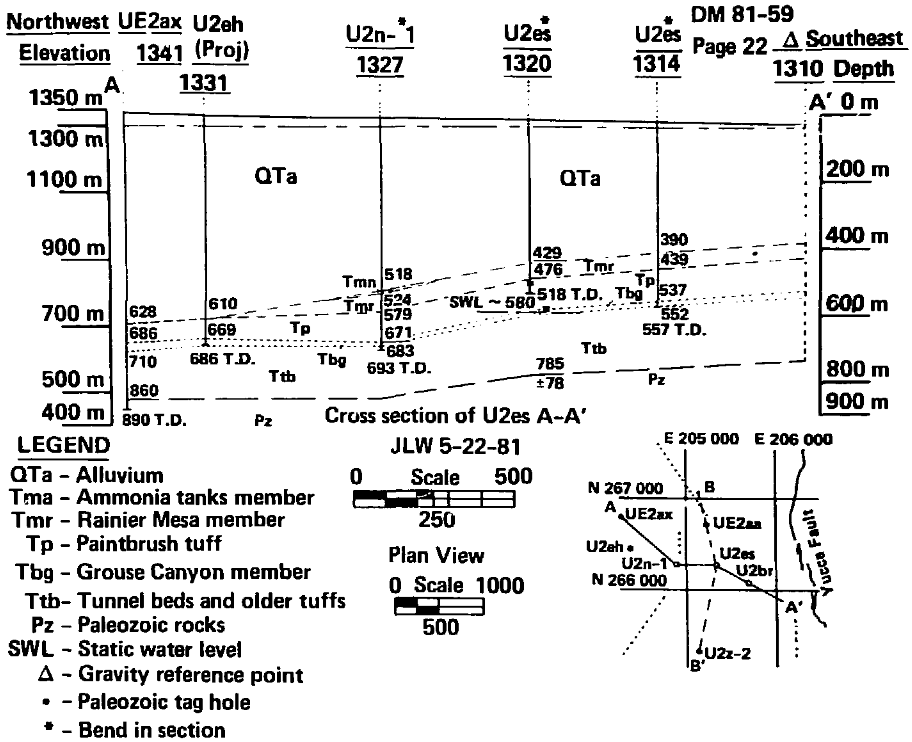
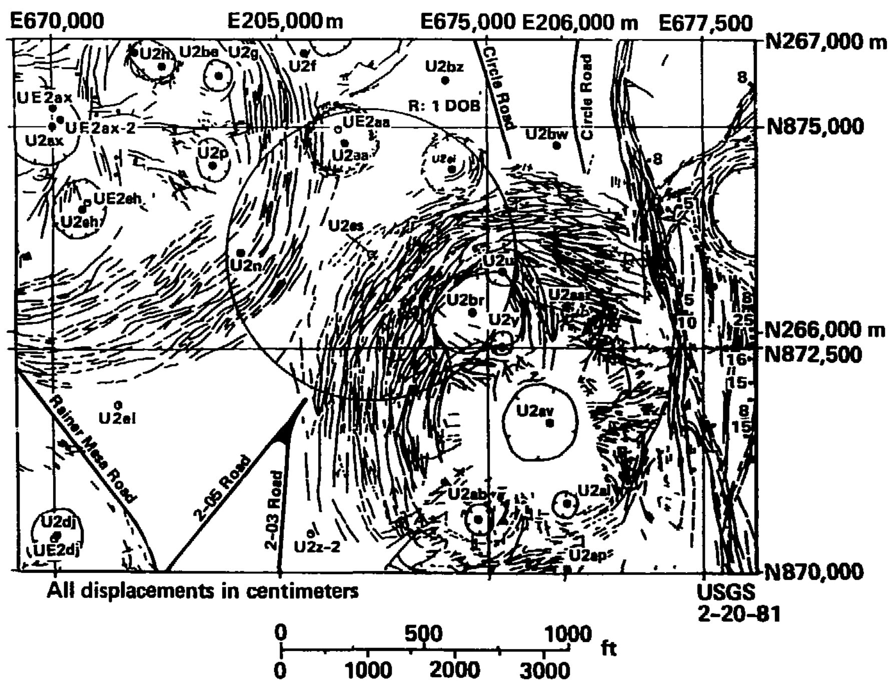


Figure A-8.



Pre-shot location map of the U2es site

Figure A-9.

DRILLING HISTORY

The emplacement hole, U2es, was spudded 1-5-81 and completed to a depth of 518 m on 1-29-81. Some sloughing has occurred, and there is fill in the hole whose depth is now 508 m. The hole's condition has been monitored with monthly caliper logs. Routine Birdwell geophysical logs were taken 1-23-81, through 3-8-81, and LLL-N logs from 3-18-81 to 8-21-81. The hole was sidewall sampled 2-15/18-81.

MEDIUM CHARACTERISTICS

Medium characteristics, tabulated in Table I, for a 19.1 radius averaging interval centered at 494 m in Paintbrush Tuff in U2es are derived from logs and samples from the emplacement hole. The sources of information and assumptions made in this analysis are shown in Table II.

Table III contains the previous experience for work points in unsaturated tuff of northern Yucca Flat, and that experience is graphically depicted in histograms on Figs. 1 and 2. All the properties are within previous experience.

Figure A-10.

