PDHonline Course C268 (4 PDH)

In Situ Subsurface Testing

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sets of a site have been identified and evaluated, their relative importance to ground water flow should be assessed. Joints and fractures can be evaluated by developing the structure and stratigraphy of the site from accessible outcrops and from borehole logs.

5-21. Pressure Tests

a. Pressure tests are performed to measure the permeability of zones within rock masses. Pressure test results are used in assessing leakage in the foundation and as a guide in estimating grouting requirements. Pressure tests are typically conducted during exploratory core drilling and are a relatively inexpensive method of obtaining important hydrogeologic information about a rock mass. Hydraulic pressure testing should be considered an integral part of the exploratory core drilling process in all cases where rock seepage characteristics could affect project safety, feasibility, or economy. The testing interval is typically 1.5 to 3 m (5 to 10 ft) but may be varied to fit specific geological conditions observed during the core drilling operations. Zones to be tested should be determined by (1) examining freshly extracted cores, (2) noting depths where drilling water was lost or gained, (3) noting drill rod drop, (4) performing borehole or TV camera surveys, and (5) conducting downhole geophysical surveys. In rock with vertical or high angle joints, inclined borings are necessary to obtain meaningful results. Types of tests and test procedures are described in Ziegler (1976), U.S. Department of Interior (1977), and Bertram (1979).

b. Pressures applied to the test section during tests should normally be limited to 23 kilo Pascals (kP) per meter (1 psi per foot) of depth above and 13 kP/meter (0.57 psi/foot) of depth below the piezometric surface. The limit was established to avoid jacking and damage to rock formations. The limit is conservative for massive igneous and metamorphic rocks. However, it should be closely adhered to for tests in horizontally bedded sedimentary and other similar types of formations. Naturally occurring excess water pressures (artesian) should be taken into account in computations for limiting test pressures. Where the test intervals are large, a reduction in total pressure may be necessary to prevent jacking of the formation within the upper portion of the test section.

c. An important, but often unrecognized, phenomenon in pressure testing is joint dilation and contraction as pressure is applied and released. In the case of a dam project, it is desirable to use pressures that will correspond to future reservoir conditions. Joint dilation can frequently be observed by conducting a "holding" test. The fall in pressure is observed and a plot of pressure versus time is made. The pressure should quickly drop to near the surrounding piezometric level if the joint openings remain the same width. The common observation of a slow pressure decay in pressure holding tests indicates joint closure with reduction in pressure.

d. Qualitative evaluations of leakage and grout requirements can be made from raw pressure test data (Ziegler 1976, U.S. Department of Interior 1977, Bertram 1979). Most analyses of this type assume laminar flow rather than turbulent flow. This assumption can be verified by conducting pressure tests on the same interval at several different pressures. If the water take is directly proportional to the total applied pressure, laminar flow can be assumed. If pressure test data are converted into values of equivalent permeability or transmissivity, calculations can be performed to estimate seepage quantities. Wherever possible, such results should be compared with data from completed projects where similar geologic conditions exist.

Section VI
In situ Testing to Determine Geotechnical Properties

5-22. In Situ Testing

In situ tests are often the best means for determining the engineering properties of subsurface materials and, in some cases, may be the only way to obtain meaningful results. Table 5-2 lists in situ tests and their purposes. In situ rock tests are performed to determine in situ stresses and deformation properties (moduli) of the jointed rock mass, shear strength of jointed rock masses or critically weak seams within the rock mass, and residual stresses along discontinuities or weak seams in the rock mass. Pressure tests have been discussed in Section V (paragraph 5-20) of this manual.

Table 5-2
In Situ Tests for Rock and Soil

		Applicability to	
Purpose of Test	**Type of Test**	**Soil**	**Rock**
Shear strength	Standard penetration test (SPT)	X	
	Field vane shear	X	
	Cone penetrometer test (CPT)	X	
	Direct shear	X	
	Plate bearing or jacking	X	X[1]
	Borehole direct shear[2]	X	
	Pressuremeter[2]		X
	Uniaxial compressive[2]		X
	Borehole jacking[2]		X
Bearing capacity	Plate bearing	X	X[1]
	Standard penetration	X	
Stress conditions	Hydraulic fracturing	X	X
	Pressuremeter	X	X[1]
	Overcoring		X
	Flatjack		X
	Uniaxial (tunnel) jacking	X	X
	Borehole jacking[2]		X
	Chamber (gallery) pressure[2]		X
Mass deformability	Geophysical (refraction)	X	X
	Pressuremeter or dilatometer	X	X[1]
	Plate bearing	X	X
	Standard penetration	X	
	Uniaxial (tunnel) jacking	X	X
	Borehole jacking[2]		X
	Chamber (gallery) pressure[2]		X
Relative density	Standard penetration	X	
	In situ sampling	X	
	Cone[2] penetrometer	X	
Liquefaction susceptibility	Standard penetration	X	
	Cone penetrometer test (CPT)[2]	X	