U2ES GENERATED 08/25/81

DATA SUMMARY FOR WP = 494.0 M

WP MEDIUM IS TO

AVERAGING RADIUS = 19.08 M (18.78 UP, 19.37 DOWN)
AVERAGING INTERVAL = 475.2 TO 513.4 M

PARAMETER	MEAN VALUE	UNITS	NO. POINTS	STD DEV	ESTIMATED ERROR	DATA-RANGE	DEPTH-RANGE	AVERAGING METHOD	LOG-TYPE	RUN NO.
BULK DENSITY	1.69	MG/M3	101	0.10	0.08	1.61 TO 1.84	475.6 TO 506.0	SPH	GRAVITY BH	2
GRAIN DENSITY	2.46	MG/M3	12	0.07	0.02	2.40 TO 2.57	475.5 TO 508.4	SPH	SAMPLE SWT	1
WATER CONTENT	17.1	WT%	12	2.9	0.9	13.6 TO 22.7	475.5 TO 508.4	SPH	SAMPLE SWT	1
POROSITY	43.2	VOL%	12	*.	2.9	*. TO *.	475.2 TO 513.4	CALC	CALCULATED	
SATURATION	66.9	VOL%	12	*.	8.1	*. TO *.	475.2 TO 513.4	CALC	CALCULATED	
GAS POROSITY	14.3	VOL%	12	*.	4.4	*. TO *.	475.2 TO 513.4	CALC	CALCULATED	
CO2 CONTENT	<0.5	WT%	0	0.	0.	0. TO 0.	0. TO 0.	INT	SAMPLE SWT	1
DUAL SR VELOCITY	1019.	M/S	6	187.	*.	799. TO 1333.	481.6 TO 506.0	INT	DUAL SR	1
SEISMIC VELOCITY	2098.	M/S	2	*.	861.	*. TO *.	442.0 TO 502.9	INT	VIBROSEIS	1

OVERBURDEN PARAMETERS (WP-TO-SURFACE AVERAGES)

BULK DENSITY	=	1.93	MG/M3	+ - 0.10
DUAL SR VELOCITY	=	1308.	M/S	+ - 0.
SEISMIC VELOCITY	=	1782.	M/S	+ - 87.
GAS POROSITY	=	12.9	VOL%	

Zones of swelling clay greater than 20%:
None detected

BULK DENSITY IS NOT WATER-CORRECTED

Figure A-11.

PHYSICAL PROPERTIES DISTRIBUTION FOR SELECTED W.P. IN UNSATURATED TUFF

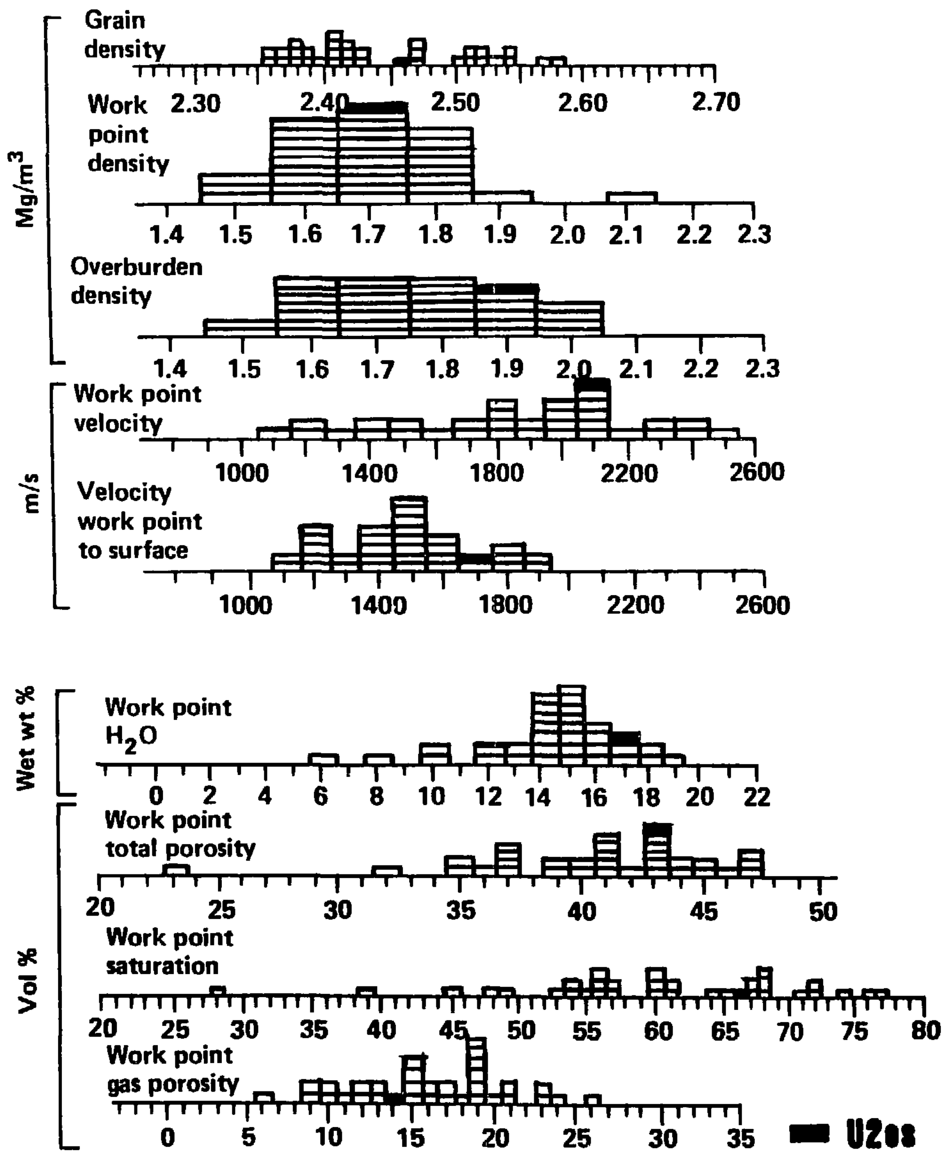


Figure A-12.