[1] Primarily for clay shales, badly decomposed, or moderately soft rocks, and rock with soft seams.
[2] Less frequently used.

a. Soils, clay shales, and moisture-sensitive rocks. Interpretation of in situ tests in soils, clay shales, and moisture-sensitive rocks requires consideration of the drainage that may occur during the test. Consolidation during testing makes it difficult to determine whether the test results correspond to unconsolidated-undrained, consolidated-undrained, consolidated-drained conditions, or intermediate conditions between these limiting states. The cone penetrometer test is very useful for detecting soft or weak layers and in quantifying undrained strength trends with depth. Interpretation of in situ test results requires complete evaluation of the test conditions and the limitations of the test procedure. ASTM D 3877-80 (ASTM 1996h) is the standard laboratory method for evaluating shrink/swell of soils due to subtraction or addition of water.

b. Rock. Rock formations are generally separated by natural joints, bedding planes, and other discontinuities resulting in a system of irregularly shaped blocks that respond as a discontinuum to various loading conditions. Response of a jointed rock mass to imposed loads involves a complex interaction of compression, sliding, wedging, rotation, and possibly fracturing of individual rock blocks. Individual blocks generally have relatively high strengths, whereas the strength along discontinuities is normally reduced and highly anisotropic. Commonly, little or no tensile strength exists across discontinuities. As a result, resolution of forces within the system generally cannot be accomplished by ordinary analytical methods. Large-scale, in situ tests tend to average out the effect of complex interactions. In situ tests in rock are generally expensive and should be reserved for projects with large, concentrated loads. Well-conducted tests, however, may be useful in reducing overly conservative, costly assumptions. Such tests should be located in the same general area as a proposed structure and test loading should be applied in the same direction as the proposed structural loading.

5-23. In situ Tests to Determine Shear Strength

Table 5-3 lists in situ tests that are useful for determining the shear strength of subsurface materials. In situ shear tests are discussed and compared by Nicholson (1983b) and Bowles (1996).

Table 5-3
In Situ Tests to Determine Shear Strength

Test	For Soils	For Rocks	Reference	Remarks
Standard penetration	X		EM 1110-2-1906 Appendix C	Use as index test only for strength. Develop local correlations. Unconfined compressive strength in tons/square foot is often 1/6 to 1/8 of N-value
Direct shear	X	X	RTH 321'	Expensive; use when representative undisturbed samples cannot be obtained
Field vane shear	X		EM 1110-2-1906, Appendix D, Al-Khafaji and Andersland (1992)	Use strength reduction factor
Plate bearing	X	X	ASTM[2] Designation D 1194 ASTM SPT **479[3]**	Evaluate consolidation effects that may occur during test
Uniaxial compression		X	RTH 324[1]	Primarily for weak rock; expensive since several sizes of specimens must be tested
Cone penetrometer test (CPT)	X		Schmertmann (1978a); Jamiolkowski et al. (1982)	Consolidated undrained strength of clays; requires estimate of bearing factor, N_c

' Rock Testing Handbook (USAEWES 1993).
[2] American Society for Testing and Materials (ASTM 1996a).
[3] Special Technical Publication 479 (ASTM 1970).

a. The Standard Penetration Test (SPT). The SPT is useful for preliminary appraisals of a site (Bowles 1996). The N-value has been empirically correlated with liquefaction susceptibility under seismic loadings (Seed 1979). The N-value is also useful for pile design. In cohesive soils, the N-value can be used to determine where undisturbed samples should be obtained. The N-value can also be used to estimate the bearing capacity (Meyerhof 1956; Parry 1977), the unconfined compressive strength of soils (Mitchell, Guzikowski, and Villet 1978), and settlement of footings in soil (Terzaghi, Peok, and Mesri 1996).