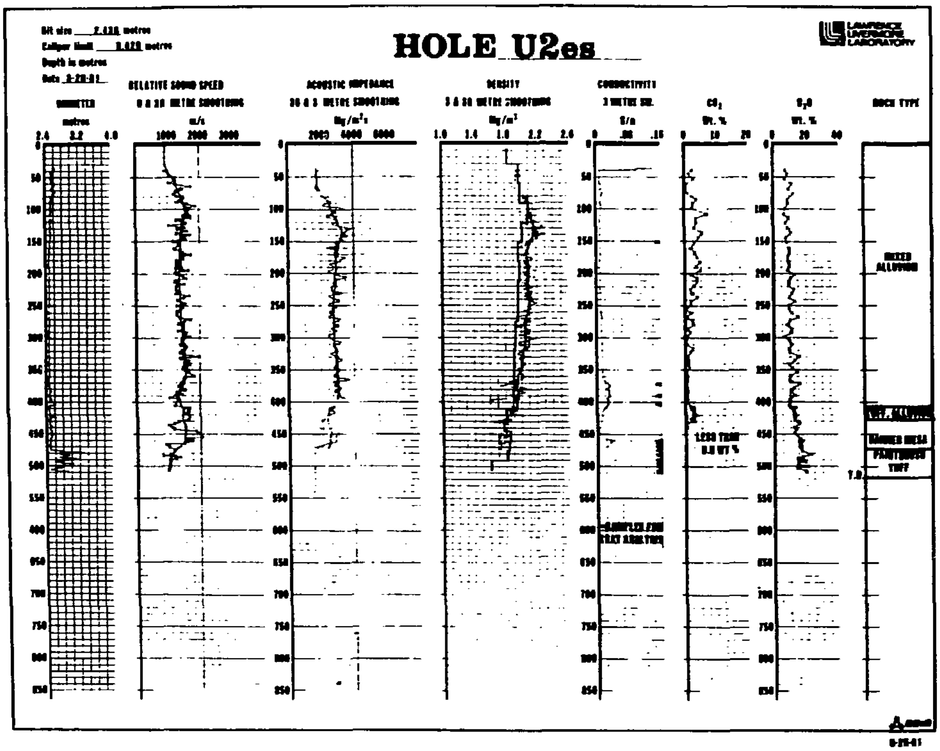


Figure A-13.

PREVIOUS GEOLOGIC EXPERIENCE
UNSATURATED TUFF
AREAS 2, 4, 8, 9, 10

Hole	m WP	Mg/m³ Density Overburden	Mg/m³ Density WP	Mg/m³ Density Grain	m/s Sonic Velocity WP to Surface	m/s Sonic Velocity WP	wt % H_2O	Vol % ϕ	Vol % Sat.	Vol % Gas ϕ
U2bc	331	1.90	1.60	2.39	1859	2134	15	43	55	19
U2br	519	2.00	1.80	2.43	1829	1798	15	37	76	10
U2dq	315	1.88	1.87*	2.52*	1504	2436	13	35	67	12
U2ds	314	2.00	2.10	2.52	1615	1829	8	23	72	6
U2dy	412	2.00	1.60*	2.42*	1560	2260	16	44	57	19
U2ei	530	1.89	1.54*	2.41*	1752	1981	17	47	56	21
U2eo	536	1.94	1.77*	2.47*	1774	2518	10	35	48	18
U4ah	331	1.81	1.65*	2.47*	1423	1880	14	43	56	19
U8c	271	1.76	1.69*	2.54*	1532	1962	15	43	57	19
U8e	420	1.87	1.73*	2.51*	1710	2096	14	41	60	16
U8k	323	1.76	1.72	2.57*	1580	2120	15	43	60	17
U8l	200	1.80	1.60*	2.41*	1484	1831	12	41	45	23
U9ch	378	1.60	1.60	2.38	1494	1829	18	45	64	15
U9ci	250	1.82	1.57*	2.36	1341	1859	10	40	39	24
U9cl	305	1.75	1.76*	2.42*	1400	2300	15	42	72	12
U9cm	326	1.78	1.85*	2.58*	1400	2210	15	39	71	11
U9cp	320	1.72	1.54*	2.40*	1540	1694	17	47	56	21
U9cq	320	1.74	1.65*	2.47*	1420	2010	16	44	61	17
U9cr	341	1.70	1.66*	2.53*	1620	1970	19	47	67	15
U9Its S-25	411	1.60	1.80	2.37	1463	2134	15	36	74	9
U9Its W-22	184	1.70	1.76*	2.43	1189	1372	14	37	65	13
U9Its W-24.5	201	1.50	1.50	2.37	1189	1189	15	46	49	23
U9Its XY-31	273	1.50	1.85	2.38	1372	1676	12	32	68	10
U9Its YZ-26	213	1.60	1.70	2.39	1189	1311	14	39	61	15
U9Its YZ-26	183	1.60	1.70	2.38	1158	1219	16	41	68	13
U9Its Z-27	244	1.60	1.70	2.51	1859	2408	6	37	28	26
U10aq	305	1.60	1.70	2.54	1463	1524	18	45	68	15
U10as	343	2.00	1.70	2.50	1494	2134	13	41	54	19
U10ax	267	1.70	1.80	2.41	1311	1585	16	37	77	9
U10bc	183	1.69	1.68*	2.41*	1129	1454	14	40	60	16
U10bd	200	1.69	1.65*	2.42*	1100	1385	14	41	54	19
U10bg	200	1.59	1.57*	2.36*	1162	1143	14	43	53	20
U2es	494	1.93	1.69*	2.46*	1730	2100	17	43	67	14

*water corrected density

MWH 7/31/81

Figure A-14.

APPENDIX B

SHEAR STRENGTH DATA FOR TUFFS AND ALLUVIUM OF THE NEVADA TEST SITE AND ITS VICINITY

Figures B-1 to B-11: tuffs.

Figures B-12 & B-13: alluviums.

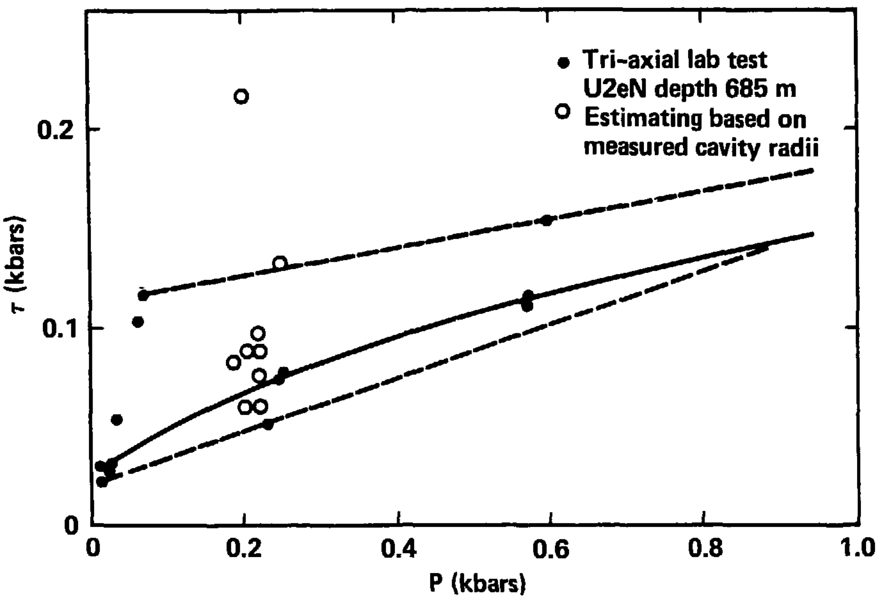


Figure B-1. Area 2.[60]

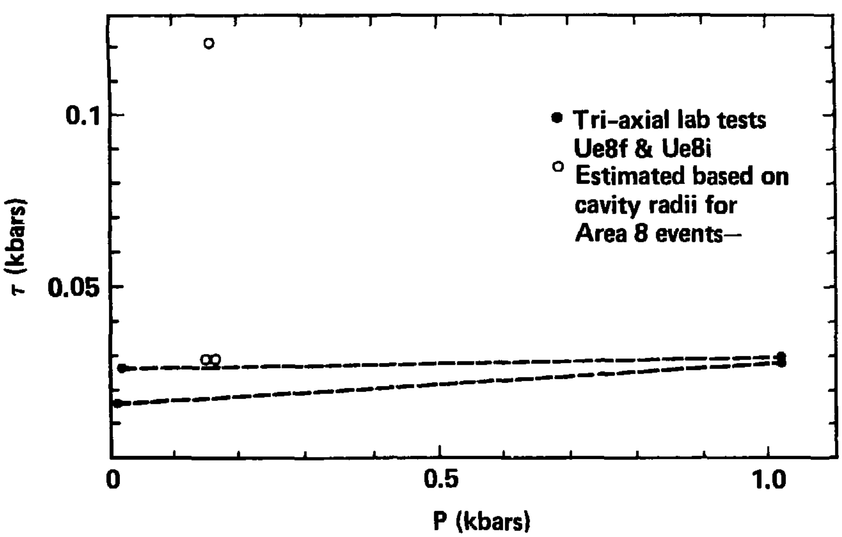


Figure B-2. Area 8.[60]

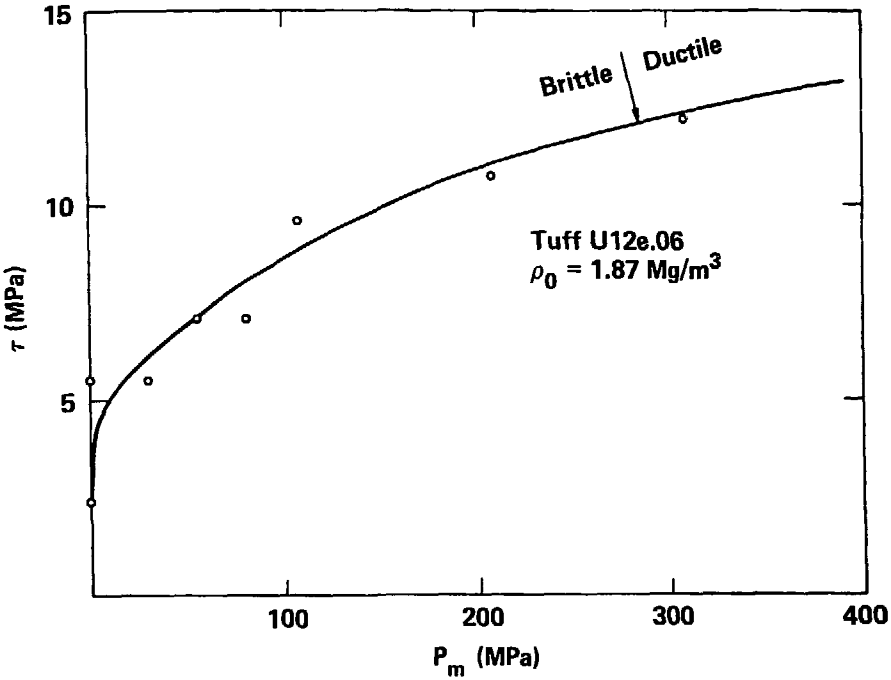


Figure B-3. Area 12.[27]