b. The Becker Penetration Test. The Becker drill (paragraph 5-5(2)) provides estimates of in situ soil strength and other properties similarly to the SPT, including coarse grained soils like gravel. The Becker penetration test was described in Harder and Seed (1986). The test consists of counting the number of hammer blows required to drive the casing 1 ft into the soil, for each foot of penetration. The test uses both open casing and plugged bits, commonly with a 14-cm (5.5-in.) or 17-cm (6.6-in.) OD casing and bits. Correlations of Becker blowcounts with SPT blowcounts have been developed to allow the use of Becker data in foundation investigations and in evaluation of liquefaction potential in coarse-grained soils under seismic loading.

c. Direct shear tests. In situ direct shear tests are expensive and are performed only where doubt exists about available shear strength data and where thin, soft, continuous layers exist within strong adjacent materials. The strength of most rock masses, and hence the stability of structures, is often controlled by the discontinuities separating two portions of the rock mass. Factors controlling the shear of a discontinuity include the loads imposed on the interface, the roughness of the discontinuity surfaces, the nature of the material between the rock blocks, and the pore water pressure within the discontinuity (Nicholson 1983b). In situ direct shear tests measure the shear strength along a discontinuity surface by isolating a block of rock and the discontinuity, subjecting the specimen to a normal load perpendicular to and another load (the shear load) parallel to the plane. The advantages of the direct shear test are: (1) its adaptability to field conditions, i.e., a trench, an adit, a tunnel, or in a calyx hole; (2) it is ideal for determining discontinuity shear strength because the failure plane and direction of failure are chosen before testing, accommodating anisotropic conditions; and (3) it allows for volume increases along the failure plane. Disadvantages of direct shear tests are their expense, the fact that they measure strength along only one potential failure plane, and the sometimes nonuniform application of normal stress during shearing. For the latter reasons, some engineers favor the triaxial compression test, which also can be performed in situ, for determination of shear strength (Ziegler 1972). The direct shear test measures peak and residual strength as a function of stress normal to the shear plane. Results are usually employed in limiting equilibrium analysis of slope stability problems or for stability analysis of foundations for large structures such as dams. Where field evidence suggests that only residual strengths can be relied on, either in a thin layer or in a mass, because of jointing, slickensiding, or old shear surfaces, in situ direct shear tests may be necessary. Few in situ direct shear tests are performed on soils, but they may be justified on clay shales, indurated clays, very soft rock, and on thin, continuous, weak seams that are difficult to sample. Ziegler (1972) and Nicholson (1983a,b) discuss the principles and methods of performing in situ direct shear tests. The Rock Testing Handbook (RTH) method RTH 321-80 (USAEWES 1993) provides the suggested method for determining in situ shear strength using the direct shear apparatus.

d. Field vane shear tests. Field vane tests performed in boreholes can be useful in soft, sensitive clays that are difficult to sample. The vane is attached to a rod and pushed into the soft soil at the bottom of the borehole. The assembly is rotated at a constant rate and the torque measured to provide the unconsolidated, undrained shear strength. The vane can be reactivated to measure the ultimate or residual strength (Hunt 1984). The vane shear test results are affected by soil anisotropy and by the presence of laminae of silt or sand (Terzaghi, Peck, and Mesri 1996). Shear is applied directionally. Failure of the soil occurs by shearing of horizontal and vertical surfaces. See Al-Khafaji and Andersland (1992) for a discussion of effects of soil anisotropy. The test may give results that are too high. Factors to correct the results are discussed in Bjerrum (1972) and Mitchell, Guzikewski, and Villet (1978). The test has been standardized as ASTM method D 2573 (ASTM 1996e).

e. Plate bearing tests. Plate bearing (plate-load) tests can be made on soil or soft rock. They are used to determine subgrade moduli and occasionally to determine strength. The usual procedure is to jack-load a 30- or 76-cm (12- or 30-in.) diam plate against a reaction to twice the design load and measure the deflection under each loading increment. The subgrade modulus is defined as the ratio of unit pressure to unit deflection, or force/length^3 (Hunt 1984). Because of their cost, such tests are normally performed during advanced design studies or during construction.

f. Cone penetrometer test or dutch cone. The Cone Penetrometer Test (CPT) can provide detailed information on soil stratigraphy and preliminary estimations of geotechnical properties. Based on the soil type as determined by the CPT (Douglas and Olsen 1981) or adjacent boring, the undrained strength can be estimated for clays (Jamiolkowski et al. 1982, Schmertmann 1970), and the relative density (and friction angle) estimated for sands (Durgunoglu and Mitchell 1975; Mitchell, Guzikewski, and Villet 1978; Schmertmann 1978b). For clays, a bearing factor, N_c, must be estimated to calculate the undrained strength from the CPT cone resistance and should be close or slightly greater than the CPT sleeve friction resistance if the soil is not sensitive or remolded (Douglas and Olsen 1981). The calculated undrained strength as well as the change of undrained strength with depth can both be used with several techniques to estimate the overconsolidation ratio (OCR) (Schmertmann 1978a). For sands, the relative density can be estimated if the overconsolidation conditions (i.e., lateral stress ratio) and vertical effective stress are known. The friction angle can also be estimated but also depends on the cone surface roughness and the assumed failure surface shape (Durgunoglu and Mitchell 1975). The mechanical (i.e., Dutch) cone is performed at a depth interval of 20 cm (8 in.) using hydraulic gages which measure the force from an inner rod directly in contact with the end of the probe. The electric (PQS or VGRO) cone is pushed at a constant speed for 1-m intervals while electronically measuring cone and friction resistance continuously.

5-24. Tests to Determine In Situ Stress

Table 5-4 lists the field tests that can be used to determine in situ stress conditions. The results are used in finite element analyses, estimating loading on tunnels, determining rock burst susceptibility in excavations, and identifying regional active and residual stresses. Stresses occur as a result of gravity forces, actively applied geologic forces such as regional tectonics, and from stored residual-strain energy. Stress is measured to determine the effect on foundations of changes in loading brought about by excavation or construction. Where a confining material has been removed by natural means or by excavation, the remaining material tends to approach a residual state of stress. In a majority of projects, the major principal stress is vertical, i.e., the weight of the overlying material. However, it has been found from measurements made throughout the world that horizontal stresses in the near-surface vicinity, defined as 30 m (100 ft) or less, can be one and one-half to three times higher than the vertical stress. Recognition of this condition during the design phase of investigations is very important. Where high horizontal stresses occur at a project site, the stability of cut slopes and tunnel excavations is affected. In situ testing is the most reliable method for obtaining the magnitude and direction of stresses. The three most common methods for determining in situ stresses are the overcoring, hydrofracture, and flatjack techniques.

Table 5-4
In Situ Tests to Determine Stress Conditions

Test	Soils	Rocks	Bibliographic Reference	Remarks
Hydraulic fracturing	X		Leach (1977) Mitchell, Guzikowski, and Villet (1978)	Only for normally consolidated or slightly consolidated soils
Hydraulic fracturing		X	RTH 344[1] Goodman (1981) Hamison (1978)	Stress measurements in deep holes for tunnels
Vane shear	X		Blight (1974)	Only for recently compacted clays, silts
Overcoring techniques		X	RTH 341[1] Goodman (1981) Rocha (1970)	Usually limited to shallow depth in rock
Flatjacks		X	RTH 343[1] Deklotz and Boisen (1970) Goodman (1981)	
Uniaxial (tunnel) jacking	X	X	RTH 365[1]	May be useful for measuring lateral stresses in clay shales and rocks, also in soils
Pressuremeter (Menard)	X		Al-Khafaji and Andersland (1992), Hunt (1984)	

[1] Rock Testing Handbook (USAEWES 1993).

a. Overcoring method. Possibly the most common method used for measuring in situ stresses in rock is overcoring, a stress-relief technique. An NW (75.7 mm (2.980 in.)) core hole is drilled, instrumented, and redrilled with a larger core barrel. The overcoring decouples the rock surrounding the instrument package from the natural stress field of the in-place formation. The change in strain recorded by the instruments is then converted to stress by using the elastic modulus of the rock determined from laboratory tests. At least three separate tests must be made in the rock mass in nonparallel boreholes. A detailed description of the field test is provided by RTH 341-80 (USAEWES 1993). The overcoring method is hampered by the necessity for many instrument lead wires that may be broken during testing. The practical maximum depth of testing is usually less than 45 m (150 ft).