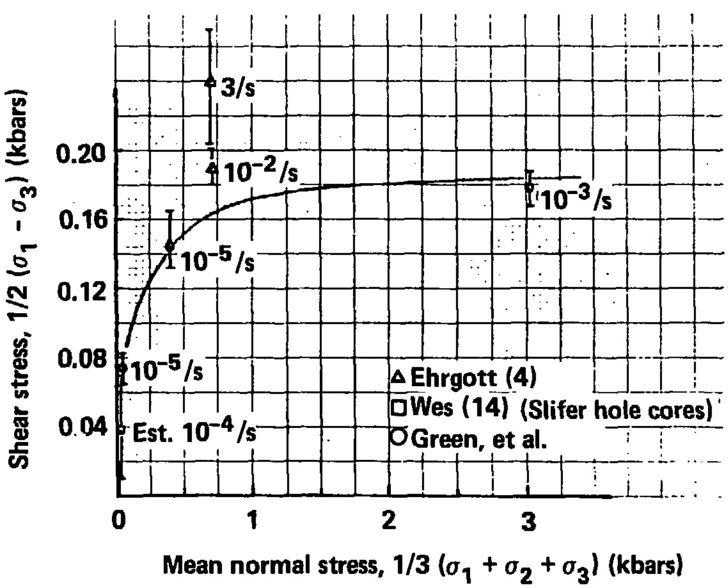


Figure B-4. Area 12.[25]

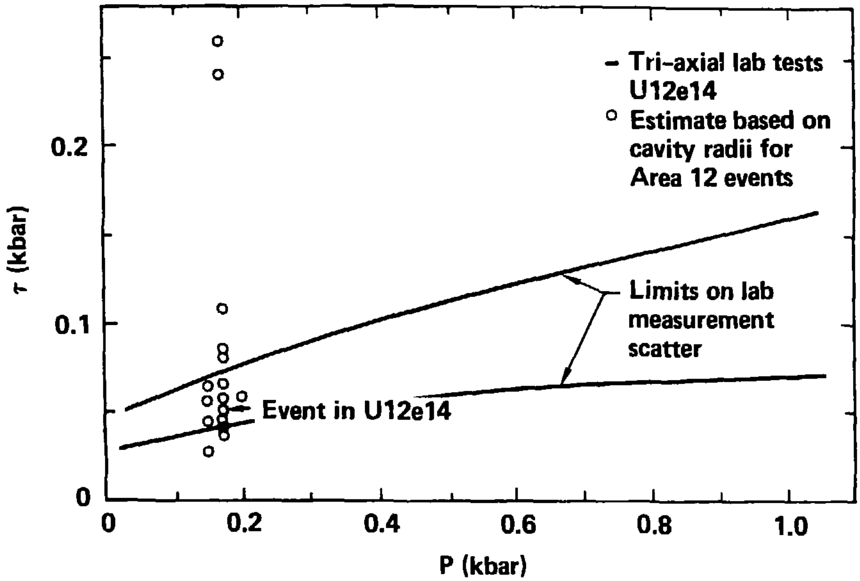


Figure B-5. Area 12.[60]

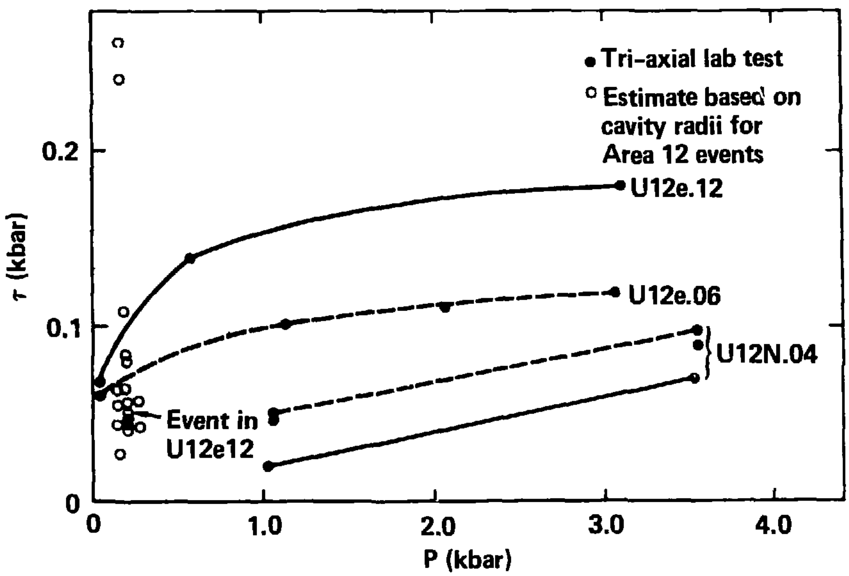


Figure B-6. Area 12.[60]

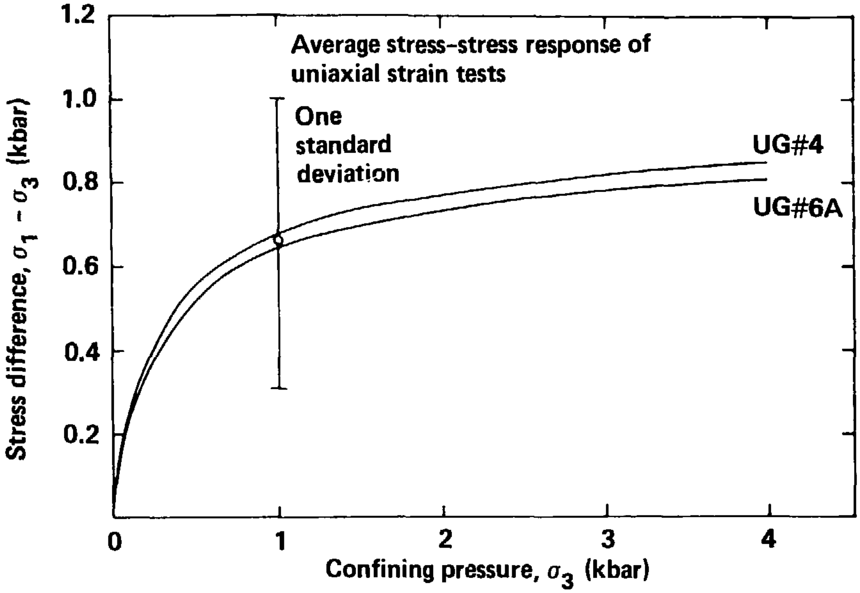


Figure B-7. Area 12.[31]

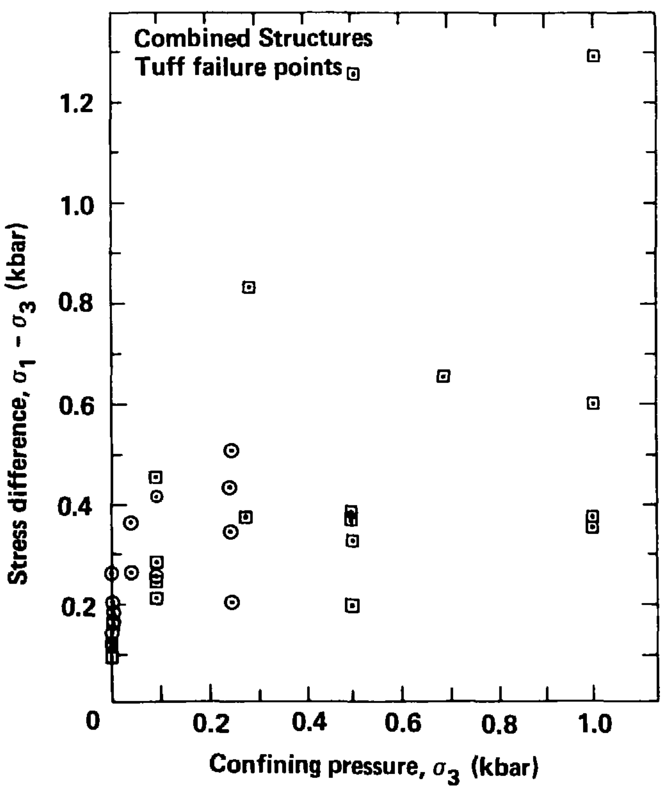


Figure B-8. Area 12.[31]