b. Flatjack method. In the flatjack method, a slot is bored or cut into the rock wall midway between two inscribed points. Stresses present in the rock will tend to partially close the slot. A hydraulic flatjack is then inserted and grouted into the slot, and the rock is jacked back to its original position as determined by the inscribed points. The unit pressure required is a measure of the in situ stress. Flatjacks installed at different orientations provide a measure of anisotropy (Hunt 1984). The value recorded must be corrected for the influence of the tunnel excavation itself. Flatjack tests require an excavation or tunnel. The high cost for constructing the opening usually precludes this technique as an indexing tool except where the size of the structure and complexity of the site dictate its use.

c. Hydrofracture method. The hydrofracture method has been used in soils and rock. A section of hole is isolated with packers at depth, and an increasingly higher water pressure is applied to the zone. A point will be reached where the pressure begins to level off, and there is a marked increase in water take.

This indicates that a crack in the formation has opened, and the threshold pressure has been reached. The threshold pressure measures the minor principal stress component. The orientation is then obtained by an impression packer. This procedure then gives the intensity and direction of the minor principal stress, which is perpendicular to the crack. The hydrofracture method has no particular depth limitation, but drilling deep holes can be very expensive. This expense can often be circumvented by using holes that have been drilled for other purposes. Evidence indicates that stresses measured within 30 m (100 ft) or more of ground surface may not always reflect the actual stress magnitude or orientation at depth. This may be true particularly in areas where closely jointed and weathered surface rock formations are decoupled from the deeper, more intact rock.

5-25. Tests to Determine In Situ Deformation

Deformation characteristics of subsurface materials are of major importance in dynamic and seismic analyses for dams and other large structures, static design of concrete gravity and arch dams, tunnels, and certain military projects. Geotechnical investigations for such purposes should be planned jointly by geotechnical personnel and structural engineers. Deformation properties are normally expressed in terms of three interdependent parameters: Young's modulus, shear modulus, and Poisson's ratio. These parameters assume that materials are linear, elastic, homogeneous, and isotropic. In spite of this limitation, these parameters are often used to describe the deformation properties of soil and rock. Large-scale tests (e.g., tunnel jacking) are frequently used because they reduce the effect of nonhomogeneity. Multiple tests, with different orientations, can be used to determine the anisotropy of the deformation properties. Soils, in particular, tend to be nonlinear and inelastic. As a result, their properties are often strain dependent, i.e., moduli determined at low strain levels can be substantially different from those determined at high strain levels. The fact that sample disturbance, particularly in soils, can substantially affect the deformation properties serves as the primary reason for using in situ tests in soils. Table 5-5 lists the in situ tests used to determine one or more of the deformation parameters. Some test results are difficult to relate to the fundamental parameters but are used directly in empirical relationships (Table 5-6). Deformation properties of a jointed rock mass are very important if highly concentrated loadings are directed into the abutments of arch dams in directions that are tangent to the arches at the abutments. In these cases, the ratio of the deformation modulus of the abutment rock to that of the concrete in the dam must not be so low as to cause adverse tensile stresses to develop within the concrete dam. One problem often encountered in conducting in situ deformation tests is the need to include representative sizes of the jointed rock mass in the test, particularly if the joint spacing is moderately large (e.g., 0.6 to 0.9 m or 2 to 3 ft). This problem has been solved in some instances by excavating a chamber in rock, lining it with an impermeable membrane, and subjecting it to hydraulic pressure to load the rock over relatively large areas.

a. Chamber tests. Chamber tests are performed in large underground openings, such as exploratory tunnels. Preexisting openings, such as caves or mine chambers, can be used if available and applicable to project conditions. The opening is lined with an impermeable membrane and subjected to hydraulic pressure. Instrumented diametrical gages are used to record increases in tunnel diameter as the pressure load increases. The test is performed through several load-unload cycles. The data are subsequently analyzed to develop load-deformation curves from which a deformation modulus can be selected. The results are usually employed in the design of dam foundations and for the proportioning of pressure shaft and tunnel linings. The chamber test method is described by RTH 361-89 (USAEWES 1993).

Table 5-5
In Situ Tests to Determine Deformation Characteristics

Test	For Soils	For Rocks	Reference	Remarks
Geophysical refraction, cross-hole and downhole	X	X	EM 1110-1-1802	For determining dynamic Young's Modulus, E, at the small strain induced by test procedure. Test values for E must be reduced to values corresponding to strain levels induced by structure or seismic loads
Pressuremeter	X	X	RTH 362[1] Baguelin, Jezequel, and Shields (1978) Mitchell, Guzikowski, and Villet (1978)	Consider test as possibly useful but not fully evaluated. For soils and soft rocks, shales, etc.
Chamber test	X	X	Hall, Newmark, and Hendron (1974) Stagg and Zienkiewicz (1968)	
Uniaxial (tunnel) jacking	X	X	RTH 365[1] Stagg and Zienkiewicz (1968)	
Flatjacking		X	RTH 343[1] Deklotz and Boisen (1970) Goodman (1981)	
Borehole jack or dilatometer		X	RTH 363[1] Stagg and Zienkiewicz (1968)	
Plate bearing		X	RTH 364[1] ASTM STP **479**[2] Stagg and Zienkiewicz (1968)	
Plate bearing	X		MIL-STD 621A, Method 104	
Standard penetration	X		Hall, Newmark, and Hendron (1974)	Correlation with static or effective shear modulus, in Mpa (psi), of sands; settlement of footings on clay. Static shear modulus of sand is approximately: G_{aff} = 1960 N^{051} in Mpa (psi); N is SPT value

[1] Rock Testing Handbook (USAEWES 1993).
[2] American Society for Testing and Materials, Special Technical Publication 479 (ASTM 1970).

Table 5-6
Correlations Between Field Tests for Soils, Material Characteristics, and Structural Behavior

Field Test	Correlation With	Remarks
1 x 1-ft plate load test	Modulus of subgrade reaction. Settlement of footings on sand	Mitchell, Guzikowski, and Villet (1978)
Load test for radar towers	Young's modulus of subgrade soils	MIL-STD-621A
Standard penetration N-value	Settlement of footings and mats on sand; shear modulus	TM 5-818-1; Hall, Newmark, and Hendron (1974) Meyerhof **(1956)** Parry (1977) U.S. Army Engineer Waterways Experience Station (1954)
Cone penetrometer test	0 of sands; settlement of footings on sand; relative density	Mitchell, Guzikowski, and Villet (1978) Mitchell and Lunne (1978) Schmertmann (1978b) Durgunoglu and Mitchell (1975) Schmertmann (1970) Schmertmann (1978a,b)

b. Uniaxial jacking test. An alternative to chamber tests is the uniaxial jacking test (RTH 365-80 (USAEWES 1993)). The test uses a set of diametrically opposed jacks to test large zones of soil and rock. This method produces nearly comparable results with chamber tests without incurring the much greater expense. The test determines how foundation materials will react to controlled loading and unloading cycles and provides data on deformation moduli, creep, and rebound. The uniaxial jacking test is the preferred method for determining deformation properties of rock masses for large projects.

c. Other deformation tests. Other methods for measuring deformation properties of in situ rock are anchored cable pull tests, flatjack tests, borehole jacking tests, and radial jacking tests. The anchored cable pull test uses cables, anchored at depth in boreholes, to provide a reaction to large slabs or beams on the surface of the rock. The test is expensive and difficult to define mathematically but offers the advantages of reduced shearing strains and larger volumes of rock being incorporated in the test. Flatjack tests are flexible, and numerous configurations may be adopted. In relation to other deformation tests, the flatjack test is relatively inexpensive and useful where direct access is available to the rock face. Limitations to the method involve the relatively small volume of rock tested and the difficulty in defining a model for calculation of deformation or failure parameters.

(1) The borehole jack ("Goodman" jack), or dilatometer, and the Menard pressuremeter (Terzaghi, Peck, and Mesri 1996; Al-Khafaji and Andersland 1992; and Hunt 1984), which are applied through a borehole, have the primary advantage that direct access to the rock or soil face is not required. The dilatometer determines the deformability of a rock mass by subjecting a section of a borehole to mechanical jack pressure and measuring the resultant wall displacements. Elastic and deformation moduli are calculated. The pressuremeter performs a similar operation in soils and soft rock. The development of a mathematical model for the methods has proved to be more difficult than with most deformation measurement techniques.