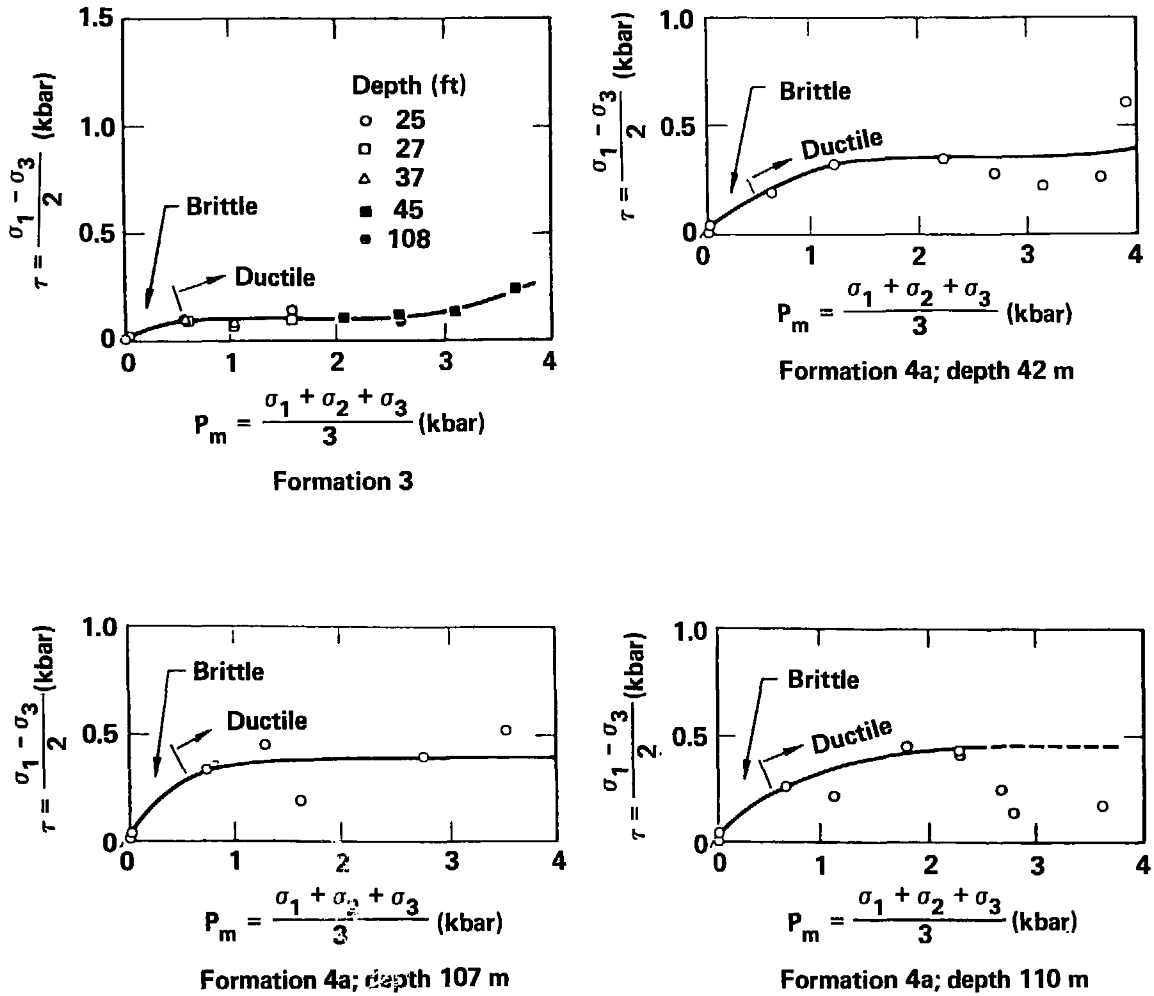
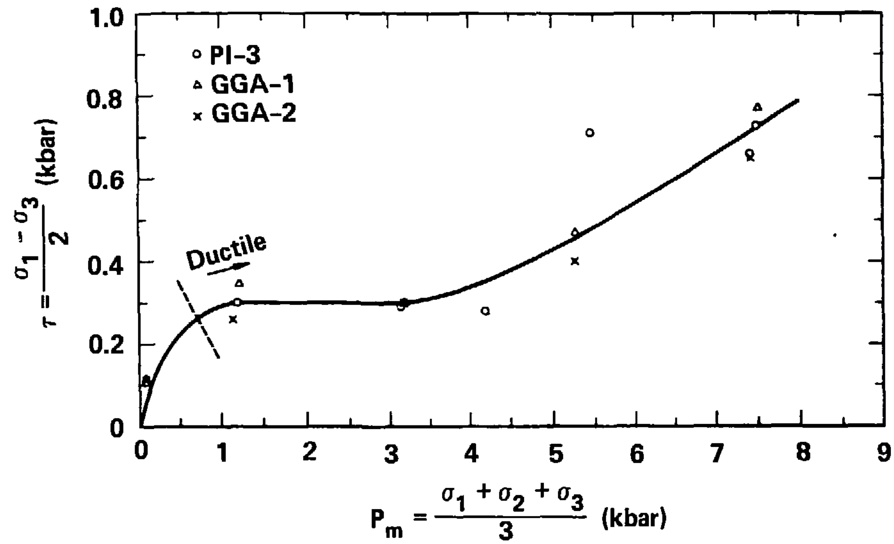
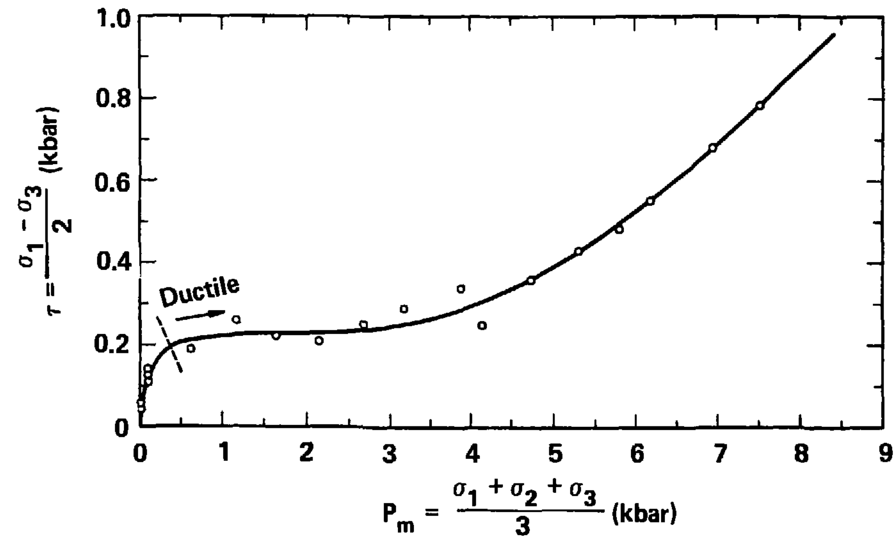


Figure B-9. Area 16 - Diamond dust.[22]



Instrument holes



Cavity

Figure B-10. Area 16 - Diamond Mine locations.[24]