(2) Radial jacking tests (RTH 367-89 (USAEWES 1993)) are similar in principle to the borehole jacking tests except that larger volumes of rock are involved in the testing. Typically, steel rings are placed within a tunnel with flatjacks placed between the rings and the tunnel surfaces. The tunnel is loaded radially and deformations are measured. The method is expensive but useful and is in the same category as chamber tests. All methods of deformation measurements have inherent advantages and disadvantages, and thus selection of test methods must be dictated by the nature of the soil or rock mass, the purpose of the test, and the magnitude of the project. Care must be exercised and limitations recognized in the interpretation and use of measurements of deformation.

5-26. Determination of Dynamic Moduli by Seismic Methods

Seismic methods, both downhole and surface, are used on occasion to determine in-place moduli of soil and rock (see Table 5-2). The compressional wave velocity is mathematically combined with the mass density to estimate a dynamic Young's modulus, and the shear wave velocity is similarly used to estimate the dynamic rigidity modulus. However, because particle displacement is so small and loading is transitory during these seismic tests, the resulting modulus values tend to be too high. The seismic method of measuring modulus should not be used in cases where a reliable static modulus value can be obtained. Even where the dynamic modulus is to be used for earthquake analyses, the modulus derived from seismic methods is too high. The moduli and damping characteristics of rock are strain dependent, and the strains imposed on the rock during seismic testing are several orders of magnitude lower than those imposed by a significant earthquake. Generally, as the strain levels increase, the shear modulus and Young's modulus decrease and the damping increases. Consideration of these factors is necessary for earthquake analyses.

Section VII
Backfilling of Holes and Disposition of Samples and Cores

5-27. Backfilling Boreholes and Exploratory Excavations

Except where the hole is being preserved for future use, all boreholes and exploratory excavations should be backfilled. The reasons for backfilling holes are to: eliminate safety hazards for personnel and animals, prevent contamination of aquifers, minimize underseepage problems of dams and levees, and minimize adverse environmental impacts. Many states have requirements regarding backfilling boreholes; therefore, appropriate state officials should be consulted. Holes preserved for the installation of instrumentation, borehole examination, or downhole geophysical work should be backfilled when no longer needed. As a minimum, borings that are preserved for future use should be protected with a short section of surface casing, capped, and identified. Test pits, trenches, and shafts should be provided with suitable covers or barricades until they are backfilled. Where conditions permit, exploratory tunnels may be sealed in lieu of backfilling. Procedures for backfilling boreholes and exploratory excavations are discussed in Appendix F, Chapter 10, of this manual.

5-28. Disposition of Soil Samples

Soil samples may be discarded once the testing program for which they were taken is complete. Geotechnical samples of soil and/or rock collected from areas suspected of containing HTRW *should be properly disposed of* Soil samples are not normally retained for long periods, because even the most careful sealing and storing procedures cannot prevent the physical and chemical changes that, in time, would invalidate any subsequent test results. Requirements for the disposition of soil samples from plant pest quarantined areas are specified in ER 1110-1-5.

5-29. Disposition of Rock Cores

All exploratory and other cores not used for test purposes shall be properly preserved, boxed, and stored in a protected storage facility until disposal. The following procedures govern the ultimate disposition of the cores.

a. Care and storage. Filled core boxes can be temporarily protected at the drilling site by wrapping them in plastic sheeting and preventing direct contact of the boxes with the ground. Exploratory or other cores, regardless of age, will be retained until the detailed logs, photographs, and test data have been made a matter of permanent record. Precautions shall be taken to ensure against the disposal, destruction, or loss of cores that may have a bearing on any unsettled claim. Such cores shall be retained until final settlement of all obligations and claims. They then will be disposed of in accordance with the procedures outlined in the following text.

b. Disposal. Cores over 15 cm (6 in.) in diameter may be discarded after they have served their special purpose. Geotechnical samples of soil and/or rock collected from areas suspected of containing HTRW *should be properly disposed of* In a case where the project is deauthorized, all associated cores may be discarded. When a project has been completed and final settlement has been made with the contractors and others concerned, all cores, except those related to future construction, and a few selected cores representative of foundation and abutment conditions, may be discarded. Selected cores, retained after the completion of a project, and additions thereto, may be discarded or otherwise disposed of 5 years after final completion of the project, provided no unforeseen foundation or abutment conditions have developed. After cores are disposed of, core boxes should be salvaged for reuse if their condition permits.