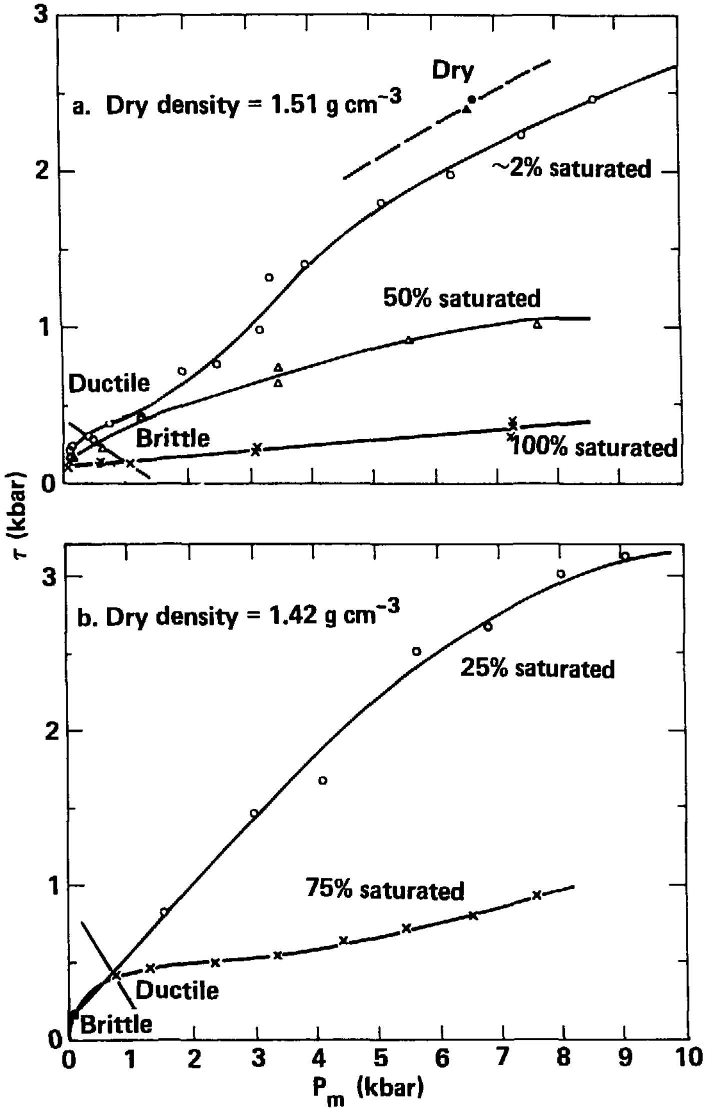


Figure B-11 Mt. Helen location.[29]

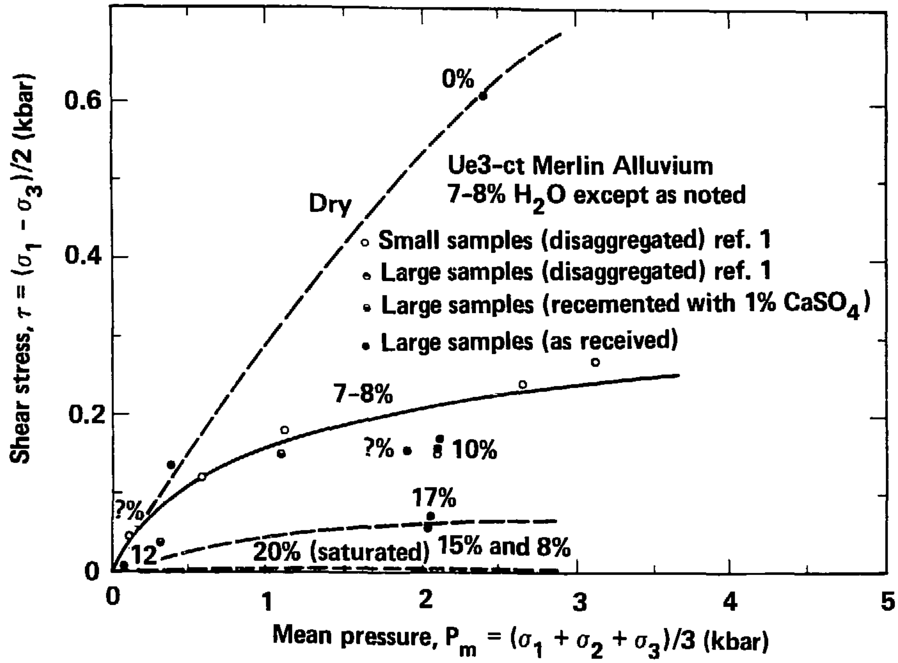


Figure B-12. Area 3.[26]

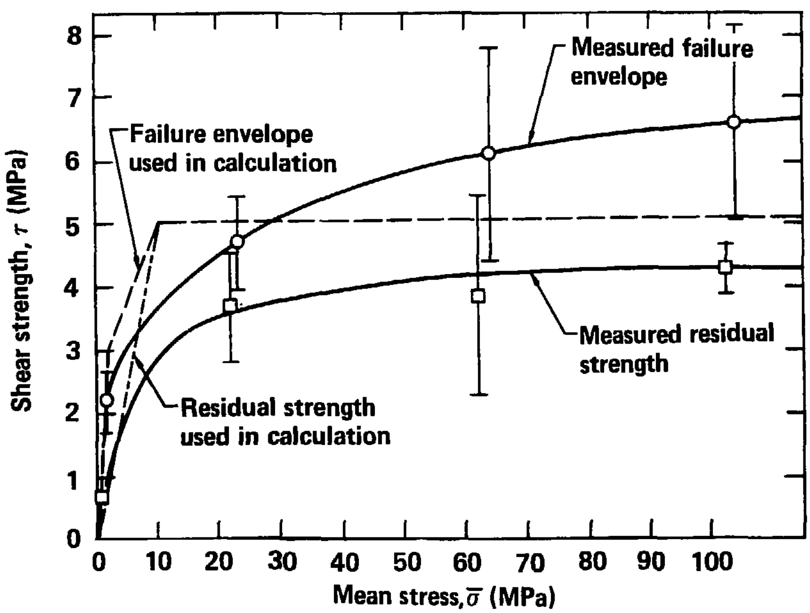


Figure B-13. Area 4.[59]

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